Appendix G

Geotechnical **Engineering**

GEOTECHNICAL APPENDIX

South San Francisco Bay Shoreline Study (SSFBS)

Contents

ATTACHMENTS

1.0 PROJECT BACKGROUND AND PURPOSE

The South San Francisco Bay Shoreline (SSFBS) Study is evaluating the feasibility of a multipurpose project to provide flood risk management and ecosystem restoration benefits to the Shoreline of the South San Francisco Bay Area including addressing increased flood risk from future sea level rise. The project study was originally scoped in the 1980s and has since been reduced in scope to focus on the most acute life safety risk in the Alviso area.

The study can be divided into three distinct stages technical stages that are shown in Table 1-1. Multiple geotechnical reports were developed to support the Feasibility Scoping Meeting held in 2010. They discussed geotechnical baseline conditions and the estimated geotechnical performance of the outer and inner levees of the project area and provide the basis for most geotechnical recommendations related to design and construction. This work was compiled and presented in USACE (2009). Additionally, the USACE Engineering, Research and Development Center (ERDC) conducted a study to characterize erosion performance estimates for hydraulic simulation modeling of the existing outer and inner levees (USACE 2008, USACE 2009). The above referenced documents have undergone both District Quality Control (DQC) and Agency Technical Review (ATR) and should be referred to for technical details not provided in this appendix.

Table 1-1: Planning miletones and associated time periods.

Stage	Time Period	Planning Milestone
1	2004 to 2011	Feasibility Scoping Meeting [F3]
2	2011 to 2013	Alternative Formulation Briefing [F4]
3	2013 to 2014	Public Release of Study

The information presented in this geotechnical appendix is simplified to highlight key design and construction constraints most likely to impact the decision on the recommended plan, and summarizing critical elements governing the geotechnical performance of existing outboard and inboard dikes. Key constraints focus on geotechnical impacts to cost (e.g. fill requirements, staged construction) and calculation of project benefits (e.g. performance of the existing features).

1.1 Study Area and Recommended Alignment

The current project study area is shown on Figure 1-1. The recommended alignment and extent for the new flood control levee is coincident with the existing inboard dike. The recommended levee is approximately 19,500 ft long (3.7 miles). The alignment includes two closure structures; one mitre gate at the railroad and one tide gate at Artesian Slough. The ends of the alignment will tie into existing flood control levees along the Guadalupe River and Coyote Creek.

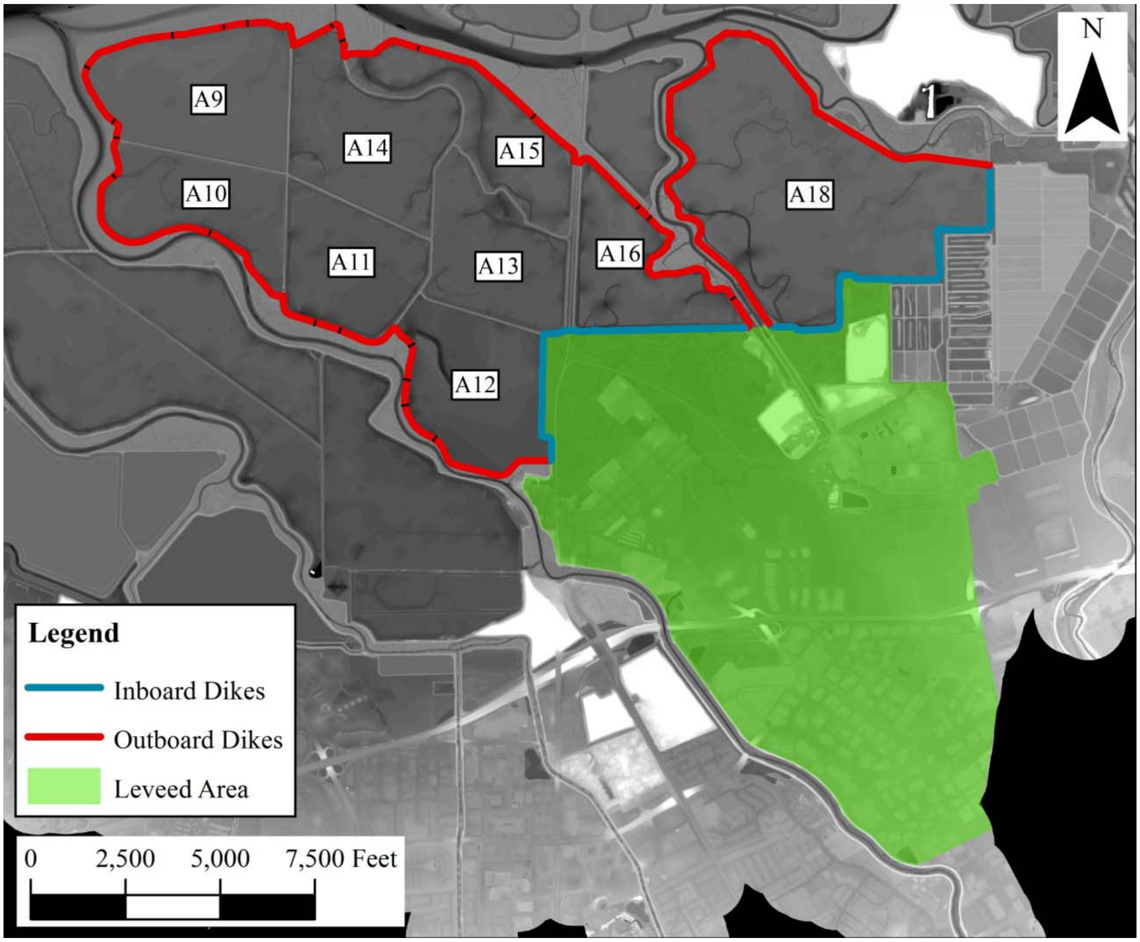


Figure 1-1: Study area vicinity map, pond locations, and existing berm features.

1.2 Geotechnical Investigations and Analysis Leading to the AFB

The primary source of geotechnical information for this summary is the 2009 F3 Milestone Appendix C: Geotechnical Investigation and Analysis for the South San Francisco Bay Shoreline Study in Study Area (USACE, 2009). The investigation included the review of 650 standard penetration test (SPT) borings and 43 cone penetrometer test (CPT) soundings performed by others. In addition, explorations were advanced on the existing outboard (14 SPT, 44 CPT) and inboard (20 SPT, 58 CPT) project levees for the study. Both laboratory testing and in-situ data was used to develop a statistical distribution of geotechnical properties for use in analyses.

Geomatrix (2008) developed fragility curves for six index points along outboard dikes in the project study area. The primary modes of failure considered were seepage and rapid drawdown. One fragility curve (i.e. Area 5) was used to model outboard dike performance for the with project condition at all index point locations prior to the AFB. This fragility curve was incorporated into a Monte Carlo simulation that studied the without project condition (Noble 2012).

Geotechnical recommendations for design and construction were developed for the Alternative Formulation Briefing (AFB). These recommendations focused on constraints most likely to impact a recommended plan (i.e. cost and constructability). Constraints were ubiquitous among all alternatives and used for screening and evaluating potential flood risk reduction measures against one another. The constraints were considered in the recommended levee alignment (Figure 1-1) and the associated national economic development (NED) and locally preferred plan (LPP) described in the Civil Design Appendix of this integrated document.

1.3 Geotechnical Recommendations since the AFB

Recommendations that were developed for the Alternative Formulation Briefing (AFB) were revised during the current effort and are discussed in Section 3.0 of this appendix. Design and construction recommendations were revised to be more specific to the recommended levee alignment and to reflect additional technical recommendations (e.g. vegetation).

The project was analyzed under the “high” sea level rise rate for the with project condition at the time of the AFB. Following the AFB the existing condition was analyzed under the historical and intermediate sea level rise rates for the without project condition. The geotechnical basis for the fragility curve was modified from a seepage and drawdown governed performance to one governed by overtopping and erosion. The basis for the modified fragility is discussed in Section 4.0 of this appendix. The results of the analysis are discussed in detail in Tidal Flood Risk Analysis Appendix of this integrated document.

2.0 SUMMARY OF GEOTECHNICAL CONDITIONS

Details regarding the subsurface explorations are presented in USACE (2009). The level of subsurface information collected and evaluated to date is judged sufficient to support conceptual alternative comparisons in terms of design, cost, and construction differences. The recommendations provided are intended for conceptual feasibility level analysis for selection and comparison of different alternatives. The recommendations are based on engineering judgment, analysis, and subsurface exploration and laboratory testing. All recommendations will be reevaluated and finalized during preconstruction engineering and design (PED).

In general, the Alviso area of the project is mapped as Bay Mud, which is recently deposited fine-grained soil of marine origin. Bay Mud is relatively thin (< 5 feet) along the existing urban/salt pond boundary and becomes deeper (35 to 40 feet thick) along the outer pond levees adjacent to the bay. Bay Mud is occasionally underlain by thin (< 5 feet) granular marine deposits of loose to medium dense consistency. More typically the Bay Mud is underlain by alluvial flood plain deposits and Old Bay Mud that range in grain size from coarse to fine. The consistency of these deep foundation soils is medium dense to dense/stiff.

The existing inboard levees for the project area are constructed from excavated alluvial deposits in the vicinity of the alignment. The outboard levees are most likely constructed of Bay Mud borrow excavated from adjacent ponds and sloughs.

Bay Mud thickness is judged to be the most important geotechnical aspect affecting the cost of proposed alternatives. The thickness of the Bay Mud using cone penetrometer testing (CPT) and standard penetration testing (SPT) explorations along the inner and outer levees, regional/site geomorphology, and engineering judgment. The interpretation is shown on Figure 2-1.

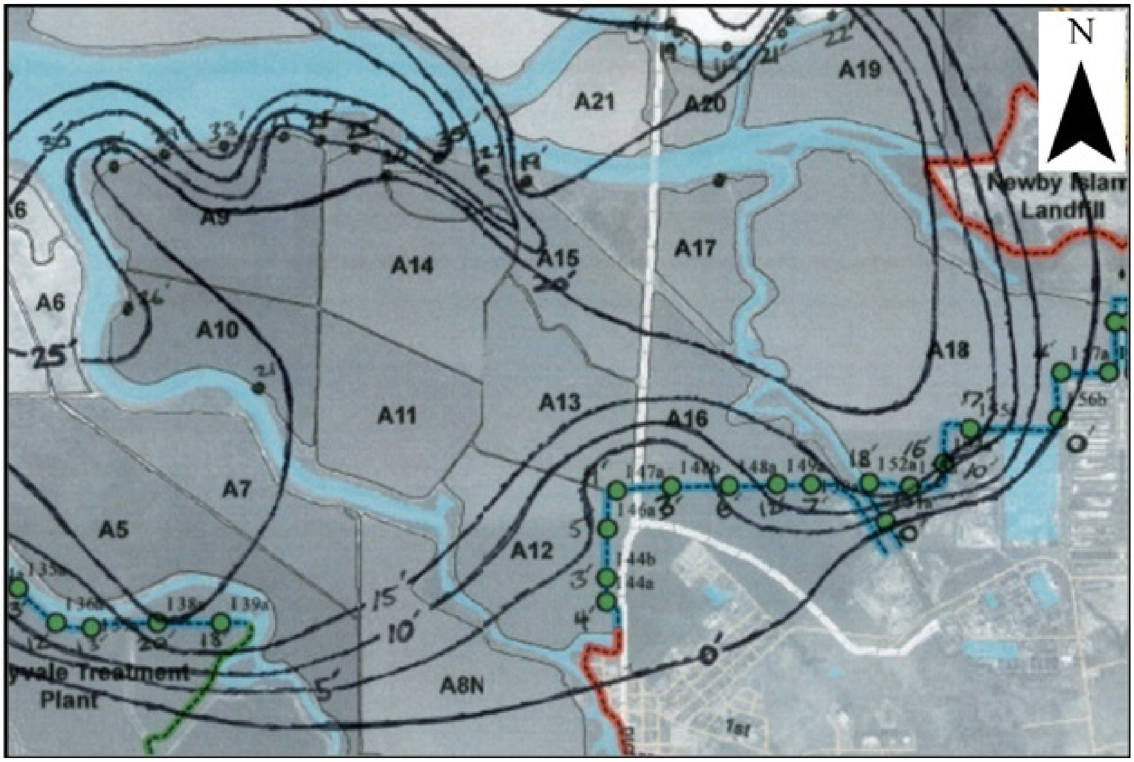


Figure 2-1: Interpreted bay mud thickness (ft) contours.

3.0 GEOTECHNICAL FINDINGS AND CONCEPTUAL DESIGN RECOMMENDATIONS

Several geotechnical explorations and analyses programs have been completed and are discussed in USACE (2009). The analyses considered multiple levee configurations for the project, the performance of existing features, and an anticipated three year period to complete all construction. The following sections summarize significant findings, geotechnical criteria, and recommendations used in the formulation of the levee alternatives.

3.1 Levee Design and Transitional Habitat Fills

The project alignment being considered includes the construction of a new levee along the existing inboard levee alignment. Various configurations of transitional habit fill are being considered along the waterside slope of the new levee. The fills range from large areal fills (> 300 ft wide) to a smaller fill bench (~ 50 ft wide) to provide an area for a variety of habitat and animal refugia to establish. The primary geotechnical constraint for fill design and construction are related to weak Bay Mud foundation soils that underlie the project area. These foundation soils may result in large magnitude settlement, bearing capacity/slope stability failures, and require special provisions for construction.

All levee and transitional habitat fill alternatives will encounter difficult conditions due to the soft surface and foundation soils, and static water elevations above work areas. Limited working/staging areas, operating on very soft soils, the use of specialized equipment (e.g. low ground pressure), and varying water management strategies are to be expected. The geotechnical site conditions most relevant to cost of a given alternative are those issues related to settlement and low strength soils. The following sections focus on these constraints which have significant cost impacts regardless of the details of the design decision (e.g. long-term staged construction, vertical wick drains, etc.). Additional analyses to identify preferred construction methods that leverage value will be needed in PED. Similarly, construction field instrumentation (e.g. piezometers, settlement/survey monuments, etc.) will be evaluated to determine necessary monitoring during the construction and operation and maintenance phase of the project.

The construction will be sequenced to maintain control of pond water surface elevations and facilitate levee construction over a three year period. The new levee will be constructed in three reaches that are divided by the new closure structures discussed in paragraph 3.2. New structures and modifications to existing structures would be completed prior to the construction of the new levee reaches. Each reach has been identified primarily based on access to existing roads and can be subdivided during construction to better manage dewatering of the levee foundation and delivery of offsite fill for construction. Initial clearing and excavation of the existing inboard dike will create berms that will isolate the new levee foundation from the adjacent ponds. Temporary berms along the outboard of the new levee alignment can provide construction access/turn-outs and the base of new transitional habitat fills.

3.1.1 New Fill Settlement Estimates

The amount of primary consolidation settlement that would occur under new fill loads for various thicknesses of Bay Mud foundation soils and assuming 1-D loading conditions is shown in Figure 3-1. Magnitudes for settlement beneath large areal fills (e.g. transitional habitat) can be expected to be equivalent to those shown in Figure 3-1. Settlements beneath levees are likely to be approximately 5 to 10% less than those beneath large areal fills depending on the thickness of Bay Mud in the foundation. However, for planning purposes the magnitudes shown are judged to be reasonable for estimating earthwork/settlement along the levee alignment. The magnitude of, and impacts to structures resulting from, settlement will be more fully evaluated during PED.

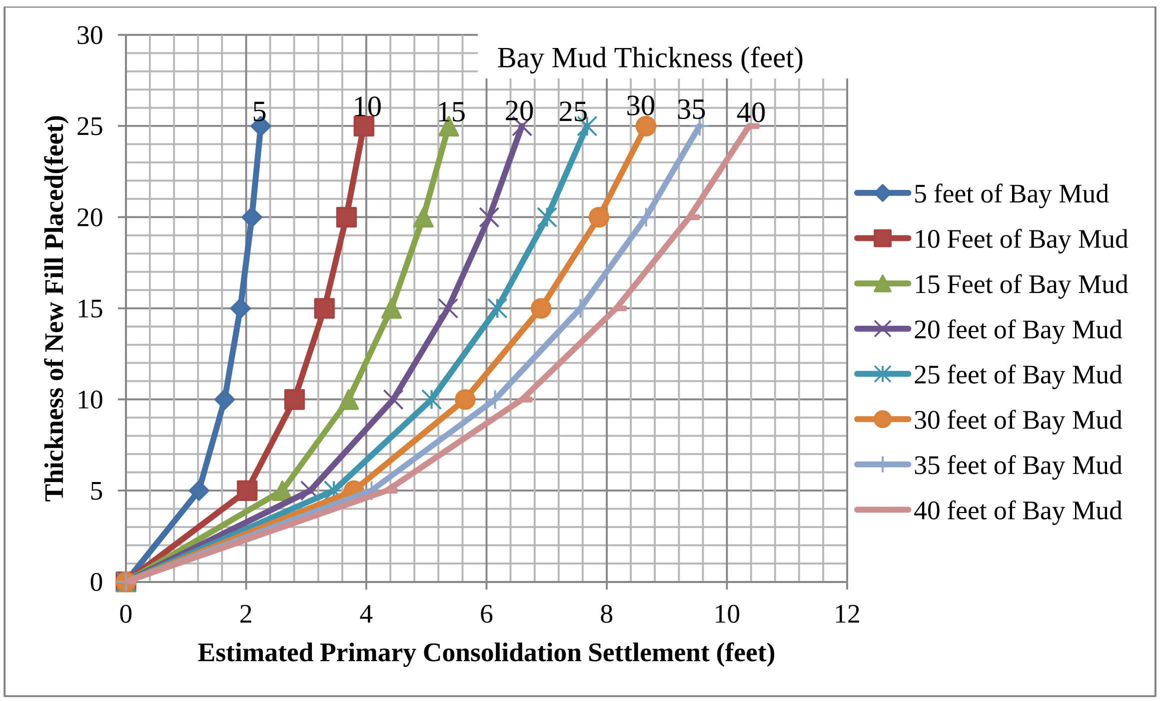


Figure 3-1: Estimated Bay Mud Consolidation Settlement for Large Areal Fills

The period to complete primary consolidation will be many years given the very low permeability of the Bay Mud. The estimated period to the completion of 50% and 90% consolidation is shown in Table 3-1.

The estimated periods assume no surcharging or subsurface drainage (i.e. wick drains) is implemented prior to or during levee fill placement. A uniform strain index of 0.32 and a new fill height of 16 ft were assumed. Double drainage is judged to prevail in the Alviso Area with the exception of a constrained area on the outboard pond berm roughly 0.5 mile east of Alviso Slough. For comparison purposes, the time to consolidation for single drainage conditions have been presented. The impact to the time required for consolidation is a factor of four.

Additional details regarding material properties and analyses assumptions are described in Attachment A. The impact of all assumptions on the large strain/settlement anticipated will be reevaluated in PED.

Table 3-1: Estimated Consolidation Rates for Bay Mud

	Double Drainage		Single Drainage	
Bay Mud Thickness (feet)	**Time for 50% consolidation (years)**	**Time for 90% consolidation (years)**	**Time for 50% consolidation (years)**	**Time for 90% consolidation (years)**
5	0.2	0.7	0.6	2.7
10	0.6	2.7	2.5	10.6
15	1.4	6.0	5.5	23.9
20	2.5	10.6	9.9	42.4
25	3.8	16.6	15.4	66.3
30	5.5	23.9	22.2	95.4
35	7.5	32.5	30.2	130
40	10.0	45.4	39.4	170

Secondary consolidation, impact of organic content, and initial distortion settlements will be analyzed in more detail during PED. Contribution from secondary consolidation is likely to be about 3% that of primary consolidation based on consolidation properties and estimates in USACE (2009). Contribution from organics is expected to be fairly uniform because the stratum with elevated organic content is typically 2 feet thick. Fills on “virgin ground” may induce localized elasto-plastic deformations typical to construction on soft soils.

More detailed analysis during PED will be needed to estimate and make recommendations to manage and accommodate elasto-plastic deformations and consolidation settlement. The use of geosynthetics (e.g. fabrics or grids) may be required for fills on virgin ground that serves as the foundation for levee fills. The use of wick drains spaced 5 to 7 feet may be used to expedite consolidation settlement of Bay Mud from many years to less than one year to accommodate a three year construction timeline for the new levee alignment. Existing strata beneath the current dikes is anticipated to be stiff enough to support against global failures and mud waves during the installation and initial wick drain service period. The need for expedited consolidation is driven by weak foundation soils and is discussed in paragraph 3.1.2.

3.1.2 Bearing Capacity and Slope Stability

New fill that is placed directly on normally consolidated Bay Mud is prone localized bearing capacity failures. Near surface Bay Mud is estimated to a cohesion of approximately 75 psf and a bearing capacity of approximately 430 psf (i.e. $q_{ult} = c*N_c = 75*5.7 = 430$) based on Terzaghi’s bearing capacity equation. The use of low ground pressure equipment (i.e. 3 psi contact pressure) will be required to place the initial lifts of new fill. The use of geosynthetics to distribute the weight of new fill and construction techniques that monolithically advance the leading edge of construction are likely to be necessary to reduce “shoving” and mud waves on virgin ground.

Slope stability was analyzed using Morgenstern-Price methods for force and moment equilibrium for circular slip surfaces along the edges of large areal fills (e.g. planned habitat islands). Material properties for each stratum are shown in Table 3-2 and are based on typical values for the study area (USACE, 2009). Parameters directly measured during this study included compacted Bay Mud, Bay Mud crust, Stiff Clay (Old Bay Mud), and strength with depth (i.e. s_u/P) trends for normally consolidated Bay Mud.

Table 3-2: Soil properties used in stability analyses (Attachment A).

		Undrained (phi = 0)		Drained	
Material	**Unit Weight (pcf)**	**Cohesion (psf)**	**S_u/P (psf/ft)**	**Phi (degrees)**	**Cohesion (psf)**
Compacted Fill	125	800	--	32	100
Bay Mud Crust	100	500	--	32	500
Normally Consol. Bay Mud	97	75 [at ground surface]	12	31	0
Stiff Clay	125	1500	--	32	0

Low undrained shear strength of the underlying Bay Mud require that new fill thicknesses be carefully planned to avoid negative impacts (e.g. bearing capacity failures, mud waves, etc.). Slope stability analysis was performed for fill slopes of 5:1 to 3:1 (H:V) to estimate the maximum fill thickness that could be placed for various Bay Mud thickness while maintaining a factor of safety (FOS) of 1.3 or greater. The minimum FOS is based on the "end of construction" condition in EM 1110-2-1913. Table 3-3 summarizes the maximum fill thickness recommendations for respective fill configurations.

Table 3-3: Estimated Fill Thickness Placement Limits for first fill stage for 3:1 to 5:1 Slopes on 5 to 40 feet of Bay Mud (Attachment A)

Bay Mud Thickness (ft)	**Side Slope of Fill (H:V)**		
	3:1	**4:1**	**5:1**
5	20 feet	20 feet	20 feet
10	14 feet	15 feet	20 feet
15	11 feet	12 feet	15 feet
20	11 feet	12 feet	13 feet
40	11 feet	11 feet	13 feet

If fill thicknesses greater than recommended are required, the fill will need to be placed in stages after pore pressures have dissipated. Wick drains will allow more rapid drainage of pore pressures. Details are discussed more in Attachment A however, a quantitative value (i.e. time savings vs. cost of installation) for wick drains cannot be accurately specified before PED.

A number of additional stability analyses were conducted assuming a 4:1 side slope fill and 20 ft of Bay Mud to verify that short term (i.e. end-of-construction) loading is the critical case. The long-term (i.e. drained condition) condition showed a factory of safety of 2.41 and 2.27 for a piezometric surface at the ground surface (0 ft) and mean higher high tide (6 ft), respectively. The addition of a tension crack for the drained condition with water at 0 ft maintained the 2.27 factor of safety with a slightly shifted critical surface geometry. Stability analyses with be reevaluated in detail during PED and may include seismic deformation analyses.

3.1.3 Seismicity and Seismic Hazards

USACE (2009) discusses the seismic hazards that could impact the project area. The project is located in a highly seismic region between the San Andreas and Hayward faults. Fault rupture within the project area is highly unlikely, however, strong ground shaking capable of inducing slope instability and liquefaction of coarse grain alluvial deposits is likely. Peak horizontal ground accelerations of around 0.5 to 0.6 g have a 10 percent chance of exceedance in 50 years. Explorations cataloged in USACE (2009) encountered discontinuous potentially liquefiable strata and sensitive clays within 50 ft of the ground surface. The effect on project levees is anticipated to be primarily related to settlement ranging from 0 to 18 inches. Due to the presences of these materials, a seismic site class F is assigned per ASCE/SEI 7-10, Chapter 20.

Detailed seismic analysis to estimate project performance should occur during PED. In general, it is anticipated that some levee distress may occur during a large seismic event, which will require repair and restoration of the levee section. Potential damage may include localized slumping, cracking, and/or seismically induced settlements at the crest. However, feasibility level analysis and past performance in the project area suggest that total loss of the levee section to significantly large liquefaction or lateral spreading it is not likely. Therefore, seismically induced damage is not anticipated to contribute significantly to an immediate post-earthquake flood risk. The compacted clay levee section is judged to be sufficiently resilient to seismic hazards with freeboard (approximately 3 feet above an event having 0.01 chance of exceedance in project year 50 which includes sea level rise), moderately flat slopes (3H:1V), and moderately wide crest (16 ft).

3.1.4 Project Fill Specifications

Levee fill shall meet the following criteria general criteria. Levee fill shall be sufficiently fine grained (e.g. CL, CH, or SC) and plastic (e.g. plasticity index of 10 to 50; liquid limit < 60) to produce a continuum of low hydraulic conductivity (i.e. $1x10^{-4}$ or less) fill. Levee fill shall be free of organic matter and particles larger than 4 inches in diameter. Past experiences of the sponsor has shown that materials meeting these specifications are commonly available from local quarries and construction projects. Levee fill specifications may be modified based on availability at the time the project enters construction.

Structural fills shall be used around new/existing structures and as a roadbase for the levee crest. Structural fills shall consist primarily of well graded sands and gravels. Fills around structures shall not free draining include 15 to 20 percent fines. Structural fills used to surface the levee crest may consist of crushed rock, quarry run, or other commercially available material capable of providing an all weather trafficable surface.

Transitional habitat fills can be constructed of materials not suitable for structural or levee fill. These materials include organic matter, material generated from clearing and grubbing, and oversize material encountered in project excavations. The top three feet of transitional habitat fill should be greater than 75% fines in order to provide the substrate necessary to support the anticipated project vegetation.

3.1.5 Potential Additional Fill Borrow Sources

The United States Fish and Wildlife Service (USFWLS) plans to import fill to the site for potential use as general fill for existing levee maintenance and for use in construction of new levees. SPN stated that if the fill material met the specifications noted in Section 3.1.4 it could be suitable for use as levee fill. An evaluation of the USFWLS proposed fill import and stockpile plan is included as Attachment B, and includes recommendations for sorting and testing of imported soil.

Additional sources of fill considered included the San Jose Wastewater treatment plant sludge pond solids and existing levees/berms. Laboratory testing of the sludge showed an organic content that precluded their use as structural fills. The sludge is geotechnical suitable for transitional habitat fills; however, additional testing to determine the environmental suitability is required. Existing inboard levee fill may be able to be reused if it meets the specifications noted or blended with suitable levee fill to improve its suitability. In all cases, levee fill should be homogeneous to provide a consistent impermeable continuum with low risk for seepage related failure or distress.

3.1.6 Vegetation and Erosion Protection

Marsh vegetation that is maintained to a height compliant with ETL 1110-2-583 is considered the only feasible vegetation at the project. Saline conditions along the alignment for the recommended levee will not support significant sod/turf. Vegetation that can be successfully installed and maintained will be a mix of native marsh vegetation. The combination of vegetation, buried stone, and/or transitional habitat fills (i.e. planting berms) are proposed to balance requirements for levee safety and regulatory limits on traditional maintenance activity (e.g. regular mowing, equipment in/near environmentally sensitive areas).

The configuration of proposed vegetation, and alternatives for maintaining vegetation, are shown and summarized in Attachment D. This vegetation will include 12 to 18 inch pickleweed from elevation 0 ft to 3 ft above the typical high water elevation. The high water elevation corresponds to approximately elevation 6 ft and 10 ft on the land and water side slope, respectively. Upland grasses will occupy the side slopes between the levee crest and the pickleweed. Combinations of buried stone protection and buried gravel may be necessary to stunt the growth of native vegetation in lieu of regular mowing in an environmentally sensitive area, or to provide erosion protection where vegetation cannot be supported. It is anticipated that a reduced need for regular mowing will still include annual mowing of the levee side slopes within 10 to 12 feet of the levee crest and above elevation 9 ft. The establishment of woody vegetation (e.g. coyote bush) on the levee prism is unlikely, but would be cleared and grubbed by hand as needed.

The recommended levee design includes vegetation as erosion protection on the water and land side slopes. Vegetation likely to establish on the project levees is described above. Vegetation is anticipated to be continuous and able to provide erosion protection from overtopping of the levee. Overtopping would be of short duration (i.e. minutes to hours) for events exceeding the design levee height. Erosion protection from 0.5 to 1 ft waves generated during frequent events will be provided by the transitional habitat fills (e.g. 50 foot bench at EL 9 feet or ecotone), buried stone protection, and existing wave break berms between the railroad and Artesian Slough.

3.2 Levee Crossings

For the feasibility design the structures recommended at levee crossings are gate closure systems. Recommendations were based on the subsurface stratigraphy shown in CPTs 47a and 48b, and boring 52a. Additional borings and design analyses will be necessary during PED to validate and finalize the feasibility dataset and assumptions.

3.2.1 Rail Road Flood Gate Closure

The recommended levee alignment will require a mitre gate closure structure across the existing railroad track near Station 34+75. The miter gate is shown in the Civil Design Appendix. The feasibility level design, construction, and operations of the proposed gate structure considered:

- Use of deep foundation system (i.e. concrete piles) to support the structure. The piles will be 20 feet long concrete piles extending to stiff soils beneath the soft Bay Mud. The pile section is 24 inches square and sufficiently oversized to bear potential down drag and seismic loads.
- Differential settlement and lateral loading between the closure structure and proposed levees.
- Availability of materials and trained personnel to respond to flood events.
- The construction of the closure structure should not require sustained interruptions in the railroad operations or modification to the railroad grade/alignment.
- A concrete cutoff through the railroad bed beneath the mitre gate to prevent seepage.

3.2.2 Tide Gate at Artesian Slough

The recommended levee alignment will require a tide gate at Artesian Slough near Station 94+75. The design and construction of the proposed tide gate considered:

- Use of deep foundation system (i.e. concrete piles) to support the structure. The piles will be 20 feet long concrete piles extending to stiff soils beneath the soft Bay Mud. The pile section is 24 inches square and sufficiently oversized to bear potential down drag and seismic loads.
- Differential settlement and lateral loading between the tide gate and proposed levees.
- The new levee should provide access for regular maintenance and operation of the tide gate. Additional width, surfacing requirements, or other provisions may be required to support equipment and light duty vehicle traffic.

3.2.3 Utilities

Four utility crossings are identified along the recommended levee alignment. An action at each crossing is described where applicable.

- A siphon near Station 76+00. The siphon was installed in 2012 and maintains flow through the existing inboard dike to New Chicago Marsh.
- Underground electric lines leading to the SCWD weir near Station 95+00. The utility will be reconfigured to an overhead configuration.
- Culverts near Station 96+00 that maintain flow from Artesian Slough to the area south of Pond A18.
- Overheard PG&E electric and appurtenant towers near Station 130+00. Overhead clearance is substantial enough to not impact levee construction. Tower bases in Pond A18 may require added erosion protection after the pond is breached to tidal action.

The siphon and culvert provide water to environmentally sensitive areas. Neither crossing has a means of positive closure and will likely need to be replaced. The design and construction of the new siphon and culvert should consider settlements induced by new levee fill. Critical components such as valves, weir board structures, etc. may require support from a deep foundation or be sized to be resilient to differential settlement.

4.0 ECONOMICS AND HYDRAULICS MODELING SUPPORT

The following section discusses geotechnical performance (i.e. fragility curve) of the existing dike-pond system that was used in hydraulic modeling of flooding in the project area. The fragility curve provides the likely performance of the outboard dike as a function of water surface elevation. Performance is characterized as the "probability of unsatisfactory performance" and is more plainly the "probability of breach". The resultant fragility curve that was input in the Flood Damage Reduction Analysis (HEC-

FDA) software to model the without project condition and identify economic benefits captured for different levels of flood protection. The effects of erosion and overtopping on geotechnical performance and breach development are also discussed.

4.1 Performance of Existing Dike-Pond System

The existing dikes in the project area are not engineered structures. The most likely source of initial flooding under more frequent flood events is through the dike-pond system that is west of Artesian Slough (Figure 4-1). By comparison, the existing condition of the west side of the project is consistently at lower elevations (i.e. > 2 ft) on both inboard and outboard dikes.

The following sections summarize geotechnical performance in the context of the dike-pond system west of Artesian Slough. Overtopping and erosion based failures are critical to the performance of the dike-pond system. Seepage and drawdown based failures were determined to be non-credible due to the short duration (i.e. hours) loading of flood events.

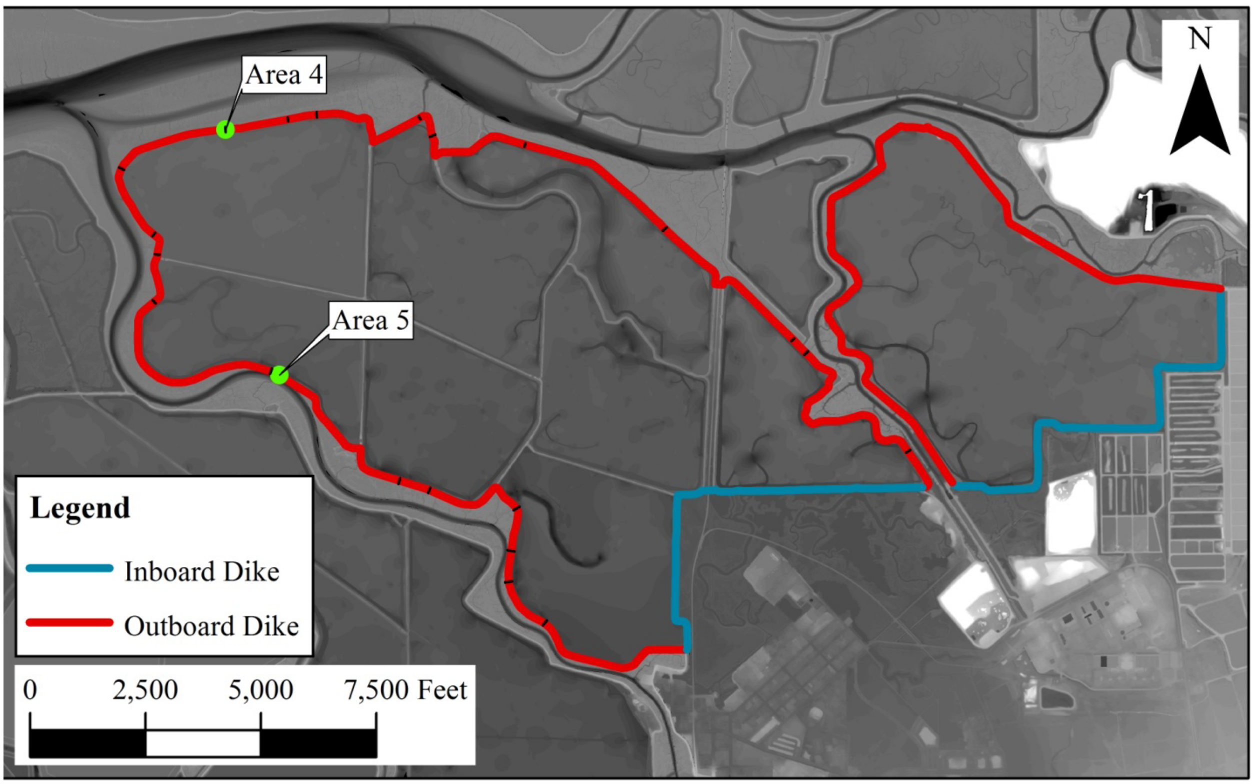


Figure 4-1: Project map of existing dikes and berms.

4.2 Outboard Dike Performance

4.2.1 Fragility Curves Prior to Alternative Formulation Briefing

Geotechnical fragility curves for the entire SSBS project were developed in USACE (2009) to characterize the condition of the existing outboard dikes. This effort leveraged data from existing (650 SPT and 43 CPT soundings), as well as new (34 SPT and 102 CPT soundings), geotechnical exploration locations along the existing inboard and outboard dikes and historical operation and maintenance efforts.

This data was used to create a total of 14 index points; six on the outboard dikes (Geomatrix, 2008) and eight on the inboard dikes (USACE 2009).

Two of the index points developed in Geomatrix (2008) are along the outboard dike that is west of Artesian Slough (Figure 4-1). A "most critical" geometry was estimated from six cross sections within 500 feet of each index point. Fragility curves were developed by varying outboard water surface elevations and reporting the minimum factor of safety under steady state seepage and rapid drawdown conditions. Probability of unsatisfactory performance (P_u), also referred to as probability of failure, was reported as a function of water surface elevation from the crest (i.e., crest elevation minus water surface elevation).

4.2.2 Fragility Curve post-Alternative Formulation Briefing

The fragility curve used prior to the AFB was based upon seepage and rapid drawdown and judged incompatible with the short duration (hours) loading of flood events. Erosion and overtopping erosion were identified as the mechanisms critical to determining the likelihood of failure/breach of the outboard dike. In addition, newer and higher resolution survey information in the study area had been collected. An additional fragility curve was developed to more accurately represent loading (i.e. erosion and overtopping) and updated dike dimensions (i.e. elevation and crest width) known to exist in the study area.

No new geotechnical analysis was performed to quantitatively support the additional curve. However, existing analysis for erosion and overtopping, as well as empirical observations of dike performance, were leveraged to support the justification for the revised fragility curve. The primary factors supporting the revised fragility curve were (i.) typical conditions along the outboard dike, (ii.) hydraulic and breach modeling already performed for the without project condition in the study area, and (iii.) observed performance relative to maintenance performed.

A 2010 USGS LiDAR survey of the study area was used to identify the typical configuration of the outboard dike. The cross-section geometry was sampled at 21 representative locations (Figure 4-2) and plotted (Figure 4-3). Cross sections were purposely concentrated in areas where overtopping is likely to occur first (i.e., saddles) and/or erosion is more likely (i.e., proximity to sloughs). Crest widths were estimated by measuring the section width 1 ft below the peak crest elevation. This method was used to avoid underestimating crest widths due to irregular topography. Factors that contribute to functionally narrower crests, such as rodent holes, irregularities from erosion, and very loose erodible soils, were not considered in the estimate of the crest width. The average crest elevation and width of the sampled cross sections was 10.8 ft NAVD88 and 18 ft, respectively.

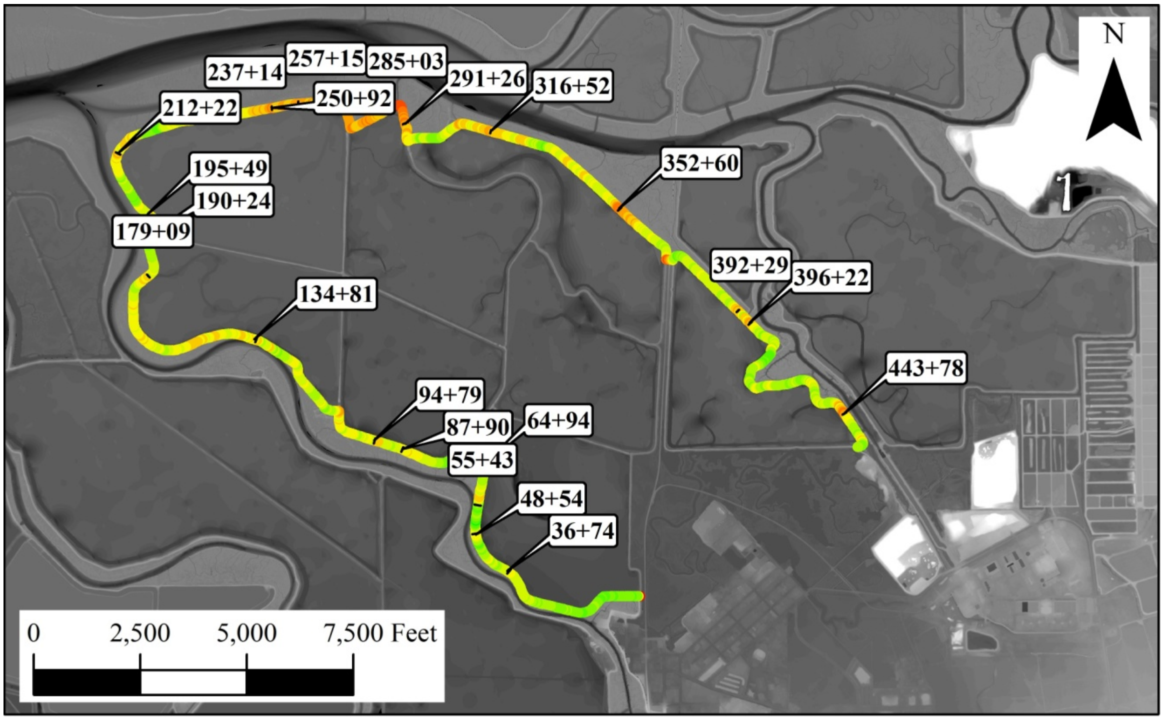


Figure 4-2: Locations of select cross-sections along the ouboard dike.

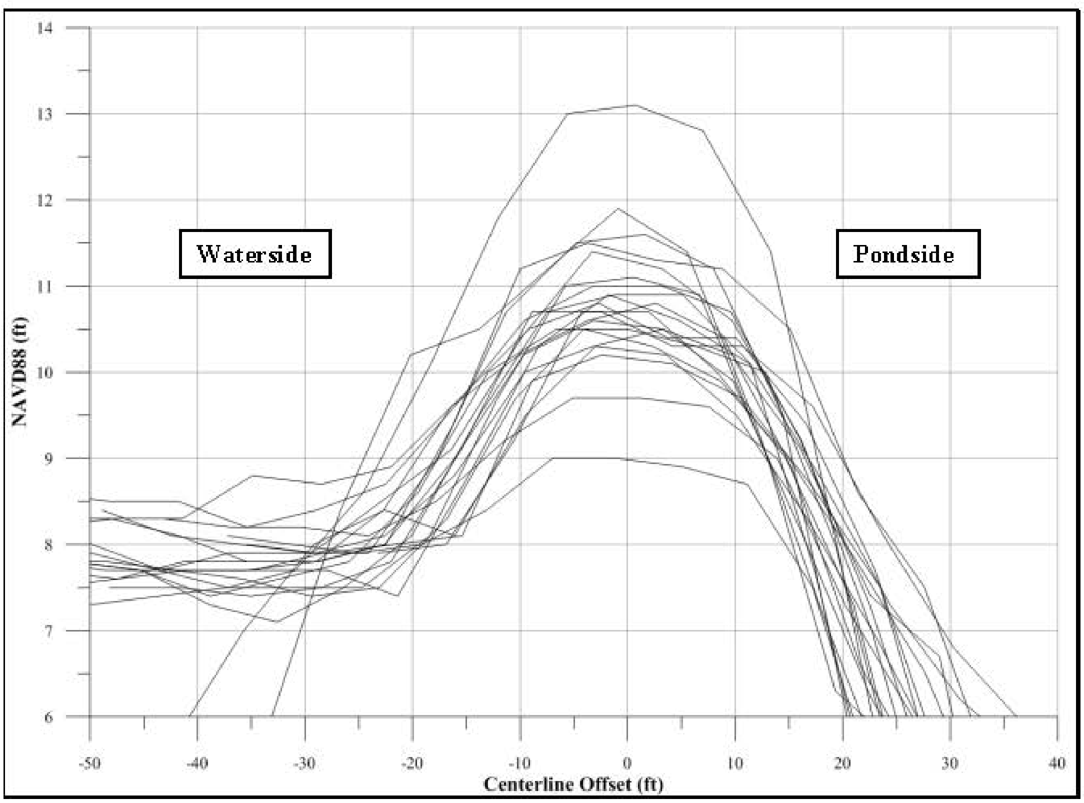


Figure 4-3: Cross-sections along the outboard dike.

4.2.3 Overtopping and Erosion Induced Breaching

Overtopping and erosion are critical to the performance of the outboard dike. Existing information duration of tidal flood events and the results of breach modeling efforts in the study area were used to estimate the thresholds at which the likelihood of breach along the outboard dike will occur. The following section discusses the basis for estimated loading duration and respective performance impacts to the outboard dike with respect to the peak water surface elevation (WSE) experience during a flood event.

The duration of flood loading was estimated using the tidal signal (i.e., shape) from the San Francisco Golden Gate tide gauge. The peak of the signal was set equal to a given WSE and the duration above lower elevations was recorded. Table 4-1 shows the approximate durations of loading above elevations incrementally lower than the peak WSE.

Table 4-1: Summary of durations exceeding elevations lower than the peak WSE.

Peak Water Level (NAVD88, ft)	WSE above (NAVD88, ft)	Duration Above WSE (hr)
12	11	4.5
	10	7
	9	9
	8	> 10
11	10	4.5
	9	7
	8	9
10	9	4.5
	8	7

USACE (2008) details the investigation and modeling effort to establish likely times to breach from wave attack, overtopping erosion, or both. Table 4-2 summarizes the overtopping scenarios likely to induce a breach at the outboard dike between Alviso and the ponds west of Artesian Slough. The table was adapted from USACE (2008) and shows the expected time to breach for overtopping scour only.

Table 4-2: Estimated time to breach versus dike crest width.

		Expected critical time to breach (hr) for respective crest width (ft)					
Height (ft) of overtopping	q (ft^3/s) per foot of dike	W = 25*	W = 20*	W = 15	W = 11	W = 7	W = 5
0.30	0.5	--	--	42.86	31.43	19.43	14.04
0.47	1	--	--	9.19	6.7	4.33	2.98
0.75	2	--	--	4.46	3.32	2.08	1.49
0.98	3	5.50	4.40	3.29	2.42	1.53	1.09
1.19	4	4.60	3.70	2.75	2.02	1.27	0.91

1. Overtopping height determined from broad crested weir equation (Henderson, 1966).
2. Overtopping flow rate from the Feasibility Scoping Meeting Geotechnical Appendix (USACE, 2009).
3. (*) Indicates time to breach interpolated from linear fit of data for dikes with W from 5 to 15 ft.

The cross-section geometry, anticipated loading duration, loading required for overtopping breach, and past performance were considered to identify possible breach locations. Figure 4-4 shows potential overtopping breaches that can be expected to occur from a given peak WSE. Point labels represent crest elevation and width at respective outboard dike station (Figure 4-2). Lines draw indicate the approximate threshold (i.e. overtopping duration vs. crest width) to which overtopping breaches are likely to occur. Of the 21 cross sections evaluated, three locations are at risk of an overtopping breach for a peak WSE of 11 ft. The number of potential overtopping breaches increases to 12 for a peak WSE of 12 ft.

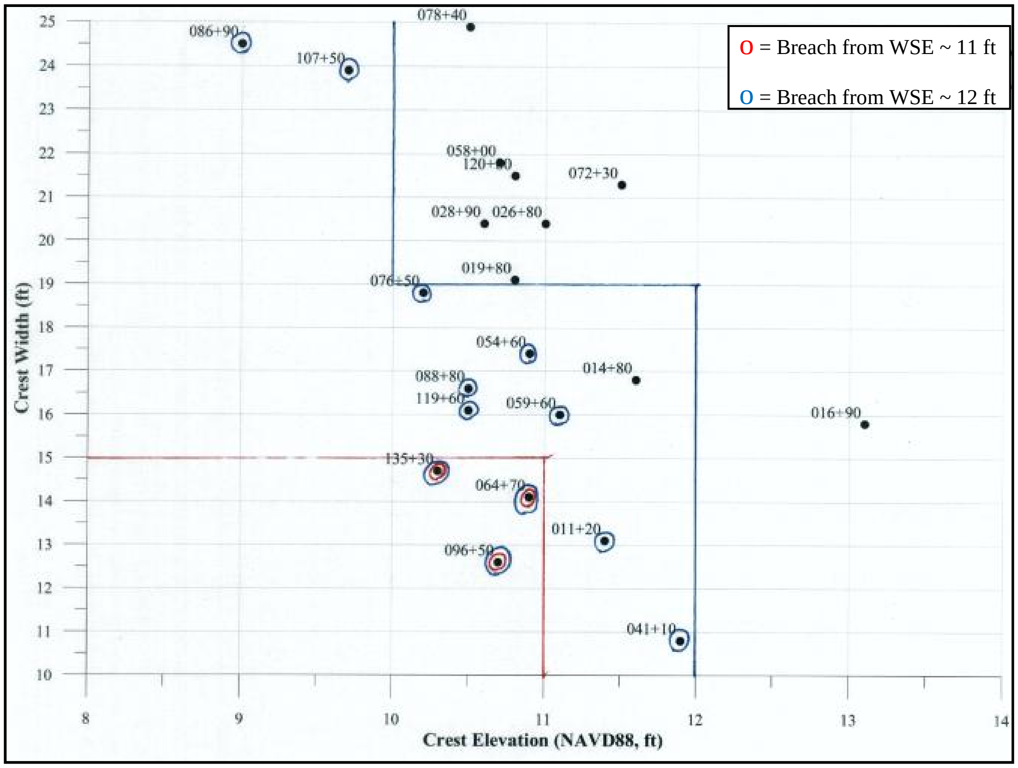


Figure 4-4: Potential overtopping breach locations for given peak WSE (Geomatrix, 2006).

The impact of wave attack and erosion on the waterside of the outboard contribute to the performance of the outboard dike. USACE (2008) modeled wave attack, however, wave height (i.e. 3 ft height or greater) was judged to be overestimated by at least 2 ft in the study area. Past performance along the outboard dike during frequent (i.e. non-overtopping) events was inferred from maintenance records for the period 1995 to 2005 (Geomatrix, 2006). These records provide a generally coarse interpretation of distress along the outboard dike. Figure 4-5 shows the number of repair episodes along the outboard dike in the period of record. Figure 4-6 shows the summed extent of repairs in the period of record when such records were available. The extent of repairs was typically described in terms of linear feet and/or cubic yards. A review of the storm frequency and annual maximum water levels showed a positive correlation between “stormier years” and increased maintenance (i.e. 1997 and 2003).

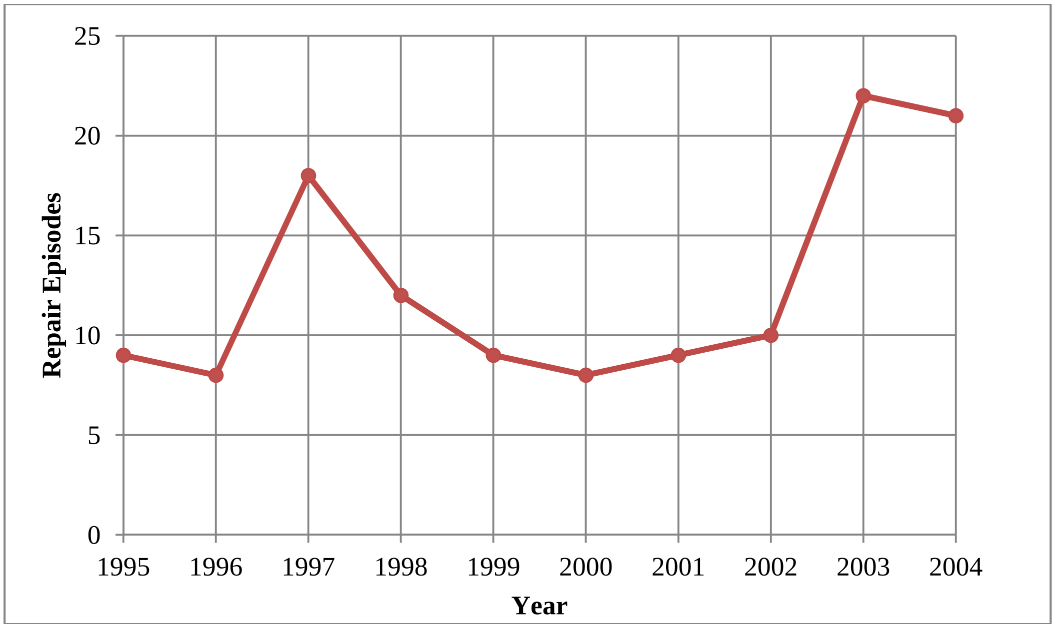


Figure 4-5: Number of maintenance episodes by year along the outboard dike.

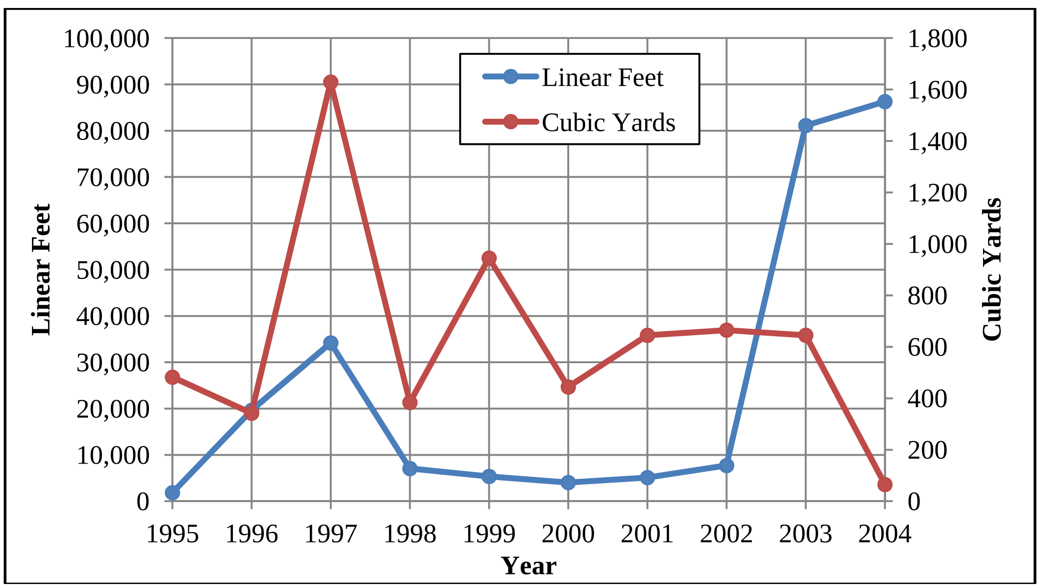


Figure 4-6: Summed total extent of repairs by year along the outboard dike (Geomatrix, 2006).

The fragility curve for outboard dike combined geotechnical investigation, numerical modeling, and maintenance record datasets to capture the primary mechanisms critical to performance along the outboard dike; overtopping and erosion. The key assumptions used to construct the fragility curve are as follows:

- Time to overtopping breach is quantitatively supported in the geotechnical analyses performed in USACE (2009a).
- Maintenance records demonstrate distress and/or damage occurring in “stormier years” with presumably higher than typical water surface elevations. Maintenance was generally ad-hoc

when the ponds and associated dikes were owned by Cargill, Inc.; however, the U.S. Fish and Wildlife Service (FWS) performs maintenance annually in the period following the wet season.

- Wave height in the project area is limited to 0.5 to 1 ft above the static WSE and does not increase with increasing static WSE. The outboard dike is assumed partially exposed to wave attack above elevation 8 ft and fully exposed above elevation 9 ft (Figure 4-3).
- The extent of resources (e.g., funding and staff) for FWS to maintain the outboard dike into the future is uncertain. To date, repairs have been prioritized to the areas of highest need and is not comprehensive to all needs (USACE, 2014a).

Figure 4-7 shows the fragility curves developed during the study for analysis pre- and post-AFB. Table 4-3 shows the estimated probability of unsatisfactory performance for the two mechanisms considered since the AFB and the combined probabilities for respective elevations. Justifications and support to the engineering judgment applied while estimating performance at each elevation are described in Table 4-3.

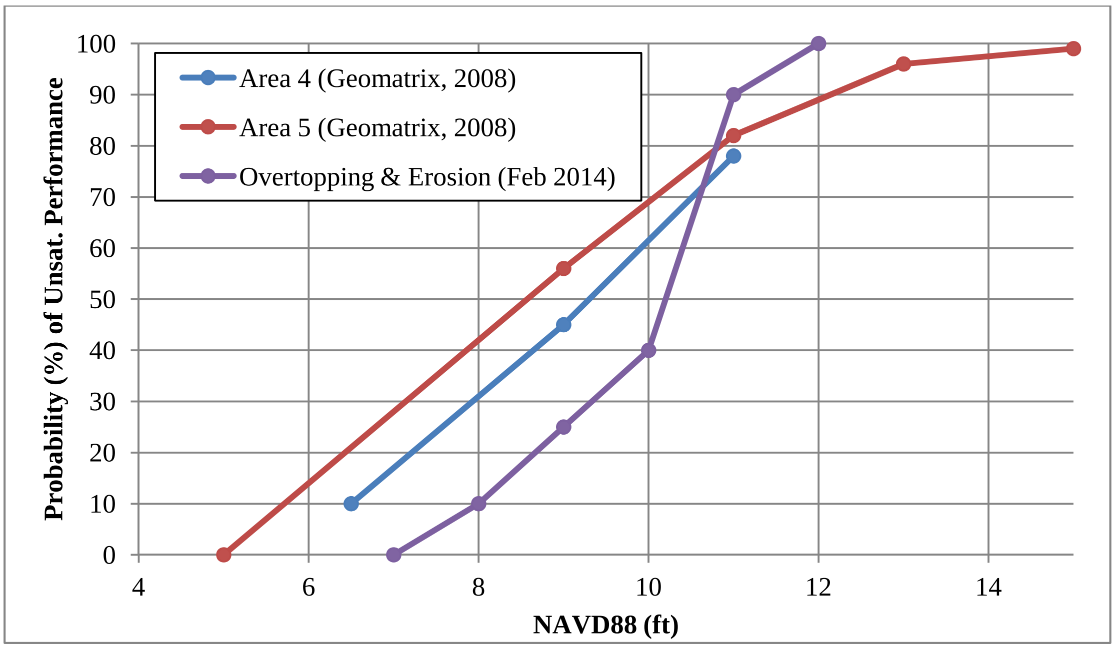


Figure 4-7: Comparison of outboard dike fragility curves

A sensitivity analysis was conducted to determine impact of the geotechnical fragility curve on calculated damages for the current and future without project condition. The analysis evaluated two additional fragility curves; (i) failure due to overtopping only, and (ii) no failure below elevation 10 feet. The additional fragility curves are discussed in detail in Tidal Flood Risk Analysis Appendix of this report.

Table 4-3: Updated probability of unsatisfactory performance (breach) based on erosion and overtopping mechanisms only.

Static WSE (NAVD88, ft)	Probability of Failure (P_u)			Comments
	Erosion	Overtopping	Combined[1]	
12	0.3	1.0	1.0	1. 32,000 ft of outboard dike (70% of length) overtops. About 21,000 ft overtops over elevation 11 ft for 4hrs, possibly inducing up to 3 overtopping breaches. 2. Overtopping of crest elevations at 10 ft for 6.5 hours, possibly inducing 9 additional overtopping breaches (Figure 4-4).
11	0.3	0.85	0.90	1. 9,250 ft of outboard dike (25% of length) overtops above elevation 10 ft for 4 hrs. Potential overtopping breaches at three locations. 2. Overtopping height is transient and the duration required to induce breaching may not occur. 3. Breach from combined erosion and overtopping increases the likelihood of breach at the three locations (Figure 4-4).
10	0.25	0.20	0.40	1. Overtopping at a limited number of locations. These locations have wide sections and sustain overtopping erosion for proportionally longer durations than narrow (< 15 feet) sections. 2. The dike crest in several reaches is composed of loose highly erodible silt with organics (USACE, 2014a). Time to overtopping breach may be substantially shorter in these reaches. 3. Rodent activity in the uppermost 1 to 3 feet of the dike section may contribute to internal erosion (USACE, 2014a) or effectively narrower crest width available during overtopping. 4. Very loose silts and organics in localized reaches of the dike crest may be substantially more erodible than assumed in USACE (2008). 5. Increased size and frequency of maintenance can be expected based on maintenance records (Geomatrix 2006). 6. The difference between the 2010 site survey and current conditions in 2014 is uncertain (e.g. potential for lower and thinner than measured crest elevations). 7. Repairs/Action to restore crest elevation from subsidence is recognized only after overtopping occurs (i.e., no periodic surveys/measurements of dikes). 8. Dike vulnerability to combined erosion and overtopping in low spots is very minor or incipient overtopping.
9	0.2	0.05	0.25	1. WSE in the range observed to have increased frequency and scope of repairs. 2. Lower WSE more frequent in a single wet season with maintenance performed annually and not ad-hoc. 3. Prioritization of repairs/maintenance relative to available resources can allow "semi-vulnerable" locations to become increasing vulnerable to loading. 4. Loss of section height and width due to normal coastal processes.
8	0.1	0	0.10	
7	0	0	0.0	1. Water levels experienced frequently (daily to weekly) with no noteworthy distress.

Notes:

1. Calculated per ETL 1110-2-547; (1 - Erosion) * (1 - Overtopping) = 1 - Combined.

4.3 Inboard Dike Performance

The inboard dike was assumed to fail due to overtopping. The inboard dike crest width is variable in the reach west of Artesian Slough. Crest widths are typically between 10 and 15 ft wide but can be as little as 8 ft along the alignment. Crest elevations vary from 6 to 11 feet suggesting substantial overtopping length (i.e. 1,000 ft) if the dike was exposed to normal high tides (i.e MHHW = 7 ft NAVD88) or greater than one mile of overtopping length for WSEs that cause an overtopping breach of the outboard dike. It can be inferred from Table 4-2 that an overtopping height of 1 ft for the duration of 3 to 4 hrs is likely to induce a breach through the inboard dike. An accumulation of overtopping high tide cycles in the days following a non-overtopping outboard dike breach, or an overtopping induced breach of the outboard dike would result in subsequent failure of the inboard dike.

Static failures prior to overtopping were not considered credible during the current effort. Water levels have been sustained for significant periods near mean tide elevation (i.e., 3.5 ft) without failure. If the outboard dike experienced a breach, normal high tide water levels (i.e., MHHW ~ 7 ft) would overtop the lowest reaches (elevation 6 to 6.5 ft) of the inboard dike. Therefore, sustained water levels that are appreciably above elevation 3 ft and do not overtop the inboard dike are highly unlikely.

4.4 Failure Mode Sequence

The geotechnical performance of the outboard dike is critical to the performance of the entire dike-pond system. The failure at the outboard dike will result in overtopping and subsequent failure at the inboard dike. Overtopping is likely to occur at as low as elevation 6.5 ft for the inboard dike. Overtopping, or a breach before overtopping, of the outboard dike will likely result in at least 2 feet of overtopping at the inboard dike. In addition, a breach of the inboard dike is assumed to occur shortly after breach of the outboard.

4.5 Breach Development

Levee failure logic requires estimates for breach dimensions that are likely to develop under variable hydraulic loading conditions. Breach dimensions were estimated using Nagy (2006) equations, which have correlated levee breach dimensions to retained water height, based on a review of 1000+ breaches. These dimensions were consistent with the more physical process breach modeling completed by USACE (2008). Table 4-4 summarizes these estimates. A memorandum summarizing the breach dimension analysis assumptions is included as Attachment C.

Table 4-4: Estimated Breach Lengths using Nagy (2006)

Approximate Water Height above Landside Toe (ft)	Estimated Fully Developed Breach Length (ft)
6.5	75
10	160
13	340
16	725
20	1530

5.0 REFERENCES

AMEC Geomatrix (2008), “Summary Report, Geotechnical Reliability Evaluation of Outboard Levees South San Francisco Bay Shoreline Study, Alameda and Santa Clara Counties, California”, Oakland, CA.

Geomatrix (2006), “South Bay Salt Pond Restoration Project Levee Assessment”, Oakland, CA.

Henderson, F. M. (1966), Open channel flow. MacMillian Publishing Co., New York, NY.

Nagy, L. (2006), “Estimating Dike Breach Length from Historical Data,” Periodica Polytechnica, Serial Civil Engineering, Vol. 50, No. 2, pp. 125-139.

Noble (2012), “Monte Carlo Simulation Under With Project Conditions for South San Francisco Bay Shoreline Study”, Noble Consultants, pgs. 23.

USACE (2008), “Erosion-induced Breaching: Reliability Assessment of San Francisco South Bay Salt Pond Levees”, *Geotechnical and Structures Laboratory*.

USACE (2009), “Appendix C: Geotechnical Investigation and Analysis South San Francisco Bay Shoreline Study F3 Milestone Without Project”, San Francisco District, San Francisco, CA.

USACE (2009a), “Reliability Assessment of San Francisco South Bay Salt Pond Inboard Levees”, Landris T. Lee Jr., Geotechnical and Structures Laboratory. Vicksburg, MS.

USACE (2014), “Memorandum for Record: South San Francisco Bay Shoreline Study Supplemental Analyses on Sea Level Change and flood risk associated with US Fish & Wildlife Service’s (FWS) Refuge Lands”, dated 12 Mar 2014.

USACE (2014a), “Geotechnical field assessment of the San Francisco South Bay dike system”, *Trip Report*, Richard Olsen, pgs. 14.

Attachment A

CESPN –ET –EG

5 AUGUST 2011

(minor revisions 15 June 2012)

PROJECT: South San Francisco Bay Shoreline Study

SUBJECT: Geotechnical Support for Alternatives Evaluation and Plan Formulation

Background

The Geo-Sciences Section of the San Francisco District of the Army Corps of Engineers (SPN) has been tasked with providing geotechnical input that will be used to develop cost estimates for various project alternatives as part of the plan formulation process. This memorandum is intended as an interim document that provides general guidance in schematic plan an alternative development. It is anticipated that additional geotechnical consultation may be required at various times during the alternative formulation process to support alternative designs and evaluation. This memo is intended to provide consolidation magnitude and time-rate settlements for various foundation Bay Mud and fill configurations.

Documents that have been relied upon in preparation of this memorandum are:

- Geotechnical Engineering Appendix in support of the Feasibility Scoping Meeting (2010). This document includes geotechnical investigation, laboratory testing and engineering analysis of the outboard levees performed by Geomatrix (under contract to the California Coastal Conservancy) and by SPN Geo-Sciences, geotechnical investigation and laboratory testing of the inboard levees performed by Geomatrix(under contract to the California Coastal Conservancy, 2010) and engineering analysis performed by the SPN Geo-Sciences Section.
- Conceptual Design information provided by the Santa Clara Valley Water District, in a June 28, 2011 email.

Scope of Work of Memorandum

A brief discussion of geotechnical needs for the current conceptual feasibility analysis was provided in a June 17, 2001 email. Six (6) items were proposed by Geosciences as tasks that would assist in the development of better project cost estimates for feasibility level planning and design. The tasks are summarized below:

1) Estimate settlement vs. fill height for various levee fill and foundation conditions
2) Estimates settlement rate for various fill and foundation conditions, including discussion of ways to increase rate of settlement, as appropriate.
3) Estimate maximum fill heights that could be placed at one time without overstressing the foundation soil for various fill height and slope configurations.
4) Typical fill specifications for levee fill.
5) Narrative discussion of geotechnical construction concerns for the proposed alternatives.

6) Narrative discussion of geotechnical concerns/considerations for proposed environmental earthwork.

This memorandum is intended to address items 1-6, above. This discussion is prepared in a DRAFT and INTERIM format and is provided to the PDT team (USACE and non-federal sponsors) for review and comment. After comments, this document will be submitted for District Quality Assurance review. The analysis, recommendations and other conclusions presented in interim technical memoranda are intended to be compiled in a geotechnical report appendix for the next major planning milestone (Alternatives Formulation Briefing).

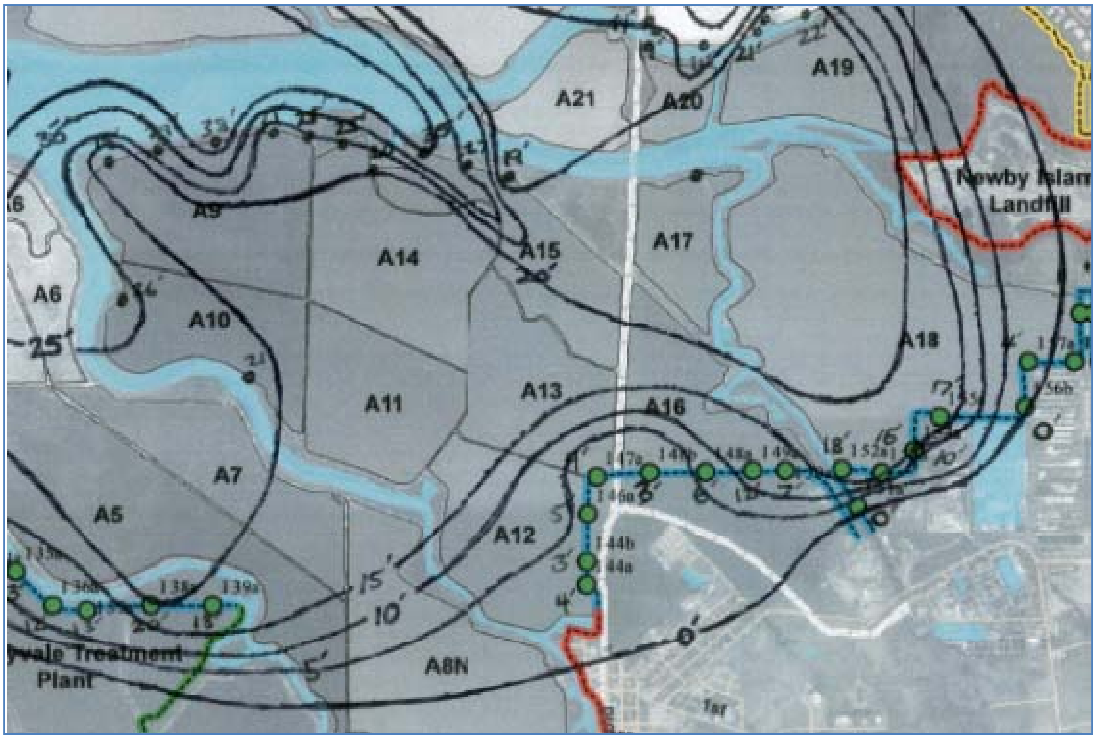


Figure 5-1 - Contours of Bay Mud Thickness

Task 1. Settlement Estimates

The project area is underlain by approximately 0 to 40 feet of marine soil deposits, locally known as Bay Mud. Bay Mud is generally normally consolidated, highly compressible and very weak clayey/silty soil. Bay Mud is commonly classified as CL/CH/ML/MH or OH depending on the location in the bay. Bay Mud was deposited underwater. Figure 1 shows the Corps' interpretation of the Bay Mud thickness for the project area. Along the edges of the deposit, the upper few feet (1-3 feet) has been observed to have slightly less compressibility, higher strength and higher over consolidation ratios, due to some desiccation drying of the soil during tidal cycles. This upper layer is commonly identified as Bay Mud "crust".

It is anticipated that the primary settlement concern for the project will be Bay Mud primary consolidation due to construction of earth or other structures on the Bay Mud. Consolidation settlement has complex soil mechanics that depends on the soil permeability, stress history, applied loads, existing loads, load geometry and other factors. The discussion below is intended to be general and detailed enough in nature to have suitable confidence in consolidation estimation for feasibility level design and cost comparisons, however more detailed settlement calculations are likely to be required once more refined designs are developed.

Three graphs estimating earthwork settlement are presented below. Several key assumptions were made in the analysis, as follows.

- Bay Mud is normally consolidated under the existing loads, and that all settlement due to existing loads is complete.
- The upper 2 feet of Bay Mud is over consolidated, with an over consolidation ratio (OCR) of 2.
- Bay Mud will generally remain in-place beneath new construction
- Bay Mud has a virgin compression index (strain based) of 0.32
- Bay Mud has a recompression index (strain based) of 0.03.
- New levees will have a crest width of 16 feet and 3:1 (H:V) side slopes on both landside and waterside of the levees (note that this may be different based on additional stability and seepage analysis).
- New fill will have a total unit weight of 125 pounds per cubic foot
- Existing levee fills are assumed to have a total unit weight of 115 pounds per cubic foot.
- Bay Mud crust has a total unit weight of 100 pounds per cubic foot
- Normally consolidated Bay Mud has a total unit weight of 97 pounds per cubic foot
- Bay Mud is 100 percent saturated at all depths

Graph 1 shows the estimated Bay Mud consolidation settlement for a large mass fill area, such as may be required for very wide environmental island construction, unusually large levees, and other large fill areas.

Graph 2 shows the estimated Bay Mud consolidation settlement for levees constructed directly on Bay Mud (no existing fills present). Settlements will be reduced if new levees can be constructed along the same alignment as existing levee fill alignments. Conceptual sketches provided by the SCVWD have indicated that some of the alternatives are proposed along the same alignment as existing levee fills. On an initial estimating basis, the design grade change should be the difference between levee crest elevations (new – existing) to estimate settlement, if the center lines of the levee crest are collinear.

To use Charts 1 and 2 below an iterative process is required, such that;

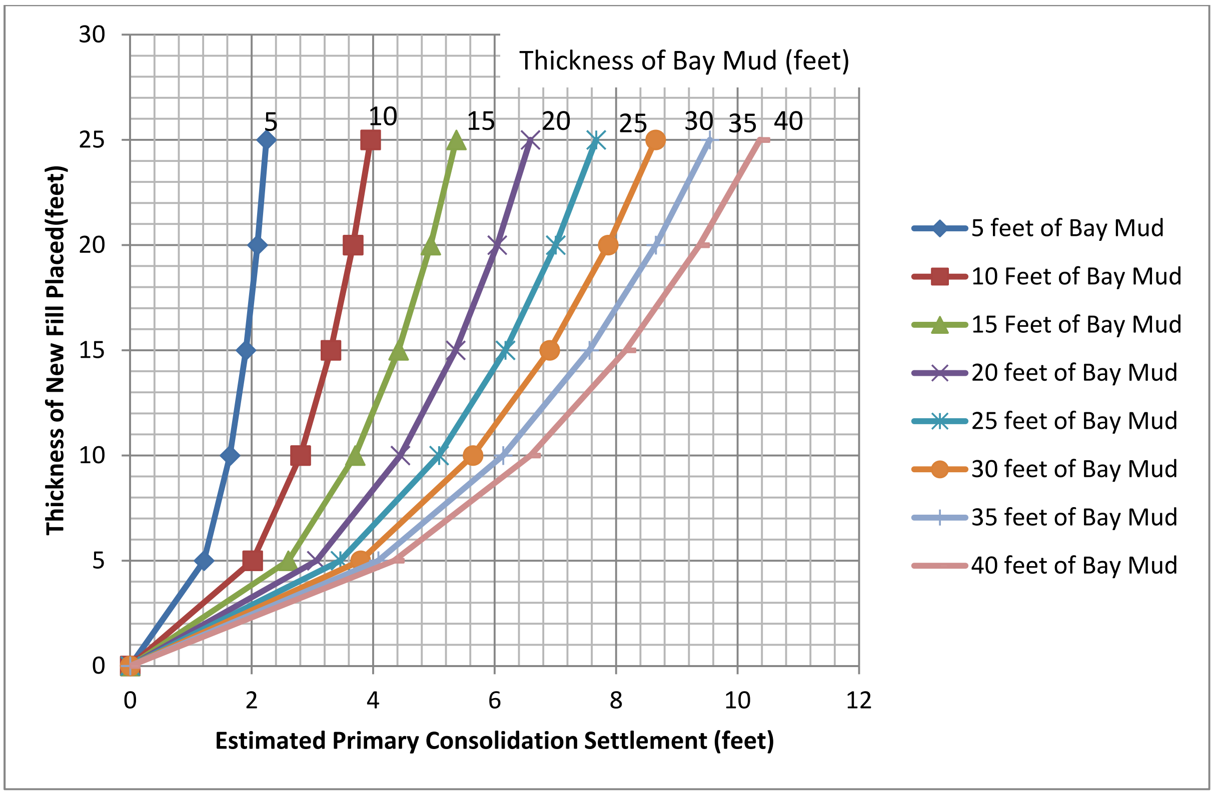
Fill thickness – settlement = design change in grade

For example: if the existing elevation = 0 feet

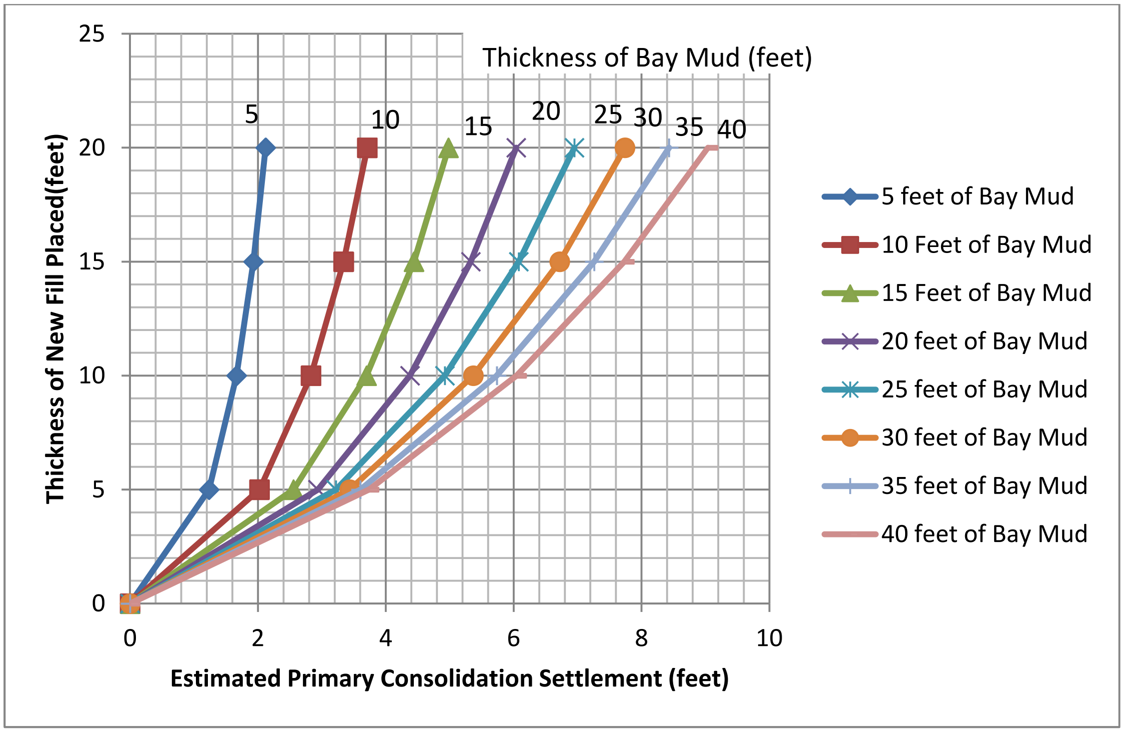
Design elevation = 10 feet

From Chart 1 , for a Bay Mud thickness of 20 feet the solution would be about 15 feet of fill

15 feet of fill - ~ 5 feet of settlement = design elevation of 10 feet.



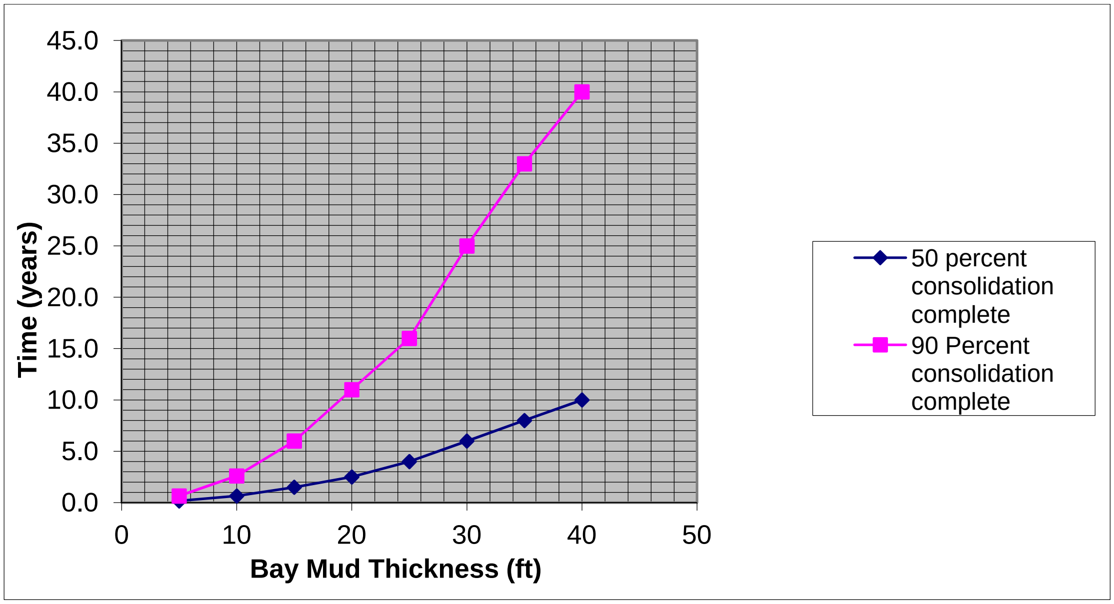
Graph 1. Estimated Bay Mud Consolidation Settlement for Large Areal Fills



Graph 2. Estimated Bay Mud Consolidation Settlement at Center of Crest for Levees with 3:1 (H:V) Slopes

Task 2. Consolidation Rates

Graph 3 and Table 1, present the estimated time for 50 percent and 90 percent consolidation for various Bay Mud thicknesses. Assumptions in the time rate consolidation include the assumption that double drainage will occur and that the coefficient of consolidation for the Bay Mud is 8 ft^2/yr. The time for consolidation is relatively short (less that 1 year for 90 percent consolidation) for thin Bay Mud thicknesses (5 feet or less). If it is desired to reduce the time for consolidation, which may be especially important if the required fill cannot be place at one time due to foundation and bearing capacity limitations of the Bay Mud, vertical drains can be installed. Typically vertical drains extend the entire thickness of the Bay Mud and are spaced on 2 to 6 foot centers depending on the drain material, and the project settlement time constraints. Additionally, surcharge fills can be placed to further reduce the time line in some situations. Vertical drains will allow the dissipation of construction pore pressures over months instead of years, which will allow additional fill stages to be placed in months rather than waiting years.



Graph 3. Estimated Consolidation Rates for Bay Mud

Table 1. Estimated Consolidation Rates for Bay Mud

Bay Mud Thickness (feet)	Time for 50% consolidation (years)	Time for 90% consolidation (years)
5	0.2	0.7
10	0.7	2.6
15	1.5	6.0
20	2.5	11.0
25	4.0	16.0
30	6.0	25.0
35	8.0	33.0
40	10.0	40.0

Task 3. Estimated Maximum Fill Thickness that Can Be Placed at One Time

Because the underlying Bay Mud for the project area is weak and slowly draining the weak Bay Mud will only support limited fill thicknesses without being overstressed. Overfilling Bay Mud will cause slope instability and bearing failures. Filling to design grades may be required in stages to allow for pore pressure dissipation before each new stress is applied. Overfilling on Bay Mud is a well documented phenomenon and should be carefully considered in design and construction activities. In addition to new structures, construction activities that may include stockpiles, heavy equipment, or excavations should be carefully planned do avoid overstressing the Bay Mud. Piezometric monitoring Bay Mud pore pressures in fill areas during construction is recommended, to determine when pore pressures have dissipated enough to allow additional filling. Table 2, below includes estimated allowable first filling thicknesses for various fill side slopes, of 3:1(H:V) to 5:1 (H:V). In areas where fills are planned where previously placed fills were/are located, allowable fill heights will be somewhat higher. The recommendations below are based on allowable end-of-construction (undrained loading) factors of safety of 1.3. Bay Mud was assumed to have an undrained strength ratio of 0.32 (S_u/σ') for the normally consolidated Bay Mud and 500 psf for the upper 2 feet (Bay Mud "crust"). Fill was assumed to have a unit weight of 125 pounds per cubic foot and an undrained shear strength of 800 psf.

From the analysis it appears that where Bay Mud is shallow (about 5 feet or less) such that all of the required fill can be placed in one stage. However, for areas of the project with more than 5 feet of Bay Mud, fill will need to be placed in stages for significant grade changes. It is assumed that undrained (end-of-construction) conditions will control slope designs, and that rapid drawdown and seepage loading will be satisfactory if end-of-construction factors of safety exceed 1.3. Additional fill stages may not be able to include as much fill thickness as the first stage, and will require careful planning.

Table 2. Estimated Fill Thickness Placement Limits for first fill stage for 3:1 to 5:1 Slopes on 5 to 40 feet of Bay Mud

Bay Mud Thickness (ft)	Side Slope of Fill (H:V)		
	3:1	**4:1**	**5:1**
5	20 feet	20 feet	20 feet
10	11 feet	11 feet	16 feet
15	9 feet	10 feet	14 feet
20	9 feet	10 feet	12 feet
40	8 feet	10 feet	12 feet

Task 4. Levee Material Specifications

Almost any soil can be used in the construction of levees, if the levee is properly designed for the fill used. In general, it is anticipated that the only on-site available borrow would be Bay Mud. Bay Mud would require significant processing (aeration, mixing, and possible chemical treatment) before it would be practical to use as a levee fill, additionally it may not meet levee fill specifications that reflect local engineering practice. In general levee fill that meet the following specifications is preferred. It is anticipated that fill materials meeting the following specifications will be available at a number of nearby quarries or construction sites. If material meeting the following specifications is not available, revisions to specifications is likely to be possible to avoid excessively long haul distances, although levee designs may require some revision to accommodate different specifications.

1) USCS soil types: CL, SC, or GC

2) At least 70 percent passing the No.4 sieve
3) 100 percent less than 4 inches in greatest dimension
4) No more than 15 percent larger than 2 ½ inches.
5) Plasticity Index of 10 to 20
6) Liquid Limit less than 40
7) Free of organic content
8) Non-dispersive clay minerals
9) Low hydraulic conductivity (less that 10^{-6} cm/sec)
10) Minimum undrained shear strength of 800 psf
11) Minimum effective friction angle of 32 degrees
12) Fill should be clean of environmental contaminants

Task 5. Discussion of Geotechnical Aspects of Proposed Cross Sections

The Santa Clara Valley Water District has performed some initial design in order to estimate costs of various levee alternatives. Figures 2 and 3 show several possible levee alignments and a typical levee cross section that the SCVWD has provided in alternative planning and discussion. USACE understands that detailed refined design has not occurred, and the provided designs are a starting point for discussion.

A brief discussion of the geotechnical considerations for each proposed alignment is presented below.

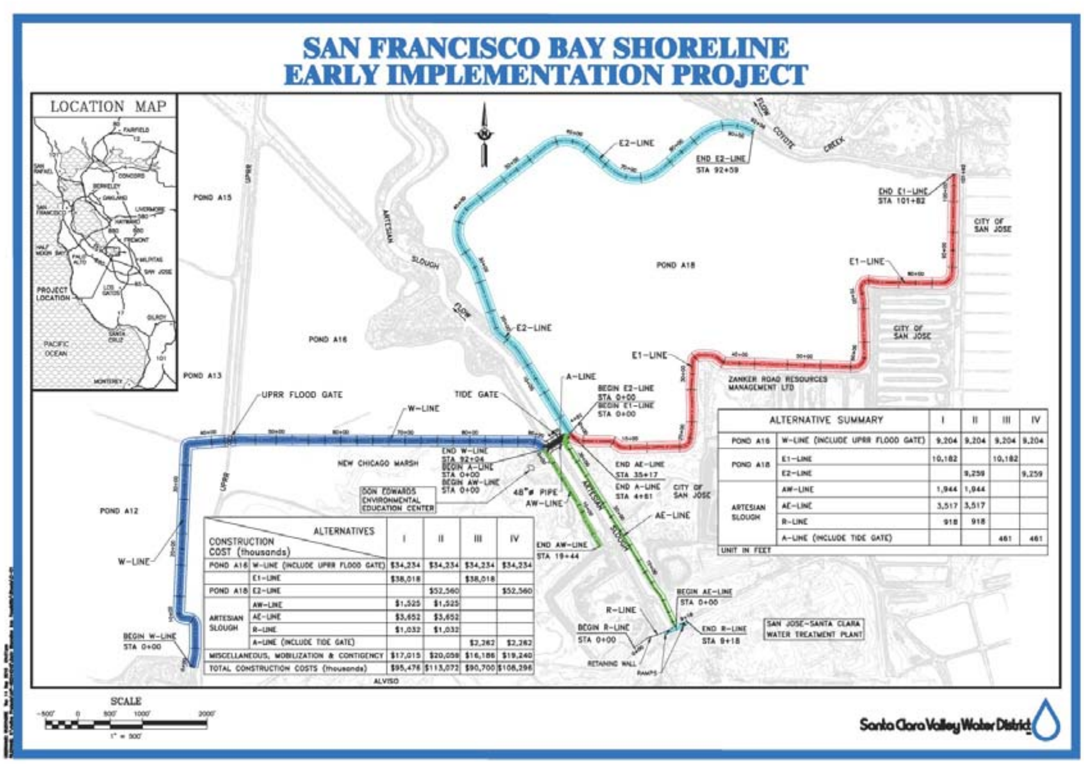


Figure 2. Alignments of Possible Flood Damage Reduction alternatives

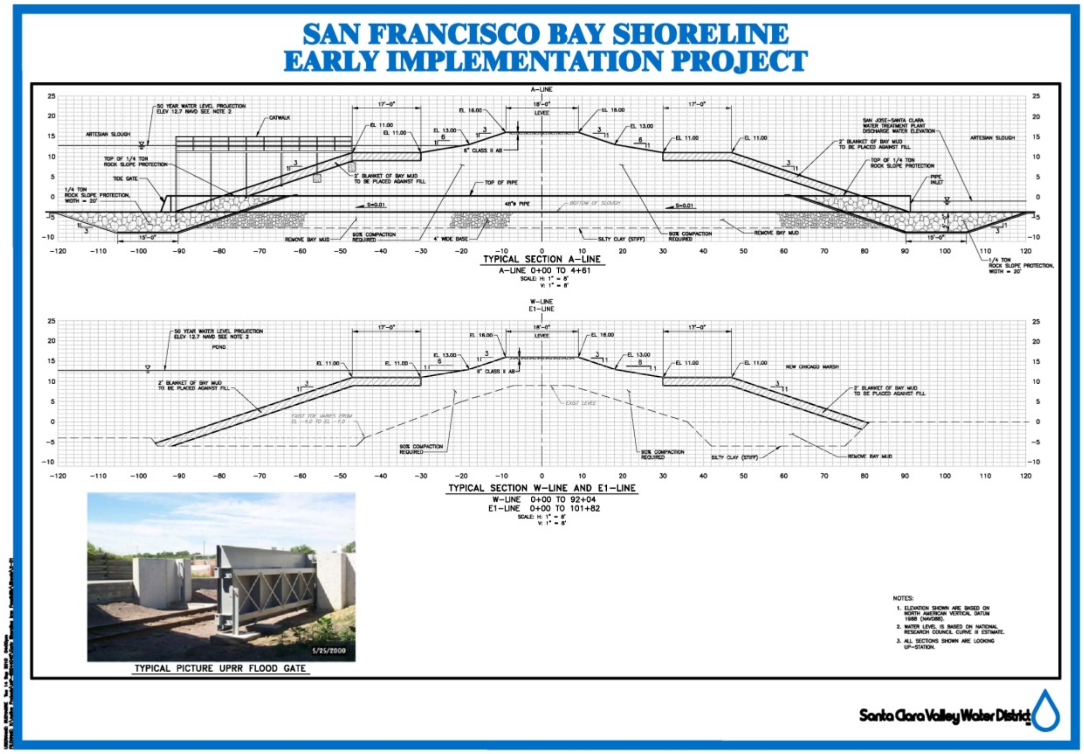


Figure 2. Typical Cross Sections

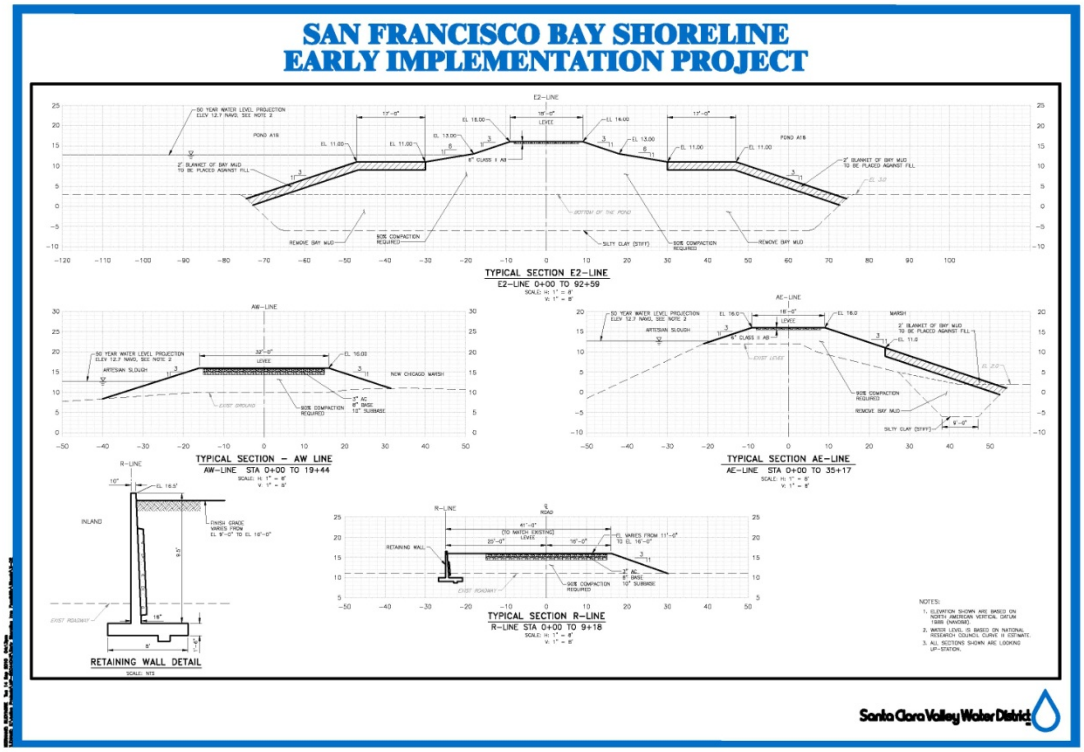


Figure 3. Typical Cross Sections

<u>Proposed alignment Line E-2</u>

- Bay Mud thickness is anticipated to be on the order of 20 feet. Excavation to deeper stiff soil would be very expensive and require very significant dewatering and soil disposal costs. Ground water is anticipated to be encountered near the ground surface (within 2 feet). Excavating to deep soil would reduce settlement and stability problem potential.
- It is anticipated that construction directly on Bay Mud with vertical drains and staged construction would be more practical.
- Total fill grade change would be about 13 feet, which would require a fill thickness on the order of 19 feet. This would require at least two fill stages to construct.
- There does not appear to be significant geotechnical value (perhaps there is a vegetation benefit to a Bay Mud levee surface?) to the 2-foot Bay Mud blanket on either side of the levee. This detail would add a construction difficulty due to handling and controlling the placement and compaction of different materials in a levee cross section.
- The slope designs appear to have included some thought that the relatively flat configuration, with slope benches would allow construction in a single stage, however it is not anticipated that in the current alignments single stage fill placement would be possible. However, if two stages of earthwork are performed steeper slopes may be practical.

- If low permeability fills are used seepage concerns are not anticipated. In order to add seepage performance reliability, a landside toe drain would add both stability and seepage performance reliability.
- Drainage features (conduits crossing from landside to bay) would need to have appropriate pipe bedding, joint flexibility and camber.

Proposed tide gate alignment Line A-2

- Bay Mud thickness is anticipated to be on the order of 20 feet. Excavation to deeper stiff soil would be very expensive and require very significant dewatering and soil disposal costs. Excavating to deep soil would reduce settlement and stability problem potential.
- It is anticipated that construction directly on Bay Mud with vertical drains and staged construction would be more practical.
- Total fill grade change would be about 13 feet, which would require a fill thickness on the order of 19 feet. This would require at least two fill stages to construct.
- There does not appear to be significant geotechnical value (perhaps there is a vegetation benefit to a Bay Mud levee surface?) to the 2-foot Bay Mud blanket on either side of the levee. This detail would add a construction difficulty due to handling and controlling the placement and compaction of different materials in a levee cross section.
- The slope designs appear to have included some thought that the relatively flat configuration, with slope benches would allow construction in a single stage, however it is not anticipated that in the current alignments single stage fill placement would be possible. However, if two stages of earthwork are performed steeper slopes may be practical.
- If low permeability fills are used seepage concerns are not anticipated. In order to add seepage performance reliability, a landside toe drain would add both stability and seepage performance reliability.
- Conduits would need to have appropriate pipe bedding, joint flexibility and camber. Possibly filling would be required first, with construction of gates, post levee settlement.
- Recommend consideration of concrete structure supported on deep foundations.
- Differential settlement will need to be considered at gate/levee joint due to differing stress histories.

Proposed AE and AW alignments

- Differing foundations will lead to differential settlement along each alignment. Design should account for this.
- There does not appear to be significant geotechnical value (perhaps there is a vegetation benefit to a Bay Mud levee surface?) to the 2-foot Bay Mud blanket on either side of the levee. This detail would add a construction difficulty due to handling and controlling the placement and compaction of different materials in a levee cross section.
- Practically, construction of thin slope wedges is very difficult. Consider how the construction benching and compaction into the existing levee will be performed in earthwork estimates.

Line R

- Retaining wall supported road appears feasible. Preliminary designs have not been checked.

Proposed alignment Line E-1 and Line W

- Bay Mud thickness is anticipated to range from less than 4 feet to about 20 feet.
- There does not appear to be significant geotechnical value (perhaps there is a vegetation benefit to a Bay Mud levee surface?) to the 2-foot Bay Mud blanket on either side of the levee. This detail would add a construction difficulty.
- The alignments share the same center line as existing levee alignments. This is anticipated to reduce the settlement potential for new levees and provide some slope stability benefit.
- The proposed levees are shown to be constructed on top of existing levees. Geotechnical analysis of the existing levees indicates that differing soil conditions are present along the existing alignment with both clayey and sandy fill soils. It is suggested that in order to improve reliability and certainty the existing levee should be removed, soils mixed to the specifications and re-built. The new levee should be located in the same alignment.
- Differential settlement will need to be considered due to differing Bay Mud thicknesses along alignments.
- Sections include excavation to stiff soil below Bay Mud. This will not be practical due to thicker Bay Mud at many locations. Designs should account for appropriate settlement and stability recommendations as discussed above. It is likely staging may be required for thicker Bay Mud deposits.
- Toe drains or other drainage features may improve seepage reliability.

Proposed RR Gate

- The closure gate across the RR lines will likely need to be supported on deep foundations.
- Differential settlement and lateral loading on tracks and foundation will need to be accounted for due to adjacent levee filling
- Reliability of gate including maintenance and operations considerations should be considered carefully in the alternatives analysis.

Geotechnical Considerations for Environmental Restoration Alternatives

The primary geotechnical considerations for environmental restoration alternatives are earthwork settlement and stability. Estimates of settlement and maximum fill thickness for various Bay Mud conditions are included above in the discussion. Fills not only cause settlement under the filled area, but also can cause settlement of nearby adjacent features. Environmental fills should be properly designed and constructed to minimize these effects on utilities, infrastructure, and flood damage reduction features.

In addition, alternatives should not impact the ability to inspect, maintain, or emergency flood fight around flood damage reduction projects.

Sample Calculation for Large Aereal Fills in the Alviso Area

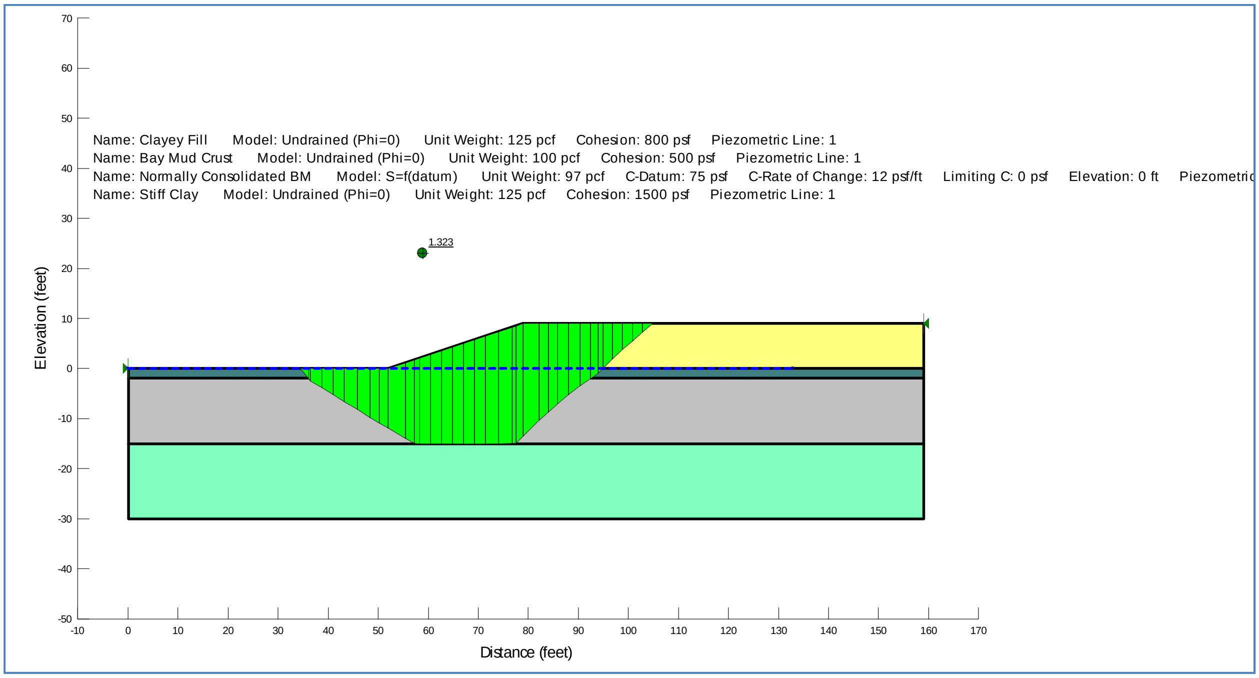
depth below existing mud line (ft)	depth below GW	u (psf)	Total unit weight	Total Stress	Initial Effective Stress (psf)	Layer thicknes	Cc (strain index)	Cr (strain index)	delta p (psf)	OCR	Pp	Primary Consolidation (ft)	Cumulative Consolidation (ft)
0	0	0	100	0	0	0	0.32	0.03	600	2	0	#DIV/0!	0
1	1	62.4	100	100	37.6	1	0.32	0.03	600	2	75.2	0.3061	0.3061
2	2	124.8	100	200	75.2	1	0.32	0.03	600	2	150.4	0.2177	0.5238
3	3	187.2	97	297	109.8	1	0.32	0.03	600	1	109.8	0.2594	0.7832
4	4	249.6	97	394	144.4	1	0.32	0.03	600	1	144.4	0.2279	1.0111
5	**5**	**312**	**97**	**491**	**179**	**1**	**0.32**	**0.03**	**600**	**1**	**179**	**0.2044**	**1.2155**
6	6	374.4	97	588	213.6	1	0.32	0.03	600	1	213.6	0.1859	1.4014
7	7	436.8	97	685	248.2	1	0.32	0.03	600	1	248.2	0.1708	1.5721
8	8	499.2	97	782	282.8	1	0.32	0.03	600	1	282.8	0.1582	1.7303
9	9	561.6	97	879	317.4	1	0.32	0.03	600	1	317.4	0.1475	1.8778
10	**10**	**624**	**97**	**976**	**352**	**1**	**0.32**	**0.03**	**600**	**1**	**352**	**0.1383**	**2.0161**
11	11	686.4	97	1073	386.6	1	0.32	0.03	600	1	386.6	0.1302	2.1463
12	12	748.8	97	1170	421.2	1	0.32	0.03	600	1	421.2	0.1231	2.2694
13	13	811.2	97	1267	455.8	1	0.32	0.03	600	1	455.8	0.1167	2.3861
14	14	873.6	97	1364	490.4	1	0.32	0.03	600	1	490.4	0.1111	2.4972
15	**15**	**936**	**97**	**1461**	**525**	**1**	**0.32**	**0.03**	**600**	**1**	**525**	**0.1059**	**2.6031**
16	16	998.4	97	1558	559.6	1	0.32	0.03	600	1	559.6	0.1013	2.7044
17	17	1060.8	97	1655	594.2	1	0.32	0.03	600	1	594.2	0.0970	2.8014
18	18	1123.2	97	1752	628.8	1	0.32	0.03	600	1	628.8	0.0931	2.8945
19	19	1185.6	97	1849	663.4	1	0.32	0.03	600	1	663.4	0.0895	2.9840
20	**20**	**1248**	**97**	**1946**	**698**	**1**	**0.32**	**0.03**	**600**	**1**	**698**	**0.0862**	**3.0702**
21	21	1310.4	97	2043	732.6	1	0.32	0.03	600	1	732.6	0.0831	3.1534
22	22	1372.8	97	2140	767.2	1	0.32	0.03	600	1	767.2	0.0803	3.2337
23	23	1435.2	97	2237	801.8	1	0.32	0.03	600	1	801.8	0.0776	3.3113
24	24	1497.6	97	2334	836.4	1	0.32	0.03	600	1	836.4	0.0752	3.3864
25	**25**	**1560**	**97**	**2431**	**871**	**1**	**0.32**	**0.03**	**600**	**1**	**871**	**0.0728**	**3.4593**
26	26	1622.4	97	2528	905.6	1	0.32	0.03	600	1	905.6	0.0706	3.5299
27	27	1684.8	97	2625	940.2	1	0.32	0.03	600	1	940.2	0.0686	3.5985
28	28	1747.2	97	2722	974.8	1	0.32	0.03	600	1	974.8	0.0667	3.6652
29	29	1809.6	97	2819	1009.4	1	0.32	0.03	600	1	1009.4	0.0648	3.7300
30	**30**	**1872**	**97**	**2916**	**1044**	**1**	**0.32**	**0.03**	**600**	**1**	**1044**	**0.0631**	**3.7931**
31	31	1934.4	97	3013	1078.6	1	0.32	0.03	600	1	1078.6	0.0615	3.8546
32	32	1996.8	97	3110	1113.2	1	0.32	0.03	600	1	1113.2	0.0599	3.9145
33	33	2059.2	97	3207	1147.8	1	0.32	0.03	600	1	1147.8	0.0584	3.9729
34	34	2121.6	97	3304	1182.4	1	0.32	0.03	600	1	1182.4	0.0570	4.0300
35	**35**	**2184**	**97**	**3401**	**1217**	**1**	**0.32**	**0.03**	**600**	**1**	**1217**	**0.0557**	**4.0857**
36	36	2246.4	97	3498	1251.6	1	0.32	0.03	600	1	1251.6	0.0544	4.1401
37	37	2308.8	97	3595	1286.2	1	0.32	0.03	600	1	1286.2	0.0532	4.1933
38	38	2371.2	97	3692	1320.8	1	0.32	0.03	600	1	1320.8	0.0520	4.2454
39	39	2433.6	97	3789	1355.4	1	0.32	0.03	600	1	1355.4	0.0509	4.2963
40	**40**	**2496**	**97**	**3886**	**1390**	**1**	**0.32**	**0.03**	**600**	**1**	**1390**	**0.0499**	**4.3462**

depth below existing mud line (ft)	depth below GW	u (psf)	Total unit weight	Total Stress	Initial Effective Stress (psf)	Layer thicknes	Cc (strain index)	Cr (strain index)	delta p (psf)	OCR	Pp	Primary Consolidation (ft)	Cumulative Consolidation (ft)
0	0	0	100	0	0	0	0.32	0.03	1200	2	0	#DIV/0!	0
1	1	62.4	100	100	37.6	1	0.32	0.03	1200	2	75.2	0.3983	0.3983
2	2	124.8	100	200	75.2	1	0.32	0.03	1200	2	150.4	0.3061	0.7044
3	3	187.2	97	297	109.8	1	0.32	0.03	1200	1	109.8	0.3445	1.0489
4	4	249.6	97	394	144.4	1	0.32	0.03	1200	1	144.4	0.3101	1.3589
5	**5**	**312**	**97**	**491**	**179**	**1**	**0.32**	**0.03**	**1200**	**1**	**179**	**0.2837**	**1.6427**
6	6	374.4	97	588	213.6	1	0.32	0.03	1200	1	213.6	0.2626	1.9053
7	7	436.8	97	685	248.2	1	0.32	0.03	1200	1	248.2	0.2451	2.1505
8	8	499.2	97	782	282.8	1	0.32	0.03	1200	1	282.8	0.2303	2.3807
9	9	561.6	97	879	317.4	1	0.32	0.03	1200	1	317.4	0.2174	2.5982
10	**10**	**624**	**97**	**976**	**352**	**1**	**0.32**	**0.03**	**1200**	**1**	**352**	**0.2062**	**2.8044**
11	11	686.4	97	1073	386.6	1	0.32	0.03	1200	1	386.6	0.1962	3.0006
12	12	748.8	97	1170	421.2	1	0.32	0.03	1200	1	421.2	0.1873	3.1879
13	13	811.2	97	1267	455.8	1	0.32	0.03	1200	1	455.8	0.1793	3.3672
14	14	873.6	97	1364	490.4	1	0.32	0.03	1200	1	490.4	0.1720	3.5391
15	**15**	**936**	**97**	**1461**	**525**	**1**	**0.32**	**0.03**	**1200**	**1**	**525**	**0.1653**	**3.7045**
16	16	998.4	97	1558	559.6	1	0.32	0.03	1200	1	559.6	0.1592	3.8637
17	17	1060.8	97	1655	594.2	1	0.32	0.03	1200	1	594.2	0.1536	4.0173
18	18	1123.2	97	1752	628.8	1	0.32	0.03	1200	1	628.8	0.1484	4.1656
19	19	1185.6	97	1849	663.4	1	0.32	0.03	1200	1	663.4	0.1435	4.3092
20	**20**	**1248**	**97**	**1946**	**698**	**1**	**0.32**	**0.03**	**1200**	**1**	**698**	**0.1390**	**4.4482**
21	21	1310.4	97	2043	732.6	1	0.32	0.03	1200	1	732.6	0.1348	4.5830
22	22	1372.8	97	2140	767.2	1	0.32	0.03	1200	1	767.2	0.1309	4.7138
23	23	1435.2	97	2237	801.8	1	0.32	0.03	1200	1	801.8	0.1272	4.8410
24	24	1497.6	97	2334	836.4	1	0.32	0.03	1200	1	836.4	0.1237	4.9647
25	**25**	**1560**	**97**	**2431**	**871**	**1**	**0.32**	**0.03**	**1200**	**1**	**871**	**0.1204**	**5.0850**
26	26	1622.4	97	2528	905.6	1	0.32	0.03	1200	1	905.6	0.1173	5.2023
27	27	1684.8	97	2625	940.2	1	0.32	0.03	1200	1	940.2	0.1143	5.3166
28	28	1747.2	97	2722	974.8	1	0.32	0.03	1200	1	974.8	0.1115	5.4281
29	29	1809.6	97	2819	1009.4	1	0.32	0.03	1200	1	1009.4	0.1089	5.5370
30	**30**	**1872**	**97**	**2916**	**1044**	**1**	**0.32**	**0.03**	**1200**	**1**	**1044**	**0.1063**	**5.6433**
31	31	1934.4	97	3013	1078.6	1	0.32	0.03	1200	1	1078.6	0.1039	5.7473
32	32	1996.8	97	3110	1113.2	1	0.32	0.03	1200	1	1113.2	0.1016	5.8489
33	33	2059.2	97	3207	1147.8	1	0.32	0.03	1200	1	1147.8	0.0995	5.9484
34	34	2121.6	97	3304	1182.4	1	0.32	0.03	1200	1	1182.4	0.0974	6.0457
35	**35**	**2184**	**97**	**3401**	**1217**	**1**	**0.32**	**0.03**	**1200**	**1**	**1217**	**0.0954**	**6.1411**
36	36	2246.4	97	3498	1251.6	1	0.32	0.03	1200	1	1251.6	0.0934	6.2345
37	37	2308.8	97	3595	1286.2	1	0.32	0.03	1200	1	1286.2	0.0916	6.3261
38	38	2371.2	97	3692	1320.8	1	0.32	0.03	1200	1	1320.8	0.0898	6.4160
39	39	2433.6	97	3789	1355.4	1	0.32	0.03	1200	1	1355.4	0.0881	6.5041
40	**40**	**2496**	**97**	**3886**	**1390**	**1**	**0.32**	**0.03**	**1200**	**1**	**1390**	**0.0865**	**6.5906**

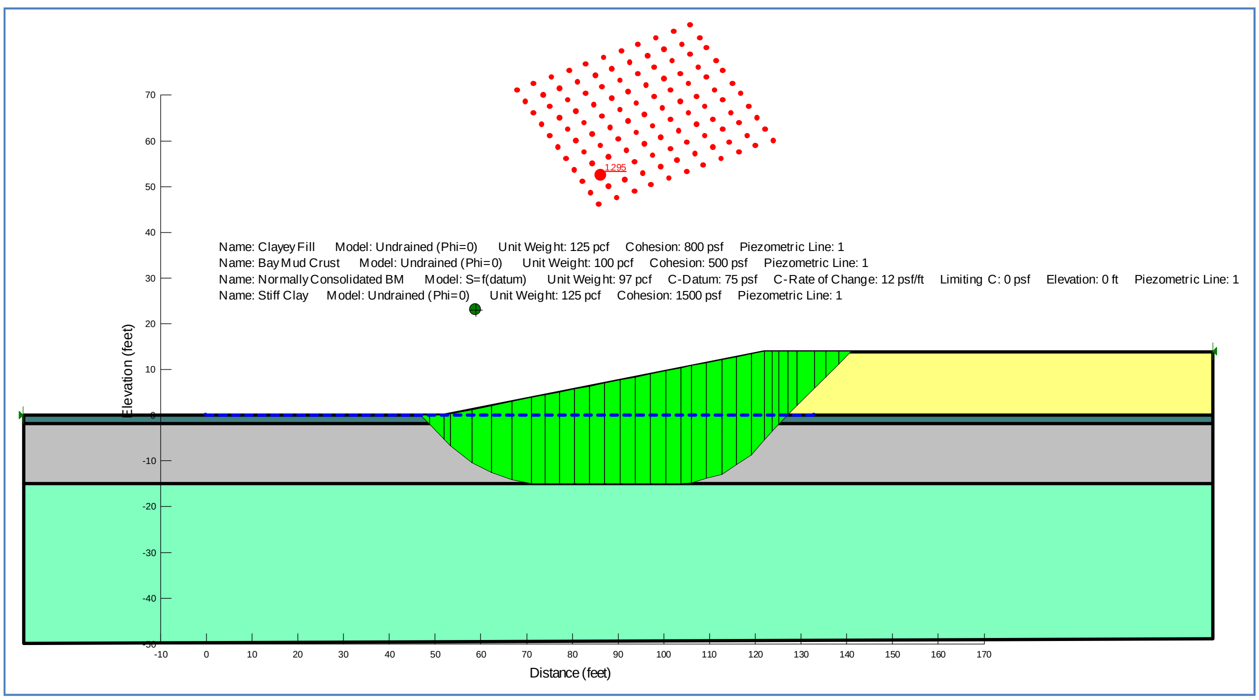
depth below existing mud line (ft)	depth below GW	u (psf)	Total unit weight	Total Stress	Initial Effective Stress (psf)	Layer thicknes	Cc (strain index)	Cr (strain index)	delta p (psf)	OCR	Pp	Primary Consolidation (ft)	Cumulative Consolidation (ft)
0	0	0	100	0	0	0	0.32	0.03	1800	2	0	#DIV/0!	0
1	1	62.4	100	100	37.6	1	0.32	0.03	1800	2	75.2	0.4532	0.4532
2	2	124.8	100	200	75.2	1	0.32	0.03	1800	2	150.4	0.3597	0.8129
3	3	187.2	97	297	109.8	1	0.32	0.03	1800	1	109.8	0.3969	1.2098
4	4	249.6	97	394	144.4	1	0.32	0.03	1800	1	144.4	0.3613	1.5712
5	**5**	**312**	**97**	**491**	**179**	**1**	**0.32**	**0.03**	**1800**	**1**	**179**	**0.3339**	**1.9051**
6	6	374.4	97	588	213.6	1	0.32	0.03	1800	1	213.6	0.3118	2.2169
7	7	436.8	97	685	248.2	1	0.32	0.03	1800	1	248.2	0.2933	2.5102
8	8	499.2	97	782	282.8	1	0.32	0.03	1800	1	282.8	0.2775	2.7877
9	9	561.6	97	879	317.4	1	0.32	0.03	1800	1	317.4	0.2637	3.0514
10	**10**	**624**	**97**	**976**	**352**	**1**	**0.32**	**0.03**	**1800**	**1**	**352**	**0.2516**	**3.3031**
11	11	686.4	97	1073	386.6	1	0.32	0.03	1800	1	386.6	0.2408	3.5439
12	12	748.8	97	1170	421.2	1	0.32	0.03	1800	1	421.2	0.2311	3.7749
13	13	811.2	97	1267	455.8	1	0.32	0.03	1800	1	455.8	0.2222	3.9972
14	14	873.6	97	1364	490.4	1	0.32	0.03	1800	1	490.4	0.2142	4.2114
15	**15**	**936**	**97**	**1461**	**525**	**1**	**0.32**	**0.03**	**1800**	**1**	**525**	**0.2068**	**4.4182**
16	16	998.4	97	1558	559.6	1	0.32	0.03	1800	1	559.6	0.2000	4.6182
17	17	1060.8	97	1655	594.2	1	0.32	0.03	1800	1	594.2	0.1937	4.8118
18	18	1123.2	97	1752	628.8	1	0.32	0.03	1800	1	628.8	0.1878	4.9996
19	19	1185.6	97	1849	663.4	1	0.32	0.03	1800	1	663.4	0.1823	5.1820
20	**20**	**1248**	**97**	**1946**	**698**	**1**	**0.32**	**0.03**	**1800**	**1**	**698**	**0.1772**	**5.3592**
21	21	1310.4	97	2043	732.6	1	0.32	0.03	1800	1	732.6	0.1724	5.5316
22	22	1372.8	97	2140	767.2	1	0.32	0.03	1800	1	767.2	0.1679	5.6994
23	23	1435.2	97	2237	801.8	1	0.32	0.03	1800	1	801.8	0.1636	5.8630
24	24	1497.6	97	2334	836.4	1	0.32	0.03	1800	1	836.4	0.1596	6.0225
25	**25**	**1560**	**97**	**2431**	**871**	**1**	**0.32**	**0.03**	**1800**	**1**	**871**	**0.1557**	**6.1783**
26	26	1622.4	97	2528	905.6	1	0.32	0.03	1800	1	905.6	0.1521	6.3304
27	27	1684.8	97	2625	940.2	1	0.32	0.03	1800	1	940.2	0.1487	6.4790
28	28	1747.2	97	2722	974.8	1	0.32	0.03	1800	1	974.8	0.1454	6.6244
29	29	1809.6	97	2819	1009.4	1	0.32	0.03	1800	1	1009.4	0.1423	6.7667
30	**30**	**1872**	**97**	**2916**	**1044**	**1**	**0.32**	**0.03**	**1800**	**1**	**1044**	**0.1393**	**6.9060**
31	31	1934.4	97	3013	1078.6	1	0.32	0.03	1800	1	1078.6	0.1364	7.0424
32	32	1996.8	97	3110	1113.2	1	0.32	0.03	1800	1	1113.2	0.1337	7.1761
33	33	2059.2	97	3207	1147.8	1	0.32	0.03	1800	1	1147.8	0.1311	7.3072
34	34	2121.6	97	3304	1182.4	1	0.32	0.03	1800	1	1182.4	0.1286	7.4357
35	**35**	**2184**	**97**	**3401**	**1217**	**1**	**0.32**	**0.03**	**1800**	**1**	**1217**	**0.1262**	**7.5619**
36	36	2246.4	97	3498	1251.6	1	0.32	0.03	1800	1	1251.6	0.1239	7.6858
37	37	2308.8	97	3595	1286.2	1	0.32	0.03	1800	1	1286.2	0.1216	7.8074
38	38	2371.2	97	3692	1320.8	1	0.32	0.03	1800	1	1320.8	0.1195	7.9269
39	39	2433.6	97	3789	1355.4	1	0.32	0.03	1800	1	1355.4	0.1174	8.0443
40	**40**	**2496**	**97**	**3886**	**1390**	**1**	**0.32**	**0.03**	**1800**	**1**	**1390**	**0.1154**	**8.1598**

depth below existing mud line (ft)	depth below GW	u (psf)	Total unit weight	Total Stress	Initial Effective Stress (psf)	Layer thicknes	Cc (strain index)	Cr (strain index)	delta p (psf)	OCR	Pp	Primary Consolidation (ft)	Cumulative Consolidation (ft)
0	0	0	100	0	0	0	0.32	0.03	2400	2	0	#DIV/0!	0
1	1	62.4	100	100	37.6	1	0.32	0.03	2400	2	75.2	0.4925	0.4925
2	2	124.8	100	200	75.2	1	0.32	0.03	2400	2	150.4	0.3983	0.8907
3	3	187.2	97	297	109.8	1	0.32	0.03	2400	1	109.8	0.4349	1.3256
4	4	249.6	97	394	144.4	1	0.32	0.03	2400	1	144.4	0.3987	1.7244
5	**5**	**312**	**97**	**491**	**179**	**1**	**0.32**	**0.03**	**2400**	**1**	**179**	**0.3708**	**2.0951**
6	6	374.4	97	588	213.6	1	0.32	0.03	2400	1	213.6	0.3480	2.4431
7	7	436.8	97	685	248.2	1	0.32	0.03	2400	1	248.2	0.3290	2.7722
8	8	499.2	97	782	282.8	1	0.32	0.03	2400	1	282.8	0.3127	3.0848
9	9	561.6	97	879	317.4	1	0.32	0.03	2400	1	317.4	0.2984	3.3832
10	**10**	**624**	**97**	**976**	**352**	**1**	**0.32**	**0.03**	**2400**	**1**	**352**	**0.2858**	**3.6690**
11	11	686.4	97	1073	386.6	1	0.32	0.03	2400	1	386.6	0.2745	3.9435
12	12	748.8	97	1170	421.2	1	0.32	0.03	2400	1	421.2	0.2643	4.2078
13	13	811.2	97	1267	455.8	1	0.32	0.03	2400	1	455.8	0.2550	4.4629
14	14	873.6	97	1364	490.4	1	0.32	0.03	2400	1	490.4	0.2465	4.7094
15	**15**	**936**	**97**	**1461**	**525**	**1**	**0.32**	**0.03**	**2400**	**1**	**525**	**0.2387**	**4.9481**
16	16	998.4	97	1558	559.6	1	0.32	0.03	2400	1	559.6	0.2315	5.1796
17	17	1060.8	97	1655	594.2	1	0.32	0.03	2400	1	594.2	0.2248	5.4043
18	18	1123.2	97	1752	628.8	1	0.32	0.03	2400	1	628.8	0.2185	5.6228
19	19	1185.6	97	1849	663.4	1	0.32	0.03	2400	1	663.4	0.2126	5.8354
20	**20**	**1248**	**97**	**1946**	**698**	**1**	**0.32**	**0.03**	**2400**	**1**	**698**	**0.2071**	**6.0425**
21	21	1310.4	97	2043	732.6	1	0.32	0.03	2400	1	732.6	0.2019	6.2445
22	22	1372.8	97	2140	767.2	1	0.32	0.03	2400	1	767.2	0.1970	6.4415
23	23	1435.2	97	2237	801.8	1	0.32	0.03	2400	1	801.8	0.1924	6.6339
24	24	1497.6	97	2334	836.4	1	0.32	0.03	2400	1	836.4	0.1880	6.8220
25	**25**	**1560**	**97**	**2431**	**871**	**1**	**0.32**	**0.03**	**2400**	**1**	**871**	**0.1839**	**7.0059**
26	26	1622.4	97	2528	905.6	1	0.32	0.03	2400	1	905.6	0.1799	7.1858
27	27	1684.8	97	2625	940.2	1	0.32	0.03	2400	1	940.2	0.1762	7.3620
28	28	1747.2	97	2722	974.8	1	0.32	0.03	2400	1	974.8	0.1726	7.5346
29	29	1809.6	97	2819	1009.4	1	0.32	0.03	2400	1	1009.4	0.1692	7.7037
30	**30**	**1872**	**97**	**2916**	**1044**	**1**	**0.32**	**0.03**	**2400**	**1**	**1044**	**0.1659**	**7.8696**
31	31	1934.4	97	3013	1078.6	1	0.32	0.03	2400	1	1078.6	0.1627	8.0324
32	32	1996.8	97	3110	1113.2	1	0.32	0.03	2400	1	1113.2	0.1597	8.1921
33	33	2059.2	97	3207	1147.8	1	0.32	0.03	2400	1	1147.8	0.1568	8.3489
34	34	2121.6	97	3304	1182.4	1	0.32	0.03	2400	1	1182.4	0.1541	8.5030
35	**35**	**2184**	**97**	**3401**	**1217**	**1**	**0.32**	**0.03**	**2400**	**1**	**1217**	**0.1514**	**8.6543**
36	36	2246.4	97	3498	1251.6	1	0.32	0.03	2400	1	1251.6	0.1488	8.8031
37	37	2308.8	97	3595	1286.2	1	0.32	0.03	2400	1	1286.2	0.1463	8.9495
38	38	2371.2	97	3692	1320.8	1	0.32	0.03	2400	1	1320.8	0.1439	9.0934
39	39	2433.6	97	3789	1355.4	1	0.32	0.03	2400	1	1355.4	0.1416	9.2350
40	**40**	**2496**	**97**	**3886**	**1390**	**1**	**0.32**	**0.03**	**2400**	**1**	**1390**	**0.1394**	**9.3744**

depth below existing mud line (ft)	depth below GW	u (psf)	Total unit weight	Total Stress	Initial Effective Stress (psf)	Layer thicknes	Cc (strain index)	Cr (strain index)	delta p (psf)	OCR	Pp	Primary Consolidation (ft)	Cumulative Consolidation (ft)
0	0	0	100	0	0	0	0.32	0.03	3000	2	0	#DIV/0!	0
1	1	62.4	100	100	37.6	1	0.32	0.03	3000	2	75.2	0.5231	0.5231
2	2	124.8	100	200	75.2	1	0.32	0.03	3000	2	150.4	0.4284	0.9515
3	3	187.2	97	297	109.8	1	0.32	0.03	3000	1	109.8	0.4647	1.4162
4	4	249.6	97	394	144.4	1	0.32	0.03	3000	1	144.4	0.4282	1.8443
5	**5**	**312**	**97**	**491**	**179**	**1**	**0.32**	**0.03**	**3000**	**1**	**179**	**0.3998**	**2.2441**
6	6	374.4	97	588	213.6	1	0.32	0.03	3000	1	213.6	0.3768	2.6209
7	7	436.8	97	685	248.2	1	0.32	0.03	3000	1	248.2	0.3574	2.9783
8	8	499.2	97	782	282.8	1	0.32	0.03	3000	1	282.8	0.3407	3.3190
9	9	561.6	97	879	317.4	1	0.32	0.03	3000	1	317.4	0.3261	3.6452



15 feet of BM, 3:1 Slope, 9 feet of fill



15 feet of BM, 5:1 slope, 14 feet of fill

Attachment B

CESPN-ET-EG

Date: 7 May 2012

Project: South San Francisco Bay Shoreline Feasibility Study

Subject: Geotechnical Review of Proposed Import Project by FWLS in Relation to Shoreline Feasibility Study Conceptual Alternatives

Summary:

We understand the United States Fish and Wildlife Service (FWLS) will be provided free import soil to Shoreline Ponds for use in pond levee maintenance and possibly for use as levee fill for future levee construction that may occur as part of the Army Corps of Engineer's Southbay Shoreline project and is planning a project to stockpile this material on-site.

The Geo-Sciences Section of the Army Corps of Engineers San Francisco District (SPN) was provided with the following documents for review:

- An April 6, 2012 technical memorandum prepared by Cornerstone Earth Group, with the subject "Ravenswood, Mountain View, and Alviso Fill Evaluation."
- 6 Plan Sheets dated April 2012, prepared by MacKay and Somps titled "Stock Pile Plan Dirt Import Project – Phase 1, Mountain View, California."
- 4 Plan Sheets dated April2002 prepared by MacKay and Somps titled "Stockpile Plan Dirt Import Project – Phase 1, Menlo Park, California."
- 4 Plan Sheets dated April2002 prepared by MacKay and Somps titled "Stockpile Plan Dirt Import Project – Phase 1, Alviso, California."

Based on review of these documents, SPN has the following comments regarding the proposed stockpile plan as described in the documents above.

Comment 1: In general, the recommendations provided by the geotechnical consultant for the project are labeled as "conceptual". Typically construction drawings are not developed using conceptual recommendations as there are often many details and uncertainties in conceptual design that are not fully developed for construction drawings. It is recommended that the geotechnical engineer provide additional exploration, lab testing and engineering analysis as necessary to support construction documents. Of particular consideration should be effects of new fill on existing infrastructure due to settlement, changes in levee crest elevation, and other factors. The Corps has provided the FWLS copies of subsurface exploration used in the feasibility analysis, this should be available to the geotechnical consultant for review. A particular discrepancy noted in the consultant geotechnical recommendations and our subsurface interpretation, is that the Corps has interpreted thicker Bay Mud deposits near Alviso as the proposed temporary fill extends toward the bay. This could impact the consultant's stability and settlement estimates.

Comment 2: The proposed fill specifications in the geotechnical recommendations appear suitable for general fill, however to be used for levee fill, we have proposed more stringent fill specifications, although there may be some room for flexibility as Corps alternative designs are in concept only at this time. In general, to date, the Corps has proposed levee fill having a PI of 10 to 20, with non-dispersive behavior, generally clayey soil (CL, SC, GC), with low permeability (<10^{-6} cm/sec). Of particular concern are the performance of fill with high plasticity and liquid limits for new levee construction. It appears that the proposed fill specifications allow for MH, CH and ML soils for import, that may not meet final design specifications for levee fill. For general use to construct environmental ecology features, the environmental designers should verify that the proposed import fill will support the required habitat species.

Comment 3: Fill imported to the site should be tested and placed so that material properties of the import fill are generally geographically known (i.e. if the project needs levee fill where can we find it?).

Comment 4: Because the fill will settle, some fill imported may not be practical to reclaim for new project purposes (i.e. it settles too much below the water level). Therefore the best fill material (meeting Corps proposed levee fill specifications) should be placed on shallower Bay Mud areas to minimize loss to settlement.

Comment 5: The proposed fill elevations in some locations are higher than existing levee crest elevations. The H&H team should review proposed fill geometry and effects the proposed filling may have on the project hydraulic performance. Note that fill settlement time-rate estimates may be required such that appropriate engineering judgments can be made.

Comment 6: FWLS should note that the Southbay Shoreline Feasibility Study has not finalized or recommended design alternatives at this point, and that all, some or none of the imported fill may be useful to the project. By thoroughly testing engineering properties and documenting fill placement locations, the potential for project beneficial re-use will be maximized.

This review does not constitute Corps approval or responsibility of performance of the proposed temporary fill stockpiles, which are the responsibility of the USFWLS and their retained consultants and contractors. If the USFWLS requires technical assistance from the Corps, Corps technical assistance can be provided under our Memorandum of Understanding with the USFWLS.

This discussion has been prepared by:

Brian Hubel, P.E., G.E.

Geotechnical Engineer

Corps of Engineers

San Francisco District Geo-Sciences Section

415-503-922

Attachment C

CESPN-ET-EG

18 April 2012

Project: San Francisco South Bay Shoreline Study

Subject: Levee Breach Dimensions

Background:

Previous levee failure and flooding analysis, performed by the Corps of Engineers Engineering Research and Development Center (ERDC) had determined levee breach dimensions based on the shear stress the water applies to the soil. The breach is extended until the shear stress applied is balanced by the bulk erosive strength of the soil which was assumed to be about 36 Pa (0.7psf). This methodology is discussed in Section 3 of the draft report titled, “Coastal Flooding Uncertainty Analysis for South San Francisco Bay Shoreline Study: without Project Conditions,” dated 24 January 2011 ***(1)***. Currently, the previous modeling performed by ERDC is undergoing validation by retained engineering contractors who will also be performing with-project modeling. SPN-Geosciences has been requested to provide estimates of breach dimensions for this modeling effort.

The estimates presented in this memo should be considered coarse approximations, as breach development and resulting dimensions is a complex process. Most predictive methods rely heavily on empirical observations and relationships. The modeling presented in the ERDC report has basis in more physical principals. The resulting dimensions of the breaches are not reported or easily checked in the referenced document. The recommendations presented in this memo are based on empirical observations.

Methodology:

Estimating breach dimensions is difficult and depends on a number of factors including the water head over the “weir”(bar), geotechnical properties of the levee soil, dimensions of the levee, hydraulic loading type (river, coastal, etc.), protected side topographic conditions, flood fight activities, and time. Our assumptions are based on a maximum breach developing (unlimited time) without flood fight, for water at the top of the levee, and that the entire levee cross section is washed away with the exception of a small bar at the water side toe of the levee. Figure 1. shows the typical shape of a levee breach.

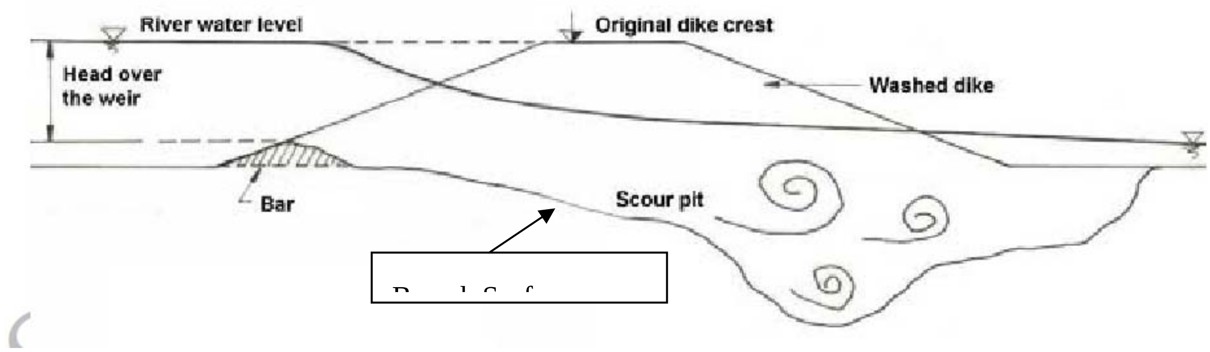


Figure 5-2. Typical Cross Section of a Dike Failure *(2)*

For most failures the "bar" on the water side could be on the order of 40cm ***(2).*** For this analysis we assumed that the top of the bar would be the same elevation as the toe of the protected side of the levee. Numerous research papers have been published regarding the development of dam breaches, which although have some similarities to levees, some important differences are also present. These differences include a water supply limited by the reservoir storage, and that embankments are generally constructed at narrow canyon constrictions rather than as long continuous structures.

Table 1 summarizes some recent levee breach dimensions reported in a 2009 report published by the Southeast Region Research Initiative (SERRI) Project ***(3)***. As can be seen in the table geometries are quite variable, as should be expected for various geometry, load, and geotechnical conditions.

Table 1. Summary of Recent US Levee Breach Geometry *(3)*

Levee Breach	Load Type	Water side Slopes (H:V)	Crest Width (ft)	Protected Side Slopes (H:V)	Levee Height (ft)	Water Height (ft)	Breach Length (ft)	Scour Depth (ft)
Feather River near Arboga, CA (1997)	River	2:1	20	3:1	29	25	623	56
Pin Oak Levee on Mississippi River near Winfield, MO (2008)	River	3:1	10	3:1	12	11	150	--
Truckee Irrigation Canal Levee, near Fearnly, NV (2008)	River	2:1	15	1.5:1	9.5	6.5	50	11
Jones Tract Levee on Middle River near Stockton, CA (2004)	River	3:1	28	3:1	16	9	344	--
Russell-Allison Levee on Wabash River near Westport, IL (2008)	River	3:1	10	3:1	8	5	173	--
Cap au Gris Levee on Mississippi River near Windfield, MO (2008)	River	3:1	10	3:1	9	11	351	15
Floodwall on Metairie Outfall Canal, New Orleans, LA (2005)	Hurricane	3:1	10	3:1	19	17	449	21
Floodwall on London Avenue Canal, New Orleans, LA (2005)	Hurricane	3:1	10	3:1	17	13	125	5

Floodwall on Inner Harbor Navigation Canal, New Orleans, LA (2005)	Hurricane	3:1	10	3:1	20	22	919	--

Several breaches have been purposely constructed in the area for restoration of Pond A6 and at Redwood Creek. The stabilized breach lengths at A6 are 106, 82,138, and 74 feet and at Redwood Creek lengths are measured from air photos at 184, 215, 85, and 147 feet. It is presumed that constructed widths were smaller and allowed to progress to these widths. These dimensions are relatively consistent with dimensions lengths that would be calculated using Nagy(2006) relationships for 7 to 12 feet of water height.

Nagy (2006) reported on over 2200 dike failures in the Carpathian-Basin (Hungary) from about 1800 to present. Of those case histories more than 1000 failures have known levee breach lengths. Although the statistical fit is loose (R^2=.39), Nagy correlated the fully developed breach length to the water height above the water side "bar" by the equation:

$$y=5.1899e^{0.7498x}$$

where:

x=water height in meters

y=breach length in meters

Recommendations:

Using the equation above, Table 2 presents estimates of fully developed breach lengths water heights of 6.5 to 20 feet. Once a breach begins it is anticipated that the entire levee section will be quickly lost, with the "bar" weir crest elevation likely similar to the elevation of the protected side toe of the levee. The water height can be taken as the difference between the water loading elevation and the toe elevation on the protected side of the levee. The depth of the scour channel is quite difficult to estimate, and varies widely in case histories. Initially we recommend a scour depth(depth of erosion below bottom of levee) of about 2 times the water height. If there is high sensitivity to this parameter, additional research may be warranted. The slopes of the levee breach may be taken as vertical for the purposes of this modeling. Breach geometry is highly uncertain and could contribute significantly to modeling uncertainty.

Table 2. Estimated Breach Lengths using Nagy (2006)

Approximate Water Height (ft)	Estimated Fully Developed Breach Length (ft)
6.5	75
10	160
13	340
16	725
20	1530

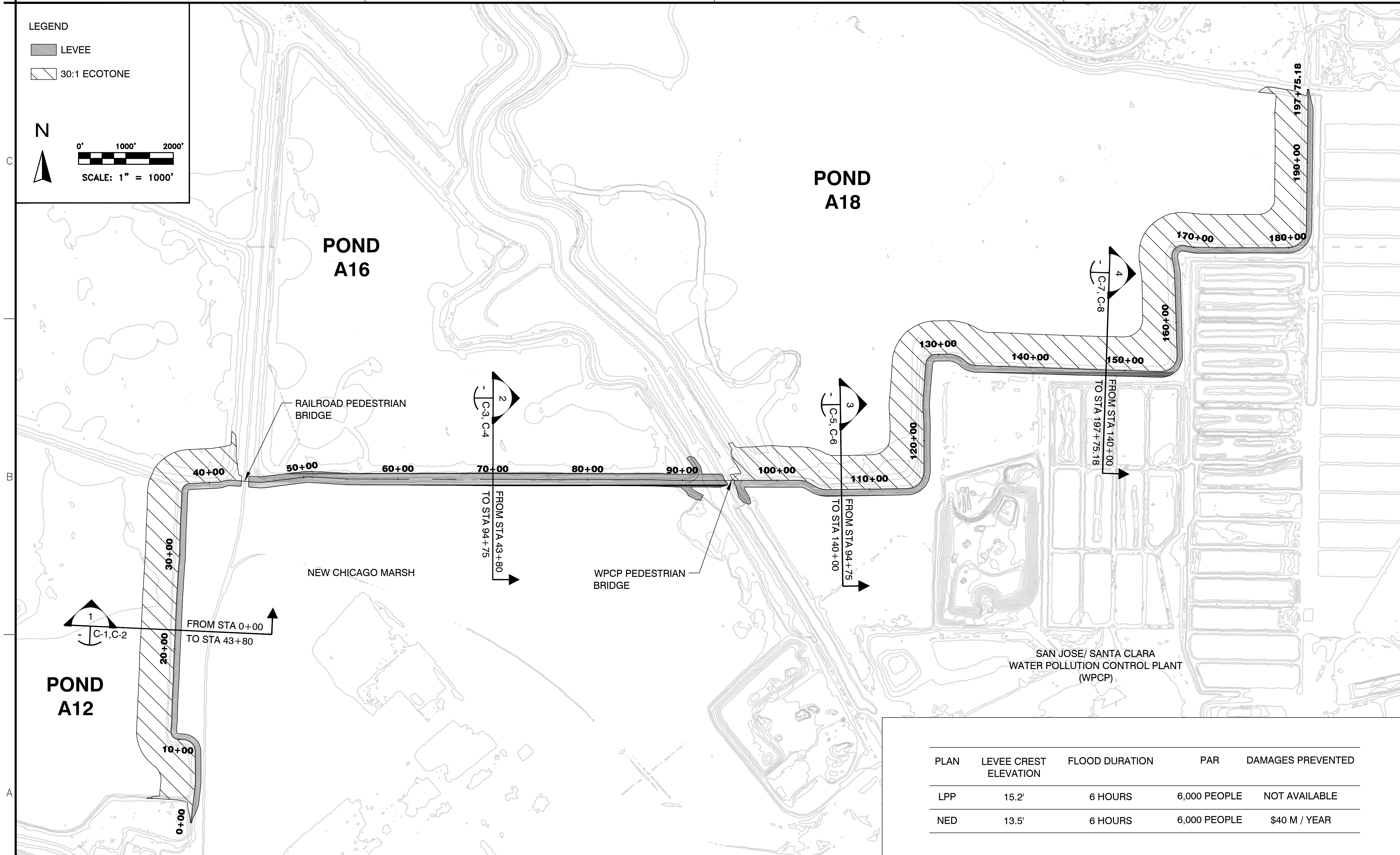
References:

(1) Letter, J.V., (2011). Draft report, "Coastal Flooding Uncertainty Analysis for South San Francsico Bay Shoreline Study: without Project Conditions," ERDC/CHL US Army Corps of Engineers.

(2) Nagy, L. (2006). "Estimating Dike Breach Length from Historical Data," Periodica Polytechnica, Serial Civil Engineering, Vol. 50, No. 2, pp. 125-139.

(3) Saucuer, C. L.; Howard, I.L.; and Tom, J. G., (2009) SERRI Report 70015-001, "Levee Breach Geometries and Algorithms to Simulate Breach Closure" prepared for US Department of Homeland Security under Department of Energy Interagency Agreement 43WT10301.

Attachment D



PLAN	LEVEE CREST ELEVATION	FLOOD DURATION	PAR	DAMAGES PREVENTED
LPP	15.2'	6 HOURS	6,000 PEOPLE	NOT AVAILABLE
NED	13.5'	6 HOURS	6,000 PEOPLE	$40 M / YEAR

SHORELINE PROJECT DATUMS

HORIZONTAL: CALIFORNIA COORDINATE SYSTEM NAD83 ZONE III
VERTICAL: NAVD88

PLAN VIEW

SCALE: 1"= 1,000'

US Army Corps of Engineers San Francisco District

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

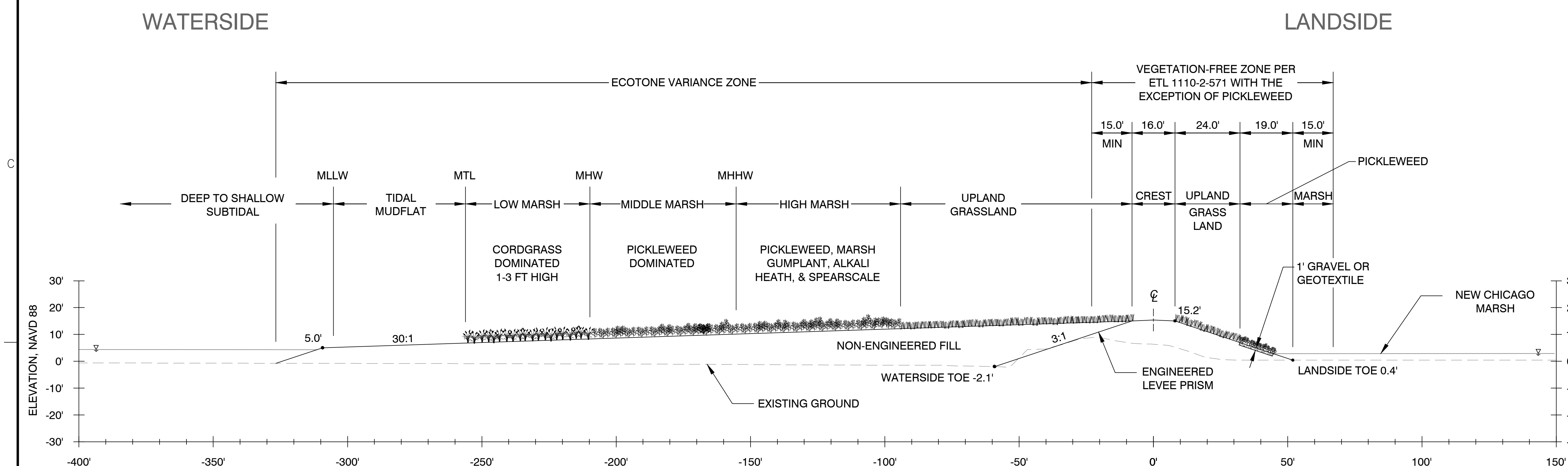
US Army Corps of Engineers
San Francisco District
1455 Market Street
San Francisco, Ca 94103

Prepared by: M. Young
Dwn by: M. Young
Spec No. 1 of 9
Reviewed by: W. DeJager
Submitted by: P. Schimelfenyg
Date: 05/01/2014
Design file no: X-XX-XXXX
Drawing Code: XXXX
File name:
Plot date:
Dwg scale:
Rev:

SAN FRANCISCO CALIFORNIA
SAN FRANCISCO BAY SHORELINE
TYPICAL CROSS SECTIONS
PLAN VIEW

Sheet reference number: G-1

DRAFT



GRAVEL OR GEOTEXTILE ALTERNATIVE
TYPICAL CROSS SECTION FROM STA 0+00 TO STA 43+80

1 / - C-1

SCALE: 1"=40' (H), 1"= 40' (V)

WATERSIDE NOTES:

1. PLANTS IN THE LOW MARSH AND MIDDLE MARSH AREAS WILL ESTABLISH ON THEIR OWN. LOW MARSH WILL CONSIST OF CORDGRASS (A TALL SPECIES OF GRASS) WHILE THE MIDDLE MARSH WILL PRIMARILY CONSIST OF PERENNIAL PICKLEWEED, A LOW SUCCULENT AND SLIGHTLY WOODY PLANT THAT GROWS TO AN AVERAGE HEIGHT OF 1 FOOT. THE LOW AND MIDDLE MARSH AREAS WOULD BE LARGELY UNMANAGED WITH NO MOWING.

2. THE HIGH MARSH AND UPLAND GRASSLAND WILL BE PLANTED. HIGH MARSH PLANTS ARE MOSTLY 1-2 FEET TALL WITH OCCASIONAL STEMS REACHING UP TO 5 FEET TALL. PLANTS IN THESE AREAS ARE SOFT TO SEMI WOODY. THE HIGH MARSH AND UPLAND GRASSLAND AREAS WOULD ALSO BE LARGELY UNMANAGED WITH NO MOWING.

3. THE 15 FEET OF ECOTONE CLOSEST TO THE FLOOD RISK MANAGEMENT LEVEE, ALONG WITH THE REST OF THE FRM LEVEE CROSS-SECTION, WOULD BE MANAGED AS A VEGETATION-FREE ZONE PER ETL 1110-2-571. SEE SECTION FOR DETAILS.

SHORELINE PROJECT DATUMS

HORIZONTAL: CALIFORNIA COORDINATE SYSTEM NAD83 ZONE III
VERTICAL: NAVD88

LANDSIDE NOTES:

1. THE LANDSIDE OF THE LEVEE AND THE ADJACENT 15 FOOT OFFSET ARE INTENDED TO BE MANAGED AS PART OF THE VEGETATION-FREE ZONE. HOWEVER, THE LOWER LANDSIDE SLOPE CAN BE EXPECTED TO DEVELOP GROWTH OF PICKLEWEED AND OTHER HIGH MARSH PLANTS DUE TO THE PRESENCE OF SALT AND SEASONAL WATER IN THE AREA. THE GROWTH OF HIGH MARSH PLANTS CAN BE EXPECTED THROUGHOUT THE ENTIRE LENGTH OF THE LEVEE. PICKLEWEED WILL NOT USUALLY GROW BEYOND THE TOE OF THE LEVEE DUE TO PERMANENT AND SEASONAL PONDED WATER. THERE WILL BE VERY MINOR LOCATIONS WHERE PICKLEWEED DOES GROW NEXT TO THE LEVEE DUE TO HIGHER GROUND.

2. ACCESS ALONG THE LAND SIDE TOE WILL NOT BE EASY DUE TO THE MARSH. THERE WILL BE VERY MINOR LOCATIONS WHERE PICKLEWEED DOES GROW NEXT TO THE LEVEE DUE TO HIGHER GROUND.

3. WE WILL GENERALLY ONLY NEED TO ADDRESS THE EXISTENCE OF PICKLEWEED ON THE LAND SIDE SLOPE DUE TO THE ECOTONE.

4. NATURAL PICKLEWEED IN TIDAL AREAS TYPICALLY GROW TO A MAXIMUM HEIGHT OF 12 TO 18 INCHES (OCCASIONALLY UP TO 24 INCHES) BETWEEN 0.0 AND 3.0 FT ABOVE THE WATER SURFACE ELEVATION. OUTSIDE OF TIDAL AREAS, CONDITIONS AT THE SHORELINE SITE ARE USUALLY NOT IDEAL; HEIGHTS WILL RANGE FROM 6 INCHES TO 24 INCHES DEPENDING ON SOIL SALINITY AND WATER AVAILABILITY. THE PICKLEWEED COULD BE STUNTED BY EXTREME SALINITY CONDITIONS (HIGH OR LOW), PROLONGED INUNDATION, OR SEVER LACK OF WATER. THE MOST RELIABLE METHOD WOULD BE TO APPLY A LAYER OF BAY MUD TO THE SURFACE OF THE LEVEE TO CREATE A COMBINATION OF HIGH SALINITY AND DRY CONDITIONS. THE ABILITY OF OTHER METHODS TO STUNT PICKLEWEED IS UNCERTAIN.

5. THE PROJECT DELIVERY TEAM (PDT) RECOMMENDS A MINIMUM LEVEE PRISM WHICH WOULD BE CONSTRUCTED PRIMARILY OF BAY MUD AS SHOWN. HOWEVER, IF THE PICKLEWEED HEIGHT NEEDS TO BE FURTHER REDUCED FOR LEVEE SAFETY ON THE LANDSIDE, AN UNDERLYING LAYER OF GRAVEL OR AN UNDERLYING GEOTEXTILE COULD BE ADDED, AS ALSO SHOWN. THE PDT DOES NOT RECOMMEND ADDING A PLANTING BERM WITH NATURAL PICKLEWEED, AS SHOWN ON SHEET C-2, BECAUSE IT WILL REDUCE THE AREA OF THE MARSH, POSSIBLY RAISE ADDITIONAL ENVIRONMENTAL ISSUES, AND IS JUDGED TO BE THE MOST EXPENSIVE SOLUTION.

6. ELEVATION OF PICKLEWEED ABOVE THE LEVEE BASE IN NON-TIDAL AREAS WILL DEPEND ON THE SOIL SOURCE USED FOR THE LEVEE FACE. REUSED BAY MUD WILL ENCOURAGE PICKLEWEED AND DISCOURAGE GRASS. UPLAND SOIL WILL GENERALLY GROW GRASS UNLESS SOIL SALINIZATION OCCURS FROM ADJACENT WATERS.

US Army Corps of Engineers
San Francisco District

Date: 05/01/2014
Design file no: X-XX-XXXX
Drawing Code: XXXX
Spec No. 2 of 9
Prepared by: M. Young
Dwn by: M. Young
Reviewed by: W. DeJager
Submitted by: P. Schmelfenyg

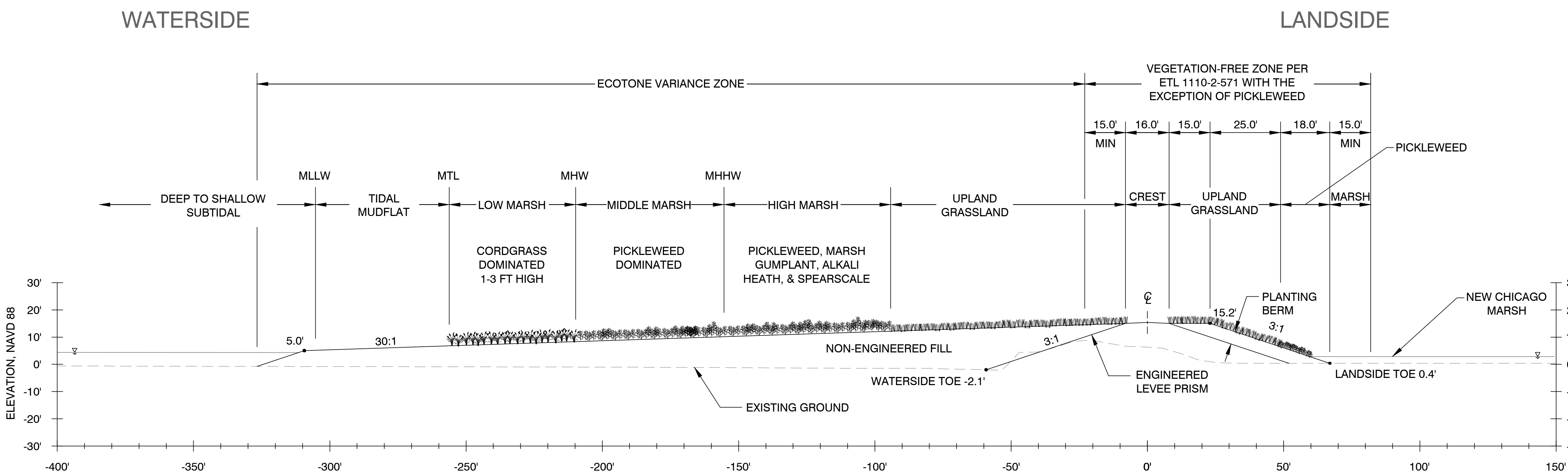
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

SAN FRANCISCO CALIFORNIA
SAN FRANCISCO BAY SHORELINE
TYPICAL CROSS SECTIONS
GRAVEL OR GEOTEXTILE ALTERNATIVE
STA 0+00 TO STA 43+80

Sheet reference number:
C-1

DRAFT

UPDATE



PLANTING BERM ALTERNATIVE
TYPICAL CROSS SECTION FROM STA 0+00 TO STA 43+80

1 - C-2

SCALE: 1"=40' (H), 1"= 40' (V)

WATERSIDE NOTES:

1. PLANTS IN THE LOW MARSH AND MIDDLE MARSH AREAS WILL ESTABLISH ON THEIR OWN. LOW MARSH WILL CONSIST OF CORDGRASS (A TALL SPECIES OF GRASS) WHILE THE MIDDLE MARSH WILL PRIMARILY CONSIST OF PERENNIAL PICKLEWEED, A LOW SUCCULENT AND SLIGHTLY WOODY PLANT THAT GROWS TO AN AVERAGE HEIGHT OF 1 FOOT. THE LOW AND MIDDLE MARSH AREAS WOULD BE LARGELY UNMANAGED WITH NO MOWING.

2. THE HIGH MARSH AND UPLAND GRASSLAND WILL BE PLANTED. HIGH MARSH PLANTS ARE MOSTLY 1-2 FEET TALL WITH OCCASIONAL STEMS REACHING UP TO 5 FEET TALL. PLANTS IN THESE AREAS ARE SOFT TO SEMI WOODY. THE HIGH MARSH AND UPLAND GRASSLAND AREAS WOULD ALSO BE LARGELY UNMANAGED WITH NO MOWING.

3. THE 15 FEET OF ECOTONE CLOSEST TO THE FLOOD RISK MANAGEMENT LEVEE, ALONG WITH THE REST OF THE FRM LEVEE CROSS-SECTION, WOULD BE MANAGED AS A VEGETATION-FREE ZONE PER ETL 1110-2-571. SEE SECTION FOR DETAILS.

SHORELINE PROJECT DATUMS

HORIZONTAL: CALIFORNIA COORDINATE SYSTEM NAD83 ZONE III
VERTICAL: NAVD88

LANDSIDE NOTES:

1. THE LANDSIDE OF THE LEVEE AND THE ADJACENT 15 FOOT OFFSET ARE INTENDED TO BE MANAGED AS PART OF THE VEGETATION-FREE ZONE. HOWEVER, THE LOWER LANDSIDE SLOPE CAN BE EXPECTED TO DEVELOP GROWTH OF PICKLEWEED AND OTHER HIGH MARSH PLANTS DUE TO THE PRESENCE OF SALT AND SEASONAL WATER IN THE AREA. THE GROWTH OF HIGH MARSH PLANTS CAN BE EXPECTED THROUGHOUT THE ENTIRE LENGTH OF THE LEVEE. PICKLEWEED WILL NOT USUALLY GROW BEYOND THE TOE OF THE LEVEE DUE TO PERMANENT AND SEASONAL PONDED WATER. THERE WILL BE VERY MINOR LOCATIONS WHERE PICKLEWEED DOES GROW NEXT TO THE LEVEE DUE TO HIGHER GROUND.

2. ACCESS ALONG THE LAND SIDE TOE WILL NOT BE EASY DUE TO THE MARSH. THERE WILL BE VERY MINOR LOCATIONS WHERE PICKLEWEED DOES GROW NEXT TO THE LEVEE DUE TO HIGHER GROUND.

3. WE WILL GENERALLY ONLY NEED TO ADDRESS THE EXISTENCE OF PICKLEWEED ON THE LAND SIDE SLOPE DUE TO THE ECOTONE.

4. NATURAL PICKLEWEED IN TIDAL AREAS TYPICALLY GROW TO A MAXIMUM HEIGHT OF 12 TO 18 INCHES (OCCASIONALLY UP TO 24 INCHES) BETWEEN 0.0 AND 3.0 FT ABOVE THE WATER SURFACE ELEVATION. OUTSIDE OF TIDAL AREAS, CONDITIONS AT THE SHORELINE SITE ARE USUALLY NOT IDEAL; HEIGHTS WILL RANGE FROM 6 INCHES TO 24 INCHES DEPENDING ON SOIL SALINITY AND WATER AVAILABILITY. THE PICKLEWEED COULD BE STUNTED BY EXTREME SALINITY CONDITIONS (HIGH OR LOW), PROLONGED INUNDATION, OR SEVER LACK OF WATER. THE MOST RELIABLE METHOD WOULD BE TO APPLY A LAYER OF BAY MUD TO THE SURFACE OF THE LEVEE TO CREATE A COMBINATION OF HIGH SALINITY AND DRY CONDITIONS. THE ABILITY OF OTHER METHODS TO STUNT PICKLEWEED IS UNCERTAIN.

5. THE PROJECT DELIVERY TEAM (PDT) RECOMMENDS A MINIMUM LEVEE PRISM WHICH WOULD BE CONSTRUCTED PRIMARILY OF BAY MUD AS SHOWN ON SHEET C-1. HOWEVER, IF THE PICKLEWEED HEIGHT NEEDS TO BE FURTHER REDUCED FOR LEVEE SAFETY ON THE LANDSIDE, AN UNDERLYING LAYER OF GRAVEL OR AN UNDERLYING GEOTEXTILE COULD BE ADDED, AS ALSO SHOWN ON SHEET C-1. THE PDT DOES NOT RECOMMEND ADDING A PLANTING BERM WITH NATURAL PICKLEWEED, AS SHOWN, BECAUSE IT WILL REDUCE THE AREA OF THE MARSH, POSSIBLY RAISE ADDITIONAL ENVIRONMENTAL ISSUES, AND IS JUDGED TO BE THE MOST EXPENSIVE SOLUTION.

6. ELEVATION OF PICKLEWEED ABOVE THE LEVEE BASE IN NON-TIDAL AREAS WILL DEPEND ON THE SOIL SOURCE USED FOR THE LEVEE FACE. REUSED BAY MUD WILL ENCOURAGE PICKLEWEED AND DISCOURAGE GRASS. UPLAND SOIL WILL GENERALLY GROW GRASS UNLESS SOIL SALINIZATION OCCURS FROM ADJACENT WATERS.

US Army Corps of Engineers
San Francisco District

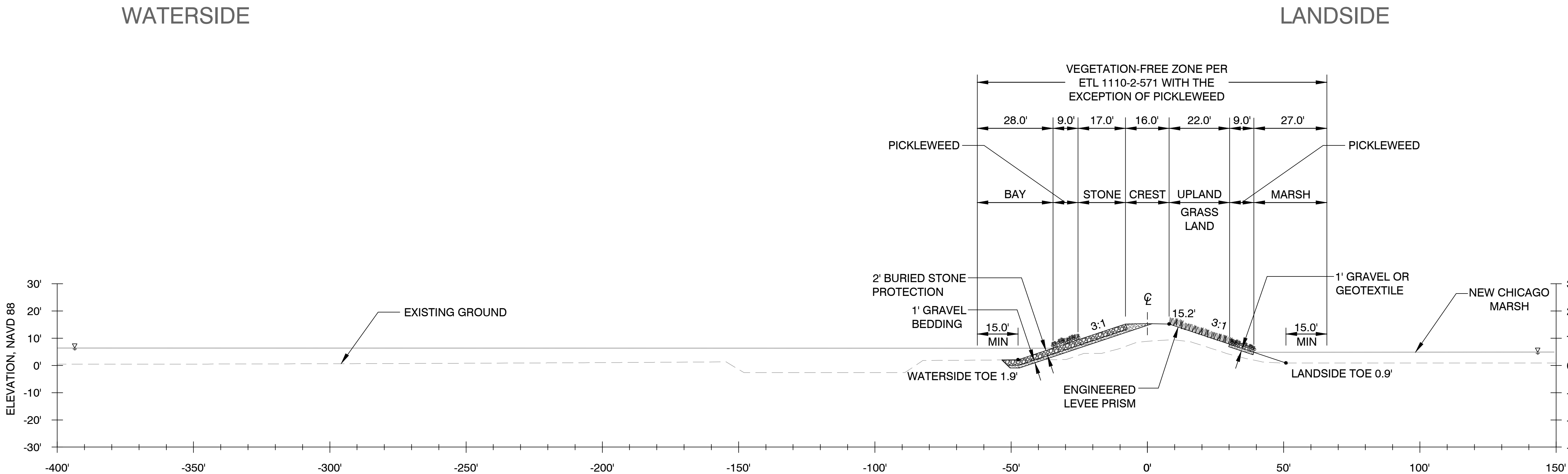
Date: 05/01/2014
Design file no: X-XX-XXXX
Drawing Code: XXXX
Prepared by: M. Young
Dwn by: M. Young
Spec No. 3 of 9
Reviewed by: W. DeJager
Submitted by: P. Schmidtenyg

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

SAN FRANCISCO BAY SHORELINE
TYPICAL CROSS SECTIONS
PLANTING BERM ALTERNATIVE
STA 0+00 TO STA 43+80

Sheet reference number: C-2

DRAFT



GRAVEL OR GEOTEXTILE ALTERNATIVE TYPICAL CROSS SECTION FROM STA 43+80 TO STA 94+75

2 - C-3

SCALE: 1"=40' (H), 1"= 40' (V)

WATERSIDE NOTES:

1. THE BURIED STONE IS NEEDED TO PROTECT AGAINST EROSION. THE SOIL PLACED IN THE INTERSTICES WILL ALLOW SMALLER ANIMALS TO TRAVEL UP AND DOWN THE SLOPES. PICKLEWEED WILL ESTABLISH ITSELF IN THE LOWER PORTION OF THE BURIED STONE. AN ADDITIONAL ECOTONE COULD ALSO BE CONSIDERED IN THIS REACH.

SHORELINE PROJECT DATUMS

HORIZONTAL: CALIFORNIA COORDINATE SYSTEM NAD83 ZONE III
VERTICAL: NAVD88

LANDSIDE NOTES:

1. THE LANDSIDE OF THE LEVEE AND THE ADJACENT 15 FOOT OFFSET ARE INTENDED TO BE MANAGED AS PART OF THE VEGETATION-FREE ZONE. HOWEVER, THE LOWER LANDSIDE SLOPE CAN BE EXPECTED TO DEVELOP GROWTH OF PICKLEWEED AND OTHER HIGH MARSH PLANTS DUE TO THE PRESENCE OF SALT AND SEASONAL WATER IN THE AREA. THE GROWTH OF HIGH MARSH PLANTS CAN BE EXPECTED THROUGHOUT THE ENTIRE LENGTH OF THE LEVEE. PICKLEWEED WILL NOT USUALLY GROW BEYOND THE TOE OF THE LEVEE DUE TO PERMANENT AND SEASONAL PONDED WATER. THERE WILL BE VERY MINOR LOCATIONS WHERE PICKLEWEED DOES GROW NEXT TO THE LEVEE DUE TO HIGHER GROUND.

2. ACCESS ALONG THE LAND SIDE TOE WILL NOT BE EASY DUE TO THE MARSH. THERE WILL BE VERY MINOR LOCATIONS WHERE PICKLEWEED DOES GROW NEXT TO THE LEVEE DUE TO HIGHER GROUND.

3. WE WILL GENERALLY NEED TO ADDRESS THE EXISTENCE OF PICKLEWEED ON BOTH THE LAND AND WATER SIDE SLOPES.

4. NATURAL PICKLEWEED IN TIDAL AREAS TYPICALLY GROW TO A MAXIMUM HEIGHT OF 12 TO 18 INCHES (OCCASIONALLY UP TO 24 INCHES) BETWEEN 0.0 AND 3.0 FT ABOVE THE WATER SURFACE ELEVATION. OUTSIDE OF TIDAL AREAS, CONDITIONS AT THE SHORELINE SITE ARE USUALLY NOT IDEAL; HEIGHTS WILL RANGE FROM 6 INCHES TO 24 INCHES DEPENDING ON SOIL SALINITY AND WATER AVAILABILITY. THE PICKLEWEED COULD BE STUNTED BY EXTREME SALINITY CONDITIONS (HIGH OR LOW), PROLONGED INUNDATION, OR SEVER LACK OF WATER. THE MOST RELIABLE METHOD WOULD BE TO APPLY A LAYER OF BAY MUD TO THE SURFACE OF THE LEVEE TO CREATE A COMBINATION OF HIGH SALINITY AND DRY CONDITIONS. THE ABILITY OF OTHER METHODS TO STUNT PICKLEWEED IS UNCERTAIN.

5. THE PROJECT DELIVERY TEAM (PDT) RECOMMENDS A MINIMUM LEVEE PRISM WHICH WOULD BE CONSTRUCTED PRIMARILY OF BAY MUD AS SHOWN. HOWEVER, IF THE PICKLEWEED HEIGHT NEEDS TO BE FURTHER REDUCED FOR LEVEE SAFETY ON THE LANDSIDE, AN UNDERLYING LAYER OF GRAVEL OR AN UNDERLYING GEOTEXTILE COULD BE ADDED, AS ALSO SHOWN. THE PDT DOES NOT RECOMMEND ADDING A PLANTING BERM WITH NATURAL PICKLEWEED, AS SHOWN ON SHEET C-4, BECAUSE IT WILL REDUCE THE AREA OF THE MARSH, POSSIBLY RAISE ADDITIONAL ENVIRONMENTAL ISSUES, AND IS JUDGED TO BE THE MOST EXPENSIVE SOLUTION.

6. ELEVATION OF PICKLEWEED ABOVE THE LEVEE BASE IN NON-TIDAL AREAS WILL DEPEND ON THE SOIL SOURCE USED FOR THE LEVEE FACE. REUSED BAY MUD WILL ENCOURAGE PICKLEWEED AND DISCOURAGE GRASS. UPLAND SOIL WILL GENERALLY GROW GRASS UNLESS SOIL SALINIZATION OCCURS FROM ADJACENT WATERS.

US Army Corps of Engineers
San Francisco District

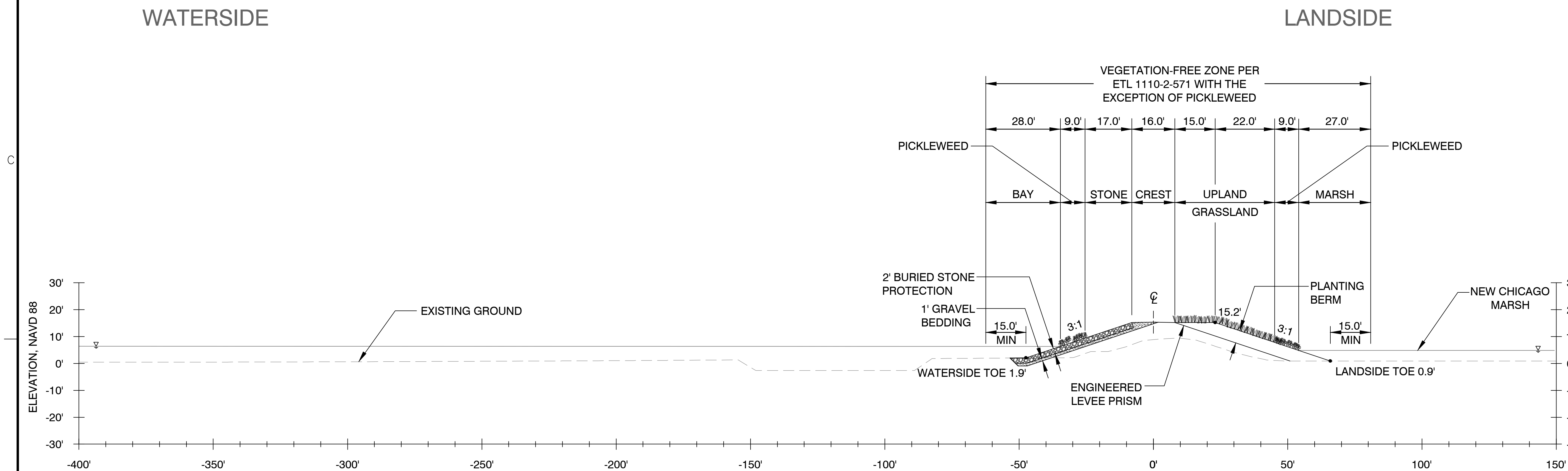
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

SAN FRANCISCO BAY SHORELINE
TYPICAL CROSS SECTIONS
GRAVEL OR GEOTEXTILE ALTERNATIVE
STA 43+8 TO STA 94+75

Sheet reference number:
C-3

DRAFT

UPDATE



PLANTING BERM ALTERNATIVE
TYPICAL CROSS SECTION FROM STA 43+80 TO STA 94+75

2 - C-4

SCALE: 1"=40' (H), 1"= 40' (V)

WATERSIDE NOTES:

1. THE BURIED STONE IS NEEDED TO PROTECT AGAINST EROSION. THE SOIL PLACED IN THE INTERSTICES WILL ALLOW SMALLER ANIMALS TO TRAVEL UP AND DOWN THE SLOPES. PICKLEWEED WILL ESTABLISH ITSELF IN THE LOWER PORTION OF THE BURIED STONE. AN ADDITIONAL ECOTONE COULD ALSO BE CONSIDERED IN THIS REACH.

LANDSIDE NOTES:

1. THE LANDSIDE OF THE LEVEE AND THE ADJACENT 15 FOOT OFFSET ARE INTENDED TO BE MANAGED AS PART OF THE VEGETATION-FREE ZONE. HOWEVER, THE LOWER LANDSIDE SLOPE CAN BE EXPECTED TO DEVELOP GROWTH OF PICKLEWEED AND OTHER HIGH MARSH PLANTS DUE TO THE PRESENCE OF SALT AND SEASONAL WATER IN THE AREA. THE GROWTH OF HIGH MARSH PLANTS CAN BE EXPECTED THROUGHOUT THE ENTIRE LENGTH OF THE LEVEE. PICKLEWEED WILL NOT USUALLY GROW BEYOND THE TOE OF THE LEVEE DUE TO PERMANENT AND SEASONAL PONDED WATER. THERE WILL BE VERY MINOR LOCATIONS WHERE PICKLEWEED DOES GROW NEXT TO THE LEVEE DUE TO HIGHER GROUND.

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5. THE PROJECT DELIVERY TEAM (PDT) RECOMMENDS A MINIMUM LEVEE PRISM WHICH WOULD BE CONSTRUCTED PRIMARILY OF BAY MUD AS SHOWN ON SHEET C-3. HOWEVER, IF THE PICKLEWEED HEIGHT NEEDS TO BE FURTHER REDUCED FOR LEVEE SAFETY ON THE LANDSIDE, AN UNDERLYING LAYER OF GRAVEL OR AN UNDERLYING GEOTEXTILE COULD BE ADDED, AS ALSO SHOWN ON SHEET C-3. THE PDT DOES NOT RECOMMEND ADDING A PLANTING BERM WITH NATURAL PICKLEWEED, AS SHOWN, BECAUSE IT WILL REDUCE THE AREA OF THE MARSH, POSSIBLY RAISE ADDITIONAL ENVIRONMENTAL ISSUES, AND IS JUDGED TO BE THE MOST EXPENSIVE SOLUTION.

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SHORELINE PROJECT DATUMS

HORIZONTAL: CALIFORNIA COORDINATE SYSTEM NAD83 ZONE III
VERTICAL: NAVD88

US Army Corps of Engineers San Francisco District

Date: 05/01/2014 | Design file no: X-XX-XXXX | Drawing Code: XXXX | Prepared by: M. Young | Dwn by: M. Young | Reviewed by: W. DeJager | Submitted by: P. Schmidtfenyg | Spec No. 5 of 9

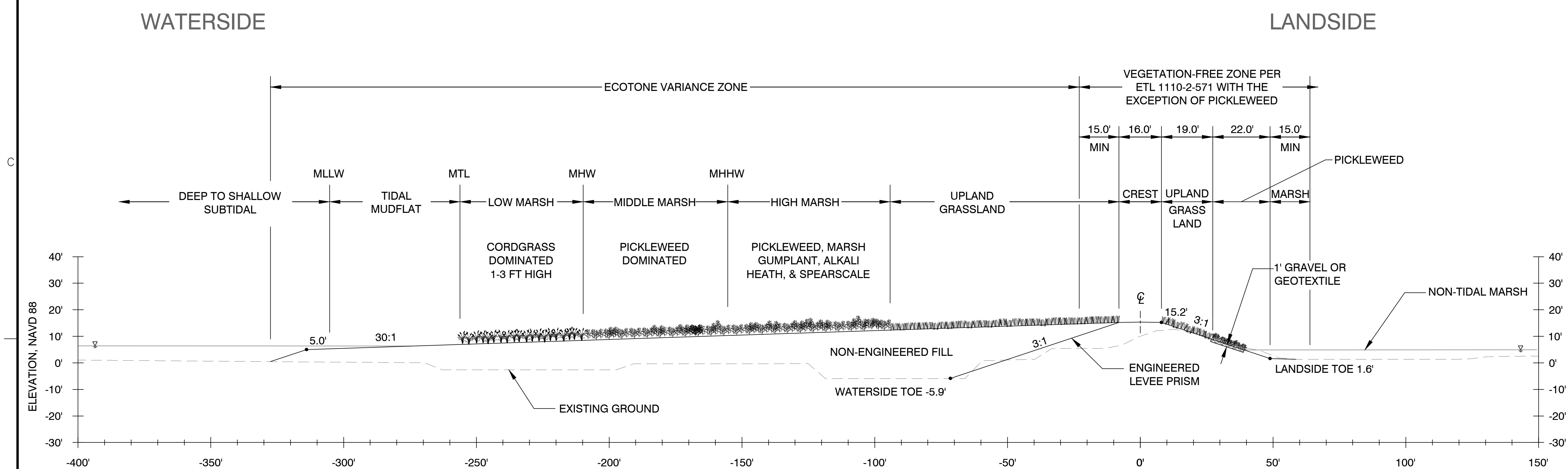
DEPARTMENT OF THE ARMY CORPS OF ENGINEERS SAN FRANCISCO, CALIFORNIA

SAN FRANCISCO BAY SHORELINE
TYPICAL CROSS SECTIONS
PLANTING BERM ALTERNATIVE
STA 43+8 TO STA 94+75
SAN FRANCISCO CALIFORNIA

Sheet reference number: C-4

DRAFT

UPDATE



GRAVEL OR GEOTEXTILE ALTERNATIVE
TYPICAL CROSS SECTION FROM STA 94+75 TO STA 140+00

3 / - C-5

SCALE: 1"=40' (H), 1"= 40' (V)

WATERSIDE NOTES:

1. PLANTS IN THE LOW MARSH AND MIDDLE MARSH AREAS WILL ESTABLISH ON THEIR OWN. LOW MARSH WILL CONSIST OF CORDGRASS (A TALL SPECIES OF GRASS) WHILE THE MIDDLE MARSH WILL PRIMARILY CONSIST OF PERENNIAL PICKLEWEED, A LOW SUCCULENT AND SLIGHTLY WOODY PLANT THAT GROWS TO AN AVERAGE HEIGHT OF 1 FOOT. THE LOW AND MIDDLE MARSH AREAS WOULD BE LARGELY UNMANAGED WITH NO MOWING.
2. THE HIGH MARSH AND UPLAND GRASSLAND WILL BE PLANTED. HIGH MARSH PLANTS ARE MOSTLY 1-2 FEET TALL WITH OCCASIONAL STEMS REACHING UP TO 5 FEET TALL. PLANTS IN THESE AREAS ARE SOFT TO SEMI WOODY. THE HIGH MARSH AND UPLAND GRASSLAND AREAS WOULD ALSO BE LARGELY UNMANAGED WITH NO MOWING.
3. THE 15 FEET OF ECOTONE CLOSEST TO THE FLOOD RISK MANAGEMENT LEVEE, ALONG WITH THE REST OF THE FRM LEVEE CROSS-SECTION, WOULD BE MANAGED AS A VEGETATION-FREE ZONE PER ETL 1110-2-571. SEE SECTION FOR DETAILS.

SHORELINE PROJECT DATUMS

HORIZONTAL: CALIFORNIA COORDINATE SYSTEM NAD83 ZONE III
VERTICAL: NAVD88

LANDSIDE NOTES:

1. THE LANDSIDE OF THE LEVEE AND THE ADJACENT 15 FOOT OFFSET ARE INTENDED TO BE MANAGED AS PART OF THE VEGETATION-FREE ZONE. HOWEVER, THE LOWER LANDSIDE SLOPE CAN BE EXPECTED TO DEVELOP GROWTH OF PICKLEWEED AND OTHER HIGH MARSH PLANTS DUE TO THE PRESENCE OF SALT AND SEASONAL WATER IN THE AREA. THE GROWTH OF HIGH MARSH PLANTS CAN BE EXPECTED THROUGHOUT THE ENTIRE LENGTH OF THE LEVEE. PICKLEWEED WILL NOT USUALLY GROW BEYOND THE TOE OF THE LEVEE DUE TO PERMANENT AND SEASONAL PONDED WATER. THERE WILL BE VERY MINOR LOCATIONS WHERE PICKLEWEED DOES GROW NEXT TO THE LEVEE DUE TO HIGHER GROUND.
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6. ELEVATION OF PICKLEWEED ABOVE THE LEVEE BASE IN NON-TIDAL AREAS WILL DEPEND ON THE SOIL SOURCE USED FOR THE LEVEE FACE. REUSED BAY MUD WILL ENCOURAGE PICKLEWEED AND DISCOURAGE GRASS. UPLAND SOIL WILL GENERALLY GROW GRASS UNLESS SOIL SALINIZATION OCCURS FROM ADJACENT WATERS.

US Army Corps of Engineers
San Francisco District

Date: 05/01/2014
Design file no: X-XX-XXXX
Drawing Code: XXXX
Prepared by: M. Young
Dwn by: M. Young
Reviewed by: W. DeJager
Submitted by: P. Schmidtenyg
Spec No. 6 of 9

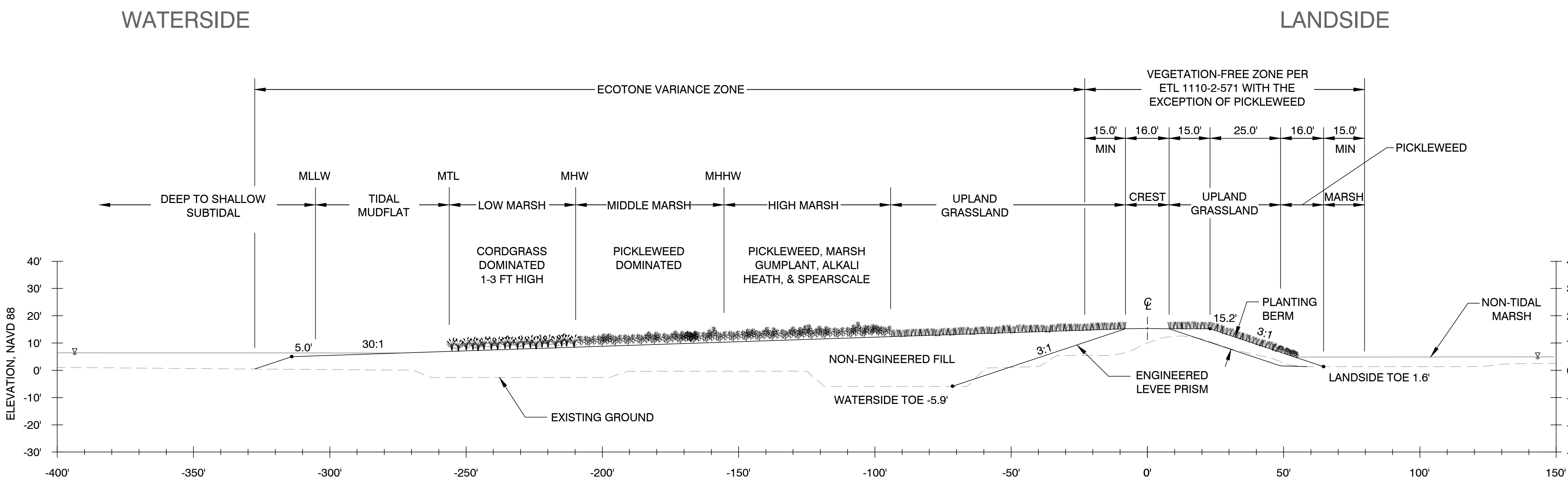
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

SAN FRANCISCO CALIFORNIA
SAN FRANCISCO BAY SHORELINE
TYPICAL CROSS SECTIONS
GRAVEL OR GEOTEXTILE ALTERNATIVE
STA 94+75 TO STA 140+00

Sheet reference number:
C-5

DRAFT

UPDATE



PLANTING BERM ALTERNATIVE
TYPICAL CROSS SECTION FROM STA 94+75 TO STA 140+00

3 - C-6

SCALE: 1"=40' (H), 1"= 40' (V)

WATERSIDE NOTES:

1. PLANTS IN THE LOW MARSH AND MIDDLE MARSH AREAS WILL ESTABLISH ON THEIR OWN. LOW MARSH WILL CONSIST OF CORDGRASS (A TALL SPECIES OF GRASS) WHILE THE MIDDLE MARSH WILL PRIMARILY CONSIST OF PERENNIAL PICKLEWEED, A LOW SUCCULENT AND SLIGHTLY WOODY PLANT THAT GROWS TO AN AVERAGE HEIGHT OF 1 FOOT. THE LOW AND MIDDLE MARSH AREAS WOULD BE LARGELY UNMANAGED WITH NO MOWING.
2. THE HIGH MARSH AND UPLAND GRASSLAND WILL BE PLANTED. HIGH MARSH PLANTS ARE MOSTLY 1-2 FEET TALL WITH OCCASIONAL STEMS REACHING UP TO 5 FEET TALL. PLANTS IN THESE AREAS ARE SOFT TO SEMI WOODY. THE HIGH MARSH AND UPLAND GRASSLAND AREAS WOULD ALSO BE LARGELY UNMANAGED WITH NO MOWING.
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SHORELINE PROJECT DATUMS

HORIZONTAL: CALIFORNIA COORDINATE SYSTEM NAD83 ZONE III
VERTICAL: NAVD88

LANDSIDE NOTES:

1. THE LANDSIDE OF THE LEVEE AND THE ADJACENT 15 FOOT OFFSET ARE INTENDED TO BE MANAGED AS PART OF THE VEGETATION-FREE ZONE. HOWEVER, THE LOWER LANDSIDE SLOPE CAN BE EXPECTED TO DEVELOP GROWTH OF PICKLEWEED AND OTHER HIGH MARSH PLANTS DUE TO THE PRESENCE OF SALT AND SEASONAL WATER IN THE AREA. THE GROWTH OF HIGH MARSH PLANTS CAN BE EXPECTED THROUGHOUT THE ENTIRE LENGTH OF THE LEVEE. PICKLEWEED WILL NOT USUALLY GROW BEYOND THE TOE OF THE LEVEE DUE TO PERMANENT AND SEASONAL PONDED WATER. THERE WILL BE VERY MINOR LOCATIONS WHERE PICKLEWEED DOES GROW NEXT TO THE LEVEE DUE TO HIGHER GROUND.
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5. THE PROJECT DELIVERY TEAM (PDT) RECOMMENDS A MINIMUM LEVEE PRISM WHICH WOULD BE CONSTRUCTED PRIMARILY OF BAY MUD AS SHOWN ON SHEET C-5. HOWEVER, IF THE PICKLEWEED HEIGHT NEEDS TO BE FURTHER REDUCED FOR LEVEE SAFETY ON THE LANDSIDE, AN UNDERLYING LAYER OF GRAVEL OR AN UNDERLYING GEOTEXTILE COULD BE ADDED, AS ALSO SHOWN ON SHEET C-5. THE PDT DOES NOT RECOMMEND ADDING A PLANTING BERM WITH NATURAL PICKLEWEED, AS SHOWN, BECAUSE IT WILL REDUCE THE AREA OF THE MARSH, POSSIBLY RAISE ADDITIONAL ENVIRONMENTAL ISSUES, AND IS JUDGED TO BE THE MOST EXPENSIVE SOLUTION.
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US Army Corps of Engineers
San Francisco District

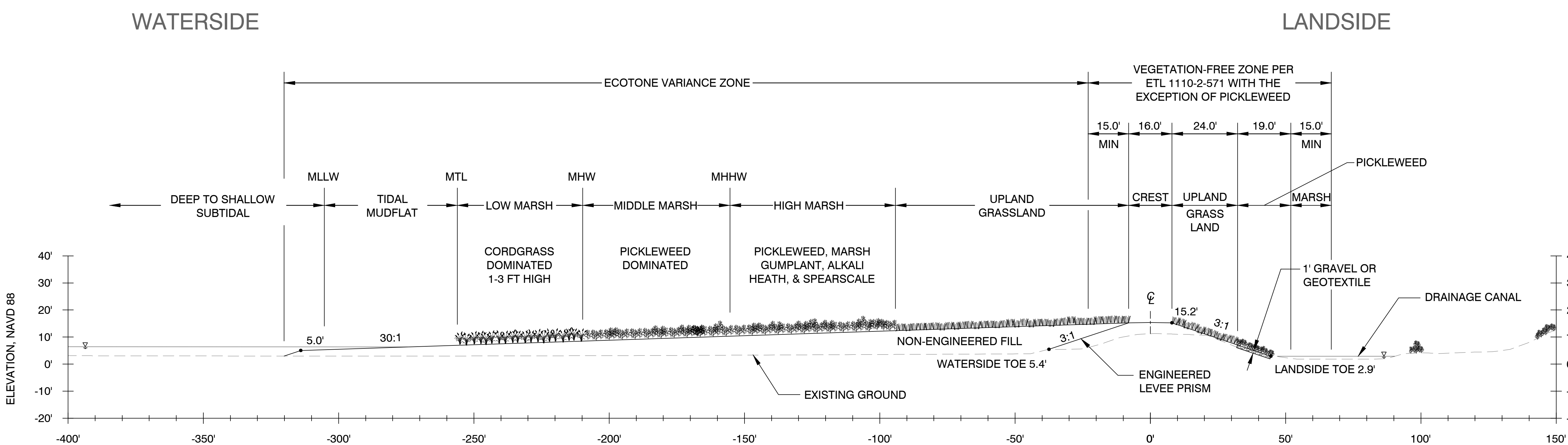
Date: 05/01/2014 | Design file no: X-XX-XXXX | Drawing Code: XXXX
Prepared by: M. Young | Dwn by: M. Young | Reviewed by: W. DeJager | Submitted by: P. Schmidtfenyg
Spec No. 7 of 9

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

SAN FRANCISCO BAY SHORELINE
TYPICAL CROSS SECTIONS
PLANTING BERM ALTERNATIVE
STA 94+75 TO STA 140+00

Sheet reference number: C-6

DRAFT



4
- C-7

GRAVEL OR GEOTEXTILE ALTERNATIVE
TYPICAL CROSS SECTION FROM STA 140+00 TO STA 197+75

SCALE: 1"=40' (H), 1"= 40' (V)

WATERSIDE NOTES:

1. PLANTS IN THE LOW MARSH AND MIDDLE MARSH AREAS WILL ESTABLISH ON THEIR OWN. LOW MARSH WILL CONSIST OF CORDGRASS (A TALL SPECIES OF GRASS) WHILE THE MIDDLE MARSH WILL PRIMARILY CONSIST OF PERENNIAL PICKLEWEED, A LOW SUCCULENT AND SLIGHTLY WOODY PLANT THAT GROWS TO AN AVERAGE HEIGHT OF 1 FOOT. THE LOW AND MIDDLE MARSH AREAS WOULD BE LARGELY UNMANAGED WITH NO MOWING.

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SHORELINE PROJECT DATUMS

HORIZONTAL: CALIFORNIA COORDINATE SYSTEM NAD83 ZONE III
VERTICAL: NAVD88

LANDSIDE NOTES:

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3. WE WILL GENERALLY ONLY NEED TO ADDRESS THE EXISTENCE OF PICKLEWEED ON THE LAND SIDE SLOPE DUE TO THE ECOTONE.

4. NATURAL PICKLEWEED IN TIDAL AREAS TYPICALLY GROW TO A MAXIMUM HEIGHT OF 12 TO 18 INCHES (OCCASIONALLY UP TO 24 INCHES) BETWEEN 0.0 AND 3.0 FT ABOVE THE WATER SURFACE ELEVATION. OUTSIDE OF TIDAL AREAS, CONDITIONS AT THE SHORELINE SITE ARE USUALLY NOT IDEAL; HEIGHTS WILL RANGE FROM 6 INCHES TO 24 INCHES DEPENDING ON SOIL SALINITY AND WATER AVAILABILITY. THE PICKLEWEED COULD BE STUNTED BY EXTREME SALINITY CONDITIONS (HIGH OR LOW), PROLONGED INUNDATION, OR SEVER LACK OF WATER. THE MOST RELIABLE METHOD WOULD BE TO APPLY A LAYER OF BAY MUD TO THE SURFACE OF THE LEVEE TO CREATE A COMBINATION OF HIGH SALINITY AND DRY CONDITIONS. THE ABILITY OF OTHER METHODS TO STUNT PICKLEWEED IS UNCERTAIN.

5. THE PROJECT DELIVERY TEAM (PDT) RECOMMENDS A MINIMUM LEVEE PRISM WHICH WOULD BE CONSTRUCTED PRIMARILY OF BAY MUD AS SHOWN. HOWEVER, IF THE PICKLEWEED HEIGHT NEEDS TO BE FURTHER REDUCED FOR LEVEE SAFETY ON THE LANDSIDE, AN UNDERLYING LAYER OF GRAVEL OR AN UNDERLYING GEOTEXTILE COULD BE ADDED, AS ALSO SHOWN. THE PDT DOES NOT RECOMMEND ADDING A PLANTING BERM WITH NATURAL PICKLEWEED, AS SHOWN ON SHEET C-8, BECAUSE IT WILL REDUCE THE AREA OF THE MARSH, POSSIBLY RAISE ADDITIONAL ENVIRONMENTAL ISSUES, AND IS JUDGED TO BE THE MOST EXPENSIVE SOLUTION.

6. ELEVATION OF PICKLEWEED ABOVE THE LEVEE BASE IN NON-TIDAL AREAS WILL DEPEND ON THE SOIL SOURCE USED FOR THE LEVEE FACE. REUSED BAY MUD WILL ENCOURAGE PICKLEWEED AND DISCOURAGE GRASS. UPLAND SOIL WILL GENERALLY GROW GRASS UNLESS SOIL SALINIZATION OCCURS FROM ADJACENT WATERS.

US Army Corps of Engineers San Francisco District

Prepared by: M. Young
Dwn by: M. Young
Reviewed by: W. DeJager
Submitted by: P. Schmidtberg
Spec No. 8 of 9
Date: 05/01/2014
Design file no: X-XX-XXXX
Drawing Code: XXXX

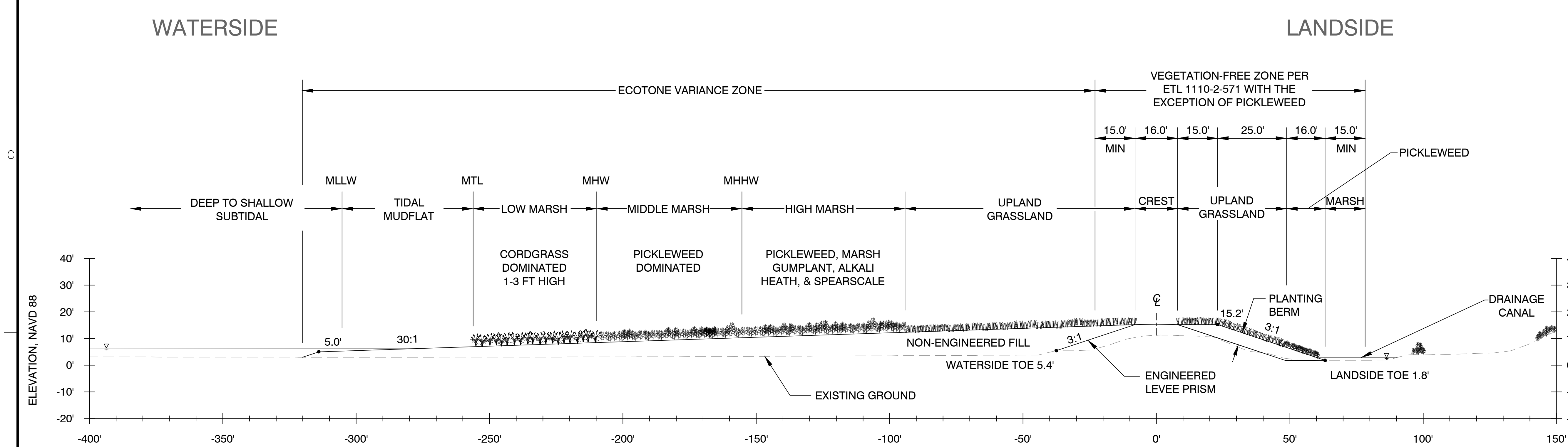
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

SAN FRANCISCO — CALIFORNIA
SAN FRANCISCO BAY SHORELINE
TYPICAL CROSS SECTIONS
GRAVEL OR GEOTEXTILE ALTERNATIVE
STA 140+00 TO STA 197+75

Sheet reference number:
C-7

DRAFT

UPDATE



PLANTING BERM ALTERNATIVE
TYPICAL CROSS SECTION FROM STA 140+00 TO STA 197+75

4
- C-8

SCALE: 1"=40' (H), 1"= 40' (V)

WATERSIDE NOTES:

1. PLANTS IN THE LOW MARSH AND MIDDLE MARSH AREAS WILL ESTABLISH ON THEIR OWN. LOW MARSH WILL CONSIST OF CORDGRASS (A TALL SPECIES OF GRASS) WHILE THE MIDDLE MARSH WILL PRIMARILY CONSIST OF PERENNIAL PICKLEWEED, A LOW SUCCULENT AND SLIGHTLY WOODY PLANT THAT GROWS TO AN AVERAGE HEIGHT OF 1 FOOT. THE LOW AND MIDDLE MARSH AREAS WOULD BE LARGELY UNMANAGED WITH NO MOWING.

2. THE HIGH MARSH AND UPLAND GRASSLAND WILL BE PLANTED. HIGH MARSH PLANTS ARE MOSTLY 1-2 FEET TALL WITH OCCASIONAL STEMS REACHING UP TO 5 FEET TALL. PLANTS IN THESE AREAS ARE SOFT TO SEMI WOODY. THE HIGH MARSH AND UPLAND GRASSLAND AREAS WOULD ALSO BE LARGELY UNMANAGED WITH NO MOWING.

3. THE 15 FEET OF ECOTONE CLOSEST TO THE FLOOD RISK MANAGEMENT LEVEE, ALONG WITH THE REST OF THE FRM LEVEE CROSS-SECTION, WOULD BE MANAGED AS A VEGETATION-FREE ZONE PER ETL 1110-2-571. SEE SECTION FOR DETAILS.

SHORELINE PROJECT DATUMS

HORIZONTAL: CALIFORNIA COORDINATE SYSTEM NAD83 ZONE III
VERTICAL: NAVD88

LANDSIDE NOTES:

1. THE LANDSIDE OF THE LEVEE AND THE ADJACENT 15 FOOT OFFSET ARE INTENDED TO BE MANAGED AS PART OF THE VEGETATION-FREE ZONE. HOWEVER, THE LOWER LANDSIDE SLOPE CAN BE EXPECTED TO DEVELOP GROWTH OF PICKLEWEED AND OTHER HIGH MARSH PLANTS DUE TO THE PRESENCE OF SALT AND SEASONAL WATER IN THE AREA. THE GROWTH OF HIGH MARSH PLANTS CAN BE EXPECTED THROUGHOUT THE ENTIRE LENGTH OF THE LEVEE. PICKLEWEED WILL NOT USUALLY GROW BEYOND THE TOE OF THE LEVEE DUE TO PERMANENT AND SEASONAL PONDED WATER. THERE WILL BE VERY MINOR LOCATIONS WHERE PICKLEWEED DOES GROW NEXT TO THE LEVEE DUE TO HIGHER GROUND.

2. ACCESS ALONG THE LAND SIDE TOE WILL NOT BE EASY DUE TO THE MARSH. THERE WILL BE VERY MINOR LOCATIONS WHERE PICKLEWEED DOES GROW NEXT TO THE LEVEE DUE TO HIGHER GROUND.

3. WE WILL GENERALLY ONLY NEED TO ADDRESS THE EXISTENCE OF PICKLEWEED ON THE LAND SIDE SLOPE DUE TO THE ECOTONE.

4. NATURAL PICKLEWEED IN TIDAL AREAS TYPICALLY GROW TO A MAXIMUM HEIGHT OF 12 TO 18 INCHES (OCCASIONALLY UP TO 24 INCHES) BETWEEN 0.0 AND 3.0 FT ABOVE THE WATER SURFACE ELEVATION. OUTSIDE OF TIDAL AREAS, CONDITIONS AT THE SHORELINE SITE ARE USUALLY NOT IDEAL; HEIGHTS WILL RANGE FROM 6 INCHES TO 24 INCHES DEPENDING ON SOIL SALINITY AND WATER AVAILABILITY. THE PICKLEWEED COULD BE STUNTED BY EXTREME SALINITY CONDITIONS (HIGH OR LOW), PROLONGED INUNDATION, OR SEVER LACK OF WATER. THE MOST RELIABLE METHOD WOULD BE TO APPLY A LAYER OF BAY MUD TO THE SURFACE OF THE LEVEE TO CREATE A COMBINATION OF HIGH SALINITY AND DRY CONDITIONS. THE ABILITY OF OTHER METHODS TO STUNT PICKLEWEED IS UNCERTAIN.

5. THE PROJECT DELIVERY TEAM (PDT) RECOMMENDS A MINIMUM LEVEE PRISM WHICH WOULD BE CONSTRUCTED PRIMARILY OF BAY MUD AS SHOWN ON SHEET C-7. HOWEVER, IF THE PICKLEWEED HEIGHT NEEDS TO BE FURTHER REDUCED FOR LEVEE SAFETY ON THE LANDSIDE, AN UNDERLYING LAYER OF GRAVEL OR AN UNDERLYING GEOTEXTILE COULD BE ADDED, AS ALSO SHOWN ON SHEET C-7. THE PDT DOES NOT RECOMMEND ADDING A PLANTING BERM WITH NATURAL PICKLEWEED, AS SHOWN, BECAUSE IT WILL REDUCE THE AREA OF THE MARSH, POSSIBLY RAISE ADDITIONAL ENVIRONMENTAL ISSUES, AND IS JUDGED TO BE THE MOST EXPENSIVE SOLUTION.

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US Army Corps of Engineers
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Date: 05/01/2014
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Prepared by: M. Young
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Spec No. 9 of 9

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

SAN FRANCISCO — CALIFORNIA
SAN FRANCISCO BAY SHORELINE
TYPICAL CROSS SECTIONS
PLANTING BERM ALTERNATIVE
STA 140+00 TO STA 197+75

Sheet reference number: C-8

DRAFT

UPDATE

Attachment E

Geotechnical field assessment of the San Francisco South Bay Dike system

Geotechnical Memo – Version 0.8 (File: Report SFSB Site14+0813--v5o.docx)
Project center: 37.447°, -121.987°   Field inspection date: 13 August 2014
Richard S. Olsen, PhD, PE, Sr. Geotechnical Engineer for USACE, HQ USACE E&C, Washington DC

Summary

The geotechnical means for potential failure of the South bay dike is crest erosion during overtopping. The 6-mile dike has a crest composed of a wide range of soil types from erodible silts to stiff clays (having low erodiblity). There is high likelihood that if overtopping occurs than some portion of the dike system will experience deep erosion of the crest.

This dike system has not experienced failure for 30+ years because of aggressive on-site efforts to maintain a marginally satisfactory crest elevation. These historic efforts have produced a system with a low margin of safety for overtopping and crest erosion. Potential for failure in the future is high for water levels of historic level as well as for lower levels.

The dike system is composed of many over geotechnical issues; settlement, rodent tunneling, drainage structures, wave erosion of exposed silt, and near surface (above pool level) sliding of the dike face. All of these issues can be addressed by properly maintaining the system.

Reason for Field Assessment

The purpose of the field visit was to observe condition of the dike to;
a) verify information from numerous project geotechnical reports,
b) observe the condition of the dike in terms of geotechnical engineering and failure modes, and
c) report on near surface conditions of the dike system (because erosion potential of the dike crest is critical).

The highest failure potential mode in terms of geotechnical engineering is erosion of the crest soils during overtopping. This memo will describe all geotechnical failure modes and related information will be examined.

Approach

Special field observation procedures as well as new reporting techniques were used for this effort and are described in Appendix A - Geotechnical field observation procedures, Appendix B – Taking photos in the field, Appendix C - Google Earth, and Appendix D – Geotechnical visualization in a report.

Observations

A visit to the levee reach was conducted by U.S. Army Corps of Engineers (USACE) personnel from HQ and SPN on 13 August 2014 by Dr. Rick Olsen (HQ USACE CW EC), Mr. Scott Nicholson (HQ USACE CW PC), Mr. Caleb Conn (USACE SPN), and Mr. Nicholas Malasavage (USACE SPN). The inspection involved driving the total dike length with frequent walking inspections at important locations.

Field observations were performed using new methods (see Appendix A and B) and displayed in this report using numerous new visualization methods (see Appendix C and D). The vehicle track and photo locations shown in Figure 1, a Google Earth visualization map (a KMZ file) which can be downloaded from http://geostaff.net//USACE/SPN/SFSB/2014aug/GE-track-photos.kmz (allow it to open in Google Earth - 6MB file will take some time to load). The photo icon groups in Figure 1 indicate the locations of vehicle stops for inspections. All project figures having site photographs will have a small map in an upper corner showing the approximate location (see right side of Figure 1).

Figure 1 - Site map

There are two methods for displaying site obtained photos using Google Earth. The first is a typical method which allows user to interact with Google Earth to see location specific photos, an example is shown in Figure 2 (see Appendix C on how to use Google Earth). A new alternative visualization method is to use Vpics (also described in Appendix C)) with an example shown in Figure 3, the Vpics Google Earth KMZ data file can be downloaded from http://geostaff.net//USACE/SPN/SFSB/2014aug/GE-Vpics.kmz (allow it to open in Google Earth – 4MB file will take some time to load).

Figure 2 - Photo view from Google Earth

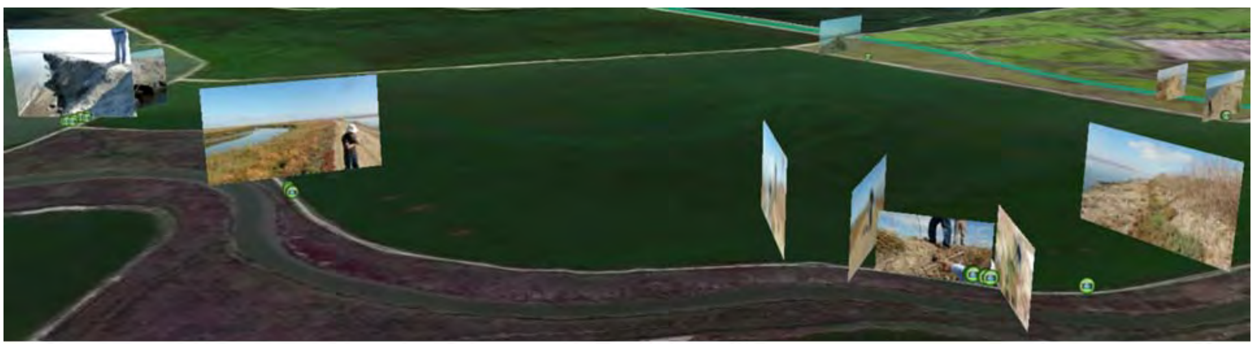

Figure 3 - Vertical pictures (Vpics) using Google Earth

Dike crest soil composition

The highest potential for dike failure is erosion of the crest soils during overtopping. There are two generalized soil locations for this dike system; soils inside the dike and the underlying soft bay mud; this section only deals with the dike soil composition.

During the field assessment most of the dike length had an observed crest soil composition ranging from loose silt behavior with high organic fiber content (termed moon dust in the field) to loose soil mixtures as shown in Figure 4. The left photo shows a silt behavior soil and the right photo shows darker high organic fiber content silt. This loose silt like behaving soil has a thickness of at least a foot in several locations (see Figure 5). These silt behaving soils have secondary influences, unfortunately generating an optimum condition for rodents to dig holes but allowing specific vegetation types to grow.

Figure 4 - Crest loose silt behavior with some to high organic fiber content

Figure 5 - Loose silt like behavior on dike crest

Characterizing crest soil types is critical for this dike system. However, none of the geotechnical engineering efforts focused on assessing the character of the dike soil types. A majority of boring effort concentrated on collection of bay mud samples. Very limited geotechnical soil sampling of the dike crest does indicate soil types ranging from silt to sandy clay, generally using only one or two soil samples in the upper 7 feet for each boring. During the CPT based exploration it’s likely that a light weight CPT truck was used because a large number of the soundings were predrilled through the stronger upper dike soils before CPT probing operations started. This pre-drilling could have been a great opportunity for retrieving soil samples from the upper portion of the dike.

The following is a summary of the boring soil classification information from the upper 7 feet for borings in dike area 2 (project area);

Clay, stiff day – Boring P8
Clay, fill – Boring P12
Silt , medium stiff to stiff dry - boring P1
Clay, very soft – Boring P13

The following information is from borings outside of area 2;

Silty clay, stiff dry to moist– boring P3
Clay, soft to medium dry – Boring P4
Clay, stiff moist – Boring P6
Clay, stiff to very stiff – Boring P7
Clay, medium stiff fill – Boring P9 area 4
Clay, medium stiff fill – Boring P11
Silt (MH), medium stiff fill – Boring A6
Sandy Clay, very loose fill – Boring A7
Sandy Clay, medium stiff fill – Boring B9

No index tests other than water content were performed. A better approach would have been to perform almost continuous soil sampling in the upper 10 feet. At a minimum, performing passing #200 sieve testing should have been performed for all soil samples. Also, no samples were obtained from the upper 2 feet of the dike crest. Only a limited number of soil samples were retrieved, and stored, therefore the potential for future index testing is impossible.

During the field inspection there is evidence that Cargel field operations would scarify or blade crest soil to the bay side of the crest as shown in Figure 6. The apparent purpose of blading was to generate a small temporary raised crest section as illustrated in Figure 7.

Figure 6 – Blading and scarifying dike crest silt to either side of the crest – to create limited crest elevation rise.

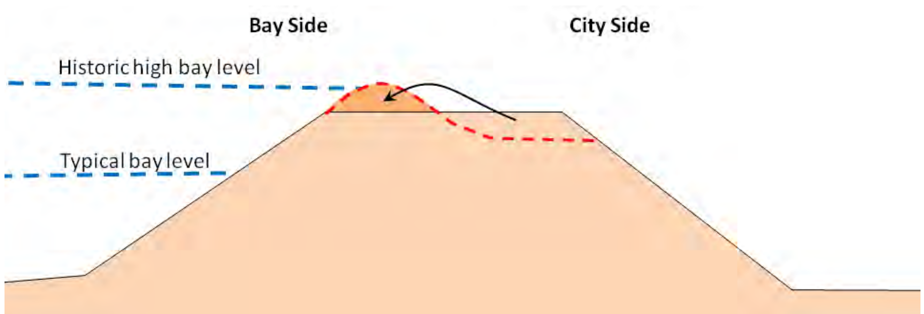

Figure 7 – Blading to create small localized mounds on the side of the crest to temporary raise the effective crest height

Crest erosion during overtopping

The crest soil type ranges from erodible silts to stiff clays (with low erodiblity). We really don't know the true range of soil types, the locations for erodible soils, or the thickness of erodible soils because there is so little information from the field. We don't know, for example, if there is only one location along the total dike length with high erosion potential. We know that at least one location along this dike system (based on borings) and likely many others (based on field observations) has significant depth of erodible silt at the crest. A high water event causing water flow over the dike crest will cause quick erosion of the crest for at least one location.

The ERDC report (2008) summarized the above geotechnical soil type data, predicted average soils (and thickness) for the levee crest, and than predicted erosion rates. It really is not possible to generalize erosion rates with so little information, specifically for soil types arranging from erodible silt to stiff clay. The most that can be predicted, with limited information, is that overtopping erosion will occur at one location having a depth that is unknown (likely as deep as 2 or 3 feet).

It is possible that only a single point along the total dike will fail due to marginal overtopping and that the depth of crest erosion will only be a few feet. We don't know the volume of breached water that could go through a shallow breach (say 2 feet) if the high water event only has a short duration.

Strength of Dike soils

Soil strength of the dike is highly variable as are most historically constructed levees and dikes around the world. Very few of the soil samples retrieved from inside the dike were laboratory tested for soil strength. A majority of the field exploration were performed using CPT soundings because it's about 3 times less expensive per foot compared to borings, but it does not provide soil samples.

Geotechnical consultants used a very simple method for estimating soil strength using CPT data. To calculate soil strength (S_u or S) the measured CPT end bearing stress required to push the tip through the ground (termed the cone resistance, q_c) was divided by N_k of 16 as shown in the equation below.

$$S_u = \frac{q_c}{N_k}$$

This method is only for clays with low silt content and is inappropriate for soil mixtures. For sandy soil mixtures the measured cone resistance will be exponentially higher and consequently an N factor higher than 16 must be used. Therefore, using N_k of 16 will generate predicted soil strength too high, and is therefore unconservative for soil mixtures in the dike.

Above waterline wave erosion of exposed silt and sliding of dike slope face soils

On the South segment of this dike system, when the dike height is high, the exposed soils on the slopes become dry and brittle. Erosion due to wave action erodes away the dike toe resulting in steeper slopes (see Figure 8). These steep sections slowly experience multipoint failures (as shown in Figure 9). The resulting debris flows into the ponds create a bench of flow material just beyond the dike toe, as illustrated in Figure 10.

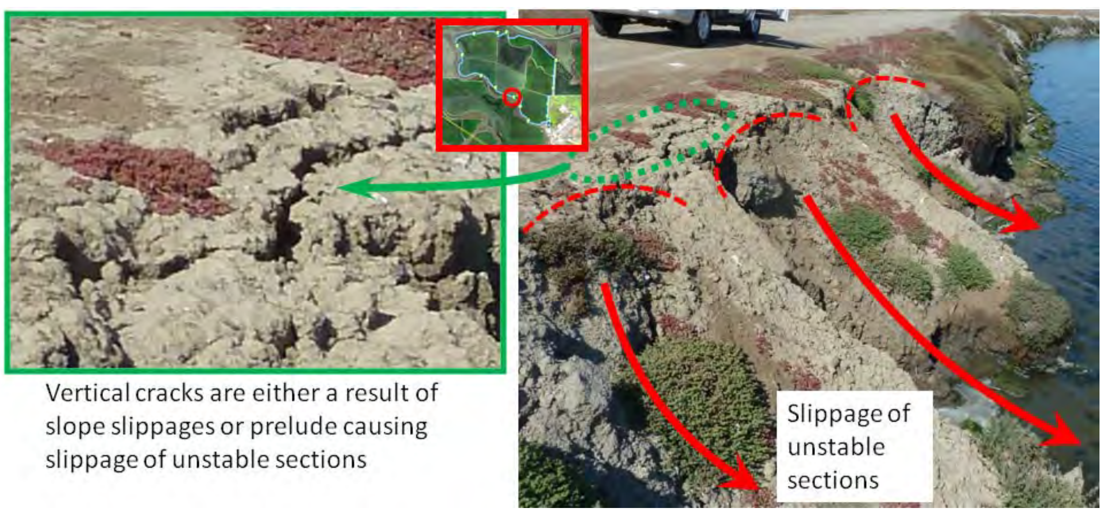

Figure 8 - toe erosion resulting in near surface failure of dike dry material

Figure 9 - Steepen dry silts and resulting sliding

Figure 10 - runout of flow material just beyond the dike toe

Rodent dug holes

Rodent holes where observed throughout the dike system - during high water events these holes can cause internal erosion based failure of the dike. The rodent holes were observed at a depth of 1 to 3 feet below the crest elevation and generally on the pool side of the dike (see Figure 11). Along an access road next to the railroad (see Figure 12) a large number of rodent holes were observed as close as one every foot. The railroad utility a few decades ago placed a large volume of small rip rap rock along the rail embankment on the south side of the road crossing, possibly because of the large number of rodent holes in this small area.

These rodent holes are important because levees have historically failed when water has risen to within 3 feet of levee crest. Figure 13 shows a levee failure in Romania likely due to rodent tunnels at a depth of 3 feet below the levee crest.

The combination of rodent holes and dry highly erodible silt soils together provide a dangerous condition because either or both in combination can cause erosion failure along an upper section of the dike (as illustrated in Figure 14).

Figure 11 - Rodent dug tunnels are throughout the dike system

Figure 12 - Rodent dug holes next to railroad crossing

Figure 13 - Failed levee in Romania due to rodent tunneling holes

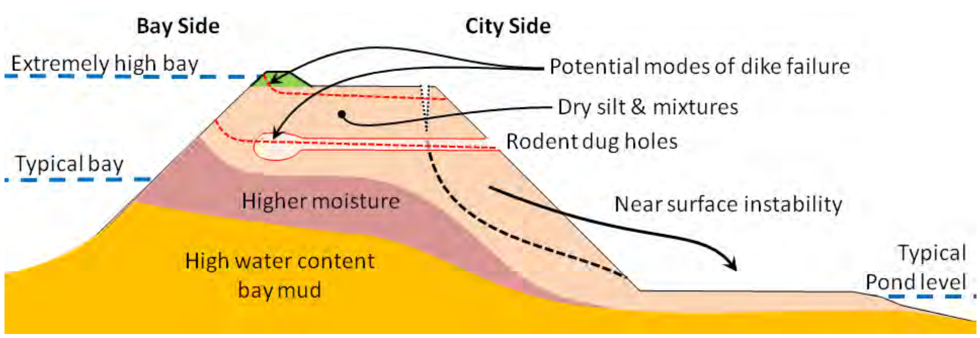

Figure 14 - Hypothetical section showing soil moisture contents, near surface instability, and rodent dug holes, and potential modes of failure.

Grass cover

Much of this dike system is covered with various types of grass cover. While grass can resist water flow the problem is that the coverage for this dike is not 100% as shown in Figure 15, there is better coverage on the bay side of the dike. There are several elevation zones of vegetation along the face of the dike likely due to the availability of water and specific soil type (see left side of Figure 16). Pickle weed was identified along upper reaches of the dike near the crest (show at the right of Figure 16). While vegetation can resist small wave action this resistance is not absolute as shown in the upper left of Figure 15.

Figure 15 - Vegetation is covering much but not all the dike slopes

Figure 16 – Elevations of vegetation

Drainage structures

Most of the dike drainage structures for this system are showing distress (as shown in Figure 17) and could be a failure point in the future - Loss of backfill next to these structures is obvious. These structures consist of a horizontal steel pipe (about 18 inch diameter with a valve) which extends through the dike near the water line and connects to wood retaining structures on both sides. Loss of soil around pipe ends could be caused by;

a) high water velocity entrance to or exiting the pipe causing erosion of adjacent soil,

b) vibrations due to pipe water flow causing loosening and transport of nearby soils, or

c) the retaining wood structure has imperfections thus allowing silt and sand to exit creating large voids.

The likely reasons are all of the above and combinations. There are many examples of complete loss of backfill (shown in Figure 17 top right and bottom left). The top left of Figure 17 is likely caused by a generated void near the bottom of the retaining wall at the water line.

An isolated pipe through a dike section is shown Figure 18. The outside of this steel pipe has experienced severe rusting. The inside of the pipe (right side of Figure 18) shows rusting but the extent of rusting could not be observed.

Figure 17 - Water Control structures are experiencing lose of soil issues

Figure 18 - Pipes through the dike are likely having rusting issues

Strength of soft Bay Mud

San Francisco bay mud strength characteristics have been well studied academically. The project geotechnical reports describe evaluation of bay mud strength based on laboratory strength tests (from boring retrieved soil samples) and predicted strengths from CPT. The soft bay mud under the dikes have experienced clay consolidation causing settlement of the dikes. This consolidation results in a strength increase under the dikes compared to bay mud deposits at same elevation beyond the dike, as shown in Figure 19. The reports don't show any analytical effort comparing strength for bay mud under the dikes to a soil column in the water beyond the dikes. It's unclear if and how the bay mud layers shown in Figure 20 were used to assign undrained strengths. A technique called the equivalent depth method could be used for comparing undrained strength versus vertical effective stress below the dike to adjacent underwater deposits. From review of the reports it cannot be discerned if the established project soil strength for the bay mud is realistic, conservative, or unconservative.

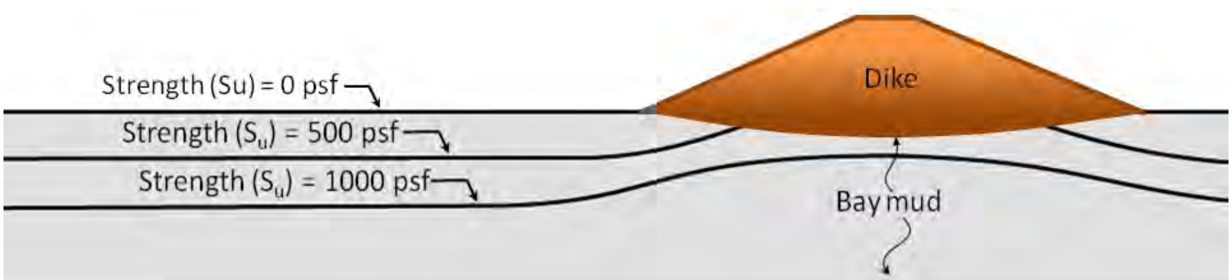

Figure 19 - Bay Mud strength beneath dike (due to consolidation and settlement) and beyond the dike

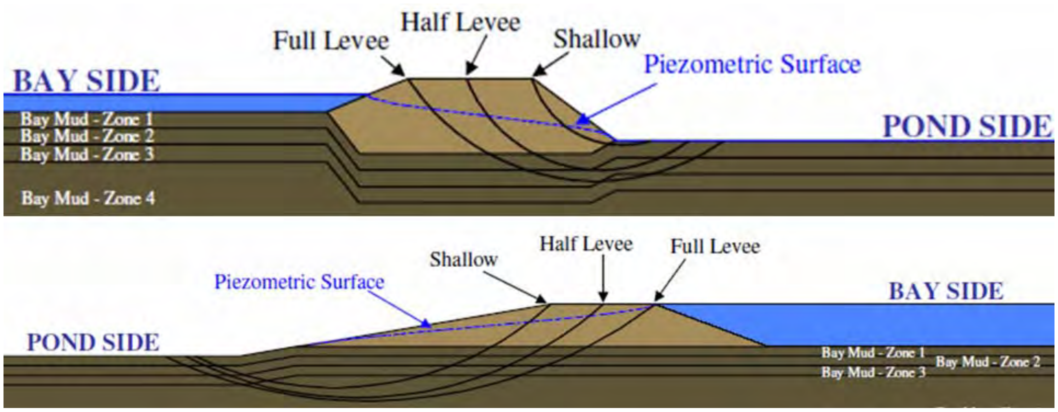

Figure 20 - Geotechnical Consultant's representing of the Bay Mud strength in terms of Bay Mud zones?

Deep Landslide Potential

A deep seated failure of this dike would require a landslide extending down and through the soft bay mud. When a landslide does occur there is a boundary between the sliding mass and non sliding soils, this is termed the failure surface. A landslide involving bay mud would cause rotational movement with the crest going down while also pushing up bay mud beyond the dike toe – generating a peninsula in the water. If such a slope failure occurred it would cause Bay mud strength level reduction along the sliding surface, and would require months to a year for strengths to be regained (if ever). The consequence is that the failure area cannot be quickly repaired. If a repair is quickly attempted it will just produce additional failures. One means of repair is to place new soil on the failure area but at a very shallow slope, such as 10:1 (10 horizontal to 1 vertical). A failure zone (and repair effort) would therefore be easy to see during a field inspection many years after the landslide.

The maintenance records from Cargel indicate no deep seated failures involving soft Bay mud. The field inspection efforts also found no evidence of historic deep seated landslides.

The geotechnical consultants spent a lot of effort performing geotechnical stability evaluation. The highly variable dike shape and crest heights were measured at numerous locations and then a generalized dike shape was used for slope stability analysis. It's unknown if the proper strengths (as discussed in previous sections on dike soil strength and bay mud strength) were used for stability evaluation. A hypothetical non circular slope instability failure surface is illustrated in Figure 21 based on realistic contours of strength under the dike. The geotechnical evaluation studies conclude that landslides are of low relative potential given the dike geometry and assuming that the soil strengths are realistic.

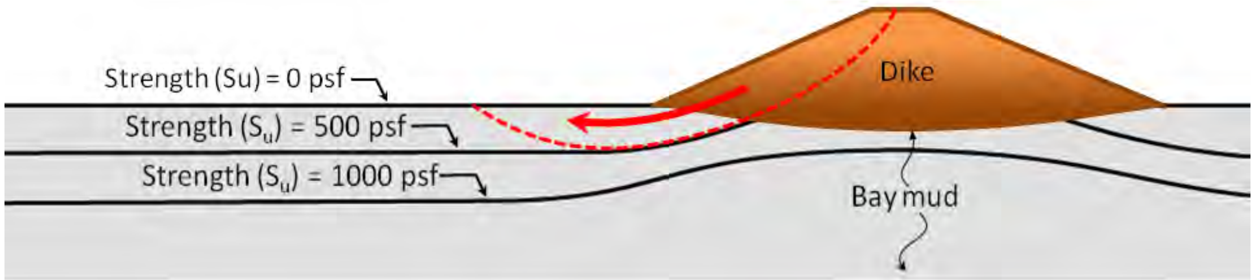

Figure 21 - Potential slip surface accounting for a realistic strength distrubution around the dike

Conclusions

There is high potential that at least one point along this dike system has highly erodible silt at the crest for a significant depth. The potential for crest erosion, during an overtopping event, is therefore high.

A large number of rodent holes were observed in this dike system (see Figure 11). There is also evidence that blading of crest soil was performed to generate small sections having a slightly higher crest elevation (see Figure 6 and Figure 7). There is unknown potential for dike failure due to rodent holes or failure through small elevated crest sections, as illustrated in Figure 14.

The potential for deep seated landslides is relatively low based on historic records as well as field observations. It could not be verified if the evaluated Bay mud strength properly accounted for strength gain under the dike and low strength immediately beyond the dike. It also could not be verified if strengths for the dike soils were unconservative based on the use of an improper simplistic evaluation method.

The historic margin of safety for this dike system has likely been very low: Failure has likely been narrowly diverted on numerous occasions. Potential for failure in the future is therefore high for water levels of historic level as well as for lower levels.