DRAFT

Design Criteria and Considerations Technical Memorandum

San Francisquito Creek
Flood Protection Capital Project

Floodwater Conveyance Improvements from East Bayshore Road to San Francisco Bay

SAN FRANCISQUITO CREEK
JOINT POWERS AUTHORITY

March 2012

HDR

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1.0 Introduction

1.1 Purpose and Scope

The purpose of this project is to provide San Francisquito Creek (SFC) conveyance of the one-percent (100-year flood event) design flood flow from the downstream face of East Bayshore Road to the San Francisco Bay. The San Francisquito Creek Joint Powers Authority (SFCJPA) has selected a design alternative at the July 23, 2009 Board of Directors meeting in response to historical flooding in Palo Alto project area, and in continuance of ongoing evaluation of the San Francisquito Creek flood protection system. The San Francisquito Creek flood protection system requires the completion of multiple projects both upstream and downstream of East Bayshore Road to make improvements in flood protection for the entire area. This project from the downstream face of East Bayshore Road to the San Francisco Bay, also referred to as the downstream project, is one of several projects needed to reduce flooding in the lower reach of the creek.

The alternative selected on July 23, 2009 is Alternative 2 proposed by Philip Williams & Associates, Ltd. (PWA, dated July 2009), which includes channel geometry modification, floodwall installation, existing levee degrade, reconfiguration of a portion of the Palo Alto Golf Course, and an outlet structure for the proposed enlargement of the Highway 101/East Bayshore Road Bridge. Figure 1, Project Site Location, illustrates the location of the project and where modifications will be implemented.

HDR Engineering, Inc. was hired by SFCJPA to develop plans, specifications, and cost estimates (PS&E) in order to construct PWA's proposed Alternative 2 design. Some refinement to PWA's hydraulic HEC-RAS model has occurred to better represent the intended design. Changes to the original Alternative 2 hydraulic model are described in Section 3.0, Proposed Project Features, that include both hydraulic modeling refinements and modification to the levee design geometry template. With this revised Alternative 2 model, HDR is proceeding forward in developing PS&E.

The purpose of this document is to summarize the design criteria and considerations that will guide development of PS&E for the flood conveyance improvement of San Francisquito Creek between East Bayshore Road and the San Francisco Bay based upon the revised Alternative 2 design. Design criteria are defined for all major project components, including survey control, development of a design water surface elevation (WSE), levee embankment and flood wall geometry and configuration, seepage, drainage, stabilization and erosion control features, project access, and project interaction with existing infrastructure, including utility modification and/or re-location. Considerations that will be incorporated into the design area also include easement, stormwater collection and discharge, tidal effects, freeboard and uncertainty, Federal Emergency Management Agency (FEMA) Conditional Letter of Map Revision (CLOMR), efficiency, effectiveness, and acceptability of the overall project.

1.2 Project Description

1.2.1 Project Area

The San Francisquito Creek watershed encompasses a 45-square-mile basin, extending from Skyline Boulevard to the west and San Francisco Bay to the east. The watershed includes public lands, Stanford University, unincorporated areas within the counties of San Mateo and Santa Clara, and numerous private landowners in the cities of East Palo Alto, Menlo Park, Palo Alto, Portola Valley and Woodside.

The San Francisquito Creek serves as a boundary between the two counties of San Mateo and Santa Clara in the lower watershed. The San Francisquito Creek is the last relatively unaltered urban creek system in the San Francisco South Bay. The headwaters of San Francisquito Creek are comprised of Corte Madera Creek and Bear Creek, and are formed just below Searsville Dam in Stanford University's Jasper Ridge Biological Preserve. The mouth of the creek opens to the San Francisco Bay adjacent to the Palo Alto Airport and the Baylands Nature Preserve. This project consists of the north and south banks of a 1.45 mile segment of San Francisquito Creek extending from East Bayshore Road to San Francisco Bay, as shown on Figure 1.

1.2.2 Project Goals and Objectives

The SFCJPA was created to support the member agencies in providing services to the citizens of the watershed and coordinate activities that improve their quality of life around the watershed they share. The founding SFCJPA Agreement identifies several purposes for the agency, including:

- "…join together for the primary purpose of managing the joint contribution of services and providing policy direction on issues of mutual concern relating to the creek…"
- Plan flood control measures and recommend funding and alternatives for flood control.
- Facilitate and perform bank stabilization, channel clearing, and other creek maintenance.
- Preserve and enhance environmental values and in-stream uses of the creek.
- Coordinate emergency mitigation and response activities.

The project goals and objectives for HDR's scope of work include:

- Work with the SFCJPA, its member agencies, and the public to develop a feasible design that is desirable and that meets flood control needs while considering possible ecosystem and recreational improvements.
- Design project features to meet requirements for the Code of Federal Regulations (CFR), Title 44, Volume 1, Chapter 1, Section 65.10 (44 CFR 65.10) and California Code of Regulation Title 23.

- Provide 100-year event flood (1% annual chance flood event) riverine protection within the project extents.
- Create project features that will complement a future California Department of Transportation (Caltrans) bridge widening project at West Bayshore Road, Highway 101 and East Bayshore Road over the San Francisquito Creek.

Based upon existing conditions, hydraulic analysis previously performed, and historical flooding events, the existing project levee does not meet a 100-year level of flood protection. Project engineering objectives include the correction of the following levee deficiencies:

- Inadequate levee height for the design water surface elevation (WSE).
- Channel capacity for the design flows.
- Vegetative and structural encroachments into the levee prism.

The following potential geotechnical and hydraulic issues will be addressed during analysis and design:

- Underseepage with excessive hydraulic exit gradients.
- Slope instability and through seepage.
- Foundation settlement and consolidation.
- Susceptibility to bank erosion and scour.
- Sea level rise.

1.3 Previous Studies

The following hydrologic, hydraulic, geotechnical, environmental and survey reports are some of the basis of design reference documents for the San Francisquito Creek Flood Protection Capital Project between East Bayshore Road and the San Francisco Bay. Section 9 contains the complete list of reference documents considered during development of this design criteria document. Note that the San Francisquito Creek mapping was completed by Towill, Inc. as part of HDR's design contract.

- Santa Clara Valley Water District (2010), Right of Way Study. May.
- Towill, Inc. (2010), San Francisquito Creek Mapping, DTM & Orthophotos, San Francisquito Creek from Highway 101 to SF Bay. March.
- USACE San Francisco District (2009), Appendix C, DRAFT Geotechnical Appendix and Reliability Analysis of Downstream Floodwalls and Levees, San Francisquito Creek, F3 Milestone without Project. December.
- Light, Air & Space Construction Environmental Services Company (2009), San Francisquito Creek Hazardous Toxic Radioactive Waste Study, San Francisco Bay to

Searsville Dam Plus Additional 5-Square Mile Study Area, Santa Clara and San Mateo Counties, California. November 20.

- Philip Williams & Associates, Ltd. (2009), San Francisquito Creek Flood Reduction Alternatives Analysis. July 17.
- Noble Consultants, Inc. (2009), Final Report, San Francisquito Creek, Development and Calibration/Verification of Hydraulic Model. May 26.
- Wang et al (2007), Santa Clara Valley Water District, San Francisquito Creek Hydrology Report. December.
- Santa Clara County (2006), LiDAR of San Francisquito Creek and surrounding areas. May. (Adjusted by HJW Geospatial in Sept. 2008 using Bestor Engineers TIN.)

2.0 Existing Site Conditions

2.1 Land Use

A range of commercial and municipal facilities are located along the south levee of the project reach, which include various commercial enterprises, the International School of the Peninsula, a US Post Office facility, the Baylands Athletic Center, the Palo Alto Municipal Golf Course, and the Palo Alto Airport of Santa Clara County. The north levee is abutted by residential housing and the Palo Alto Baylands Nature Preserve, with some commercial buildings immediately adjacent to East Bayshore Road. The Baylands Trail runs along the crown of the south levee from the Geng Road access point to the mouth of the creek, including the crossing over Friendship Bridge. The San Francisquito Creek Stormwater Pump Station is located toward the upstream end of the reach along the south levee, and discharges stormwater originating upstream of Highway 101 collected via culvert, into a man-made off-stream channel that leads into the creek. O'Connor Street Pump Station is located on the north side of the creek, adjacent to and upstream of Friendship Bridge, and discharges stormwater collected from East Palo Alto into San Francisquito Creek. The area upstream of Highway 101 is also highly developed with both commercial and residential housing along both the left and right overbanks of the creek.

2.2 General Site Features

2.2.1 Levee

The existing levees vary in height relative to the landside toe, ranging from approximately 4 feet (ft) to 13 ft. Crown widths range from 10 to 26 ft for the left bank levee and from 8 to 16 feet for the right bank levee. Within the middle reach, the levees are generally over-steepened, with an approximate slope of 2 horizontal to 1 vertical (2H:1V) or steeper. The San Francisquito Creek meanders between the right and left bank levees, resulting in terraces, or benches, between 20 and 80 ft wide on the left bank and 20 and 50 ft wide on the right bank between the levee toe and low flow channel. On the landside of the right bank levee, for the reach upstream of Friendship Bridge, significant amounts of construction debris are mounded in close proximity to, and in some cases overlying, the levee prism's landside slope. In addition, community gardens and a neighborhood bicycle motor cross course have been constructed in this area.

The channel and earthen levees in the Levee Restoration area were constructed in 1958 in a cooperative effort between San Mateo County and the Santa Clara County Flood Control and Water Conservation District to provide flood protection (the Santa Clara County Flood Control District was the predecessor of the SCVWD.) The levees are no longer at their 1958 "As-Built" elevations due to land subsidence, settlement and erosion. The channel contains significant deposits of sediment, both from upstream sediments transported downstream and tidal backwater action from San Francisco Bay. A combination of increased siltation of the channel and settlement of the levees has resulted in a decrease in channel capacity.

There are numerous vegetative and man-made encroachments on the landside of the existing slopes. These encroachments include vegetation of various sizes, residential and commercial structures, underground utility piping, and structural towers supporting overhead high voltage electrical transmission lines.

2.2.2 Utilities

An inventory of known existing utilities within the project footprint has been assembled and will be considered during design and construction. The location of utilities were either obtained from an aerial survey completed by Towill in March 2010, a ground location completed by California Utility Surveys (CUS) in January 2010, data received from utility companies, or as-built plans provided by the member agencies of the SFCJPA.

2.2.3 Bridge Structure Crossings

There are two bridge structures impacting the development of the Alternative 2 design, which include Hwy 101/ Bayshore Road and Friendship Bridge. Although the Hwy 101/ Bayshore Road is located at the upstream limit of the project study, the future construction of the bridge structure has major impacts upon the design WSE downstream.

Caltrans is currently at a 30% design level for the reconfiguration of Bayshore Road, Hwy 101, and East Bayshore Road, combining all three structures into one bridge spanning over the San Francisquito Creek. Changes to their design may have significant impact to WSEs downstream.

Downstream of Hwy 101 near station 29+88, Friendship Bridge, a pedestrian bridge, spans the existing creek. The geometry of the bridge constricts the channel significantly. The Alternative 2 design recommends an overflow bypass terrace into the golf course to widen the channel floodplain.

2.2.4 Baylands Trail

The Baylands Trail runs along the crown of the left bank levee from the Geng Road access point downstream to the mouth of the creek, including the pedestrian crossing over Friendship Bridge. Access to the trail will be disrupted during construction, however an aspect of the design is to restore access and routes for pedestrian and bicycle traffic after the project is complete to a pre-project usage condition.

2.2.5 Pumping Stations

The San Francisquito Creek Storm Water Pump Station is located approximately 75 ft northeast of East Bayshore Road on the left bank of San Francisquito Creek, setback approximately 270 ft from the creek centerline. The Storm Water Pump Station discharge outlet channel is approximately 400 feet downstream of East Bayshore Road. An 8-ft by 8-ft reinforced concrete box (RCB) culvert serves as an inlet on the south face of the pump station, connecting from a 96-inch diameter gravity drain under East Bayshore Road. The pump station discharge channel extends 300 ft from the north face of the pump station to the San Francisquito Creek centerline. Both banks corresponding to the pump station discharge channel are set equal to the levee crest elevations at the points of intersection, and the channel invert has been graded to match the channel elevation at their point of intersection.

The O'Connor Street Pump Station is located on the right bank of San Francisquito Creek, directly across from the northwest corner of the Palo Alto Municipal Golf Course and immediately upstream of Friendship Bridge. The structure extends from the channel to the landside; the structure is situated within the entire levee prism. A 60-inch diameter inlet pipe connects to the landward side of the structure. The pump station discharges to San Francisquito Creek via one 14-inch and four 30-inch diameter steel pipes. These pipes outfall to a concrete discharge structure constructed within the waterside slope of the levee prism.

2.2.6 Palo Alto Baylands Nature Preserve

A portion of the Palo Alto Baylands Nature Preserve, also known as the Faber Tract, is located on the land side of the right bank levee between Friendship Bridge and the mouth of the Creek. This area of land is owned by the City of Palo Alto and managed by the U.S. Fish and Wildlife Service.

2.2.7 Palo Alto Airport

The Palo Alto Airport is adjacent to the left levee near Station 10+00. Coordination with the Airport will be required to ensure that proposed design and associated construction activities will comply with Federal Aviation Administration (FAA) and airport regulations.

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3.0 Proposed Project Features

The SFCJPA has determined that the San Francisquito Creek Flood Protection Capital Project Downstream Section from East Bayshore Road to the San Francisco Bay shall be designed in accordance with Alternative 2, as detailed in the Alternatives Analysis conducted by Philip Williams & Associates (PWA), in July 2009. Alternative 2 consists of setback floodwalls in the upper reach, levee setbacks in the middle reach, an overflow terrace near Friendship Bridge, and a degrade of the existing right bank levee between the Faber Tract and the creek.

The upper reach extends from East Bayshore Road to the Palo Alto Municipal Golf Course, the middle reach extends from the golf course to Friendship Bridge and lower reach extends from Friendship Bridge to the San Francisco Bay. See Appendix A which illustrates the Conceptual Layout of Alternative 2, as prepared by PWA, as well as several sections along the project reaches.

As previously mentioned in Section 1.1, changes to PWA's Alternative 2 hydraulic model were needed to better represent SFCJPA's design intentions. Changes to the hydraulic model include removing multiple downstream cross-sections, reducing roughness coefficients for a downstream cross-section, adding ineffective flow areas to more appropriately model the tidal marsh area, and narrowing cross-section geometry to avoid structure acquisitions (a minor design modification to avoid impacts with private property).

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4.0 Survey and Mapping Criteria

4.1 Project Datum

Bestor Engineers, Inc. (Bestor) completed a survey in 2008 that included San Francisquito Creek to the extent of its top of bank elevations, and was developed for the USACE San Francisco District. This survey was supplemented by aerial mapping performed by Towill, Inc. (Towill) for the purposes of this project in 2010. The horizontal datum of both surveys were based on the North American Datum of 1983 (NAD83), California Zone 3. The vertical datum of the survey was based on the North American Vertical Datum of 1988 (NAVD88). Both datums are the official datums used for the primary geodetic network in North America, and are established by the National Geodetic Survey.

4.2 Datum Conversion

The vertical datum of the survey for this project is the North American Vertical Datum of 1988 (NAVD88). Should any conversions be required, any values recorded in the National Geodetic Vertical Datum of 1929 (NGVD29) shall be converted to NAVD88 using a conversion factor of 2.75 feet (0ft NGVD = +2.75ft NAVD), based upon the PWA Alternatives Analysis Report, and confirmation by Towill, Inc. in 2010.

4.3 Topographic Survey and Aerial Photo

In 2006, Bestor was hired by the USACE San Francisco District to provide mapping, for hydraulic modeling purposes, of San Francisquito Creek, from San Francisco Bay to approximately 10 miles upstream of the mouth. The survey included a detailed topographic survey of the creek channel using ground survey methods. A bathymetric survey was also performed along the creek channel between the mouth of the creek and the bridge at Highway 101. The bathymetric survey and conventional survey were merged, and were supplemented by an existing LiDAR (Light Detection and Ranging) set provided by the Santa Clara Valley Water District (SCVWD) in order to form a single continuous digital terrain model (DTM) of the entire San Francisquito Creek floodplain.

In 2010, Towill performed aerial mapping of the project area and Faber Tract in order to cover the entire project area downstream of East Bayshore Road. By compiling the previously mentioned surveys, the compiled survey provides planimetric detail sufficient for engineering design. Aerial photography was flown to support a scale of 1”=40’, with 1 ft contours. Contours were generated from the Bestor DTM.

A color digital ortho-rectified aerial photo was taken with a pixel resolution of 0.2 ft.

At this time, no topographic certification describing the level of accuracy of the existing data has been provided. HDR has assumed that the data is acceptable for this level of design and is moving forward with design per the direction of the SFCJPA.

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5.0 Levee Design Criteria

Criteria that will be applied to the design of flood protection improvements for this project will be based on published Federal and State guidelines and design standards, included in the following regulations:

- Code of Federal Regulations (CFR), Title 44, Volume 1, Chapter I, Section 65.10 (44CFR65.10)
- California Code of Regulations (CCR) Title 23

While Title 23 is specific to California's Central Valley, applicable standards of practice will be referenced from this document wherever practicable. In addition to these publications, HDR recommends adhering to applicable USACE and DWR guidelines for the design of flood protection systems. While the CFR and CCR reference several USACE publications, there are several additional documents that provide standards and guidance for analyses and practices relevant to levee construction and maintenance.

For levees to be accredited by FEMA, they must be certified by a licensed professional engineer or a Federal agency responsible for their design. Certification is a finding that, with reasonable assurance, sufficient data exists that the system in question provides protection from the 100-year flood event. These requirements are outlined in 44 CFR 65.10. The USACE and DWR criteria will be followed for the design of levees based on the requirements of 44 CFR 65.10 as well as recent publications not cited in 44 CFR 65.10. This includes design criteria for levee geometry, seepage, slope stability, and levee settlement. Additionally, requirements for freeboard, closure structures, embankment erosion protection, interior drainage, and the requirements for an O&M plan are addressed in 44 CFR 65.10.

The most recent USACE published documents, including Engineer Manuals (EM), Engineer Regulations (ER), Engineer Circulars (EC) and Engineer Technical Letters (ETL), will be the basis for the design criteria. These references can be found in Section 9.1.

5.1 General Levee Cross-Sectional Geometry

USACE EM 1110-2-1913, Design and Construction of Levees (USACE 2000) and California Department of Water Resources, California Code of Regulations, Title 23, were consulted for determining the minimum levee geometry. Based on this document, the following levee section will be used for levee design:

- Minimum levee crown width of 16 ft (USACE requires a minimum 12 ft crown for minor tributaries, but discussions with SFCJPA have resulted in a wider crown for maintenance and inspection activities). Narrower crown widths may be required based on right-of-way limitations, on a case-by-case basis, as approved by the SFCJPA.
- Landside slope 2H:1V or flatter.

- Levee waterside slope 3H:1V or flatter.

5.2 Levee Fill Material

EM 1110-2-1913 does not provide specific requirements for levee fill material. Fill material requirements are based on California Department of Water Resources, California Code of Regulations, Title 23and the Geotechnical Practice Document USACE, SPK 2008 and on the available geotechnical data, previous experience and industry standard practice. The following material type is recommended:

- Levee Embankment Fill
  - Liquid Limit (LL) is less than or equal to 45
  - Plasticity Index (PI) is greater than or equal to 8 and is less than or equal to 40
  - Fines content (Passing no. 200 sieve) is greater than or equal to 20%

The maximum particle size for all material types is 2 inches. These criteria may be updated if the potential borrow sources include material with higher plasticity, which can be placed within the inner core of the levee cross section to maximize usage of borrow material and on site excavations.

Levee embankment material is recommended to be compacted to 97% of the maximum density per American Society of Testing and Materials (ASTM) D 698, with a moisture content between -1 and +3% of optimum.

5.3 Underseepage and Through Seepage

Levee embankment stability can be compromised if hydraulic exit gradients caused by relatively high underseepage pressures exceed allowable values. Excessive hydraulic exit gradients can result in the formation of sand boils, piping, and levee failure if left unrepaired. Similarly, seepage through the levee embankment can result in seepage breakouts on the landside levee slope and reduce levee slope stability.

Levees where the phreatic surface emerges on the landside slope need to be checked for piping and erosion.

The following USACE publications are used to evaluate underseepage and through seepage for the project levee:

- EM 1110-2-1913, Design and Construction of Levees (USACE 2000)
- ETL 1110-2-569, Design Guidance for Levee Underseepage (USACE 2005)
- EM 1110-2-1901, Seepage Analysis and Control for Dams (USACE 1993)
- EM 1110-2-1908, Instrumentation of Embankment Dams and Levees (USACE 1995)

- REFP10L0.DOC, Geotechnical Levee Practice (USACE, 2008)
- Urban Levee Design Criteria (DWR, 2011)

Based on these publications, the average hydraulic exit gradients must be equal to or less than the following values for a water level at the design WSE:

- Landside levee toe 0.5
- Bottom of empty ditch at landside toe 0.5
- Bottom of empty ditch 150 ft or more from landside toe 0.8
- For ditches between the landside toe and 150 ft from the landside toe, linearly interpolate between 0.5 and 0.8

The average exit gradients summarized above are based on the assumption that the saturated unit weights of the "in situ" landside blanket soils must be at or above 112 pounds per cubic foot. If the saturated unit weight of the landside blanket soils is less than 112 pounds per cubic foot, the exit gradient would be reduced to achieve the required minimum factor of safety (FS). The minimum FS at the levee toe is 1.6 . Previous experience and guidance from the USACE suggests that the maximum allowable exit gradient should be lowered to 0.3 at critical locations such as pump stations, sumps, swimming pools, areas difficult to flood fight and areas with insufficient subsurface information.

In addition to the criteria outlined above for the design WSE, DWR has published draft criteria for levees in urban and urbanizing areas. These criteria are included in the Draft Urban Levee Design Criteria (November 15, 2011). These criteria are similar to the criteria above with respect to design WSE loading, but include an additional condition of a free water surface at the hydraulic top of levee (HTOL) on the water side of the levee. The HTOL is defined as the lowest of the following elevations:

- The expected water surface elevation plus a minimum of 3 feet
- The physical top of levee (or the water surface profile that matches the physical top of the levee at its lowest point) if interim criteria are met, or
- The expected 500-year water surface elevation.

5.4 Static Slope Stability

Design criteria applicable to slope stability are as follows:

- EM 1110-2-1913, Design and Construction of Levees (USACE 2000)
- EM 1110-2-1902, Slope Stability (USACE 2003)

The required minimum slope stability Factor of Safety (FS) presented in EM 1110-2-1913 are:

- End of Construction 1.3
- Steady State 1.4
- Rapid Drawdown 1.0 to 1.2 (Depending on duration of pre-drawdown loading) (only applicable to waterside slope)

5.5 Earthquake Loading

The following documents serve as design criteria for earthquake (seismic) loading:

- EM 1110-2-1913, Design and Construction of Levees (USACE 2000)
- ER 1110-2-1806, Earthquake Design and Evaluation for Civil Works Projects (USACE 1995)
- EC 1110-2-6067 *USACE Process for the National Flood Insurance Program (NFIP) Levee Systems Evaluation. D*ated 31 Aug. 2010
- Urban Levee Design Criteria (DWR, November 15, 2011)

In regards to earthquake loading, EM 1110-2-1913 states the following:

EC 1110-2-6067 states that levee design is to consider the 100-year frequency earthquake. Analyses for this project considered a 200-year frequency earthquake which is more conservative and will provide additional flood protection.

“Earthquake loadings are not normally considered in analyzing the stability of levees because of the low probability of earthquake coinciding with periods of high water. Levees constructed of loose cohesionless materials or founded on loose cohesionless materials are particularly susceptible to failure due to liquefaction during earthquakes. Depending on the severity of the expected earthquake and the importance of the levee, seismic analyses to determine liquefaction susceptibility may be required.”

Although there is a relatively low probability of an earthquake occurring simultaneously with the design WSE, the time required to repair the levee after the seismic event needs to be considered.

Furthermore, ER 1110-2-1806 states “Appropriate methods should be used to analyze the liquefaction and/or estimate deformities for embankment (dams, dikes, levees that retain pools), slope and foundation materials when subjected to ground motions corresponding to the Maximum Design Earthquake (MDE) and the Operating Basis Earthquake (OBE).” Therefore, liquefaction and associated foundation and/or embankment deformities should be evaluated with the following minimum factors of safety:

- Liquefaction Hazard 1.0
- Post-earthquake slope stability 1.0

5.6 Levee Settlement

Foundation settlement due to levee construction should be accounted for when establishing the top of levee (TOL) elevation. The levee design TOL should be increased to account for the calculated post-construction consolidation settlement. Preliminary values were assumed for planning purposes based on the selected levee cross sections. Areas to include a floodwall were assumed to have a settlement value of 0 feet. Where the existing levee would be raised a value of 0.5 feet was assumed, and for areas where a new levee would be constructed on a previously untested foundation a value of 1.0 foot was used.

5.7 Penetrations and Encroachments

Penetrations and encroachments into the levee prism are generally not recommended, although in discussions with the USACE San Francisco District, reviewers may be more lenient with those located in the freeboard area, i.e. within the three feet of minimum additional height above the design WSE. The levee prism is defined as a cross-sectional shape with a top elevation equal to the design TOL and slope projections that extend downward no steeper than 3H:1V on the waterside and 2H:1V on the landside.

5.7.1 Pipes and Conduits

All existing pressure pipes and conduits beneath the levee prism or within 10 ft of the toe of the levee will be removed and relocated as necessary to meet the following criteria:

- Pressure pipes/conduits crossing beneath the levee crown must be above the 100-year design water surface elevation and outside of the landside and waterside slope of the levee prism. These pipes must be equipped with a positive cutoff valve waterside of the levee crown.
- An existing sanitary sewer line runs beneath the creek near the O'Connor Pump Station and Friendship Bridge. This line runs deep below the creek as a siphon. The siphon ends near the landside toe of the left bank levee with a manhole near the Friendship bridge abutment. HDR is coordinating with East Palo Alto Sanitary District, and the USACE regarding how this line will be modified during construction to meet applicable design criteria of the USACE and the Sanitary District.

The following documents shall be referenced in design:

- EM 1110-2-2705, Structural Design of Closure Structures for Local Flood Control Protection Projects (USACE 1994)
- EM 1110-2-2902, Conduits, Culverts and Pipes (Change 1) (USACE 1998)

5.7.2 Utility Poles/Towers and Supports

The location of utility poles and supports that interfere with the proposed levee construction must be approved by the SFCJPA if they are to remain in place. In general, utility pole foundations are not allowed to penetrate the levee prism, per USACE levee encroachment guidelines. Utility tower or pole foundations will be located outside of the levee prism unless approved by the SFCJPA and coordinated with the USACE San Francisco District and the utility in question for acceptability. If tower foundations or other encroachments must remain in the levee prism, seepage must be reduced to acceptable levels, and their location must not interfere with normal or flood-fighting maintenance and operations. Coordination with the appropriate utility will be required to ensure towers that are left within the channel are modified as required for the design flow.

The following document shall be referenced:

- ER 1110-2-100, Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures (USACE 1995)

5.7.3 Levee Vegetation

Design criteria for vegetation management in the project area shall be found in:

- ETL 1110-2-571, Guidelines for Landscape Planting and Vegetation Management at Levee, Floodwalls, Embankment Dams, and Appurtenant Structures (USACE 2009)
- Urban Levee Design Criteria (DWR, November 15, 2011)

As required by the USACE, a “vegetation free zone” must be retained on and adjacent to levees and floodwalls, with the exception of certain grass species, for erosion control. The purpose of this zone is to provide access for inspection, maintenance, monitoring and flood-fighting. The vegetation-free-zone contains the levee crown, the side slopes and a 15-foot setback from the landside and waterside toes. For floodwalls, special consideration must also be given to the distance between large trees and the wall, and the potential for damage by root systems.

It is our understanding that the SFCJPA intends to pursue a variance from the aforementioned criteria through the USACE. Should a variance be granted, alterations to the design criteria will be made accordingly.

6.0 Flood Wall Design Criteria

6.1 General

An I-wall type flood wall will be utilized to increase the level of flood protection along both the right and left banks of the San Francisquito Creek. The I-wall will consist of a cantilevered sheet pile driven into the levee and foundation soils with a reinforced concrete cap. The design criteria are applicable to both steel and pre-stressed concrete sheet piles.

The I-wall will be designed in accordance with USACE design procedures and DWR design recommendations in regards to seismic vulnerability. These procedures are embodied in the following documents:

- EC 1110-2-6066, Design of I-Walls (USACE, 2011)
- EM 1110-2-2504, Design of Sheet Pile Walls (USACE 1994)
- EC 1110-2-6067 USACE Process for the National Flood Insurance Program (NFIP) Levee Systems Evaluation (August 31, 2010).
- Urban Levee Design Criteria (DWR, November 15, 2011)

The EC 1110-2-6066 is an engineering circular (EC) that expires March 31, 2013. This document was prepared based on the evaluation of I-walls that failed during hurricane Katrina. The EC is an interim guidance document pending publication of the final EM.

6.2 Load Conditions

The five load conditions that were evaluated for this project for the design of I-walls are described in Table 6.1. For Load Case 1, the 10 year water surface elevation approximately XX to XX ft. This water surface elevation is considerably less than the top of levee (i.e. bottom of exposed flood wall) and therefore, this load case is not applicable for the San Francisquito Creek flood wall design.

For the Usual Retaining Wall load case shown in Table 6.1, the wall is designed utilizing a cantilevered sheet pile without consideration of flood loading, because the net active landside earth pressure would be largest when the SFC water level is low. To be conservative, the soils in the San Francisquito Creek (SFC) providing passive resistance are assumed to be submerged.

For the Extreme Seismic load case, DWR has recommended a 200 year return period for levee designs within the Sacramento-San Joaquin Valley. Although the SFC project is not within the Sacramento-San Joaquin Valley, the seismic design criteria should be applicable to the SFC flood protection system. Since the return period for this project is 100 years, the horizontal ground acceleration could also be reduced. For the 60% level design, the 200 year horizontal ground acceleration value will be used and if the seismic load case governs the design, the horizontal ground acceleration value will be reevaluated.

6.3 Design Requirements

The requirements for the design of the flood wall are shown in Tables 6.2. For the SFC project the flood wall serves two purposes:

- Provide additional height above top of levee for SFC water surface at the 100-yr design water surface level and can support water to the top of the flood wall (extreme event) per USACE requirements.
- Support the landside levee due to excavation within the SFC to the bench elevation as shown on the plans.

The design of the wall shall consider:

- Rotational stability (i.e. sufficient wall penetration)
- Global slope stability
- Structural capacity
- Allowable lateral deflection

The wall shall have sufficient penetration below the lowest adjacent grade to satisfy the requirements for rotational stability as shown in Table 6.2. The rotational stability shall be determined using the Windows version of the USACE CWALSHT computer program. Analyses shall be performed for both undrained loading (Q-Case) and drained loading (S-Case) if there is soils with a hydraulic conductivity less than 1 x 10^{-3} cm/sec. If all soils have a hydraulic conductivity greater than 1 x 10^{-3} cm/sec, only the S-Case is required.

Global slope stability calculations shall be performed to confirm that failures surfaces beneath the tip of the sheet pile wall determined utilizing a limit equilibrium slope stability program achieve the factors of safety in table 6.2.

The extreme load cases include water level at the top of the flood wall and seismic loading. For both of these load cases the factor of safety on both the active and passive pressure is 1.1. For the seismic load case, the Mononobe-Okobe method (EM 1110-2-2100) was used to determine the seismic earth force to apply to the sheet pile wall. The load was applied at a distance of 0.6 times the unsupported wall height above the adjacent lowest ground elevation. For instance, if the landside ground elevation was 18 ft and the SFC ground elevation was 10 ft, the load was applied at an elevation of 14.8 ft in concert with the active soil pressure.

The requirements in EM 1110-2-2504 were followed to determine the structural capacity of the wall. The CWALSHT analysis was performed with factors of safety of 1.0 on both active and passive pressure. The allowable stress design method was used to determine the minimum sheet pile cross-section required to support the wall. For combined axial and bending load the allowable stress is 0.5 times the yield stress of the steel. The allowable stress may be increased by 33% and 75% for the unusual and extreme load cases, respectively.

The soil and environmental conditions at the project site may cause corrosion of the steel sheet piles. A corrosion analysis is currently being performed by HDR Schiff and the results will be available for the 90% design submittal. The steel sheet pile cross-section may need to be increased to allow for corrosion during the design life of the flood protection system; analyses are being performed for both a 50 year and 75 year design life.

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Table 6.1 Load Conditions for I-Walls

Load Case Condition and Number	Load Description	Guidance Document	Comment
Usual, LC-1	San Francisquito Creek with water at highest level with a 10-yr return period with corresponding protected side water level	EC 1110-2-6066	Full hydrostatic pressure on wall without seepage reduction (i.e. assume gap along wall).
Usual, Retaining Wall	San Francisquito Creek at MLLW and landside water level at 10 yr high level to maximize active pressure on wall	EM 1110-2-2504/ EC 1110-2-6066	Load case is for the evaluation of the sheet pile wall as a retaining structure not a flood control structure
Unusual, LC-2	SFC at 100-yr design water level (DWL) with corresponding protected side water level	EC 1110-2-6066	Full hydrostatic pressure on wall without seepage reduction (i.e. assume gap along wall)
Extreme, LC-3	Water level at top of flood wall	EC 1110-2-6066	Full hydrostatic pressure on wall without seepage reduction (i.e. assume gap along wall)
Extreme, Seismic	Seismic with SFC at normal operating water level	DWR ILDC, 11/15/11	200-yr return period

Table 6.2 Design Requirements for I-Walls Design Requirement	Load Condition	Shear Strength Requirement	Slope Stability FOS	FOS on Strength Parameters		Comment
				Active Pressure	Passive Pressure	
Rotational Stability to Determine Min. Penetration	Usual, LC-1	Q &/or S	-	1.7	1.7	
	Usual, Retaining Wall	Q &/or S	-	1.0 Q-Case 1.0 S-Case	2.0 Q-Case 1.5 S-Case	Lower water le
	Unusual, LC-2	Q &/or S	-	1.3	1.3	
	Extreme, LC-3	Q &/or S	-	1.1	1.1	
	Extreme, Seismic	Q	-	1.1	1.1	
Global Slope Stability	Usual, LC-1	Q &/or S	1.8	-	-	
	Usual, Retaining Wall	Q &/or S	1.4	-	-	Wall designed as retaining structure
	Unusual, LC-2	Q &/or S	1.5	-	-	
	Extreme, LC-3	Q &/or S	1.4	-	-	
	Extreme, Seismic	Q	1.4	-	-	
Deflection Check	Usual, LC-1A	Q &/or S	-	1.0	1.0	Top of wall deflection less than 1.0 in.
	Usual, Retaining Wall	Q &/or S	-	1.0	1.0	Top of wall deflection less than 1.0 in.

7.0 Hydraulic Design Criteria

The criteria that will be applied to the development of the hydraulic design for this project will be based on, but not limited to, the following guidelines set by USACE:

- EM 1110-2-1601, Hydraulic Design of Flood Control Channels (USACE 1994)
- ER 1110-2-1405, Hydraulic Design for Local Flood Protection Projects (USACE 1982)
- EM 1110-2-1416, River Hydraulics (USACE 1993)

7.1 Hydraulic Modeling

Hydraulic modeling is to be completed using Hydrologic Engineering Centers River Analysis System (HEC- RAS) Version 4.1 developed by USACE. Three recent hydraulic analyses have been conducted within the project reach and are to be considered for this design of this project. The previous hydraulic analyses that have been developed are as follows:

- Noble Consultants, Inc. (2009) – A hydraulic analysis was prepared for USACE. The modeling provides the existing conditions of the channel capacity.
- Philip Williams & Associates, Ltd. (2009) – An alternatives analysis was prepared for SFCJPA. The modeling provided proposed alternatives, from which Alternative 2 was selected for the basis of design. Hydraulic models used Noble models as a base model.
- Caltrans (2010 – not yet released) – An analysis is under development for the design of a new bridge at West Bayshore Road, Highway 101, and East Bayshore Road.

7.2 Hydraulic Model Topographic Information and Workmap

The existing geometric configuration of the hydraulic models is to be consistent with and based on the topographic data described in Section 4.0. Workmaps are to be created showing all pertinent information to the hydraulic modeling; topographic data and aerial imagery must be consistent with Section 4.0.

7.3 Design Flows

The design flows that are to be used in the design of the flood protection system, per the request of the SFCJPA, are based on a rainfall runoff hydrologic model that was constructed using "HEC-1Watershed Modeling Computer Program" developed by USACE (1990). The rainfall runoff hydrologic model was developed by Wang et al for USACE. The methodologies and results of the model were documented in the report entitled "Santa Clara Valley Water District San Francisquito Creek Hydrology Report" (Wang et al 2007). The SFCJPA has selected the design flow of 9,400 cubic feet per second (cfs) to be used in the hydraulic analysis of the study reach. The recommended flow is estimated to be equivalent to the 100-year flood event.

7.4 Design Water Surface Profile

The Design Water Surface Profile will be determined from hydraulic modeling that will be conducted under the previously mentioned criteria. The Design Water Surface Profiles will be set using the conservative elevations from the evaluation of coincidental events (normal depth must be reviewed to ensure that it does not produce a higher profile) plus the developed sea level rise. The HEC-RAS hydraulic model will combine the 100-year fluvial event of 9,400 cfs with the 100-year tide elevation plus sea level rise (11.3) feet to develop the design water surface profile. The tidal considerations are discussed in the next section.

PWA presented the methodology and hydraulic model results for the 100-year WSE in their "San Francisquito Creek Flood Reduction Alternatives Analysis" (PWA 2009). Included in the Alternatives Analysis were WSEs for the existing conditions and their three alternative designs.

7.5 Tidal Effects

The downstream water surface elevation at the mouth of the San Francisquito Creek is represented by the tidal stage occurring in the South San Francisco Bay; therefore, hydraulic impacts of the tidal effects must be considered. Multiple studies have been conducted to determine what tidal effects impact the San Francisquito Creek.

7.5.1 Starting Water Surface Elevation

A study conducted by the USACE in October 1984, *San Francisco Bay, Tidal Stage vs. Frequency Study*, determined the 100-Year tidal water surface elevation for the San Francisco Bay near East Palo Alto as 10.35 feet (NAVD 88). Upon further investigation, the USACE discouraged the use of data in this document due to the dated nature of the report.

In 2000, the USACE determined that the Mean Higher High Water (MHHW) tidal level to be 7.1 feet (NAVD 88). The MHHW is the average of the higher high water height of each tidal day observed over the National Tidal Datum Epoch. The tidal level of 7.1 feet was determined using the long-term record data from the Redwood City tidal level Station (Station ID: 9414523), administrated by the National Oceanic and Atmospheric Administration (NOAA). The Redwood City Station is the closest station to the creek mouth. For the historical flood events, the water levels at the creek mouth were assigned to be the highest tidal elevations measured at the NOAA Redwood City Station during these flood events.

The SFCJPA has directed HDR to assume a sea level rise equal to 26 inches. The 26 inches was added to the 100-Year tidal elevation of 10.35 feet to produce a total tidal elevation of 12.52 feet.

In May 2010, the SFCJPA directed HDR to analyze the starting water surface elevation of 11.30 feet (NAVD 88) in preliminary calculations and design. This is an estimate of what may be concluded in the USACE's Shoreline Study determination of the 100-year tidal elevation, which is not yet released.

Table 1 summarizes the range of tidal elevations or starting water surface elevations that will be evaluated during the hydraulic modeling.

Table 1 – Starting Water Surface Elevations Comparison

Starting Water Surface Elevations (NAVD 88, feet)	
Downstream Boundary Description	Elevation (feet)
Mean Higher High Water (MHHW)	7.10
100-Year Tidal Elevation	10.35
100-Year Tidal Elevation + 26” Sea Level Rise	12.52
USACE Shoreline Study, 100-Yr Tidal Elevation + Sea Level Rise	11.30

USACE has published guidance for incorporating sea level rise considerations into Civil Works projects in the following document:

- EC 1165-2-212, Sea-level Change Considerations for Civil Works Programs 01 Oct 11 (exp. 30 Sep 13).

Per FEMA’s Guidelines and Specifications for Flood Hazard Mapping Partners, Appendix C, page C-36, November 2009, the following tidal boundary condition is applicable for flood insurance studies:

> When the downstream boundary of a modeled stream is within a coastal tidal reach, the tidal boundary of the model is taken as equal to the Mean Higher High Water (MHHW) level of the nearby tide station. Location of the tide station(s) must be verified to represent true downstream conditions. The tide level can be transferable to other locations along open coast…

Note that MHHW is the average of the higher high water height of each tidal day observed over the National Tidal Datum Epoch (approximately 19 years) at the gage station.

SFCJPA wishes to incorporate consideration of a coincident 100-year tide elevation and sea level rise into the Project design. SFCJPA has requested HDR to move forward using the results from USACE Shoreline Study. At the time this technical memorandum was released, the SFCJPA has directed the use of starting water surface elevation of 11.30 feet (NAVD 88), which is the USACE’s Shoreline Study determination of the 100-year tidal elevation including a yet undetermined value of sea level rise.

7.5.2 Wind Setup and Wave Runup Analysis

A determination of wind setup and wave runup must be completed to verify that levee system will not be overtopped or compromised. The effects of waves generated by wind on surfaces of levees and floodwalls are an important consideration when determining the freeboard. The USACE has published guidance for incorporating wind setup and wave runup heights into Civil Works projects in the following document:

- EM 1110-2-1607, Tidal Hydraulics (USACE 1991)
- EC 1110-2-6055 Design of I-Walls (USACE 1 Apr 2011)

7.6 Channel Configuration

Where appropriate, the existing channel will be modified to increase capacity for flood flows. The bottom width will be enlarged, waterside levee slopes will be constructed at 3H:1V, and access will be provided by a 16 foot wide patrol road on each side of the channel. A setback terrace must be incorporated between the creek and the floodwall or levee. This configuration will be determined by hydraulic capacity requirements, and to the greatest extent possible, terrace elevation specifications conducive to achieving restoration objectives.

7.7 Energy Losses

Manning's "n" values are to be estimated following the procedures described in the "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains" United States Geological Survey Water-Supply Paper 2339. Manning's "n" values should represent the cover that will be implemented in the design (vegetation, channel armoring, protective measures for levee, levee wall material, etc.).

Two types of transition losses should be included in the hydraulic model. Contraction and expansion loss coefficients for gradual transitions of 0.1 and 0.3, respectively, are commonly used. For losses between bridge cross sections and drastic changes in channel configuration, contraction and expansion loss coefficients of 0.3 and 0.5, respectively, are more representative. These values are sometimes adjusted during model calibration. The table below provides some guidance on the loss parameters.

Table 2 – Modeling Loss Parameters

Modeling Loss Parameters	
Feature	Coefficient
Manning's n Values	
Grass Lined	0.035
Concrete Lined	0.020
Expansion Coefficient	
Channel	0.30
Bridge	0.50
Contraction Coefficient	
Channel	0.10
Bridge	0.30

Manning's "n" values to be used in hydraulic model are consistent with the vegetation desired by the SFCJPA in the channel. Inclusion of such vegetation may require a variance from the USACE's ETL 1110-02-571.

7.8 Bridges

Consideration of the effects that the existing West Bayshore Road, Highway 101, East Bayshore Road, and Friendship Bridge will have on the conveyance of flow must be included. In addition, the proposed configuration of the expansion of Highway 101 Bridge (the proposed bridge will combine the existing West Bayshore Road, Highway 101, and East Bayshore Road into one structure and also increase the capacity through the section) must be evaluated to determine the impacts that the proposed bridge will have on the conveyance of flows.

Bridge scour analysis must be conducted for all bridge structures to verify stability of the structure. Effective flow areas must be used to account for areas of zero conveyance near structures. All modeling is to be conducted under the user guidelines developed for HEC-RAS under its User Manual specifications.

7.9 Deposition/Scour and Erosion

Evaluation of the potential for deposition and scour in the channel and erosion of embankments must be conducted. Analysis must include deposition and scour in the channel, scour around bridge structures, erosion of levee embankments and scour at levee and floodwall foundations.

Prediction of significant deposition of sediment in the channel would indicate the need for additional freeboard allowance or a periodic maintenance requirement. Scour in the channel has the potential to undermine levee embankments and levee or bridge foundations and impact structure and slope stability. Typical scour components include potential long-term degradation, general scour, bend scour, contraction scour, bridge pier and abutment scour. Scour is also caused by stream instability such as channel migration.

The following documents shall be used for channel stability criteria:

- Bureau of Reclamation, Computing Degradation and Local Scour (January 1984)
- Federal Highway Administration, Evaluating Scour at Bridges, 4th Edition, HEC-18, (May 2001)
- EM 1110-2-1601, Hydraulic Design of Flood Control Channels (USACE 1994)
- EM 1110-2-1418, Channel Stability Assessment for Flood Control Projects (USACE 1994)
- EM 1110-2-2007, Structural Design of Concrete Lined Flood Control Channels (USACE 1995)

The following is a reproduction of Table 2-5 of EM 1110-2-1601, and lists maximum channel velocities that may be allowed without paving or bank revetment.

Table 3 – Suggested Maximum Permissible Mean Channel Velocities

Suggested Maximum Permissible Mean Channel Velocities	
Channel Material	Mean Channel Velocity, fps
Fine Sand	2.0
Coarse Sand	4.0
Fine Gravel[1]	6.0
Earth	
Sandy Silt	2.0
Silt Clay	3.5
Clay	6.0
Grass-lined earth (slopes < 5%2)	
Bermuda Grass	
Sandy Silt	6.0
Silt Clay	8.0
Kentucky Blue Grass	
Sandy Silt	5.0
Silt Clay	7.0

Suggested Maximum Permissible Mean Channel Velocities	
Channel Material	Mean Channel Velocity, fps
Poor Rock (usually sedimentary)	10.0
Soft Sandstone	8.0
Soft Shale	3.5
Good Rock (usually igneous or hard metamorphic)	20.0

- For particles larger than fine gravel (about 20mm = ¾”), see Plates 29 and 30 of the EM.
- Keep velocities less than 5.0 fps unless good cover and proper maintenance can be obtained.

It is assumed that bridge scour at Bayshore Road, Highway 101, and East Bayshore Road will be addressed as part of the Caltrans widening project. Appropriate design values from Caltrans will be used to determine the need for scour protection in this area. Floodwalls adjacent to bridge abutments will also be evaluated for scour protection.

Contraction and abutment scour at Friendship Bridge will be evaluated when considering different design alternatives. Levees adjacent to bridge abutments will also be evaluated for scour protection.

7.10 Freeboard / Uncertainty

The levee design water surface elevation (WSE) plus an additional height is required to establish the TOL to reduce the risk of overtopping. Three feet of height is added to the 100-year design WSE, in accordance with 44 CFR 65.10, throughout the Project reach. Also in accordance with 44 CFR 65.10, four feet of height (in lieu of three feet) is added to the design WSE for 100 feet upstream and downstream of constrictions, such as bridges.

The SFCJPA understands that recent USACE guidance calls for Risk and Uncertainty analysis in lieu of a TOL crown elevation based on the design WSE plus allowances for freeboard. However, it is the SFCJPA’s determination that this project shall be designed to meet 100-year flood event water surface elevations, in accordance with current FEMA and 44 CFR 65.10 guidelines.

Per Corps guidance, a limited amount of wave overtopping can be allowed without armoring, depending on levee geometry, soil conditions, and ground cover; typically ranging between 0.01 cubic feet per second per foot (cfs/ft) and 0.1 cfs/ft (section 7.17 ULDC).

7.11 Top of Levee and Floodwall Crown Elevation

The TOL is established to reduce the risk of overtopping by first determining the design WSE and then adding the required additional levee freeboard height in accordance with Federal and State design practice. The TOL elevation should also include an allowance for future levee and foundation settlement, as defined in the following publication:

- EM 1110-2-1904, Settlement Analysis (USACE 1990)

Preliminary settlement values were selected for planning purposes as defined in Section 5.6. These values will be updated once additional analyses are completed and incorporated into the TOL.

8.0 Other Considerations

8.1 Easements

As a standard of practice, a minimum easement for maintenance, inspection and flood fighting of 10 to 20 feet is required on the landside of levees. This is advisable, and it is recommended that minimum 10 ft easements allowing access to the levees are secured along the landside toe of the project, where the land is not already held in fee title by a member agency of the SFCJPA. As an alternative to this in areas where there are space limitations, an access road along the levee crown with intermittent access ramps to access points may suffice.

8.2 Stormwater Collection and Discharge

No modifications to stormwater collection and discharge are proposed for this project. Community drainage problems have been identified by the City of East Palo Alto in the area north of O'Connor Pump Station, and north of the creek levees, flowing into the Faber Tract. Newly refined project area topography should provide information regarding low points in the system adjacent to the project area. The SFCJPA should then be able to identify whether the drainage issues are elevation-related (i.e. no gravity flow possible), or if the issues are maintenance-related (i.e. improperly operating flap gates) and therefore outside the scope of this project.

Areas adjacent to levees will drain away from the levee toes for a minimum distance of 10 feet. Engineering analyses have determined that the new levees will not significantly increase runoff onto neighboring landscapes.

8.3 Alternative Formulation and Planning

In ER 1105-2-100, Planning Guidance Notebook (USACE 2000), Section 5(d) states the following:

> Each alternative plan is to be formulated in consideration of four criteria: completeness, effectiveness, efficiency and acceptability. Appropriate mitigation of adverse effects is to be an integral part of each alternative plan.

This capital improvement project, for flood protection design between Highway 101 and the San Francisco Bay, is one aspect of a larger program to meet flood control, ecosystem and recreation needs along the entirety of San Francisquito creek. HDR's design is guided by the project materials completed by agencies and other consultants that are complete and available from the SFCJPA at present. It is our assumption that considering the design criteria outlined above, and building on conceptual project elements and studies determined by previous consultants and SFCJPA member agencies, this project will contribute to a complete and effective program for the entirety of San Francisquito Creek under consideration by the SFCJPA.

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9.0 References

There are two types of references for this project; design criteria documents and basis of design reference documents. The design criteria documents are guidelines developed by regulatory agencies, and basis of design reference documents are other agency or consultant reports that contain information utilized in the Project.

9.1 Design Criteria Documents

9.1.1 US Army Corps of Engineers

Table 4 – USACE Design Criteria Documents

Number	Title
EM 1110-2-1405	USACE, Hydraulic Design for Local Flood Protection Projects, 1982.
EM 1110-2-1416	USACE, Rivers Hydraulics, 1993.
EM 1110-2-1418	USACE, Channel Stability Assessment for Flood Control Projects, 1994.
EM 1110-2-1601	USACE, Hdyraulic Design of Flood Control Channels, 1994.
EM 1110-2-1607	USACE, Tidal Hydraulics, 1991.
EM 1110-1-1804	USACE, Geotechnical Investigations, January 1, 2001
EM 1110-1-1807	USACE, Procedures for Drilling in Earth Embankments, March 1, 2006
EM 1110-1-1904	USACE, Settlement Analyses, September 30, 1990
EM 1110-2-1901	USACE, Seepage Analysis and Control for Dams, April 30, 1993
EM 1110-2-1902	USACE, Slope Stability, October 31, 2003
EM 1110-2-1908	USACE, Instrumentation of Embankment Dams and Levees, June 30, 1995
EM 1110-2-1913	USACE, Design & Construction of Levees, April 30, 2000
EM 1165-2-212	USACE, Sea-level Change Considerations for Civil Works Programs 01 Oct 11, (exp. 30 Sept 13).
EM 1110-2-2007	USACE, Structural Design of Concrete Lined Flood Control Channels, 1995.
EM 1110-2-2502	USACE, Retaining Walls and Floodwalls, September 29, 1989
EM 1110-2-2504	Engineering and Design - Design of Sheet Pile Walls, March 31, 1994
EM 1110-2-2705	USACE, Structural Design of Closure Structures for Local Flood Control Protection Projects, 1994.
EM 1110-2-2902	USACE, Conduits, Culverts, and Pipes, March 30, 1998
ER 1105-2-100	USACE, Planning Guidance Notebook, June 30, 2004

Number	Title
ER 1110-1-8100	USACE, Laboratory Investigations and Testing, December 31, 1997
ER 1110-2-100	USACE, Periodic Inspection and Continuing Evaluation of Completed Civil Work Structures, 1995.
ER 1110-2-1150	USACE, Engineering and Design for Civil Works Projects, August 30 , 1999
ER 1110-2-12	USACE, Quality Management, September 30, 2006
ER 1110-2-1806	USACE, Earthquake Design and Evaluation for Civil Works Projects, July 31, 1995
ER 1130-2-530	USACE, Flood Control Operations & Maintenance Policies, October 30, 1996
ETL 1110-2-569	USACE, Design Guidance for Levee Underseepage, May 1, 2005
ETL 1110-2-571	USACE, Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures, April 10, 2009.
EC 1110-2-6055	USACE, Design of I-Walls, April 2011.
EC 1110-2-6066	Engineering and Design: Design of I-Walls, April 11, 2011
EC 1110-2-6067	USACE, Process for the National Flood Insurance Program (NFIP) Levee Systems Evaluation, August 31, 2010.
REFPIOLO.DOC	USACE, Geotechnical Levee Practices, 2008.
ULDC	Urban Levee Design Criteria, DWR, November 15, 2011

9.1.2 Other Federal Agencies

Federal Emergency Management Agency, Guidance on Levee Certification for the National Flood Insurance Program, March 25, 1997.

Federal Emergency Management Agency, Requirements of 44 CFR Section 65.10: Mapping of Areas Protected by Levee Systems, March 2007.

Federal Emergency Management Agency, Title 44 Emergency Management and Assistance. Chapter 1, Federal Emergency Management Agency Part 65 – Identification and Mapping of Special Hazard Areas, October 1, 2002.

Federal Emergency Management Agency, Guidelines and Specifications for Flood Hazard Mapping Partners, Appendix C: Guidance for Riverine Flooding Analyses and Mapping, November 2009.

Federal Highway Administration, Evaluating Scour at Bridges, 4th Edition, HEC-18, May 2001.

Bureau of Reclamation, Computing Degradation and Local Scour (January 1984).

9.1.3 State / County Agencies

California Department of Water Resources, California Code of Regulations, Title 23, Volume 32, October 4, 1996.

California Department of Water Resources, Proposed Interim Levee Design Criteria for Urban and Urbanizing Area State-Federal Project Levees, May 13, 2009.

California Department of Water Resources /URS, Guidance Document for Geotechnical Analyses, Revision 6, March 2008.

9.2 Basis of Design Reference Documents

Philip Williams & Associates, Ltd. (2009), San Francisquito Creek Flood Reduction Alternatives Analysis. July 17.

Noble Consultants, Inc. (2009), Final Report, San Francisquito Creek, Development and Calibration/Verification of Hydraulic Model. May 26.

USACE San Francisco District (2005), San Francisquito Creek Flood Damage Reduction & Ecosystem Restoration General Investigations Program, 905(b) Analysis Reconnaissance Study. March 14.

USACE San Francisco District (2005), San Francisquito Creek Flood Damage Reduction & Ecosystem Restoration General Investigations Program, San Mateo & Santa Clara Counties, CA, Proposed Feasibility Phase Project Management Plan. September 27.

USACE San Francisco Bay (1984), Tidal Stage vs. Frequency Study, determined the 100-Year Tidal Water Surface Elevation.

USACE San Francisco District (2009), Appendix C, DRAFT Geotechnical Appendix and Reliability Analysis of Downstream Floodwalls and Levees, San Francisquito Creek, F3 Milestone Without Project. December.

Santa Clara Valley Water District (2010 pending), Right of Way Study. May.

Towill, Inc. (2010), San Francisquito Creek Mapping, DTM & Orthophotos, San Francisquito Creek from Highway 101 to SF Bay. March.

Towill, Inc. (2010), Location of Utilities Aerial Survey Mapping. March.

Caltrans Utility Surveys, Location of Utilities Ground Locating, January 2010.

National Geodetic Survey, North American Datum of 1983 (NAD83), California Zone 3.

National Geodetic Survey, North American Vertical Datum of 1988 (NAVD88).

Santa Clara County (2006), LiDAR of San Francisquito Creek and surrounding areas. May. (Adjusted by HJW Geospatial in Sept. 2008 using Bestor Engineers TIN.)

Wang et al (2007), Santa Clara Valley Water District, San Francisquito Creek Hydrology Report. December.

Federal Emergency Management Agency, Flood Insurance Study, San Mateo County, California and Incorporated Areas, Volumes 1 & 2, Number 06081CV001A, Preliminary.

Federal Emergency Management Agency, Flood Insurance Study, Santa Clara County, California and Incorporated Areas, Volumes 1-4, Number 06085CV001A, May 18, 2009.

Light, Air & Space Construction Environmental Services Company (2009), San Francisquito Creek Hazardous Toxic Radioactive Waste Study, San Francisco Bay to Searsville Dam Plus Additional 5-Square Mile Study Area, Santa Clara and San Mateo Counties, California. November 20.

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Figures

Figure 1. San Francisquito Creek Overall Map

SAN FRANCISQUITO CREEK
JOINT POWER AUTHORITY

SAN FRANCISQUITO CREEK
OVERALL MAP
NOT TO SCALE

FIGURE
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Figure 2 and 3. San Francisquito Creek Typical Cross-Sections

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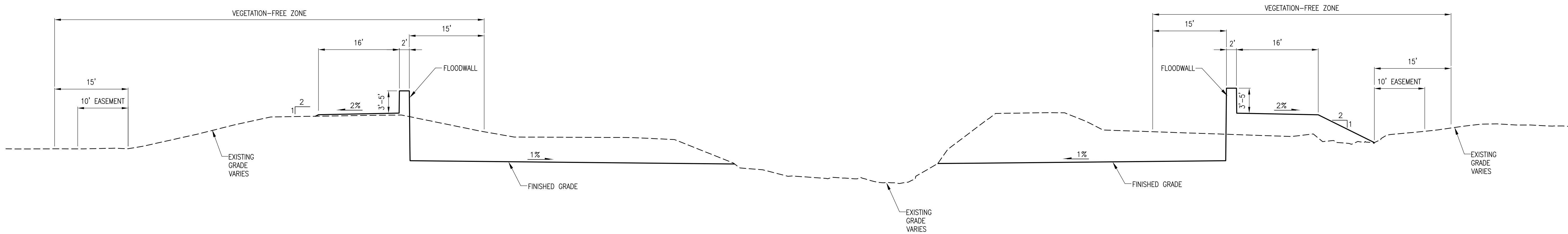
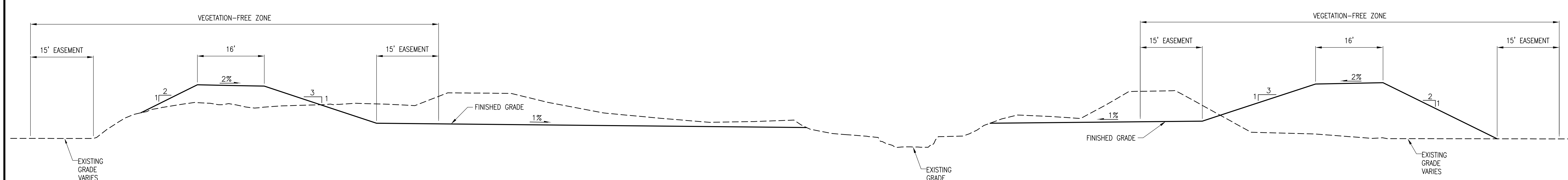


TYPICAL CROSS SECTION FOR FLOODWALL ON BOTH BANKS

NTS

NOTE:
SECTIONS ARE LOOKING DOWNSTREAM



NOTE:
SECTIONS ARE LOOKING DOWNSTREAM

TYPICAL CROSS-SECTION FOR NEW LEVEE ON BOTH BANKS

NTS

NOTE: CROSS SECTIONS ARE ADAPTED FROM PWA ALTERNATIVE 2 HYDRAULIC MODEL

SAN FRANCISQUITO CREEK
JOINT POWER AUTHORITY

SAN FRANCISQUITO CREEK TYPICAL CROSS SECTIONS	FIGURE 2

HDR

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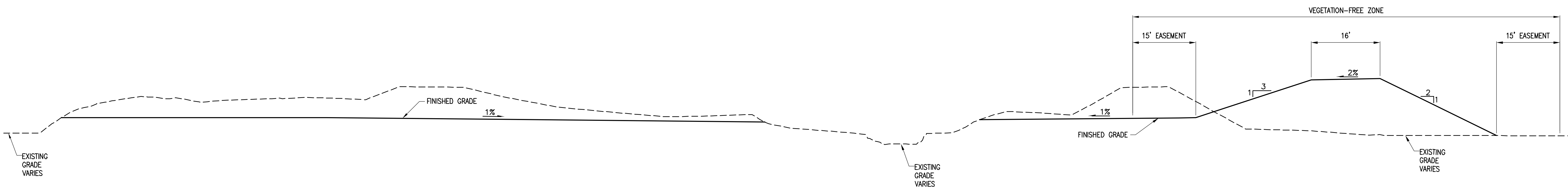

TYPICAL CROSS–SECTION FOR NEW LEVEE ON RIGHT AND LEVEE DEGRADE ON LEFT BANK

NTS

NOTE: CROSS SECTIONS ARE ADAPTED FROM PWA ALTERNATIVE 2 HYDRAULIC MODEL

SAN FRANCISQUITO CREEK JOINT POWER AUTHORITY

SAN FRANCISQUITO CREEK TYPICAL CROSS SECTIONS

FIGURE 3

C:/pwworking/sac/d0171342/TYPICAL X-SEC ALT 2-FIG2.dwg
05-20-10 AJACKSON 08:51:11

Appendix A. PWA's Alternative 2 Plan View and Typical Cross-Sections

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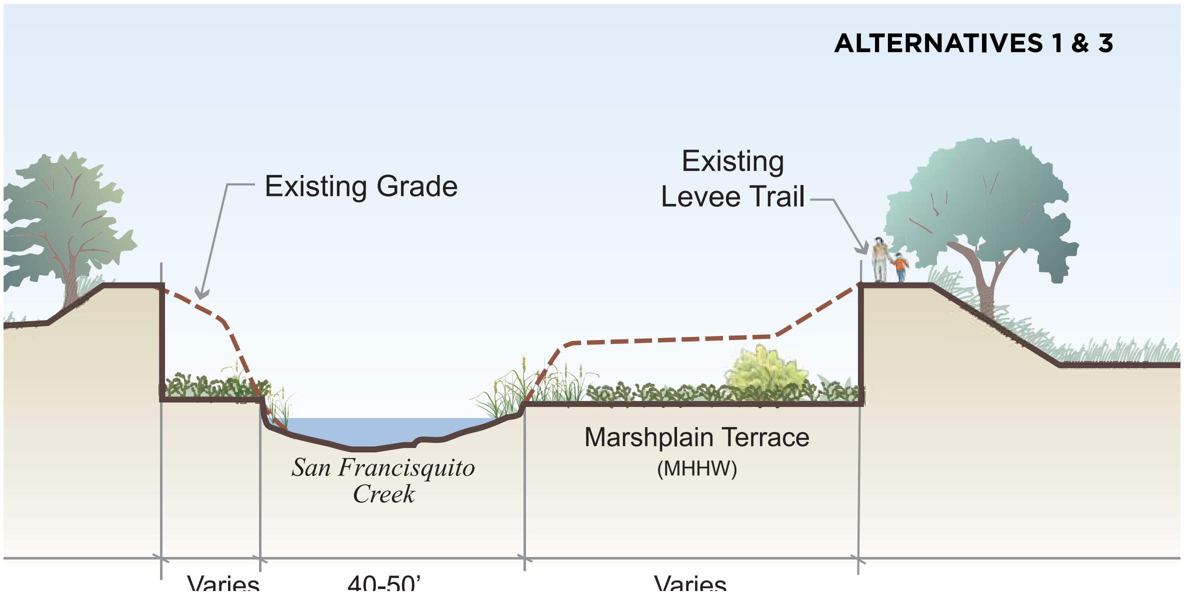
FIGURE 3 | **ALTERNATIVE 2, CONCEPTUAL LAYOUT**



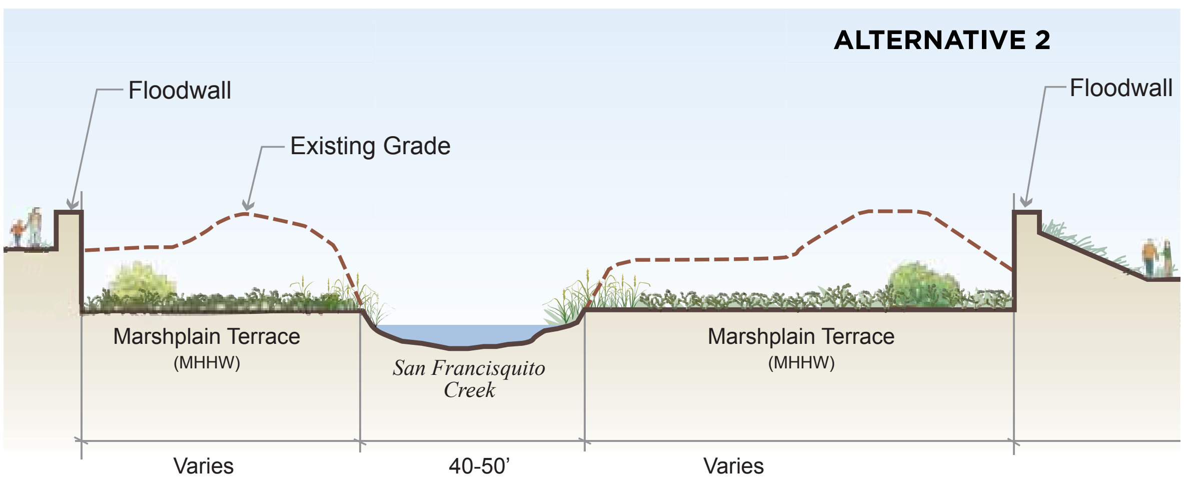
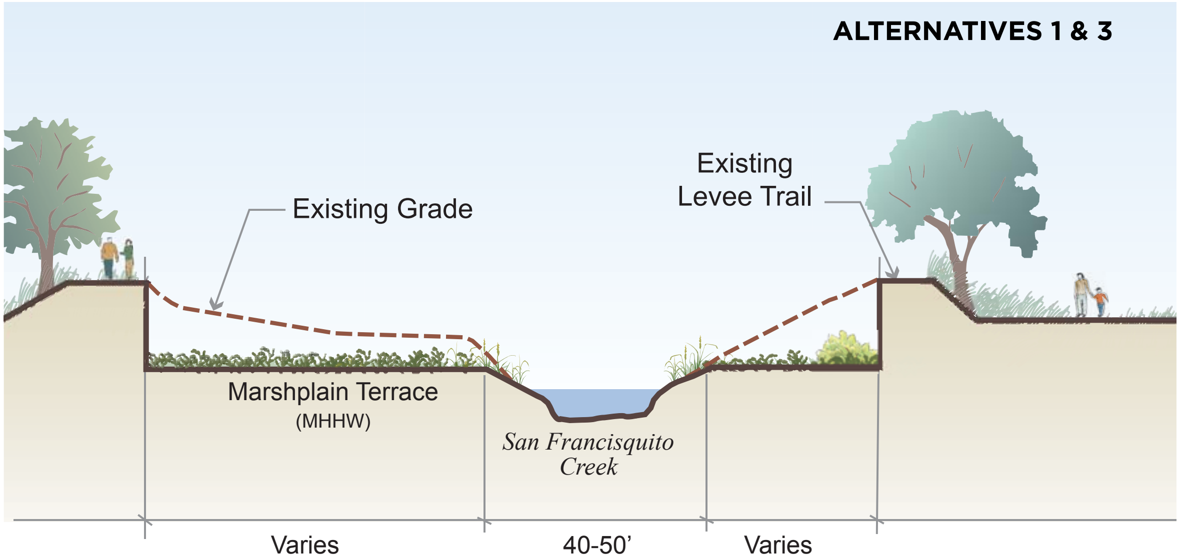


FIGURE 5 | **CROSS SECTION A, VIEW LOOKING DOWNTREAM**



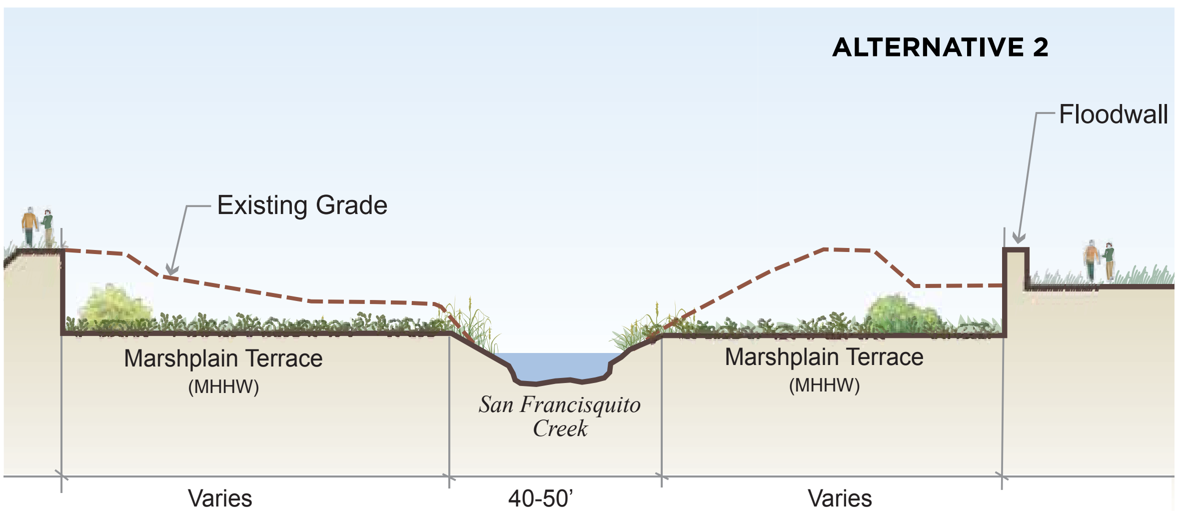
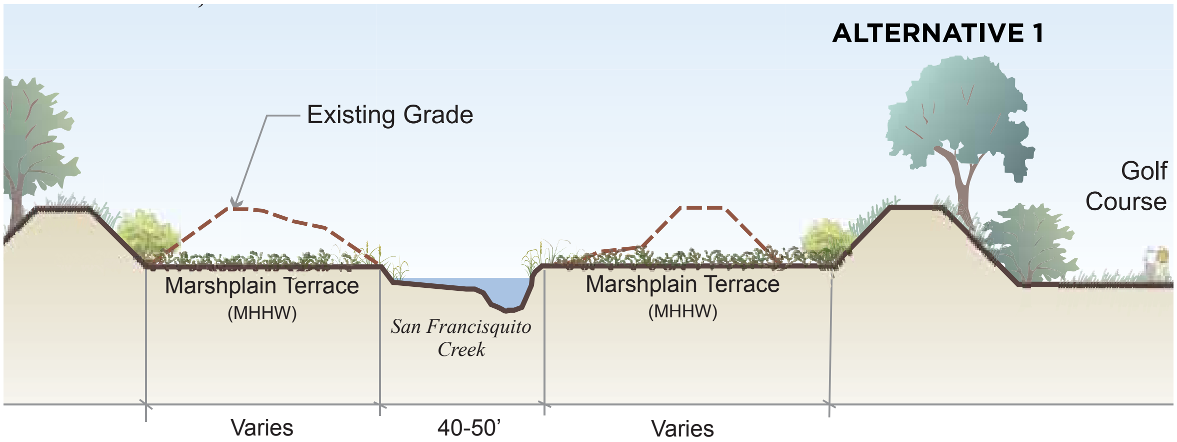


FIGURE 6 | **CROSS SECTION B, VIEW LOOKING DOWNTREAM**



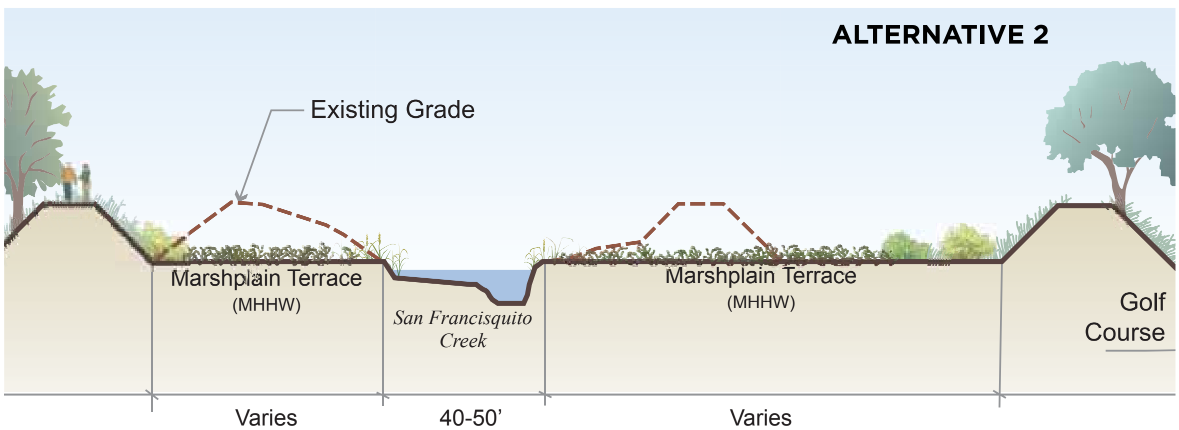


FIGURE 7 | **CROSS SECTION C, VIEW LOOKING DOWNTREAM**

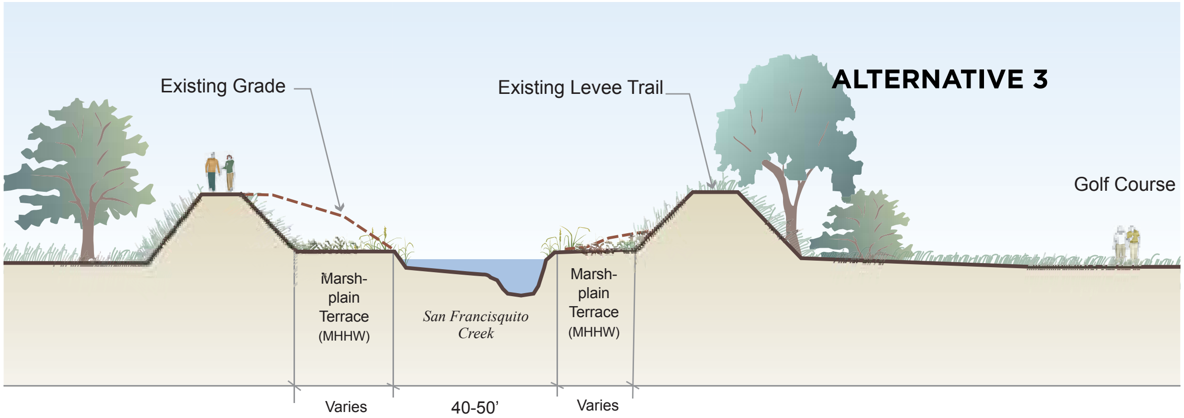


FIGURE 8 | **CROSS SECTION C, VIEW LOOKING DOWNTREAM**

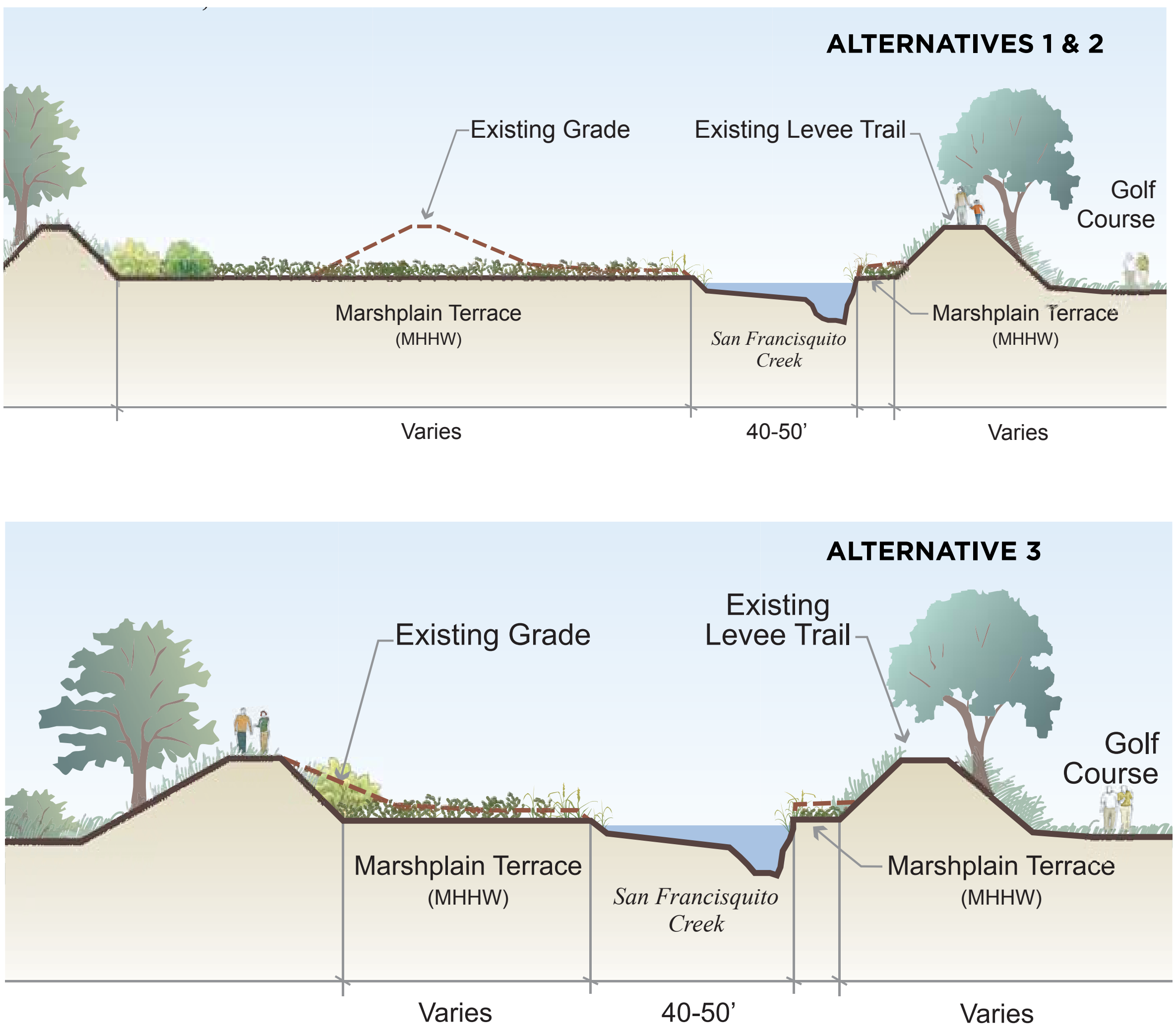


FIGURE 9 | **CROSS SECTION D, VIEW LOOKING DOWNTREAM**