



**US Army Corps
of Engineers®**
San Francisco District

Appendix I

Geotechnical Engineering



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Appendix A

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DRAFT

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. This 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 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 at 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. It is bordered by the cities of Brisbane to the north and San Bruno to the south. It 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.

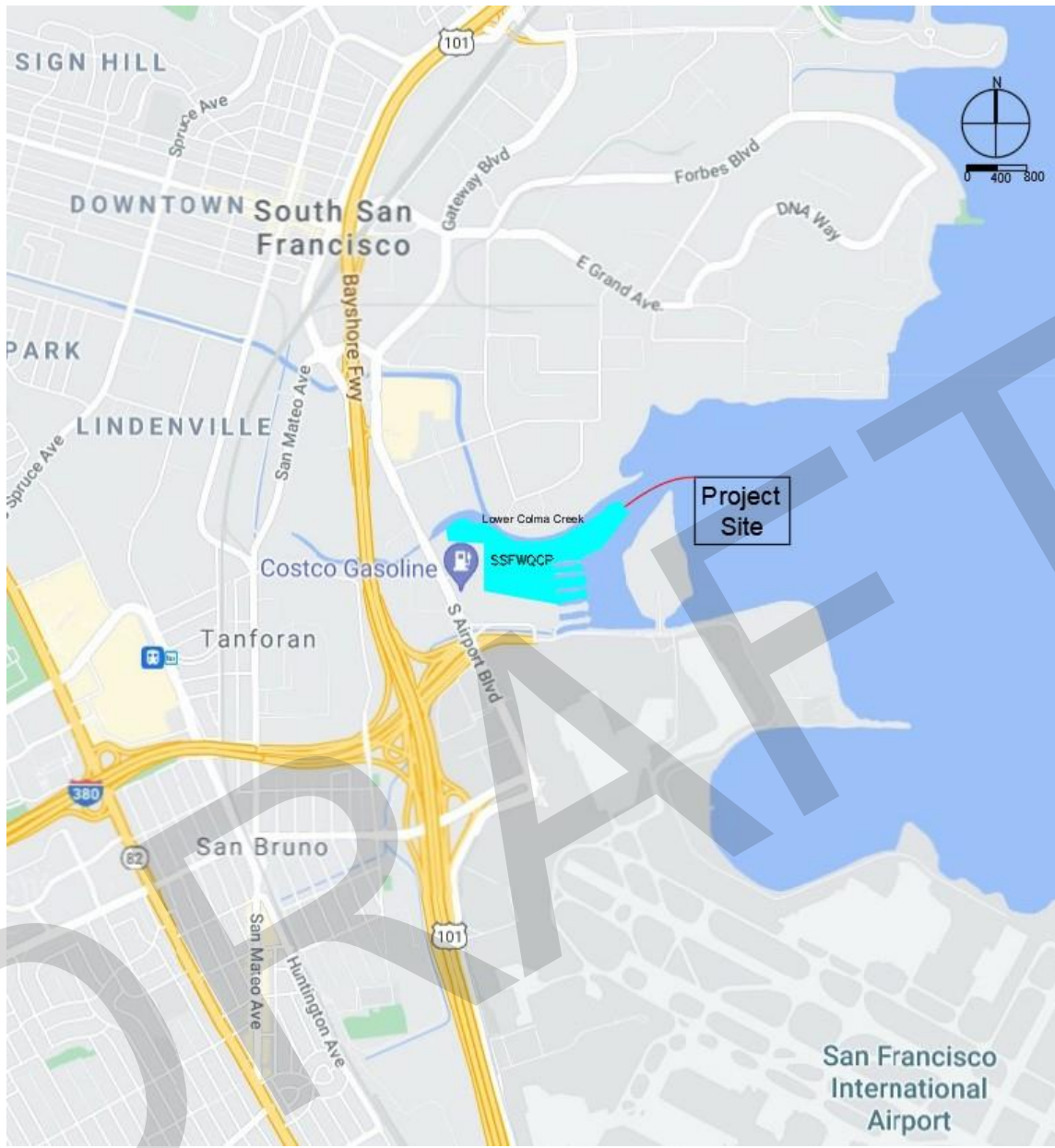


Figure1 - Location of the South San Francisco Water Quality Control Plant

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 the evaluation of the project geotechnical and geologic conditions. The scope of the geotechnical work for this project includes the following:

- Review of the existing geotechnical reports and the geotechnical 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 present geotechnical recommendations for evaluation of flood protection alternatives and CSRMs 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 States Geological Survey (USGS, 1998) shown in Figure 2. The site is primarily underlain by artificial fill over tidal flats (Qaf/ta) in the north and northeast of the project site along the creek bank. This artificial fill (Qaf/ta) 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.

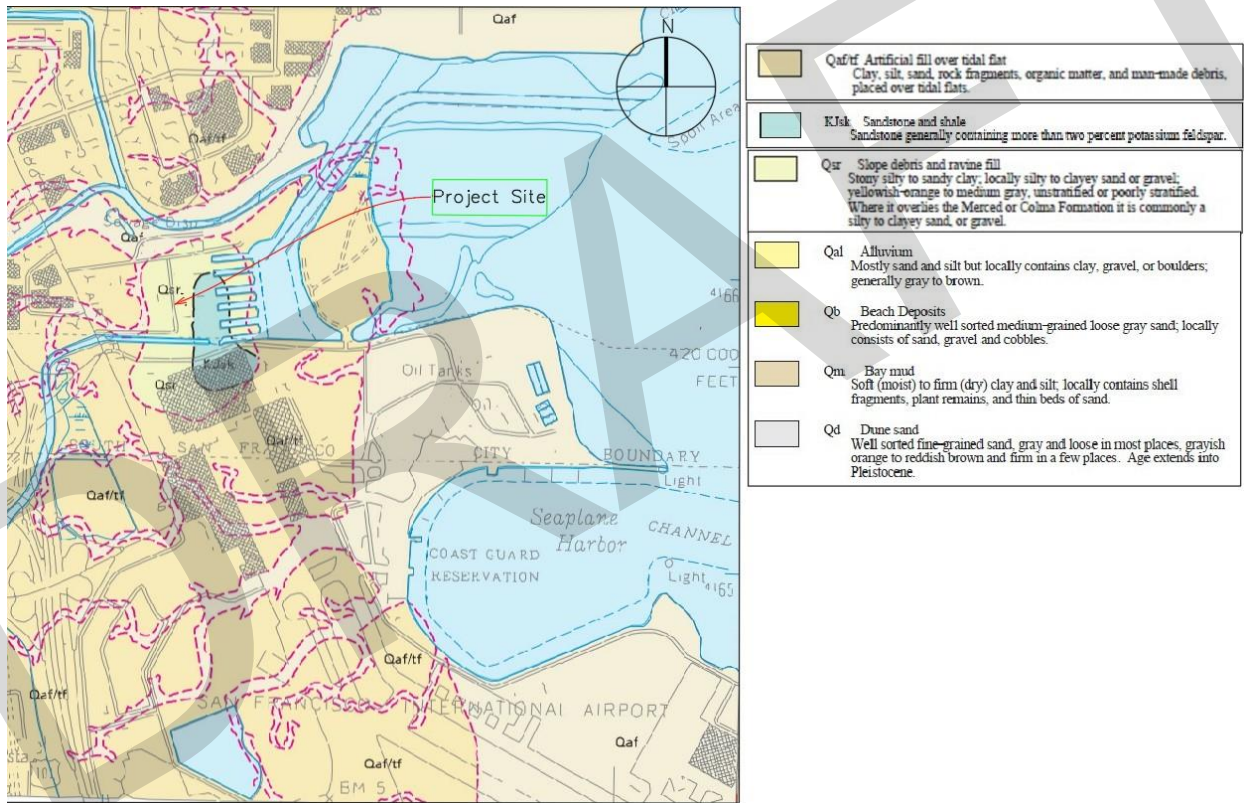


Figure 2 - Geological map adapted from the United States Geological Survey (1998)

3.0 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.

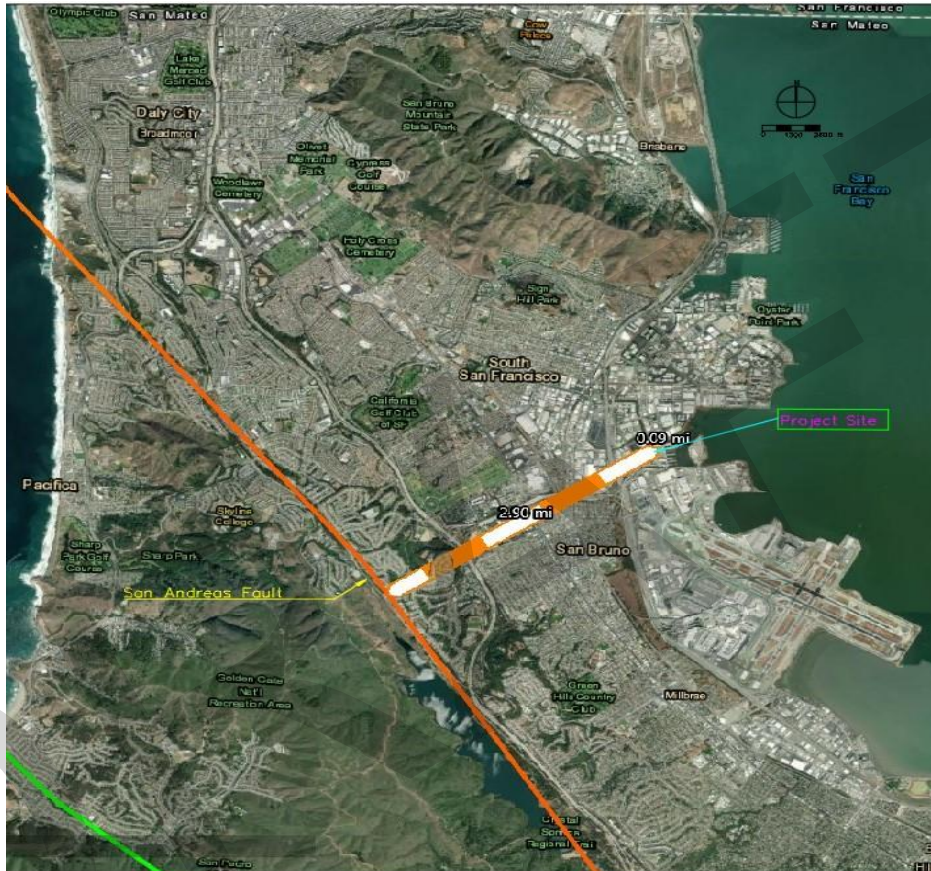


Figure 3 - Major Faults near the project Site

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>), new model for estimating earthquake probabilities developed by the 2014 Working Group on California Earthquake Probabilities 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 5 below represent the likelihood of having one or more earthquakes in the next 30 years (starting from 2014).

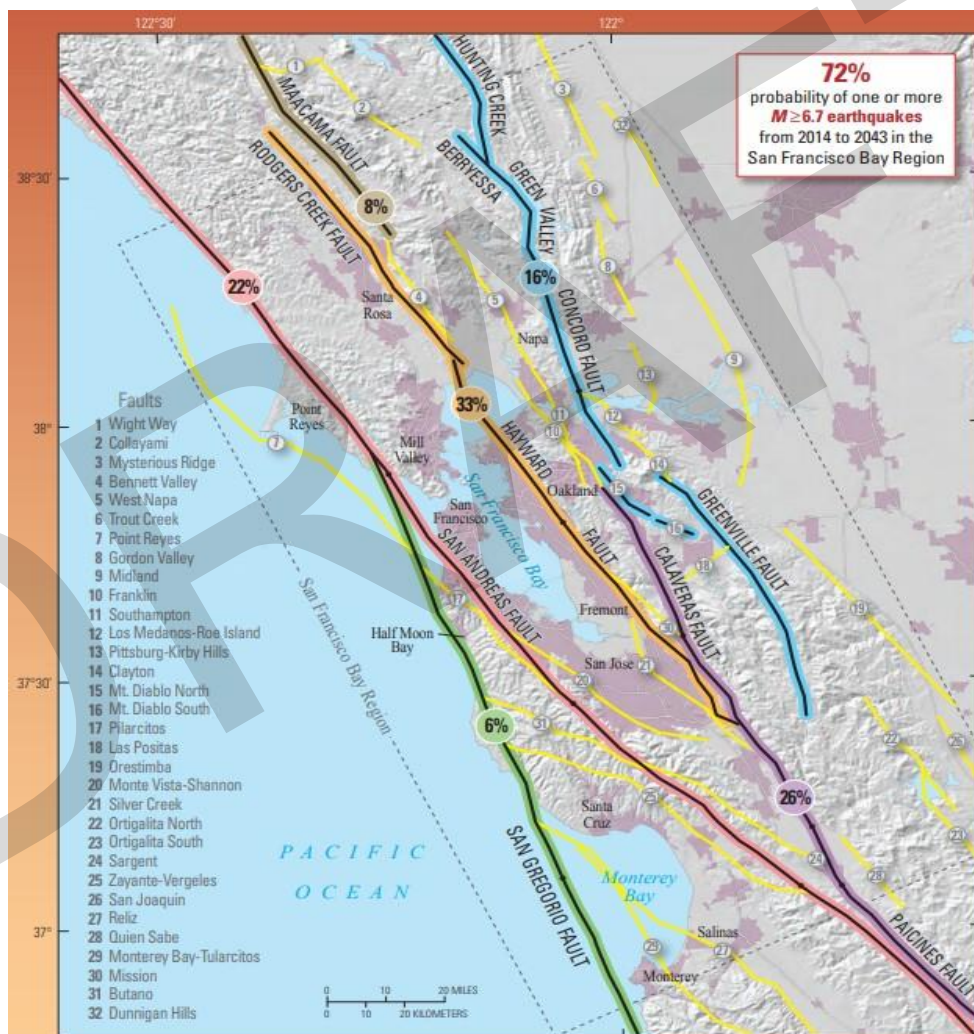


Figure 4- Earthquake Outlook for the San Francisco Bay Region 2014–2043

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.

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 quaternary 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 located in very high liquefaction-susceptible zone. Liquefaction analyses based on the available subsurface data collected from previous geotechnical reports will be performed in the subsequent section to determine the potential risk of liquefaction.

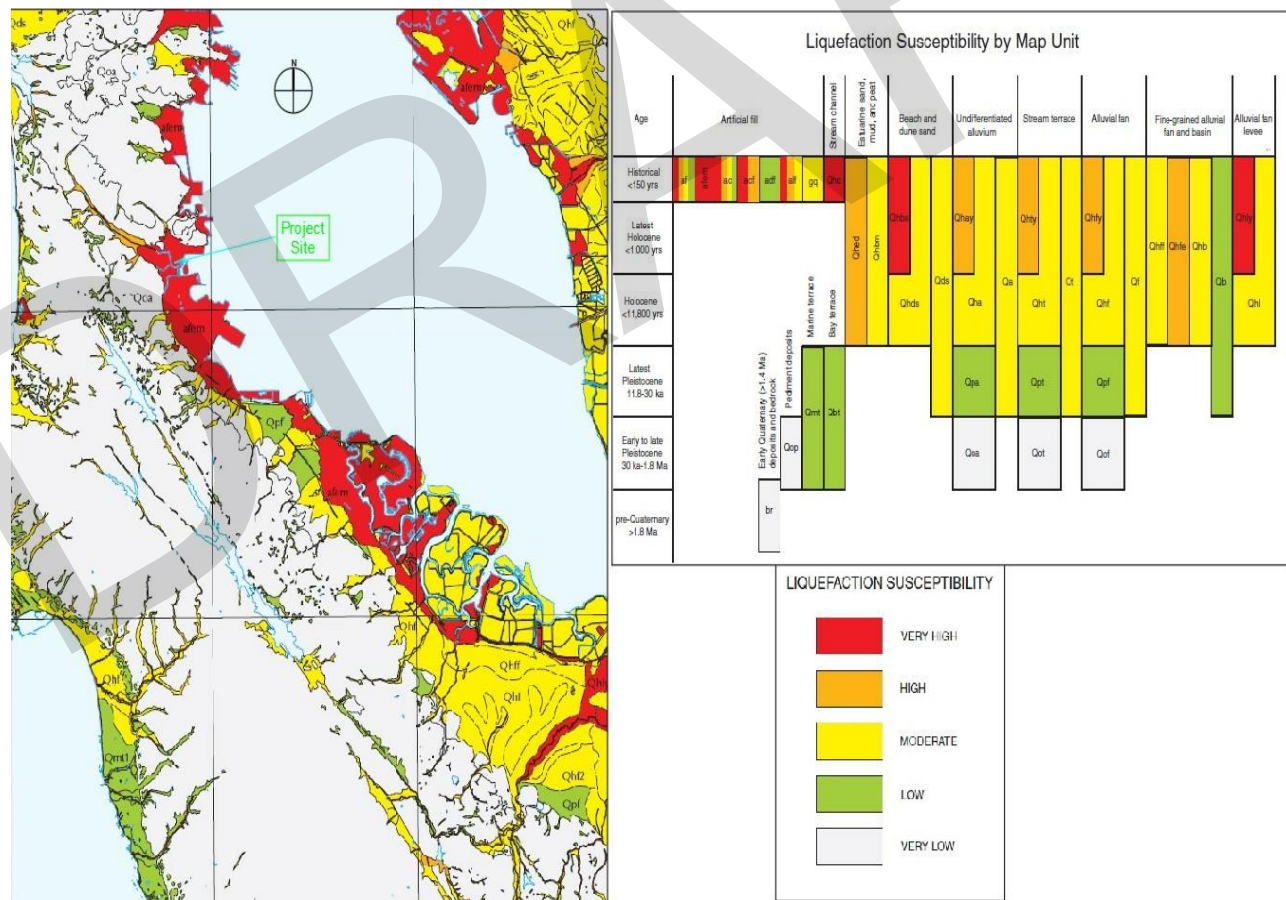


Figure 5-USGS LIQUIFACTION SUSCEPTIBILITY MAP, 2006

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. 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, it is most often caused by removal of ground water groundwater, oil, natural gas, or 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 surveys¹ 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 survey results (2019) by Santa Clara Valley Water revealed that subsidence (for San Francisco shoreline did not exceed 0.01 feet per year.

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>

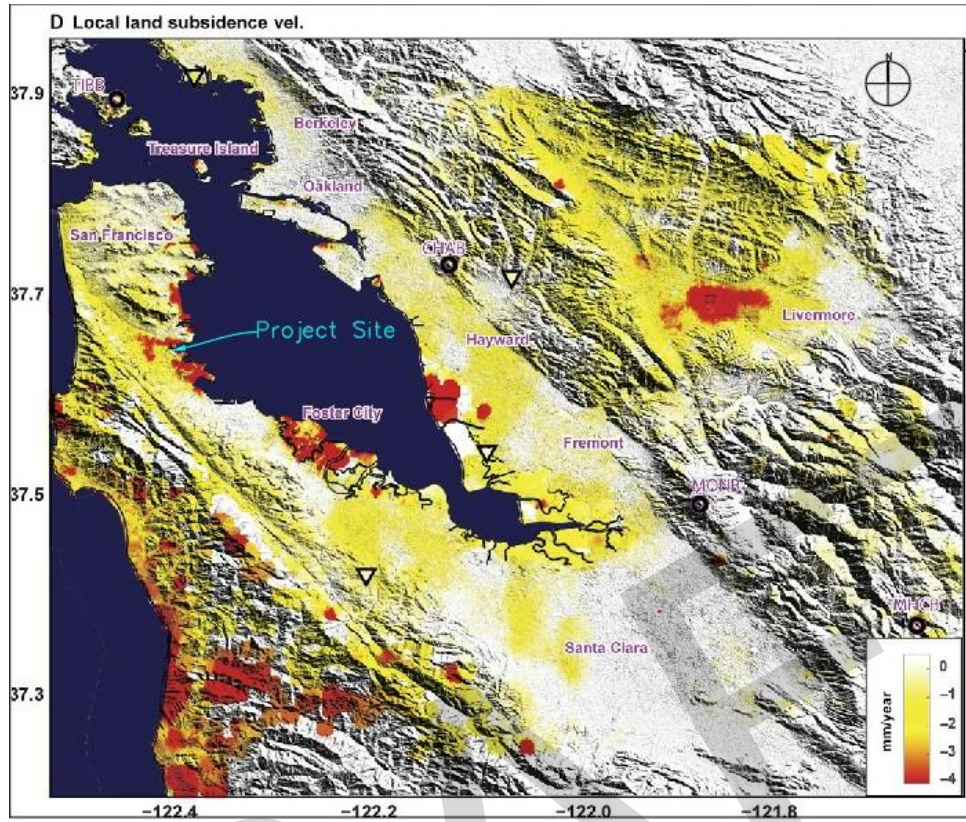


Figure 6- NASA Subsidence Map

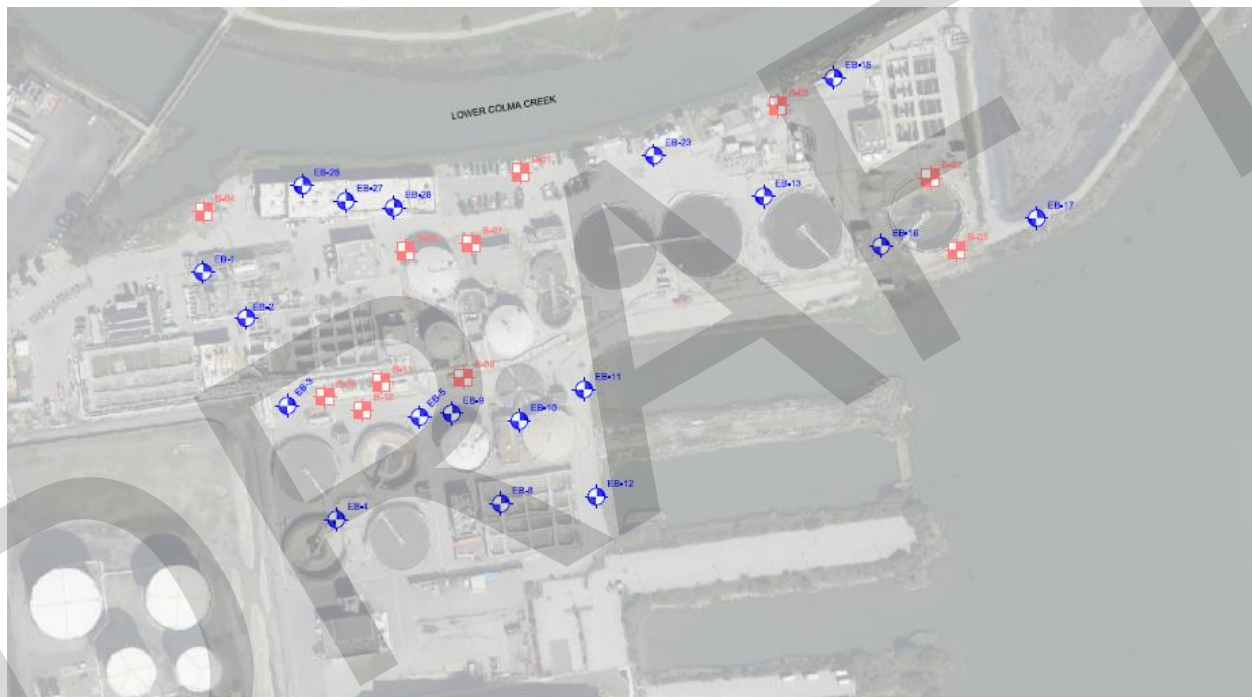
4.0 Subsurface Data and Site Condition

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

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, overlies 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), extended to depths ranging from about 5 to 11 feet. The existing fill generally consists 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 at the borings was extended to depths ranging from 3 to 6 feet below the ground surface. For the East portion of site, the near surface soils condition consists of fill extended to depths ranging from 2 ½ to 10 feet. These fills generally consist 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 one area to another in the site, generally, the thickness of BM increases to the northern portion (along the creek bank) to the eastern portion closer to the bay. The thickness of bay mud is ranging for 13 to 16 feet at/near boring EB-28 and EB- 15 respectively. The locations of the exploratory borings drilled by Harza, Fugro and others are shown in figure 7.

Alluvial deposits were encountered beneath the YBM and extended to the maximum depth explored. These deposits generally consist 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 with those encountered in the Fugro (2012) explorations to the depths explored in material type.



LEGEND
 EB-XX APPROXIMATE LOCATION OF EXPLORATORY BORINGS BY FUGRO (2018)
 EB-XX APPROXIMATE LOCATION OF EXPLORATORY BORINGS BY HARZA (1990)

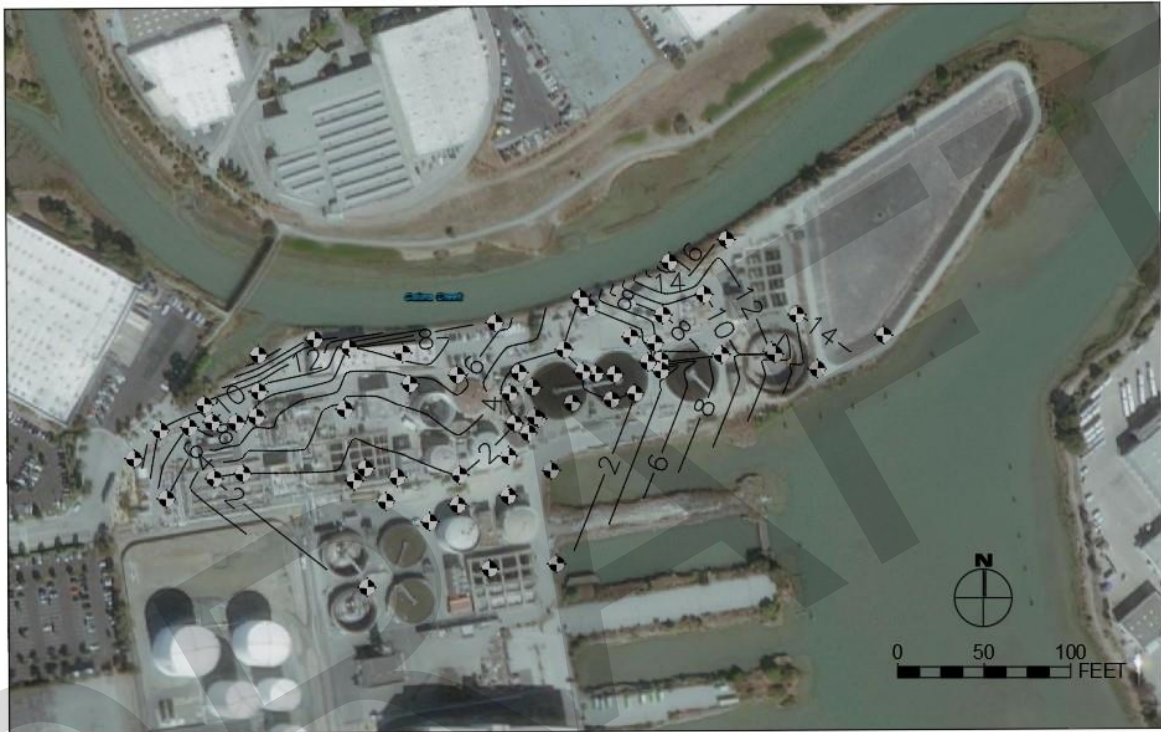
Figure 7- Approximate Boring Locations

4.1 Soil Engineering Parameters

Due to limited laboratory testing data available in the geotechnical reports Harza (1998), and (Fugro 2012 & 2018) provided by the non-federal sponsor, some of the soil engineering parameters were not present in the geotechnical reports. Therefore, these parameters were estimated using the published standard correlation charts and empirical formulas. Table 2 summarizes the key estimated soil engineering parameters used for the current geotechnical design and analyses.

4.2 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

4.3 Groundwater

Along the San Francisco Bay and Pacific Ocean shorelines as well as adjoining flatlands historic high groundwater levels are shallow (0-10 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 within 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.

5.0 Settlement

The project area is underlain by approximately 2 to 20 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 10 shows the approximate thickness of the bay mud for the Palo Alto Flood Basin. The thickness of Bay Mud varies at different locations, generally it increases towards the bay side. Along the edges of the deposit, the upper few feet (1-3 feet) have 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”.

5.1 Liquefaction Induced Settlement.

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

6.0 Proposed CSRMs Measures

Future with project conditions consist of non-structural measures and Coastal Storm Risk Management (CSRMs) measures which include concrete cantilever floodwalls, combination of floodwall over the compacted engineered fill berm/levee (Hybrid Flood wall), 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 CSRMs 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, 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 would be 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 reduce the risk of flood damages. Installation of waterproofed doors/windows, use of waterproof materials or membranes to watertight structure 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 spaciously 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, therefore, 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 an 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 providing the structures 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,

6.2.1 Floodwalls

A Floodwall is considered to provide a 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, I-wall floodwall needs to be constructed. Figure 9 shows areas where floodwall need to be constructed.

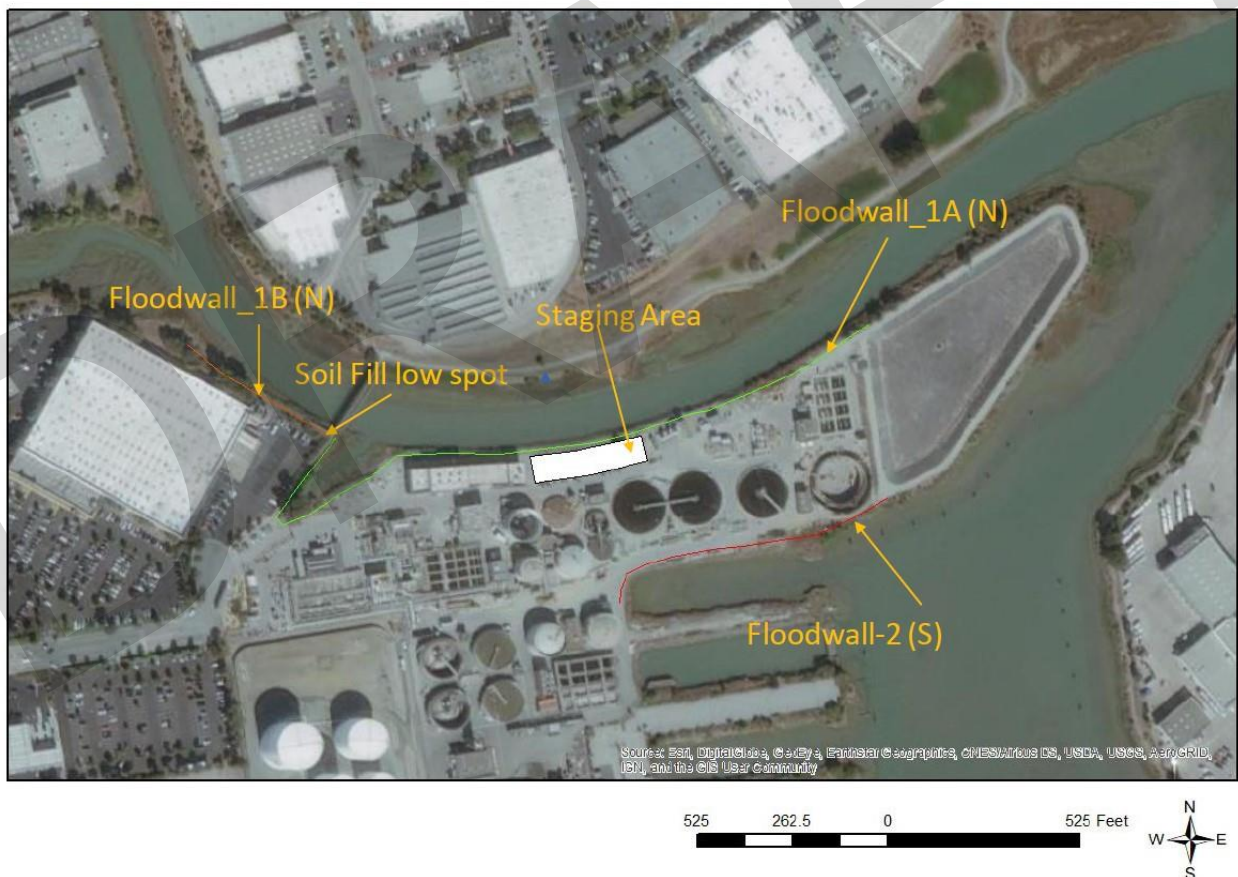


Figure 9 - Floodwall Alternatives

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 in weak and soft clays.

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 constraints for the wall footprint. The designed 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 to 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 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

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

6.2.1.1.3 Seepage

A Seep/W analysis was performed 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.

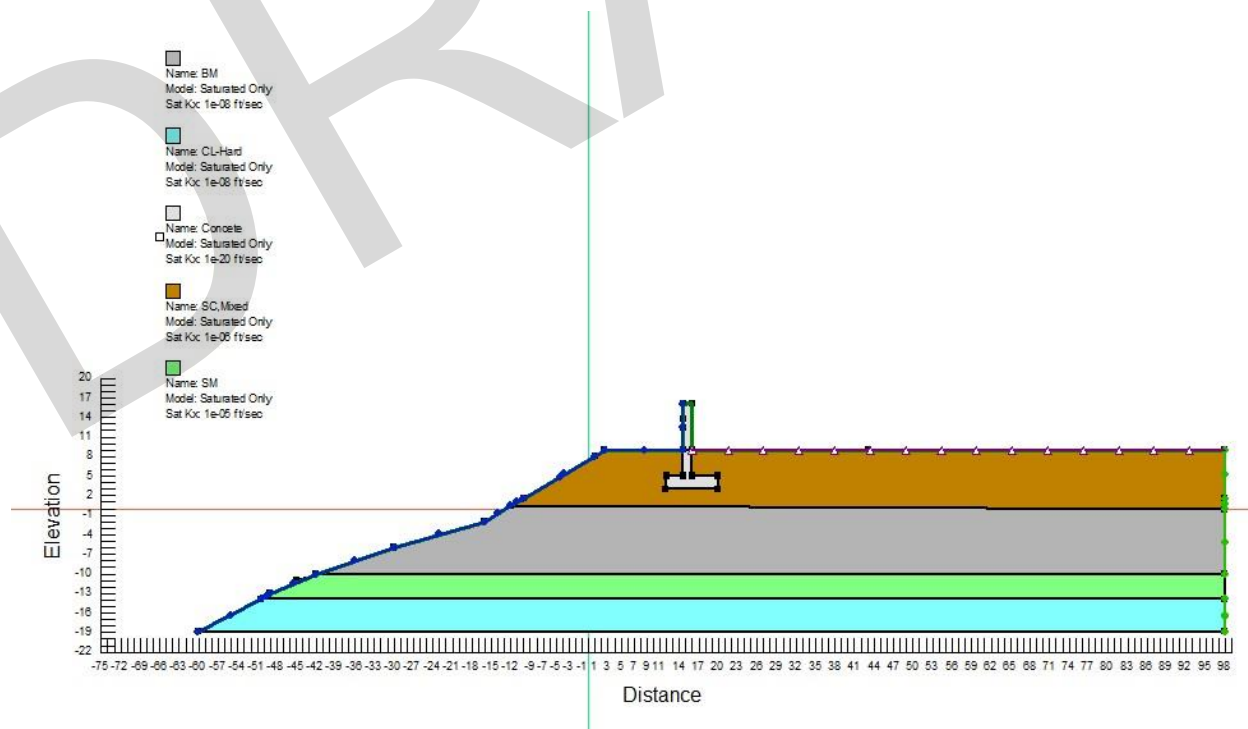


Figure 10- Seepage Model and Soil Profile

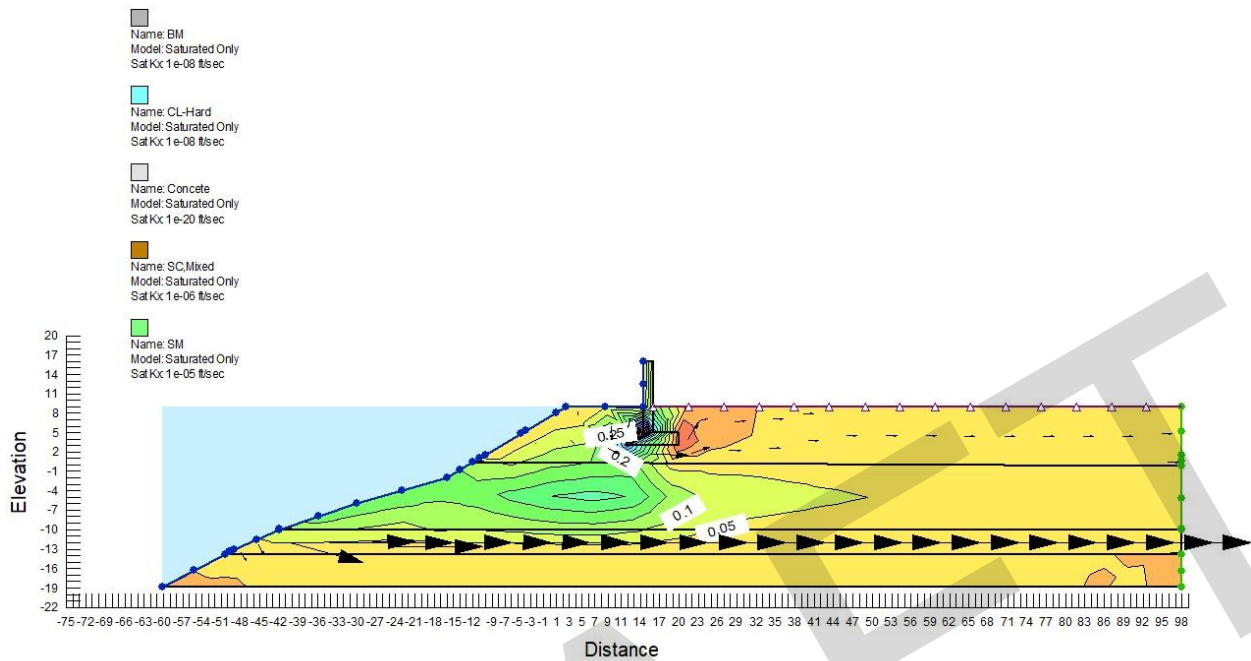


Figure 11- Seepage Model Analysis

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. I-flood walls need to have concrete encasement to provide corrosion protection for the sheet piling in accordance with EC 1110-2-606. Most of the I-flood walls, 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 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 (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 seep/W analysis was performed at a cross section where I type floodwall is required to be constructed 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 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 15.

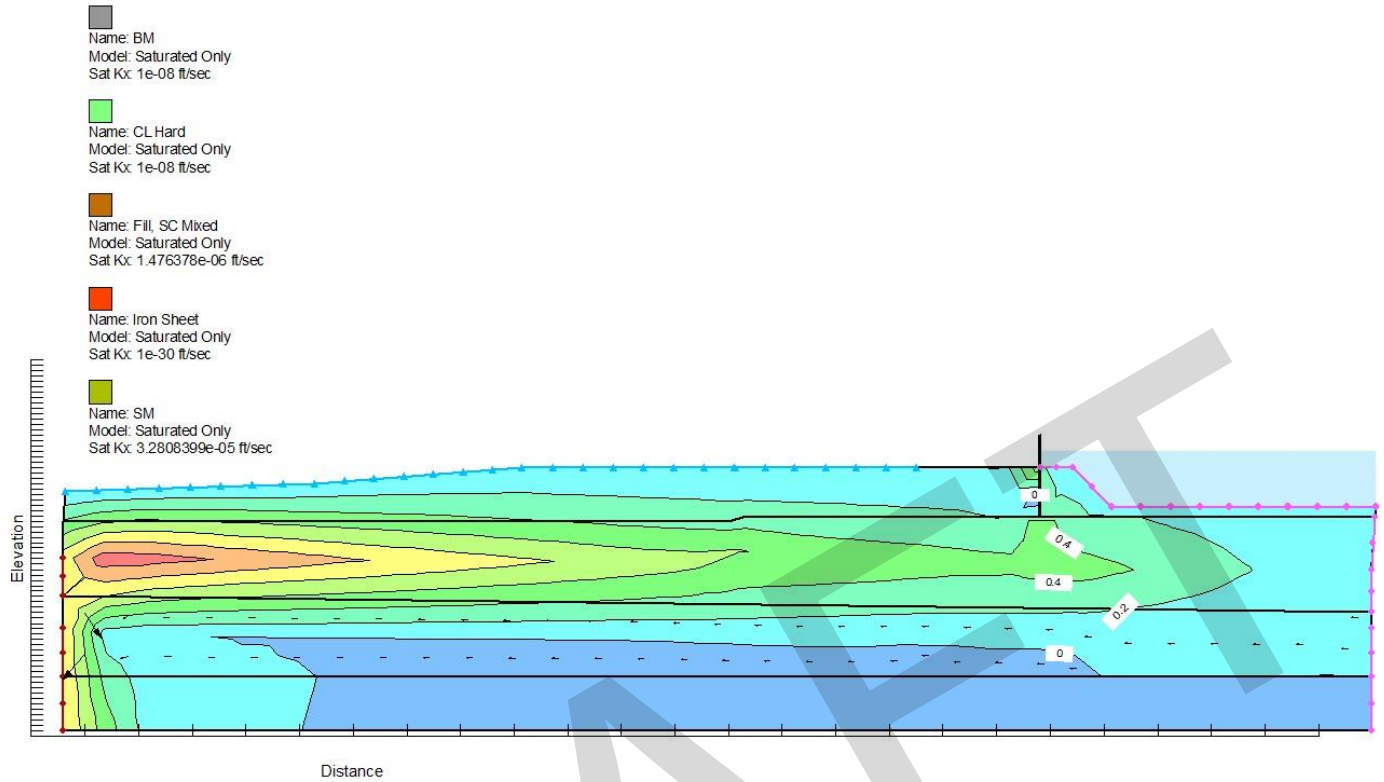


Figure 12- Seepage Analysis at Gap-Soil Sheet Pile

6.2.2 Floodwall/Retaining Wall around Pump Station No. 4

An Inverted T-shape floodwall/retaining wall is 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, High SLR scenario) or a 50-year flood event as shown in Figure 14.



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

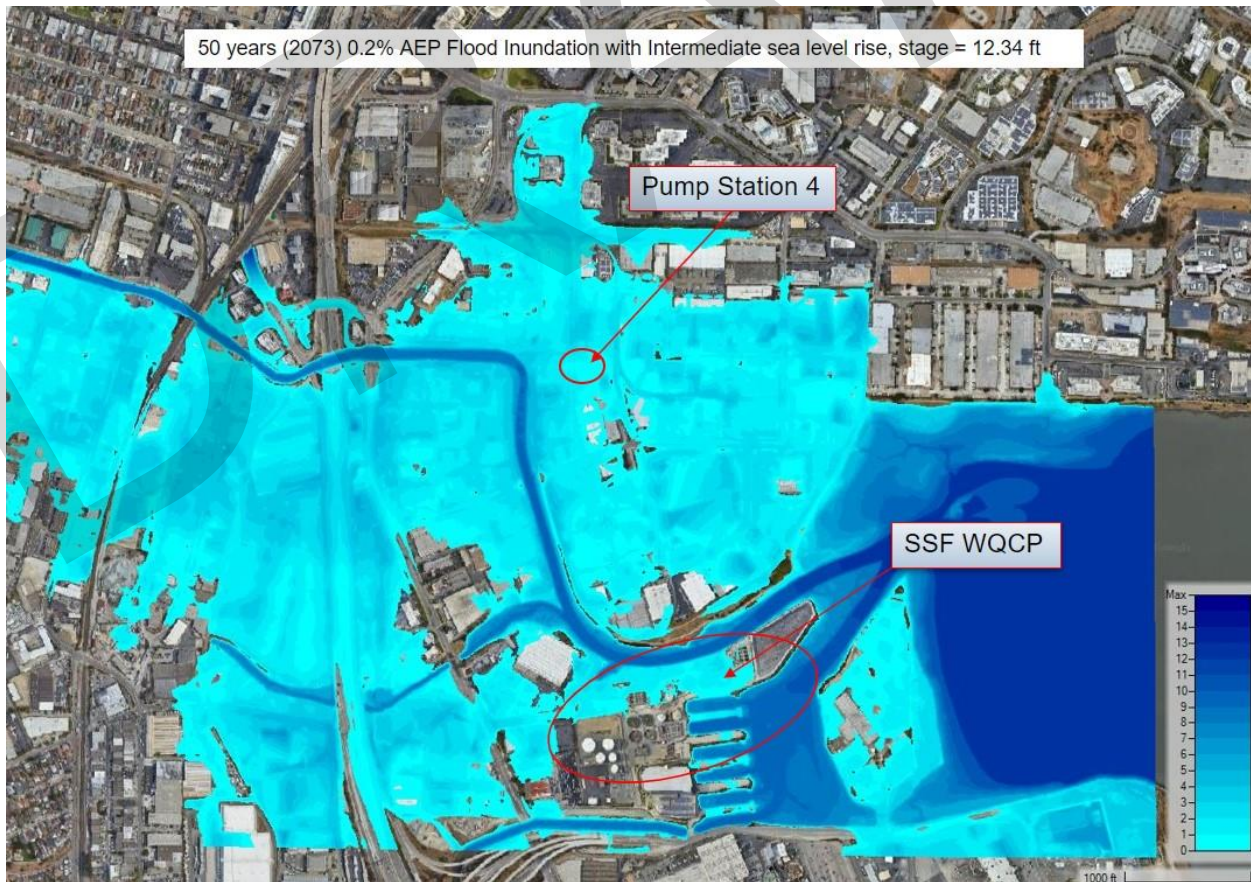


Figure 14- Map of Flood Inundation at Station 4 (50-year Flood Event)

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.

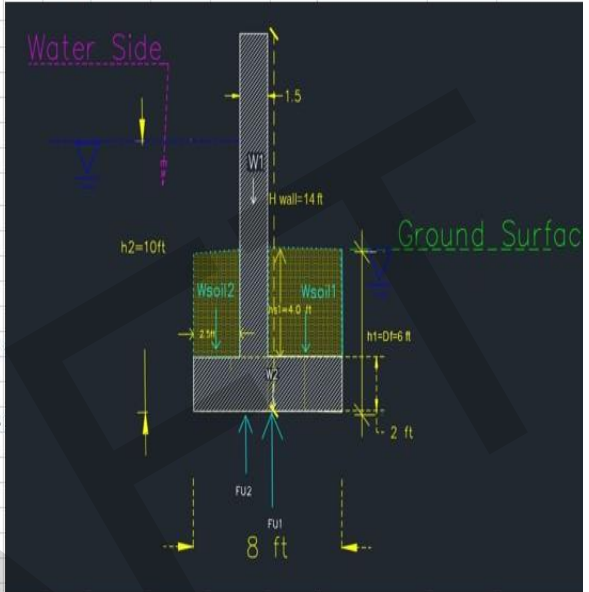
Appendix A

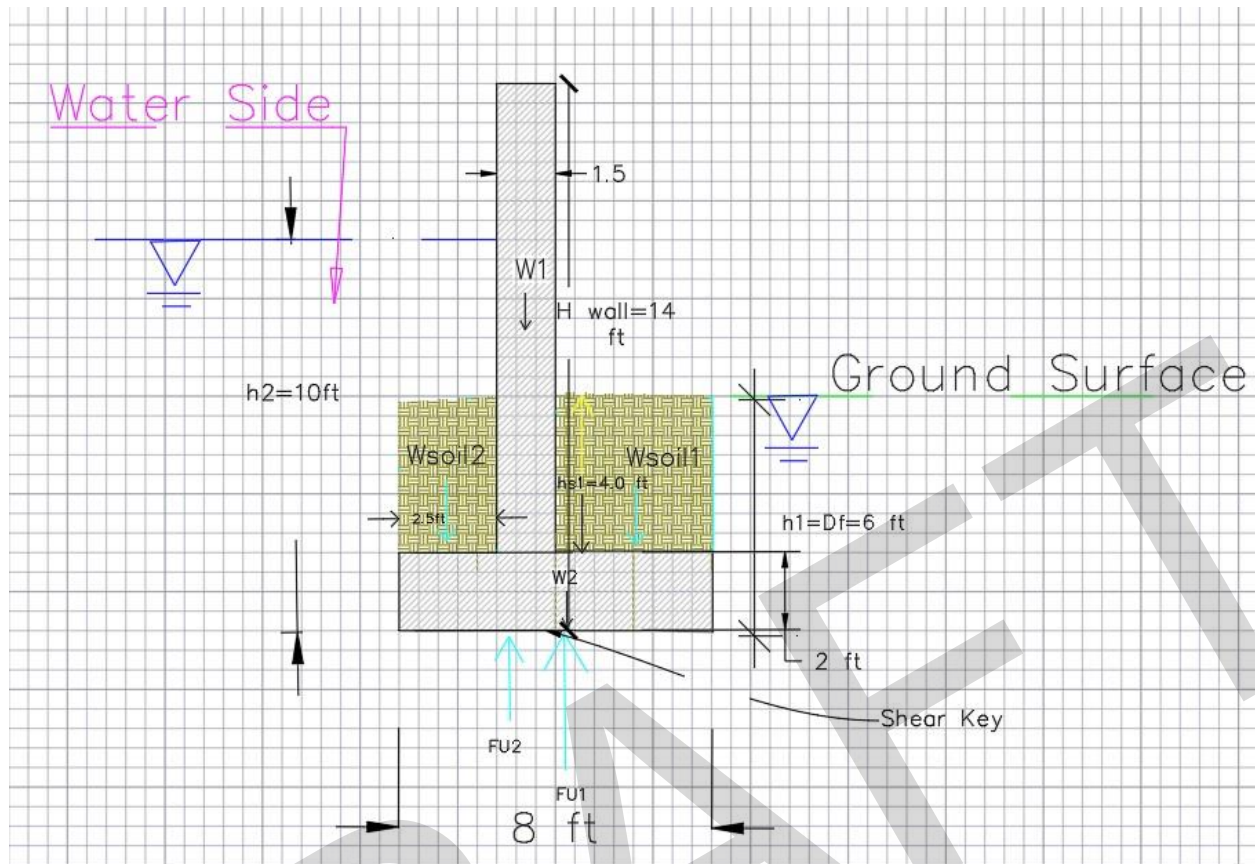
- Stability Calculations for Cantilever Floodwall
- Settlement Calculations for Cantilever Floodwall
- Cantilever Seepage Analysis with CWALSHT

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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)				XR		4.00351 OK		B'		7.99298							
W1	2700	At Right	Pp	3667.298	passive ultimate ca	e	0.00351												
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	EF right		4790.498															
W1 (water)	998.4	EF left		3250.275															
W2 (water)	1248	Net		1560.145	(not counting passive)														
N'=EV	4850.4	passive required to		1560.145	OK	FS (bearing)		40.0409 OK											
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														
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		Ratio XR/B									
W2 (water)	1.25			1560	clockwise	Other Factors													
Soil Pressur	2			3120.289167	counter clo	Fci=Fqi		1											
Pw(Right)	2			2246.4	counter clo	Fcs		1.00001											
Soil pressur	2			-260.55007	clockwise	Fqs		1											
Pw(Left)	3.3			-10296	clockwise	Fys		1											
						Fqd		1											
						Fcd		1											
						Fyd		1											
						Fyi		1											
						q		-104.4		1.33									
						Y'		38.6											
										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.

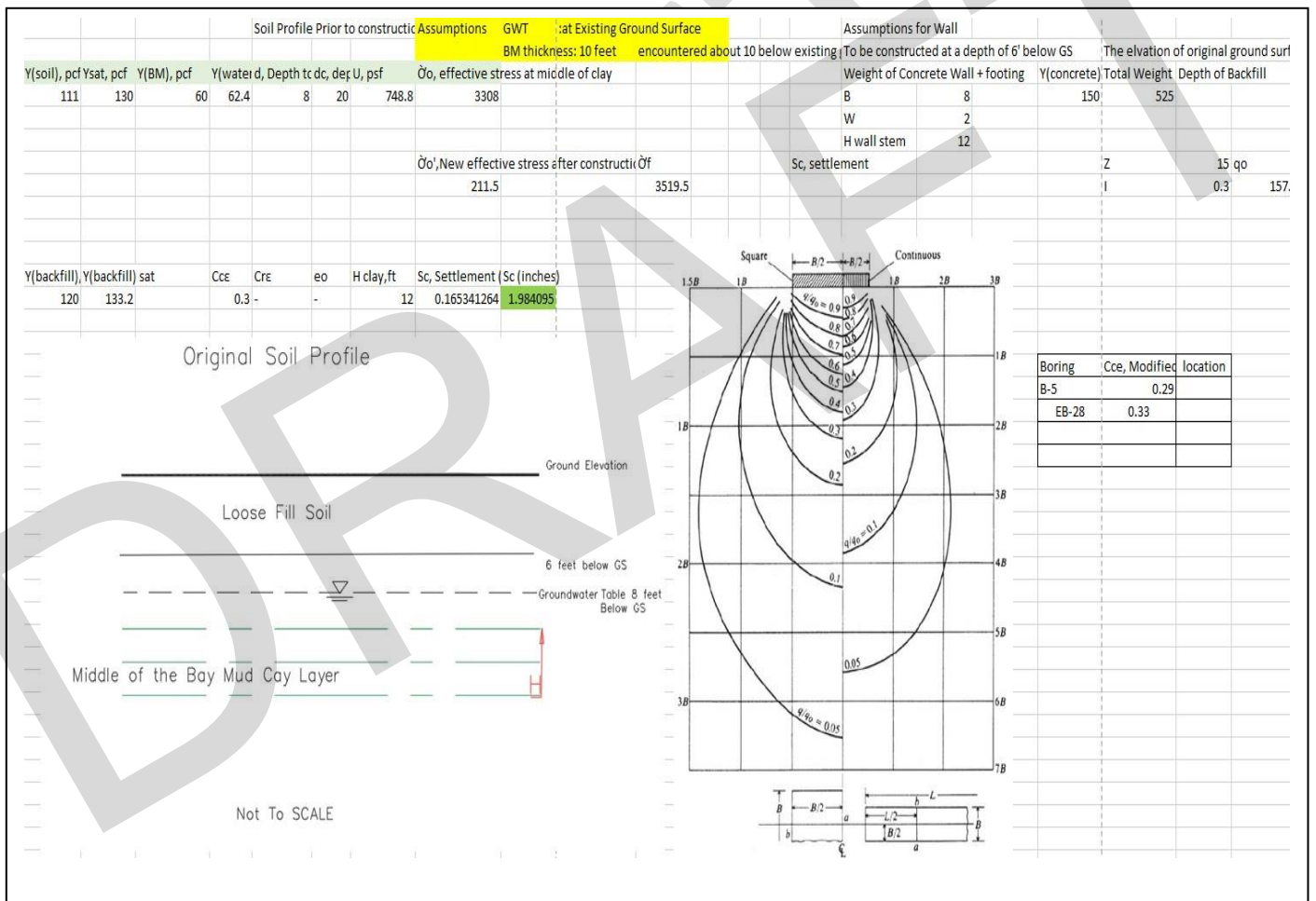
Footing depth 4 feet below the existing ground. For soft ground (marsh land) is 6 feet

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).

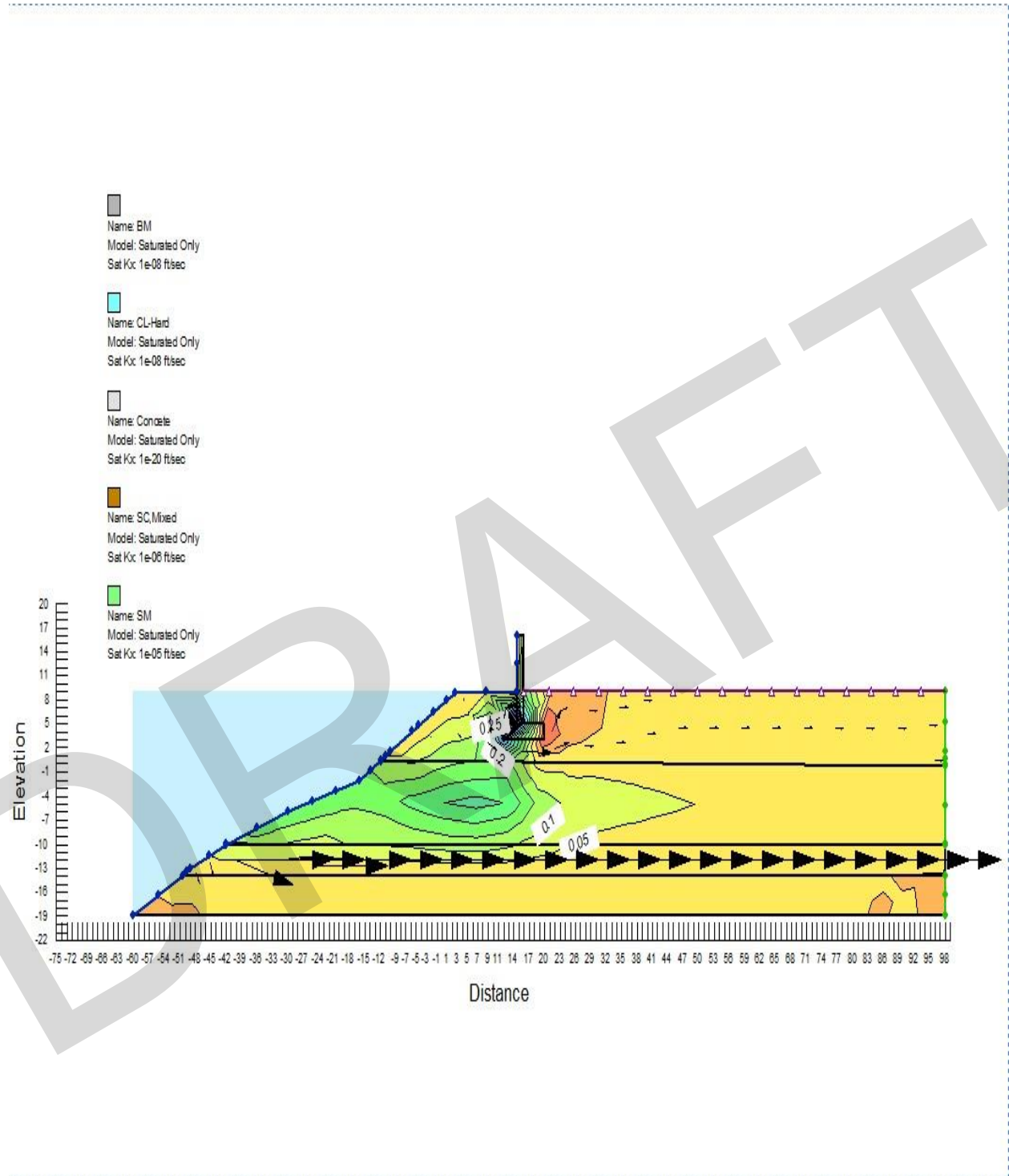
Thickness of the wall is to be 1.5 feet.

Concrete wall needs to be reinforced to avoid tension cracks.

Settlement Calculations for Cantilever Floodwall



Cantilever Seepage Analysis



CWALSHT computations for I-floodwall

BM

Hydraulic

Model: Saturated Only

Sat Kx: 1e-08 ft/sec

Ky'/Kx' Ratio: 1

Rotation: 0 °

Volumetric Water Content: 0

Compressibility: 0 /psf

SM

Hydraulic

Model: Saturated Only

Sat Kx: 1e-05 ft/sec

Ky'/Kx' Ratio: 1

Rotation: 0 °

Volumetric Water Content: 0

Compressibility: 0 /psf

CL-Hard

Hydraulic

Model: Saturated Only

Sat Kx: 1e-08 ft/sec

Ky'/Kx' Ratio: 1

Rotation: 0 °

Volumetric Water Content: 0

Compressibility: 0 /psf

* SUMMARY OF RESULTS FOR *

* CANTILEVER WALL DESIGN *

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 ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

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

WALL BOTTOM ELEV. (FT) : -5.30

PENETRATION (FT) : 5.30 *Note: 5.30 ft below the Mudline elevation (Mudline elev~2)*

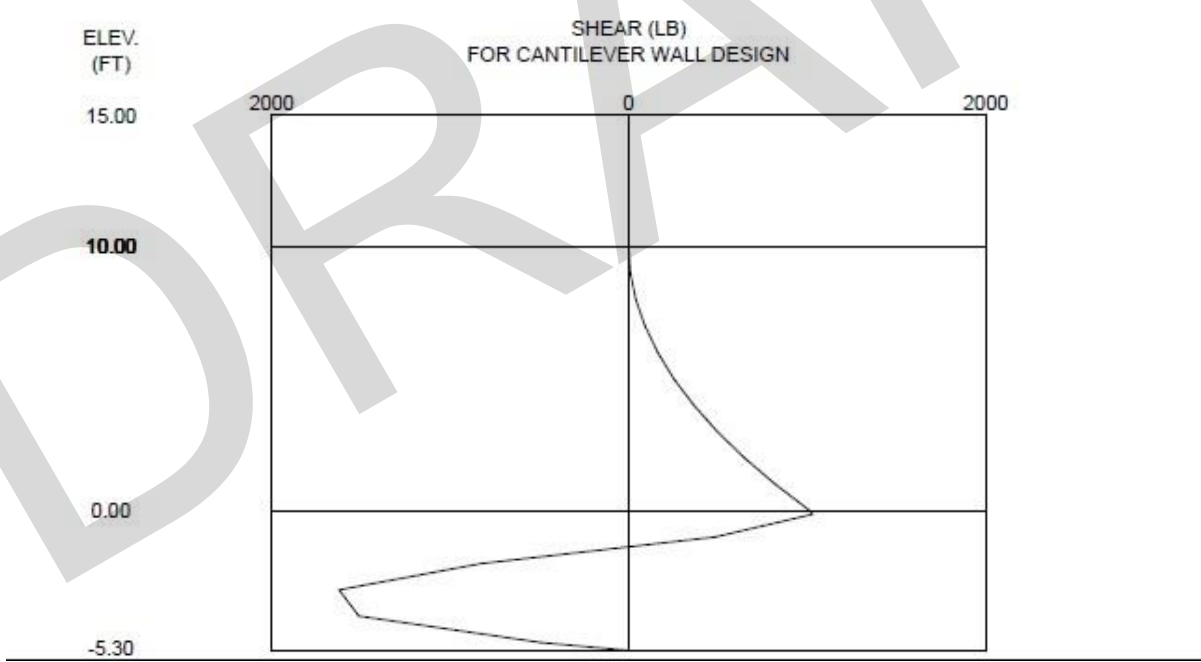
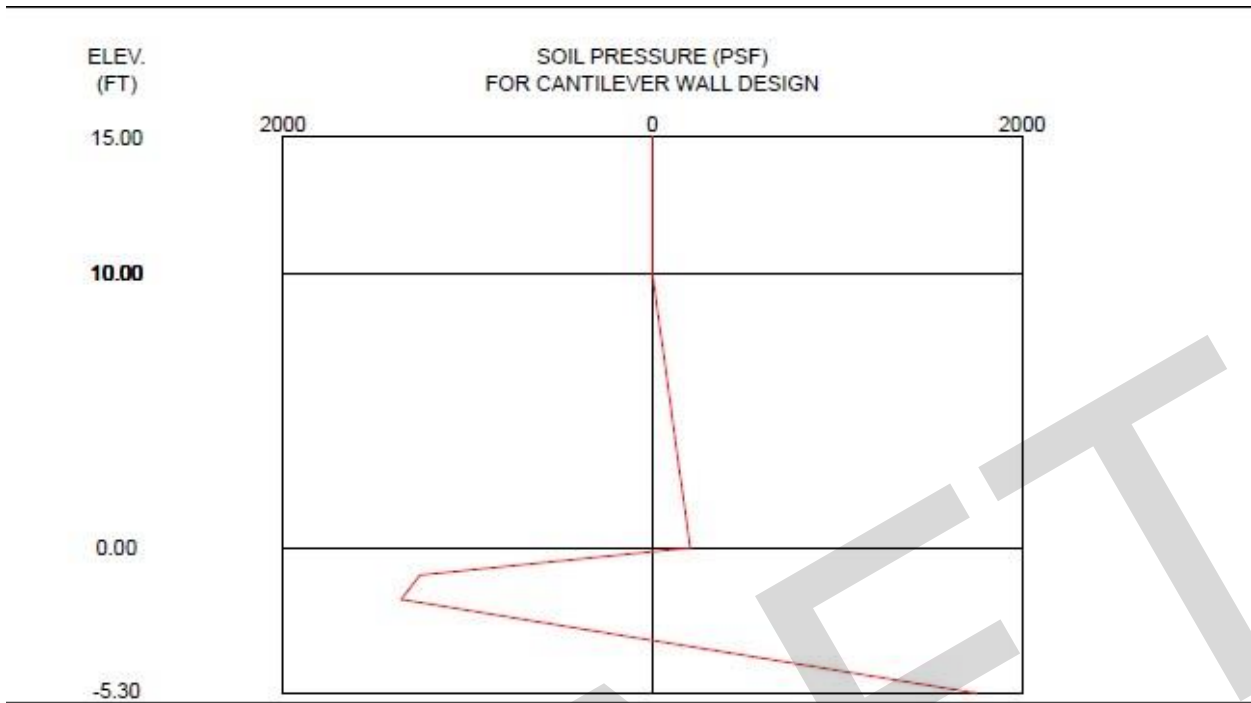
MAX. BEND. MOMENT (LB-FT) : 4.3478E+03

AT ELEVATION (FT) : -1.38

MAX. SCALED DEFL. (LB-IN³): 6.0223E+08

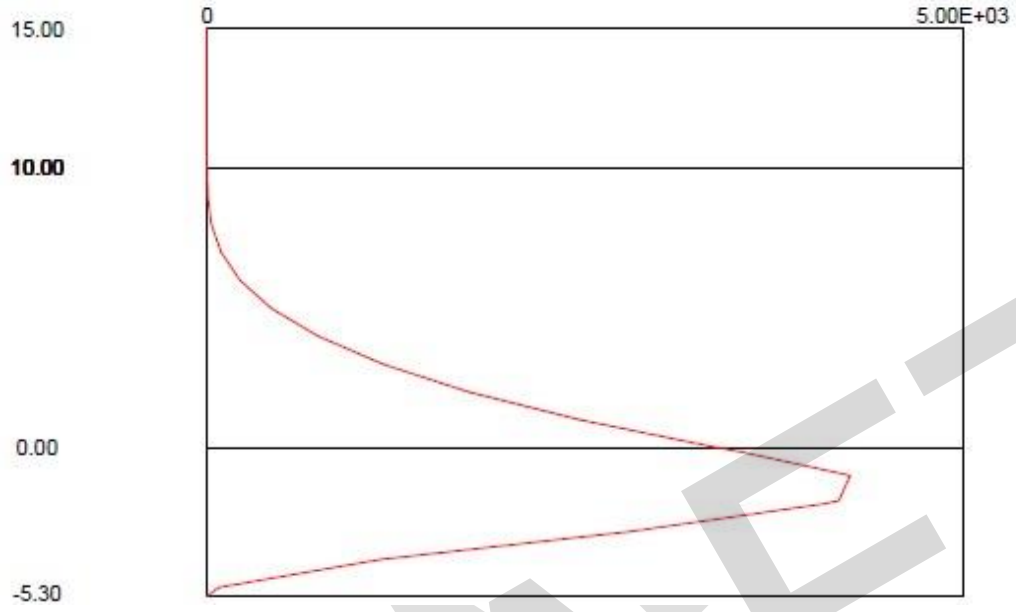
AT ELEVATION (FT) : 15.00

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF
ELASTICITY IN PSI TIMES PILE MOMENT
OF INERTIA IN IN⁴ TO OBTAIN DEFLECTION



ELEV.
(FT)

BENDING MOMENT (LB-FT)
FOR CANTILEVER WALL DESIGN



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Cantilever Floodwall/Retaining wall around Pump Station #4

