US Army Corps of Engineers®

Appendix I

Geotechnical Engineering

South Pacific Division, Continuing Authorities Program
San Francisco District

Continuing Authorities Program (CAP), Section 103

Table of Contents

1 INTRODUCTION

The Lower Colma Creek study is being conducted under Section 103 of the Continuing Authorities Program (CAP). The project implemented under this authority is formulated for protecting multiple public and private properties and facilities, and single non-federal public properties and facilities against damages caused by storm driven waves and currents. Besides flood risk management, this project provides ecosystem restoration benefits.

The risk of storm events and future sea level rise threaten some of the critical infrastructures such as San Francisco International Airport (SFO) and San Bruno/South San Francisco Water Quality Control Plant (SSFWQCP). The Colma Creek flows to the north of SSFWQCP and drains to the San Francisco Bay. Fluvial and coastal storm impacts due to future sea level rise pose a threat to the existing infrastructures and communities living adjacent to the creek. The SSFWQCP facility provides secondary wastewater treatment for the cities of South San Francisco, San Bruno, and Colma. It also provides the de-chlorination treatment of chlorinated effluent for the cities of Burlingame, Millbrae, and the San Francisco International Airport prior to discharging the treated wastewater into the San Francisco Bay. The average dry weather flow through the facility is 9 million gallons per day (MGD). Peak wet weather flows can exceed 60 MGD.

A reach of Lower Colma Creek upstream of the SSFWQCP was reported to have existing deficiencies for flood risk. The County of San Mateo is considering raising the southern (right bank) floodwalls for this reach. The low-lying areas are mostly located at the back of the existing facility buildings in the north and northeast of the project site which are more prone to flooding during the high-water events.

1.1 Project Description

The SSFWQCP located at 195 Belle Air Road, at Lat. 37.6416°N and Long. 122.3982°W in South San Francisco. South San Francisco is bordered by the cities of Brisbane to the north and San Bruno to the south. The SSFWQCP is located adjacent to San Francisco Bay on Lower Colma Creek, in the City of South San Francisco, which is part of San Mateo County. Figure 1 shows the location of the project site and the study limits.

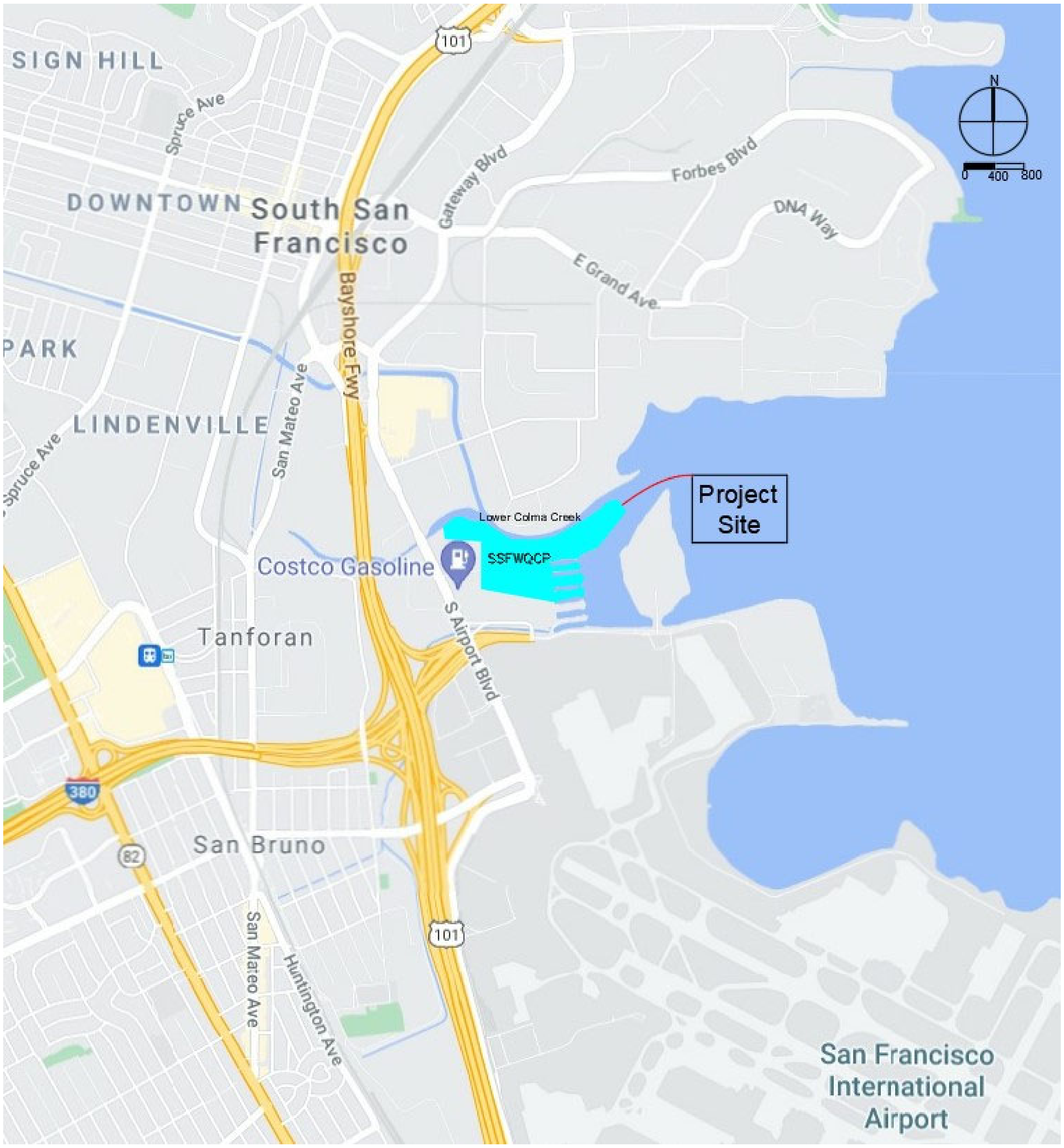


Figure1 - Location of the South San Francisco Water Quality Control Plant

1.2 Project Scope

Geotechnical engineering for this project has been performed by the Geo-Sciences Section of Army Corps of Engineers San Francisco District and input by the non-federal local sponsor, SSFWQCP. The geotechnical reports provided by the local sponsor were used as the main source in evaluation of the project geotechnical and geological conditions. The scope of the geotechnical work for this project includes the following:

- Review of the existing geotechnical reports and geological information pertinent to the project, including geologic maps and reports, boring logs, and laboratory test data.
- Engineering analysis including slope stability, seepage, settlement, and seismic hazards.
- Preparation of this geotechnical feasibility report appendix to present the geotechnical findings and provide geotechnical recommendations for evaluation of flood protection alternatives and CSRM structures.

The available project geotechnical reports discussed the geotechnical baseline conditions and provided the geotechnical recommendations related to the design and construction of the existing facilities and features. The extent of subsurface data in these reports is limited to the areas where the current existing structures were planned to be constructed. No subsurface explorations were found or reported to be conducted at the east side of the project site.

2 GEOLOGY

2.1 Regional Geology

The site is in the northeast portion of the San Francisco Peninsula, which lies within the Coast Ranges geomorphic province. The San Francisco Bay depression resulted from interaction between the major faults of the San Andreas fault zone, particularly the Hayward and San Andreas faults east and west of the bay, respectively (Atwater, 1979).

San Francisco's topography is characterized by relatively rugged hills formed by Jurassic to Cretaceous-aged bedrock (Schlocker, 1974). The bedrock consists of highly deformed and fractured sedimentary rocks of the Franciscan complex. The present topography resulted mainly from east-west compression of coastal California during the late Pliocene and Pleistocene epochs (Norris and Webb, 1990).

The low-lying areas of the San Francisco Peninsula are underlain by Quaternary sediments deposited on eroded Franciscan bedrock. Sediment deposition within the prehistoric bay margin was influenced by oscillating late-Quaternary Sea levels that resulted from the advance and retreat of glaciers worldwide. The resulting sequence of alternating estuarine and terrestrial sediments corresponds to high and low sea-level stands, respectively. In contrast, Quaternary sediments in the plains landward of the bay are predominantly terrestrial. By late Pleistocene time, the high sea level associated with the Sangamon (about 125,000 years ago) interglacial resulted in deposition of the Yerba Buena Mud (Sloan, 1992). The Yerba Buena Mud was deposited in an estuarine environment similar in character and extent to the present bay. Sea level lowering associated with the onset of Wisconsin glaciation exposed the bay floor and resulted in terrestrial sedimentation, such as the Colma formation, on the Yerba Buena Mud. Sea level rose again starting roughly 20,000 years ago, fed by the melting of Wisconsin-age glaciers. The sea re-entered the Golden Gate about 10,000 years ago (Atwater, 1979). Inundation of the present bay resulted in deposition of estuarine sediments, called Bay Mud, which continue to accumulate. Historical development of the San Francisco Bay area resulted in placement of artificial fill material over substantial portions of modern estuaries, marshlands, tributaries, and creek beds in an effort to reclaim land (Nichols and Wright, 1971).

2.2 Site Geology

General geologic features pertaining to the project site were evaluated by reference to the Geologic Map of the United Stated Geological Survey (USGS, 1998), as shown in Figure 2. The site is primarily underlain by artificial fill over tidal flats (Qaf/tf) in the north and northeast of the project site along the creek bank. This artificial fill (Qaf/tf) consists of clay, sand, silt, rock fragments and man-made debris. Close to the central portion to the west of the project site, there is the ravine type of fill formation (Qsr) which consists of stony silty to sandy clay, mostly moderately consolidated sandstone, siltstone, shale, and conglomerate. Franciscan Formation sandstone and shale (KJsk) exists from the middle to the south of the project site.

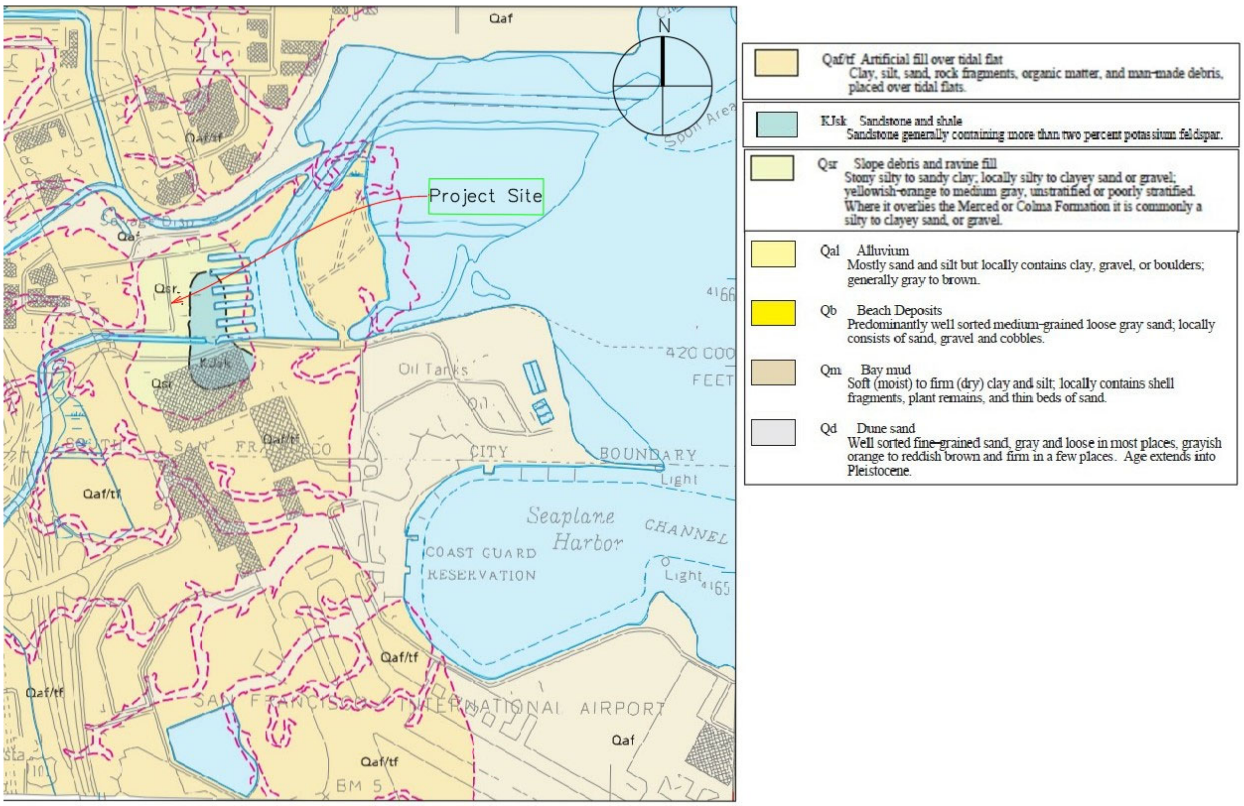


Figure 2 - Geological map adapted from the United States Geological Survey (1998)

3 GEOLOGIC HAZARDS

Geologic hazards in the region include earthquake faulting, ground shaking, and ground liquefaction.

3.1 Fault Rupture Hazard

The major Bay Area faults are shown on Figure 3. The closest active fault to the project site is the San Andreas fault, located approximately 3 miles southwest of the site. The site is not located within a state designated Alquist-Priolo Earthquake Fault Zone or a Santa Clara County fault rupture hazard zone. No known surface expression of active faults is believed to cross the site, and therefore fault rupture hazard is not anticipated.

Figure 3 - Major faults near the project Site

3.2 Strong Ground Shaking

The San Francisco Bay Area is one of the most seismically active regions in the United States. Significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas fault system, which regionally trend in a northwesterly direction. The Andreas Fault, which generated the great San Francisco earthquake of 1906, is located approximately 3 miles southwest of the site.

According to Earthquake Outlook for the San Francisco Bay Region 2014–2043, shown on Figure 4, published by USGS (https://pubs.usgs.gov/fs/2016/3020/fs20163020.pdf), a new model for estimating earthquake probabilities was developed by the 2014 Working Group on California Earthquake Probabilities that updated the 30-year earthquake forecast.

According to this model, the likelihood of earthquakes with the magnitude of 6.7 or greater in the next 30 years (starting from 2014) has been anticipated. The tabulated values on Figure 4 below represent the likelihood of having one or more earthquakes in the next 30 years (starting from 2014).

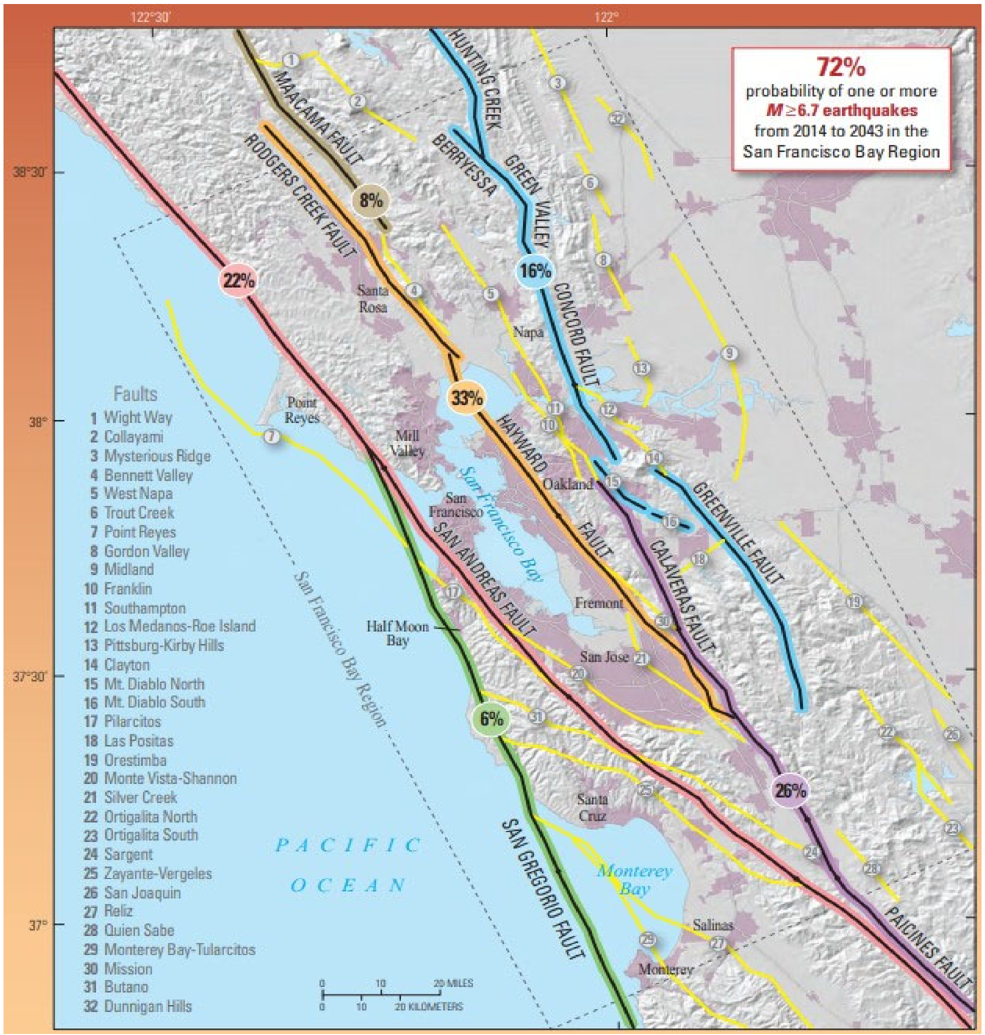


Figure 4- Earthquake Outlook for the San Francisco Bay Region 2014–2043

The peak earthquake ground motions were estimated by Ground Motion Interpreter (CGS on-line tool) for return periods of 2% and 10% in 50 years as 0.931g and 0.545g respectively. (https://www.conservation.ca.gov/cgs/ground-motion-interpolator)

3.3 Liquefaction

Liquefaction is defined as a loss of strength of saturated cohesionless soil caused by seismic shaking. Soil types most susceptible to liquefaction are loose, saturated silt to fine clean sands. The United States Geological Survey (USGS) has mapped the site area as having quintenary unit afem (artificial fill estuarine mud) with a very high potential for seismically induced liquefaction. In general, it is anticipated that liquefaction may occur at some locations. Bay Mud sediments (shells, etc.) and some of the loose and medium dense coarse grain fill and alluvium may liquefy during a large seismic event. The USGS liquefaction susceptibility map shown on figure 5 indicates that the subject site is locate in very high liquefaction-susceptible zone. A discussion on liquefaction analyses based on the available subsurface data collected from previous geotechnical reports is provided in the subsequent section.

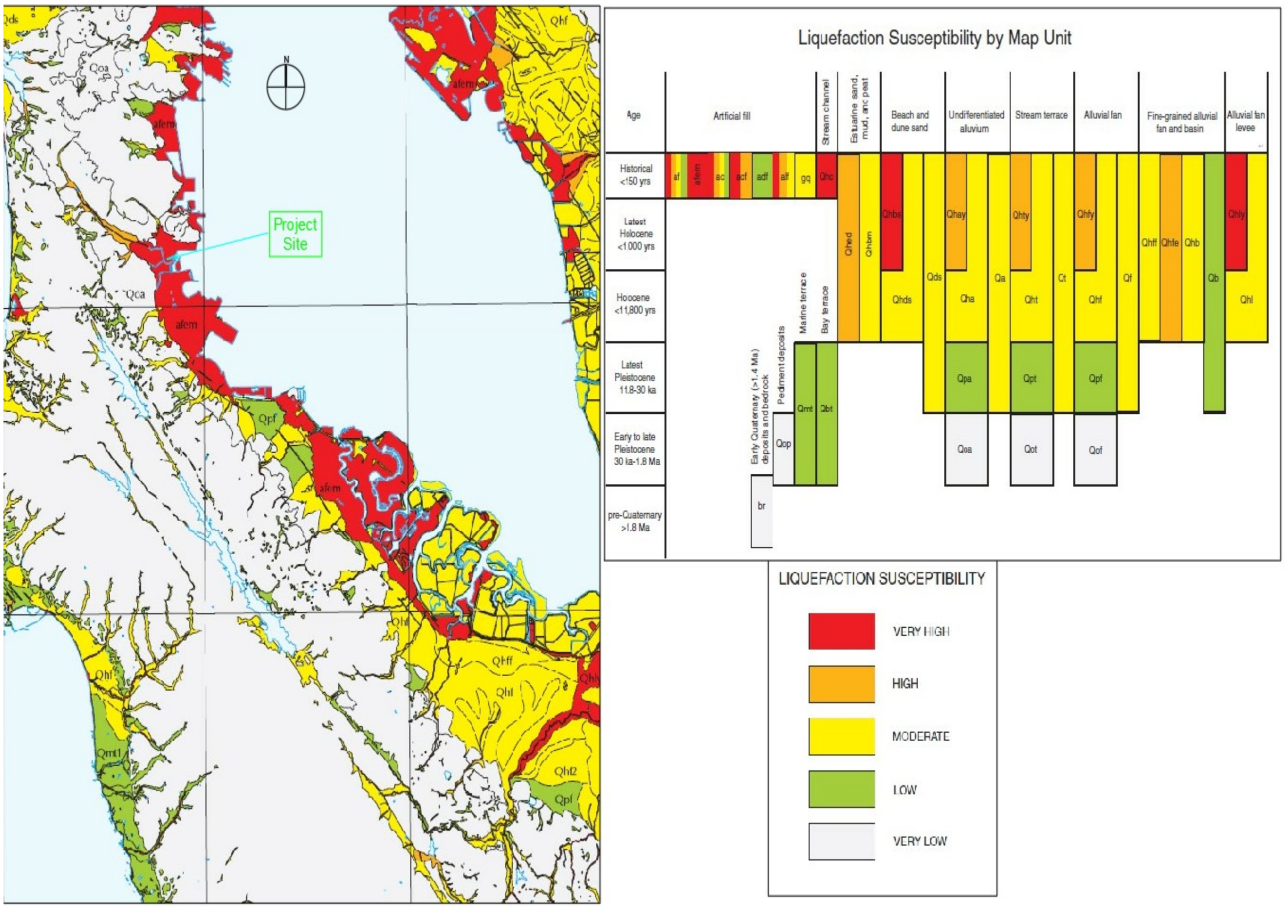


Figure 5-USGS Liquefaction Susceptibility Map, 2006

3.4 Landslide Hazard

The topography map of the subject site indicates that the site is generally located in flat areas around the perimeter of the Lower Colma Creek and the bay margin; therefore, the slope instability is not likely to occur. The site is not located within an area zoned by the State of California as having potential for seismically induced landslide hazards. Therefore, seismic landslide hazards are not anticipated to impact the project area.

3.5 Subsidence Hazard

Subsidence is a gradual sinking or caving in an area and it is most often caused by removal of groundwater, oil, natural gas, or other mineral resources out of the ground by pumping, fracking, or mining activities. Subsidence can lead to an increased risk of flooding, saltwater intrusion into groundwater, and damage settlement-sensitive infrastructure and utilities. The subsidence in the

South Bay area was generally a result of the over-extraction of groundwater (Freeze and Cherry 1979) largely due to agricultural pumping in the early part of the 1900s.

Santa Clara Valley Water (SCVW) conducted a benchmark elevation survey[1] in 2019 which includes surface elevation data from 138 benchmarks to evaluate the spatial variability of land subsidence. The survey results revealed that subsidence did not exceed 0.01 feet per year.

Another study conducted by NASA Earth-Science and Research program2 (2018) reveals that local land subsidence exacerbates inundation risk to the San Francisco Bay Area. Figure 6 is the subsidence map developed by NASA (2018) shows the subsided areas in the bay area. According to this map, the estimated vertical subsidence per year for the project site is about 2 to 3 mm (0.07-0.11 inches). This is consistent with the SCVW survey results (2019).

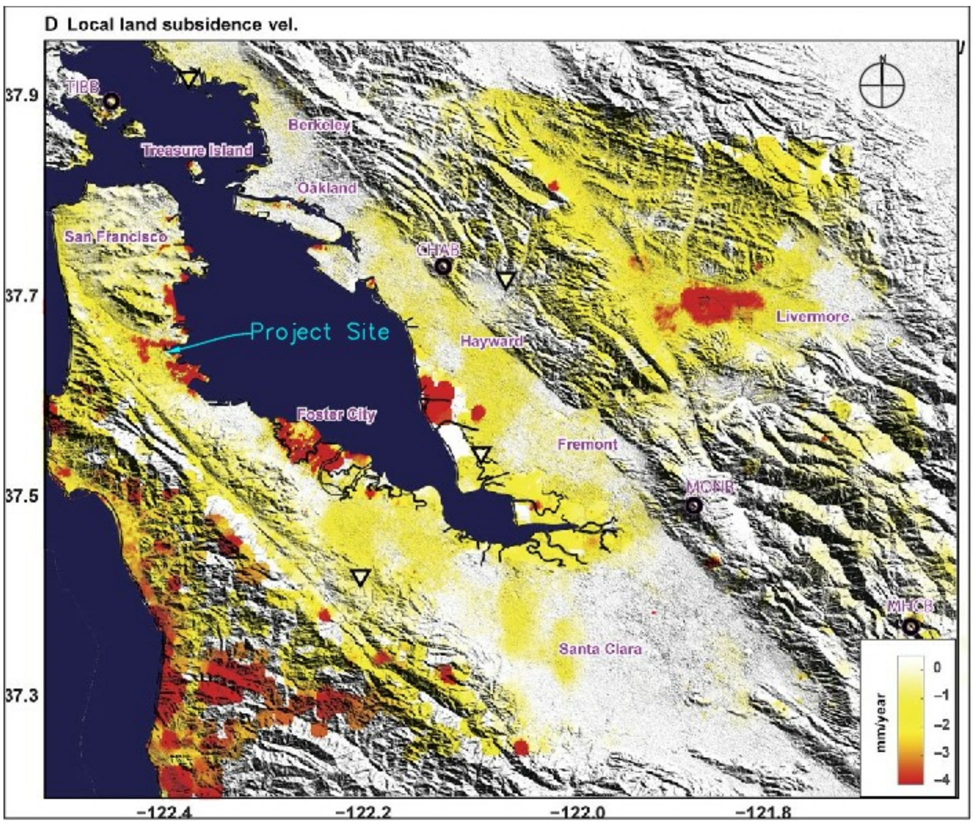


Figure 6- NASA Subsidence Map

4 SUBSURFACE DATA AND SITE CONDITION

- Geotechnical Investigation by Harza Engineering Company, 1998.
- Geotechnical Study Update by Fugro Consultants Inc, 2012.
- Geotechnical Study by Fugro Consultants Inc, 2018

[1] Annual Groundwater Report 2019 by Santa Clara Valley Water, Ch.4, page 42
[2] Source: https://science.nasa.gov/earth-science/programs/research-analysis/year-in-review-2018/san-francisco-flood-risk

The subsurface conditions at the northern portion of the site (along the Colma creek right bank) consist of existing fill (af) overlying YBM deposits (Qybm), which in turn, overlie alluvium (Qal). Fill was encountered in the Harza borings (EB-28, EB-26, EB-23, EB-15) and Fugro (B-04, B-01, B-05), extending to depths ranging from about 5 to 11 feet. The existing fill generally consisted of medium stiff to very stiff lean clay, with varying amounts of sand and gravel. At the south and southwest portions of the site, the fill encountered within the borings extended to depths ranging from about 3 to 6 feet below the ground surface. At the East portion of site, the near surface soils condition consists of fill extended to depths ranging from about 2½ to 10 feet. These fills generally consisted of heterogenous mix of dense gravelly sands with varying amounts of silts and clays. The Plasticity Index (PI) within the fill typically ranged from about 5 to 24. Underlying the fills, soft silty clay locally known as Bay Mud (BM) was encountered. The thickness of the bay mud varies from one area to another within the site. Generally, the thickness of BM increases from the northern portion (along the creek bank) to the eastern portion closer to the bay. The thickness of bay mud ranged from about 13 to 16 feet near borings EB-28 and EB-15. The locations of the exploratory borings drilled by Harza, Fugro and Woodward-Clyde-Sherard are shown in Figure 7.

Alluvial deposits were encountered beneath the YBM and generally extended to the bedrock depth explored. These deposits generally consisted of over-consolidated medium stiff to very stiff lean and fat clay to sandy lean clay with some relatively thin, isolated layers of loose to dense silty sand and clayey sand. The soils encountered in Harza's borings were generally consistent in material type with those encountered in the Fugro (2012) explorations to the depths explored.

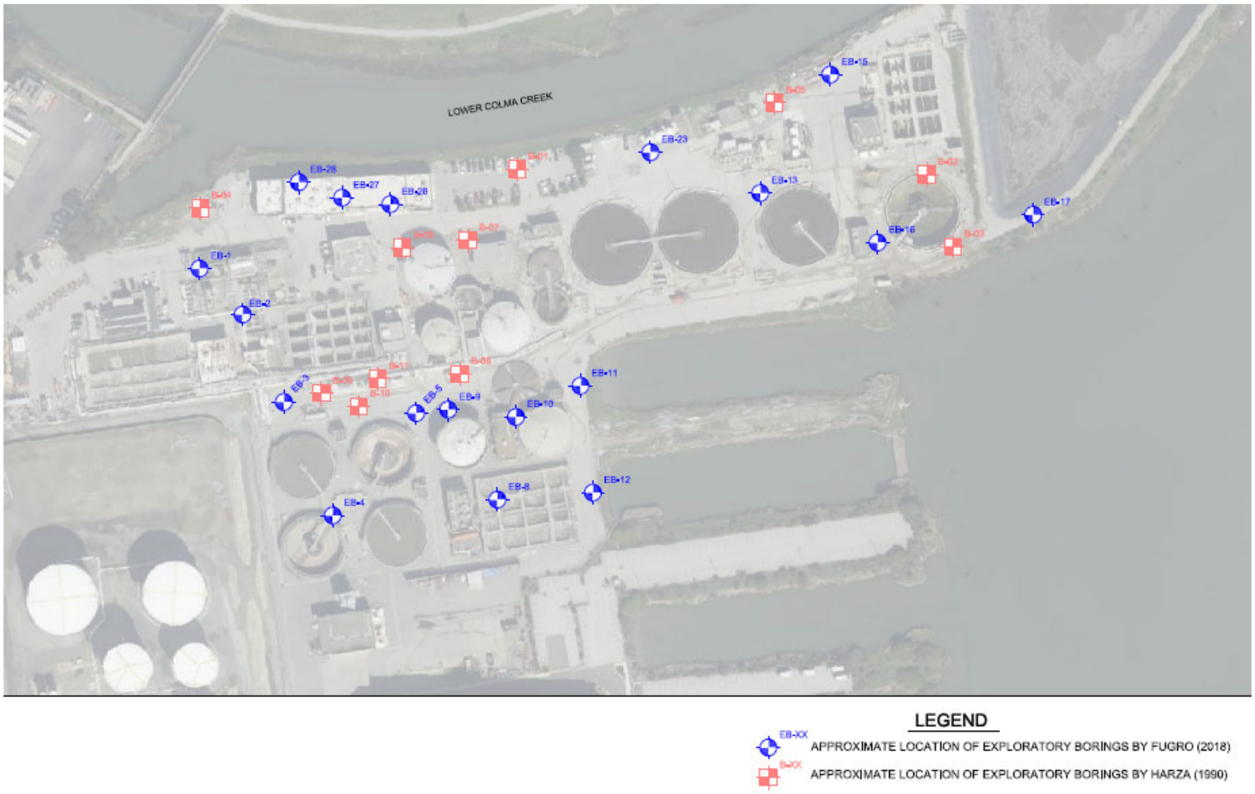
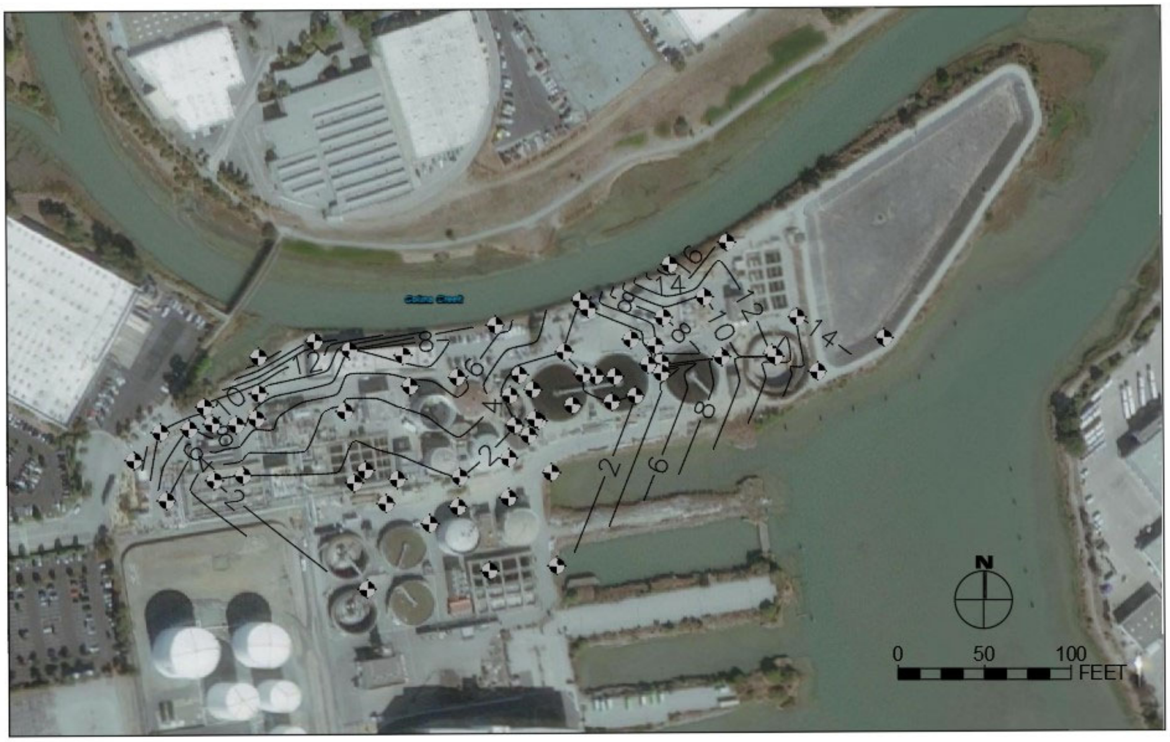


Figure 7 - Approximate Boring Locations

4.1 Bay Mud

Bay Mud thickness is judged to be the most important geotechnical aspect affecting the cost of proposed alternatives. The thickness of the Bay Mud was estimated based on the data collected from standard penetration testing (SPT) explorations previously conducted by Fugro and others, and engineering judgment. The interpretation is shown on Figure 8.



BAY MUD THICKNESS CONTOUR MAP

Figure 8- Preliminary Bay Mud Contour Map

4.2 Groundwater

Along the San Francisco Bay and Pacific Ocean shorelines as well as adjoining flatlands, historic high groundwater levels are shallow (0 to10 feet below the surface) reflecting the neighboring open water. Groundwater near the northern portion of the site (along the creek) was encountered in the borings B-4, B-1, and B-5 at depths of about 9 to 15 feet deep below ground surface. The groundwater in the southern portion of the site was encountered deeper (about 30 feet below the ground) than the areas adjacent to the creek and the Bay. However, groundwater may fluctuate over time due to rains, tides, nearby construction, irrigation, and other man-made and natural influences. This is not problematic to the selected alternative, since the use of pumps to drain groundwater would be sufficient in this scenario.

5 SETTLEMENT

The project area is underlain by approximately 2 to 16 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. The amount of settlement directly depends on the thickness of the bay mud. Figure 8 shows the approximate thickness of the bay mud for the project site. The thickness of Bay Mud varies at different locations. Generally, it increases towards the bay side.

As discussed in section 3.3, there is a potential for liquefaction at the site. A preliminary liquefaction analysis indicates that the site may experience 2 to 6 inches of liquefaction-induced settlement.

6 PROPOSED CSRM MEASURES

Proposed construction consists of non-structural measures and Coastal Storm Risk Management (CSRM) measures which include concrete cantilever floodwalls, combination of floodwall over the compacted engineered fill berm/levee (Hybrid floodwall), etc. Several non-structural and structural alternatives were proposed during the feasibility study. Some of these alternatives such as flood net were screened out due to the cost and possibility of providing reliable line of flood protection. The concrete cantilever floodwall appears to be one of the CSRM alternatives that is considered along the right bank of lower Colma creek. Construction of a new levee with 11 feet of height and slopes of 3:1 is not feasible due to the site space limitation.

6.1 Nonstructural Measures

Nonstructural measures consist of various measures to reduce flood risk and flood damages incurred within floodplains. Some of the non-structural measures includes elevation, fill basement with main floor addition, relocation, acquisition, dry flood proof, wet flood proofing, ring levees, flood insurance, and flood warning system, etc. Some of these non-structural measures may not be considered feasible or suitable in reducing risk of flood and damages. Therefore, only non-structural measures that are likely to reduce flood damages or flood risk have been considered in this report.

6.1.1 Elevation of Critical Assets

This non-structural measure is a viable measure in reducing the flood damage by raising some of the critical assets above the flood inundation plane. However, some of the critical and sensitive infrastructures such as the pump station and the electrical equipment placed and operated at low lying areas (Elev. – 2 to + 4 ft.) are more prone to the risk and such structures may not be feasible to be elevated above the flood plain. It is recommended that a positive drainage be considered when building an elevated earthen platform.

6.1.2 Wet/Dry Proofing

Waterproofing the key and essential structures could reduce the risk of flood damages. Installation of waterproofed doors/windows, use of waterproof materials or membranes to watertight structures and installation of drainage with submersible pumps for discharging the floodwater is feasible. Such measures may considerably reduce the damage to the key structures during flooding events.

6.1.3 Relocation

The relocation of the entire treatment facility plant or the partial relocation of the components of the plant to a higher ground location is not economically and practically feasible. Any vulnerable components of the plant to the flood event must stay in place for the system process.

6.1.4 Flood Warning System

A flood warning system, when properly installed and calibrated can identify the amount of time available for residents to implement emergency measures to protect valuables or to evacuate the area during serious flood events. Stream monitoring stations on Colma creek and San Bruno creek were improved to provide data on rainfall and real-time in-channel conditions in the region. A flood early warning webpage will be fully operational by the end of year 2021. Once it is ready, the emergency personnel and the community would be able to receive flood condition notifications via text or email.

6.1.5 Ring Levees

Construction of earthen ring levees around the entire project site is considered a non-structural measure, and it may reduce the risk of flood. This measure requires importing soil from nearby borrow sources. The amount of borrow soil material needed for construction of the levee system depends on the height, footprint, and topography of the site. Such non-structural measure may not provide a line of protection against coastal flooding and is not considered to be cost effective; however, building a ring of levees comprised of low permeable soil around critical equipment or structures such as a pump station may prevent any flood damage, and it would be economically feasible.

6.2 Structural Measures

CSRM alternative measures are the structures that provide a relatively reliable line of protection against coastal flooding as long as the structures are designed and constructed according to the current standard engineering design criteria. There are various CSRM alternatives proposed for the project site which include Concrete Cantilever Flood Walls and Sheet Pile Walls.

6.2.1 Floodwalls

A floodwall is considered to provide CSRM benefits to the feasibility study. The alignment for this floodwall starts from an area at the northwest near the San Francisco Trail Bridge to a higher ground area near the retention pond to the east. The proposed floodwall is considered to reduce the risk of 100-year flood and provides a line of protection to the treatment plant facility buildings and equipment. In areas where the construction of an inverted T-shape cantilever floodwall is not feasible due to space constraints, an I-wall type floodwall could be constructed. Figure 9 shows areas where floodwall need to be constructed.

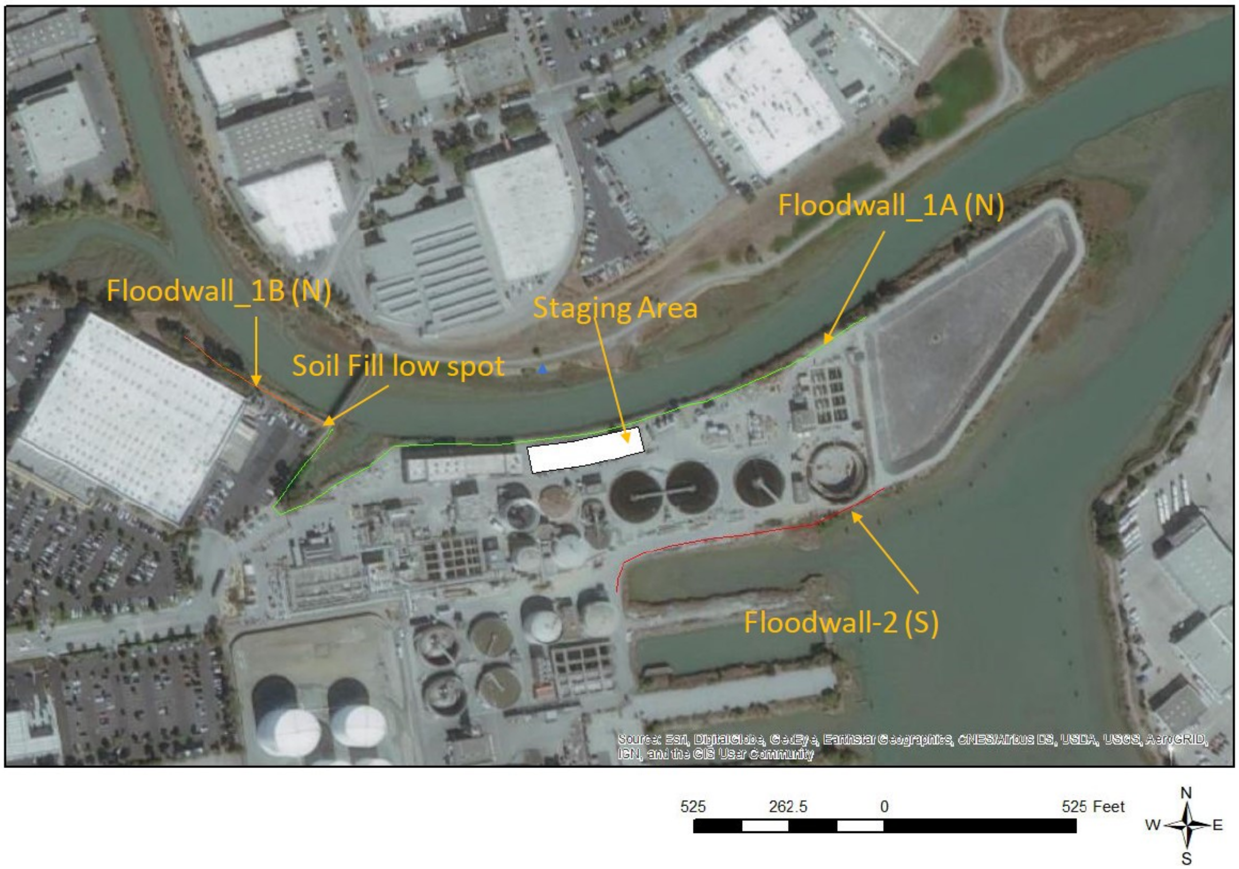


Figure 9 - Floodwall Alternatives

6.2.1.1 Inverted T-Cantilever Wall

Inverted T-cantilever flood walls are reinforced concrete walls (cast-in-place) that utilize cantilever action to retain the mass behind the wall. Reinforcement of the wall is attained by steel bars embedded within the concrete or block core of the wall. Stability of this type of wall is partially achieved from the weight of the soil on the heel portion of the base, and the weight of the wall. This type of flood wall may be supported on a deep foundation when weak and soft clays are encountered. However, this type of wall is only considered for the pump station #4 due to the presence of existing underground utilities at the pump station.

6.2.1.1.1 Stability Analysis

This type of floodwall may be considered along the Lower Colma Creek right bank and around the project site where there is no space constrains for the wall footprint. The design calculations and dimensions of this type of floodwall are presented in Appendix A. The stability of wall against overturning and sliding were checked and the Factor of Safety for the bearing capacity was determined to be over 3.

The static methods used in stability analyses of the floodwall structures and combination of applied loads, including uplift forces due to hydrostatic pressures in the foundation material, were considered in the stability analysis. Higher factors of safety would be considered for the areas with less degree of confidence where there is no or little subsurface data (at the northeast side of the project) available. The factor of safety may also be adjusted based on the probability of loading conditions. Since the PMF (or 90% PMF) event is not usual, the factor of safety would be less than any usual or normal events.

6.2.1.1.2 Settlement

The consolidation analysis revealed that the inverted T-cantilever flood walls would settle up to 2 inches depending on the thickness of Bay Mud. Any settlement greater than 2 inches would develop cracks in the concrete wall which would adversely affect the function of the flood wall. Table 1 shows approximate settlements of cantilever wall under various bay mud thickness and fill load conditions for a wall cross section along the Lower Colma right bank. The depth of excavated continuous foundation footing for the proposed flood wall along the Lower Colma Creek, where the thickness of the young bay mud is expected to be about 10 feet, should be least 4 feet. The width of the footing is designed to be 8 feet and the height of the wall from the existing ground elevation is about 7 feet (about Elevation 16 feet). The design assumptions and calculations are attached to this report in Appendix A. For the pump station No.4, the proposed floodwall around the perimeter may be either an I-wall sheet pile or a conventional cantilever floodwall or retaining wall. The schematic with the footing dimensions is included in Appendix A.

Table 1- Estimated Settlement

Thickness of Bay Mud (ft.)	Settlement (in)
2	0.3
4	0.6
6	1.0
8	1.3
10	1.7
12	2.0
16	2.6

The soils at the excavated bottom of the continuous footing for the floodwall needs to be compacted. The compaction and densification of fill soils is anticipated to reduce the amount of settlement.

6.2.1.1.3 Seepage

Seepage analysis was performed using SEEP/W computer software program at a cross section of the continuous floodwall footing. The result revealed that the seepage was not significant and would not be an issue due to low exit gradient and low permeability of Bay Mud blanket thickness as shown in Figure 8.

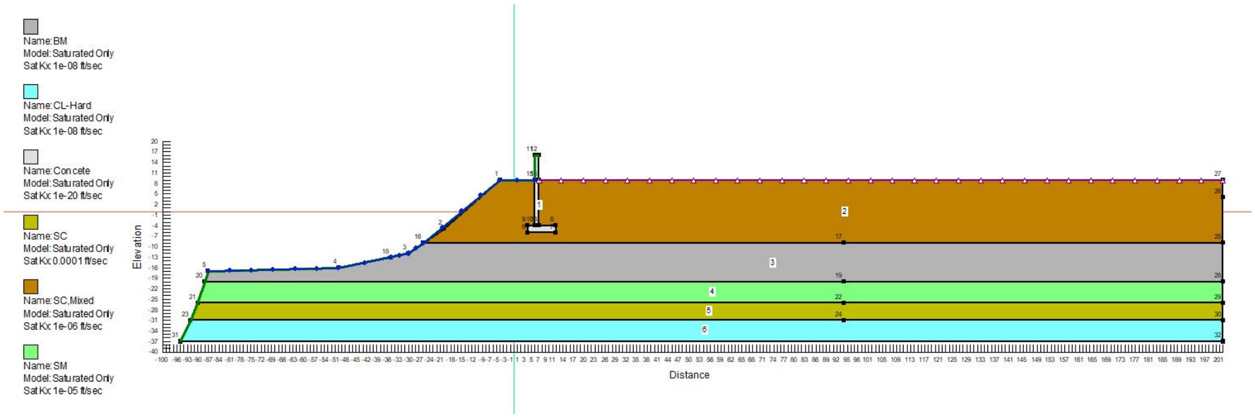


Figure 10 - Seepage Model and Soil Profile

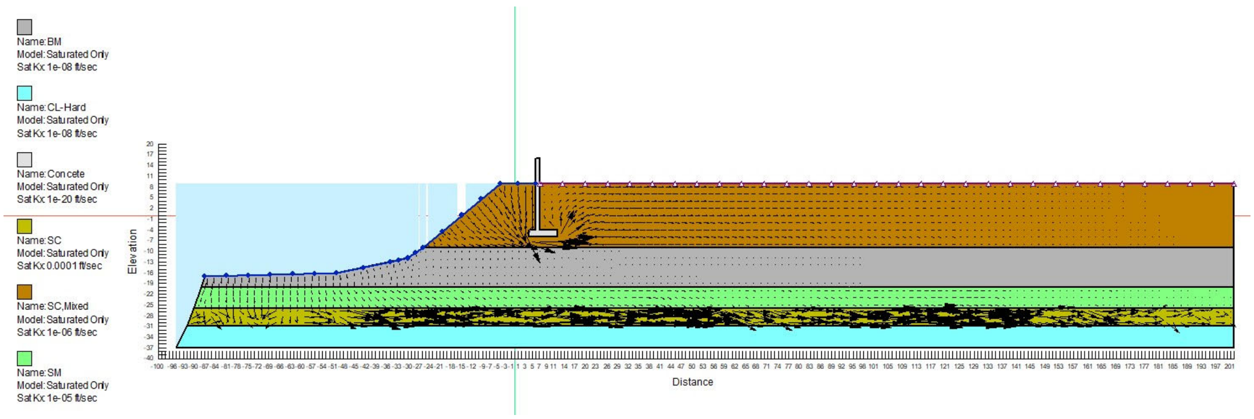


Figure 11- Seepage Model Analysis

6.2.1.2 I-Floodwall with Sheet Piles

An I-wall is defined as a slender cantilever wall, deeply embedded in the ground or in an embankment. The wall rotates when loaded and is thereby stabilized by reactive lateral earth pressures. The minimum embedment depth in accordance with EC 1110-2-6066 was calculated as the exposed landside height of the wall times 2.5. Therefore, the embedment depths should be at least 7.5 feet below the highest ground elevation and 10 feet below the lowest ground elevation along the proposed alignment. I-floodwalls need to have concrete encasement to provide corrosion protection for the sheet piling in accordance with EC 1110-2-606. Most of the I-floodwalls, including NOLA I-walls, have concrete cap/encasement that provides protection against the corrosion for the sheet piling. The stability and seepage analyses for the I-wall are evaluated in the subsequent sections.

6.2.1.2.1 Stability

CWALSHT was used to compute the depth of embedment, bending moment, and scaled deflection of I-wall sheet pile. I-Wall stability was analyzed for potential failure modes in accordance with Engineering and Design of I-Walls (EC 1110-2-6066).

Two load condition categories based on return periods were considered:

1) Unusual loading refers to operating loads and load conditions that are of infrequent occurrence. For example, Hurricane loading is classified as unusual for most I-walls. The unusual event corresponds to an annual exceedance probability less than 0.10 but greater than or equal to 0.00133. The water level was assumed to be lower than the top of I-wall elevation (at about 13 feet).

2) Extreme Loading refer to events, which are highly improbable and can be regarded as emergency conditions. The extreme event corresponds to an annual exceedance probability less than 0.00133. The water level was assumed to be at the top of the I-wall elevation (about 15 ft) for such an extreme loading condition.

According to EC 1110-2-606, the embedment of I-wall for the exposed landside height of 6 feet will be at least 15 feet. The CWALSHT computation results reveal that the wall deflections will be acceptable, and the section of PZ-27 seemed to be appropriate based on the calculated required section modulus. The detail CWALSHT calculations are included in Appendix A.

6.2.1.2.2 Seepage

A seepage analysis was performed using SEEP/W program at a cross section where I type floodwall would likely be required due to the site space limitation. The sheet pile floodwall will be constructed in between and tied into the inverted cantilever concrete walls. The gaps in the tie-in locations or between the structure and soils may create seepage paths and piping issues. The gaps need to be properly filled with competent material or soil, and soil fill needs to be compacted.

6.2.1.2.3 Gap-Soil-Sheet Pile analysis

I-wall stability should be analyzed for a full range of failure modes. One of the failure modes includes a flood-side gap between the soil and I-wall with hydrostatic pressure acting along the full gap depth. Computer software Slope/W was used to determine the slope stability and hydrostatic pressure between the soil and I-wall. Since the ground at the landside of the I-wall is relatively flat, the slope stability is not a concern. The result of SEEP/W analysis performed for this failure under an unusual hydraulic loading condition shows that the seepage is not significant and will not be an issue due to low exit gradient (less than 0.5) and low permeability of bay mud blanket as shown in Figure 12.

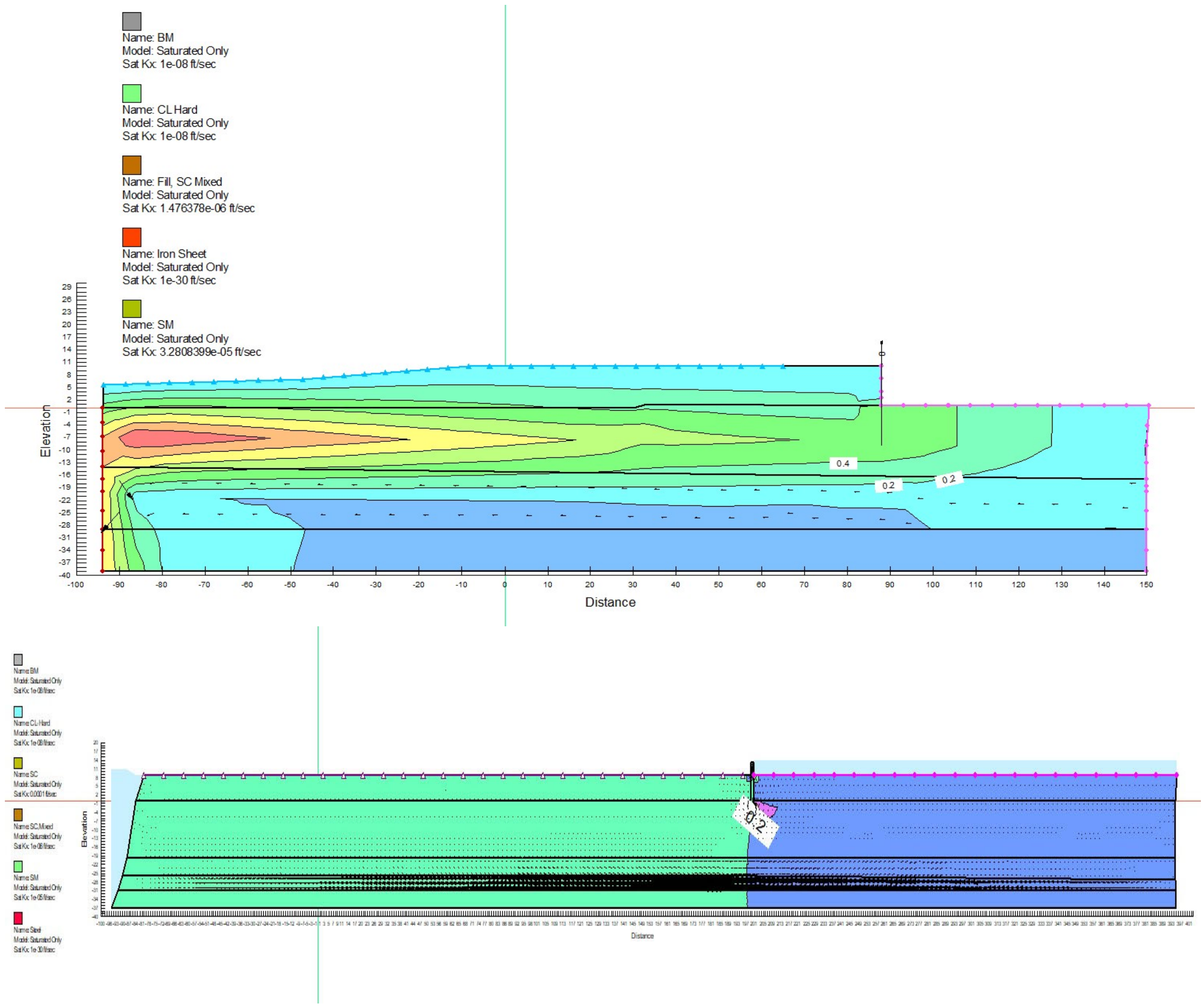


Figure 12 - Seepage Analysis at Gap-Soil Sheet Pile

6.2.2 Floodwall/Retaining Wall around Pump Station No. 4

An Inverted T-shape floodwall/retaining wall could be considered at the perimeter of Pump Station no. 4 as shown in Figure 13. The dimensions of the shallow foundation for the wall are included in Appendix A. The height of the wall should be at least 2 feet above the ground to prevent any overtopping expected during the medium scenario of SLR; however, the height of the wall can be extended to 4 feet to prevent the overtopping due to the most severe high-water events (500-year) or a 50-year flood event with intermediate sea level rise as shown in Figure 14.

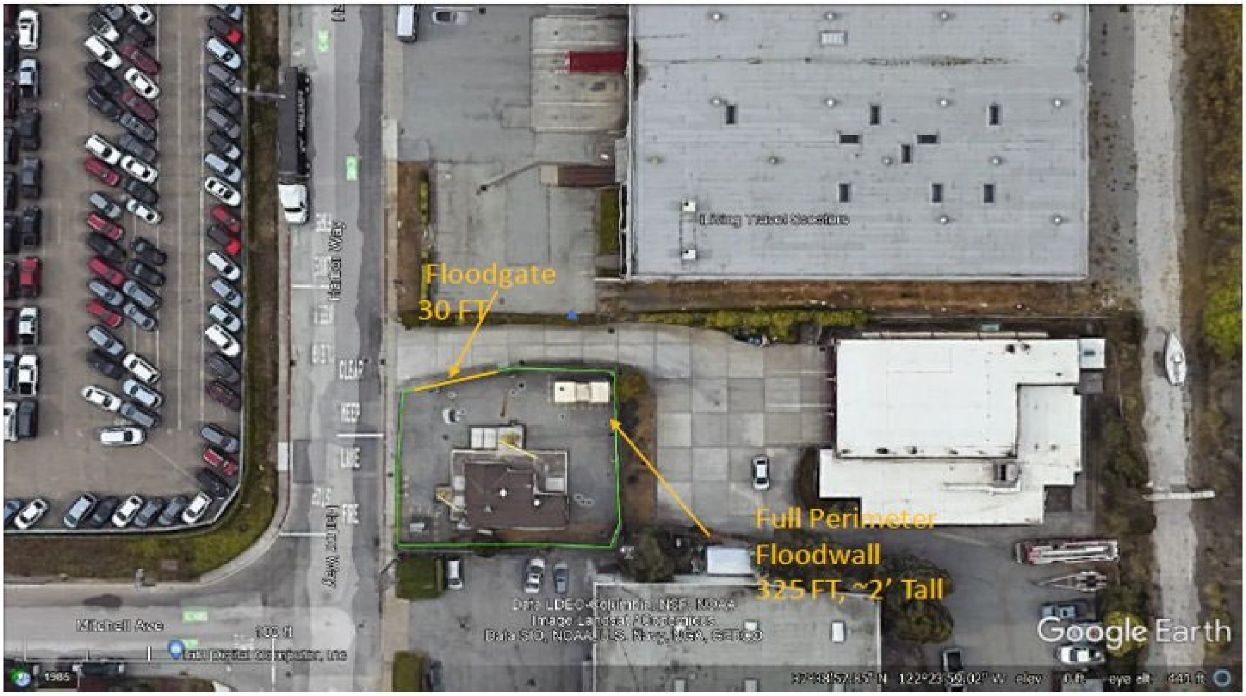


Figure 13-Pump Station No.4 with Proposed Floodwall/Retaining wall

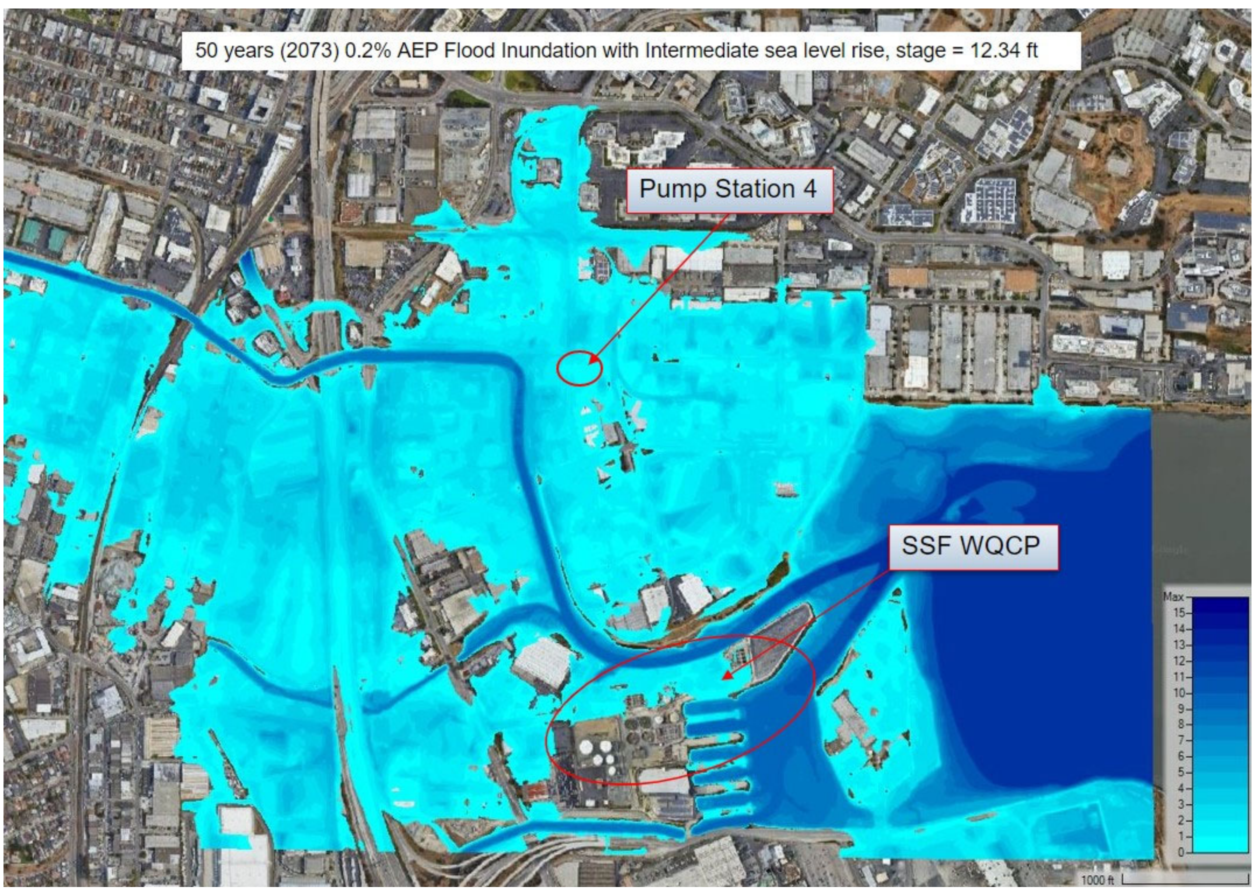


Figure 14- Map of Flood Inundation at Pump Station No. 4 (50-year Flood Event)

6.2.2.1 Constructability of Floodwall/Retaining Wall around Pump Station No. 4

The existing underground utility lines crossing the proposed floodwall alignment are expected to be as shallow as 2 feet (electrical conduits) below the ground surface (bgs) and as deep as 9 feet bgs for the other utility lines (sewer) according to the project utility plans provided by the local sponsor. If any existing utility lines encountered during the shallow foundation excavation (within 2 feet), they should be either relocated or properly encased to avoid any damage. Prior to any excavation, the existing utilities should be properly located and marked by Underground Service Alert (USA). The construction of sheet pile floodwall may not be economically feasible due the existing utilities crossing the proposed foundation alignment at depths from 2 to 9 which interfere with the depth of embedment of sheet piles. The risk and cost to relocate or bypass the existing utilities interfering with the sheet pile alignment depth (about 10 ft) is not justified for this study.

APPENDIX A

- Stability Calculations for Cantilever Floodwall
- Settlement Calculations for Cantilever Floodwall
- Sheet Pile Stability Analysis with CWALSHT

Stability Calculations for Cantilever Floodwall

Soil Parameters Input																Water Heads			
Yt (LF)	ϕ (LF)	Ka	Kp	Ysat (pcf)	Y' (pcf)	Yt (BM)	ϕ (BM)	Ysat (BM)	δ (deg)	C (lbs/ft^2	Ycon (pcf	B (ft)	Thickne	Df (ft)	H (ft)	h1 (ft)	h2 (ft)	Yw (pcf)	tw
101	34	0.28271492	3.537132	120	57.6	50	10	45	6.66667	500	150	8	2	6	14	6	10	62.4	1.5

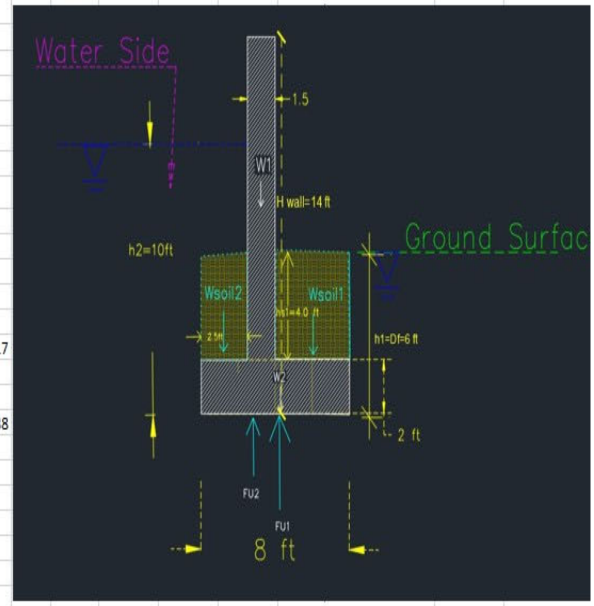
Vertical Forces (lbf)		Horizontal Forces (lbf)								
W1	2700	At Right Pp	3667.298	passive ultimate ca		e	0.00351			1.5
W2	2400	Pw	1123.2			Pmax	607.896	These formulas need correcting		
Fu1	2995.2	sliding	566.9305	1833.65	allowable	Pmin	604.704			
Fu2	998.4	At Left Pa	130.275							
W1 (soil)	921.6	Pw	3120							
W2 (soil)	576	ƐF right	4790.498							
W1 (water)	998.4	ƐF left	3250.275							
W2 (water)	1248	Net	1560.145	(not counting passive)						
N'=ƐV	4850.4	passive required to	1560.145	**OK**		FS (bearing	40.0409	OK		

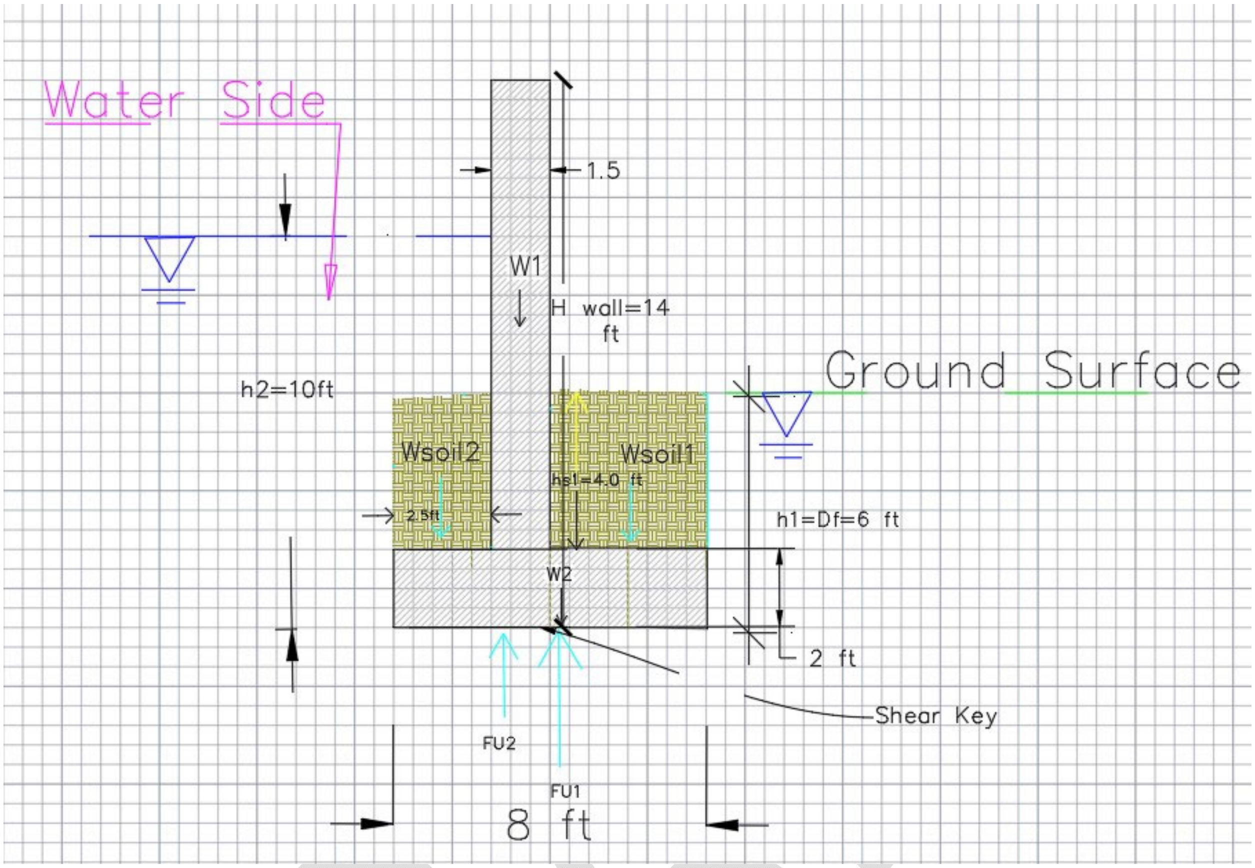
XR	4.00351 OK
B'	7.99298

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

Forces	M. Arm (ft)	Moments			
W1	3.25	-8775	clockwise		
W2	4	-9600	clockwise		
Fu1	4	11980.8	counter clockwise		
Fu2	2.60	2595.84	counter clo	Bearing Factors for Phi=34	
W1 (soil)	6	-5529.6	clockwise	Nc	42.16
W2 (soil)	1.25	-720	clockwise	Nq	29.44
W1 (water)	5.75	-5740.8	clockwise	Ny	41.06
W2 (water)	1.25	1560	clockwise	Other Factors	
Soil Pressu	2	3120.289167	counter clo	Fci=Fqi	1
Pw(Right)	2	2246.4	counter clo	Fcs	1.00001
Soil pressu	2	-260.55007	clockwise	Fqs	1
Pw(Left)	3.3	-10296	clockwise	Fys	1
				Fqd	1
	Sum	-19418.6209	Clockwise	Fcd	1
				Fyd	1
				Fyi	1
				q	-104.4
				Y'	38.6

qu	24340.7
Ratio XR/B	0.50088
e	
qmin	
	1.33
	34205.6
FS (against overturning)	1.17171





Notes:

Width of the straight footing base is to be a minimum of 7 feet. The 8-foot footing was evaluated for global instability and seepage; the same analysis and evaluation need to be performed for a 7-foot footing to ensure its stability.

Footing depth varies depending on the bay mud depth, anticipated to be around 4 feet below the existing ground. For soft ground (marsh land) is 6 feet. If bay mud is encountered at the anticipated footing depth, it should be removed and replaced with a layer of gravel.

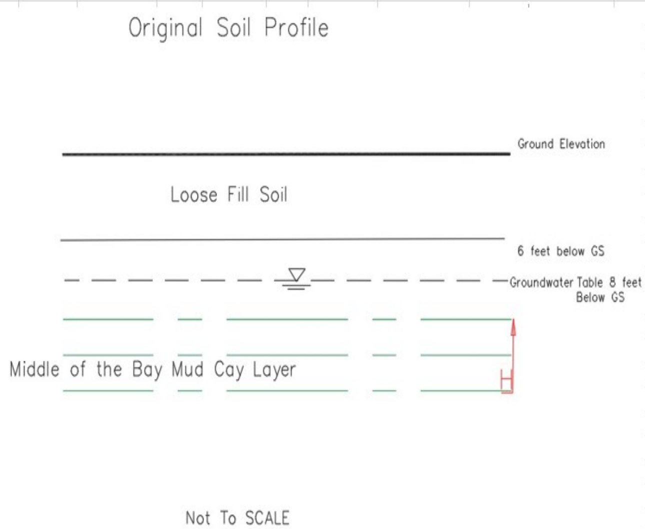
Height of the wall from the bottom of the footing to the top of the wall varies (top of the wall elevation varies from 8 to 11 feet).

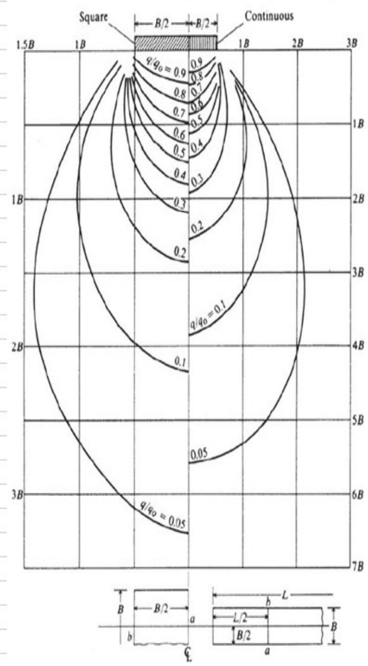
Thickness of the wall is to be 1.5 feet.

Concrete wall needs to be reinforced to avoid tension cracks.

Settlement Calculations for Cantilever Floodwall

				Soil Profile Prior to constructic			Assumptions	GWT	:at Existing Ground Surface				Assumptions for Wall					
								BM thickness: 10 feet		encountered about 10 below existing (			To be constructed at a depth of 6' below GS				The elvation of original ground surf	
Y(soil), pcf	Ysat, pcf	Y(BM), pcf	Y(water	d, Depth tc	dc, der	U, psf	Ờo, effective stress at middle of clay						Weight of Concrete Wall + footing			Y(concrete)	Total Weight	Depth of Backfill
111	130	60	62.4	8	20	748.8	3308						B	8		150	525	
													W	2				
													H wall stem	12				
							Ờo',New effective stress after constructi			Ờf		Sc, settlement					Z	15 qo
							211.5			3519.5							I	0.3 157.
Y(backfill),	Y(backfill) sat		Ccε	Crε	eo	H clay,ft	Sc, Settlement (	Sc (inches)										
120	133.2		0.3	-	-	12	0.165341264	1.984095										





Boring	Cce, Modified	location
B-5	0.29	
EB-28	0.33	

CWALSHT computations for I-floodwall

Analysis Settings
SEEP/W Analysis
Kind: SEEP/W
Method: Steady-State
Physics
Water Transfer
Free convection: thermal effects: No
Free convection: solute effects: No
Vapor transfer: isothermal: No
Vapor transfer: thermal: No
Water Settings
Maximum Number of Iterations: 500
Maximum Difference: 0.005
Significant Digits: 2
Max # of Reviews: 10
Under-Relaxation Criteria
Initial Rate: 1
Minimum Rate: 0.1
Rate Reduction Factor: 0.65
Reduction Frequency (iterations): 10
Unit Weight of Water: 62.430189 pcf
Bulk Modulus of Pore-Fluid: 43,511,321 psf
Time
Starting Time: 0 d
Duration: 0 d
Ending Time: 0 d
Materials
SC, Mixed
Hydraulic
Model: Saturated Only
Sat Kx: 1e-06 ft/sec
Ky'/Kx' Ratio: 1
Rotation: 0 °
Volumetric Water Content: 0
Compressibility: 0 /psf

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* SUMMARY OF RESULTS FOR *
* CANTILEVER WALL DESIGN *

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I.--HEADING

'CANTILEVER WALL in Fill Soil, FS for both active and passive is 1.5

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
AND THEORY OF ELLASTICITY EQUATIONS FOR SURCHARGE LOADS.

LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
AND THEORY OF ELLASTICITY EQUATIONS FOR SURCHARGE LOADS.

WALL BOTTOM ELEV. (FT) : -5.30
PENETRATION (FT) : 5.30 Note: 5.30 ft below the Mudline elevation

MAX. BEND. MOMENT (LB-FT) : 4.3478E+03
AT ELEVATION (FT) : -1.38

MAX. SCALED DEFL. (LB-IN^3): 6.0223E+08
AT ELEVATION (FT) : 15.00
NOTE: DIVIDE SCALED DEFLECTION MODULUS OF
ELLASTICITY IN PSI TIMES PILE MOMENT
OF INERTIA IN IN^4 TO OBTAIN DEFLECTION
IN INCHES.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHOREDOR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 27-OCTOBER-2021 TIME: 13:49:14

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* COMPLETE OF RESULTS FOR *
* CANTILEVER WALL DESIGN *

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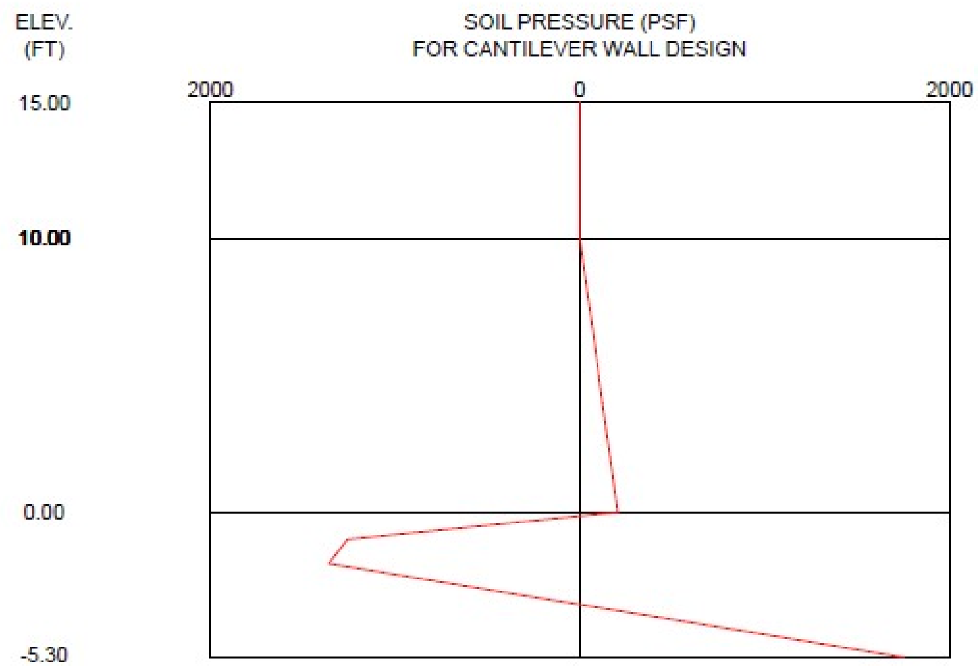
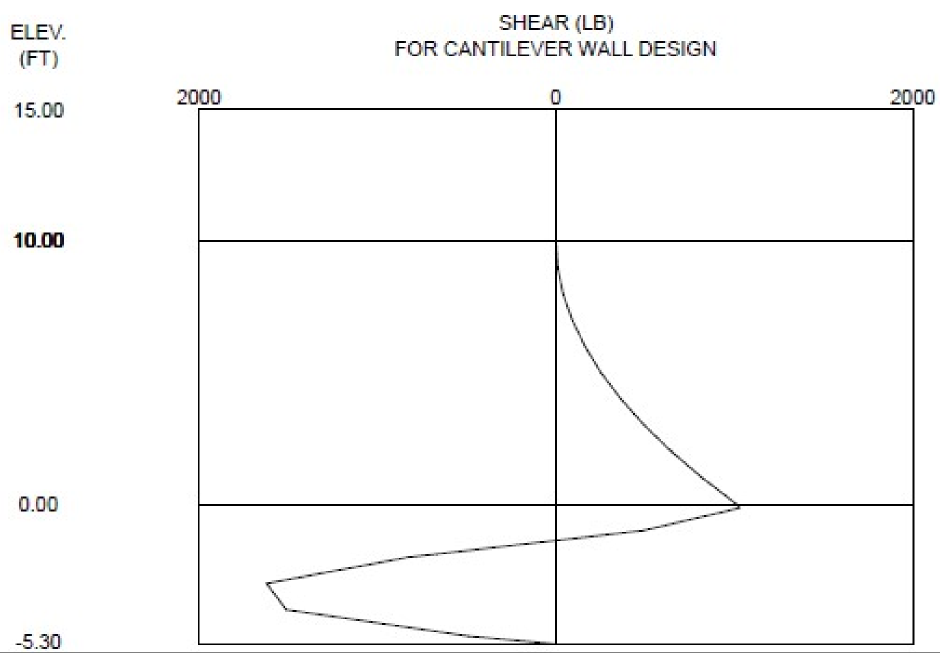
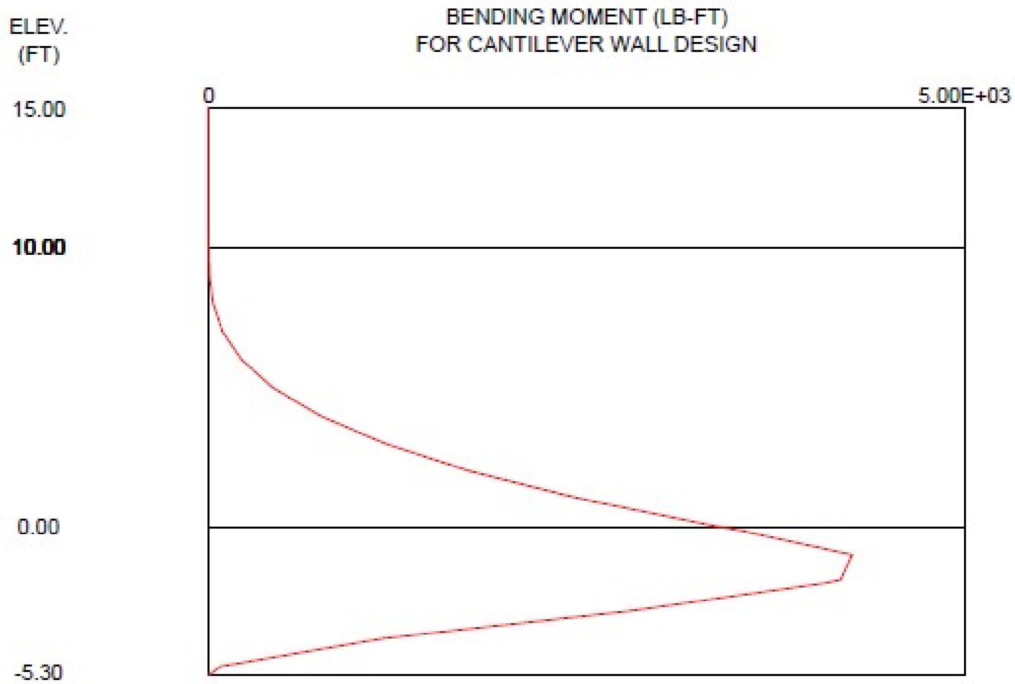
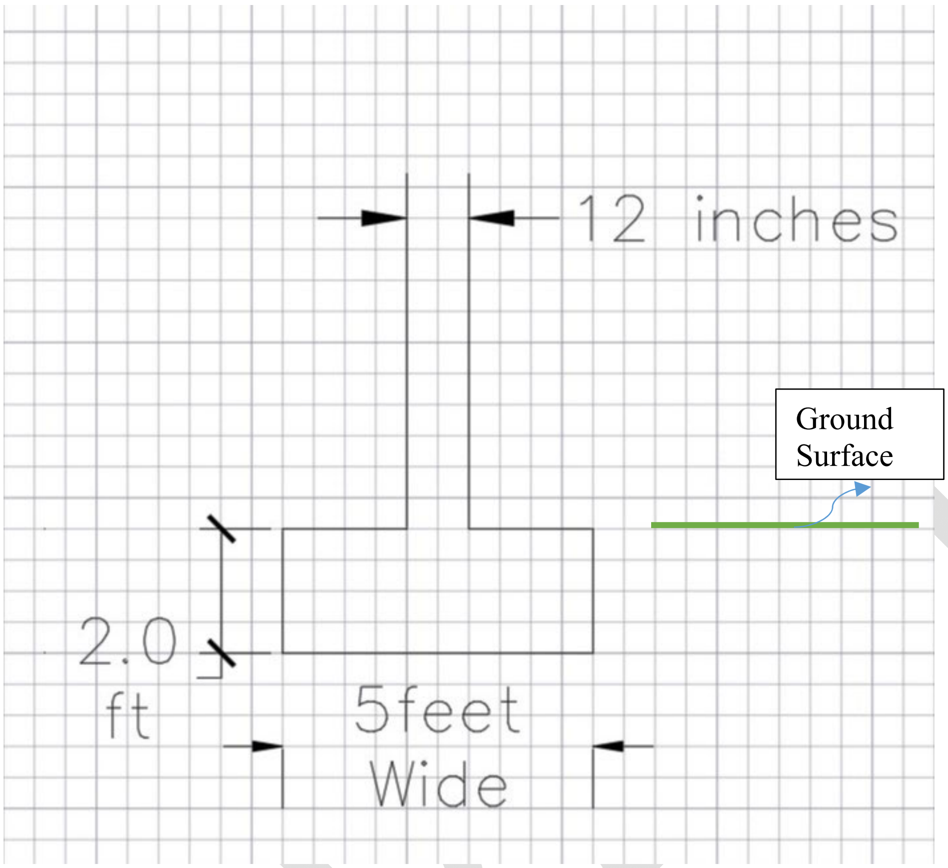

ELEV.
(FT)
SOIL PRESSURE (PSF)
FOR CANTILEVER WALL DESIGN
2000
0
2000
15.00
10.00
0.00
-5.30


ELEV.
(FT)
SHEAR (LB)
FOR CANTILEVER WALL DESIGN
2000
0
2000
15.00
10.00
0.00
-5.30


ELEV.
(FT)
BENDING MOMENT (LB-FT)
FOR CANTILEVER WALL DESIGN
0
5.00E+03
15.00
10.00
0.00
-5.30

Cantilever Floodwall/Retaining wall around Pump Station #4.[3]



3 The 5' base of the footing may be reduced to 4' due to limited easement.