US Army Corps of Engineers®

Oakland Harbor Turning Basins Widening

Revised Draft Integrated Feasibility Report and Environmental Assessment

APPENDIX B2:

Geotechnical Engineering

December 2021

Revised March 2023

1. Introduction

This draft appendix was developed as part of the Oakland Harbor Turning Basin Widening Navigation feasibility study. This appendix summarizes exiting geotechnical conditions at the site and presents the findings of the engineering analysis conducted to support the development of recommend improvements to the Inner and Outer Harbor Turning Basins.

This Appendix is based on review of plans and design documents from previous projects, consultant and agency geotechnical reports, and published geologic reports. This appendix was prepared in accordance with USACE Civil Works policy and SMART planning process (ER 1110-2-1150, ER 1105-2-100, etc.). The project is currently at the Tentatively Selected Plan Milestone. The geotechnical design and appendix will be further developed prior to the Agency Decision Milestone.

1.1. Project Description

The Port of Oakland Outer Harbor Turning Basin (OHTB) is located in the outer harbor channel near berths 25 through 30. The OHTB has a diameter of 1,650 feet; the bottom elevation of -50 feet (NAVD88) is maintained by annual dredging.

The Inner Harbor Turning Basin (IHTB) is located approximately 18,000 feet to the east of the Oakland Harbor entrance near the Howard Terminal. The IHTB basin had a diameter of 1,500 feet; the bottom elevation of -50 feet is maintained by annual dredging.

This study considered several alternative geometries for both the OHTB and the IHTB. The Tentatively Selected Plan consists of widening both the Inner and Outer Harbor Turning Basins to 1,835 feet and 1,965 feet, respectively. The Turning Basin bottom elevations would remain at Elevation -50 feet. The OHTB Variation 2.1 would not require impacts to the land. The IHTB Variation A would require excavation into the Howard Terminal on the north side of the channel and private property on the south side of the channel. The proposed footprints for the OHTB and IHTB are shown on Figures 2 and 3, respectively. Refer to the Channel Design Appendix B1 for descriptions of the variations that were considered during the alternative analysis process.

The TSP includes construction of new bulkhead walls at Howard Terminal and on the Fisk Property in Alameda. The TSP also includes a below-grade, in-water retaining structure in front of the Schnitzer Steel property to the northwest of the IHTB. The wall will be approximately 300 to 400 feet long, and will be entirely submerged. The wall will likely be a concrete secant wall or driven pile structure. The wall will be offset 10 to 20 feet from the existing Schnitzer Steel wall in the direction of the turning basin. The top of the wall will be flush with the existing grade (mudline) at the base of the Schnitzer wall. The proposed wall will retain approximately 20 to 25 feet of soil.

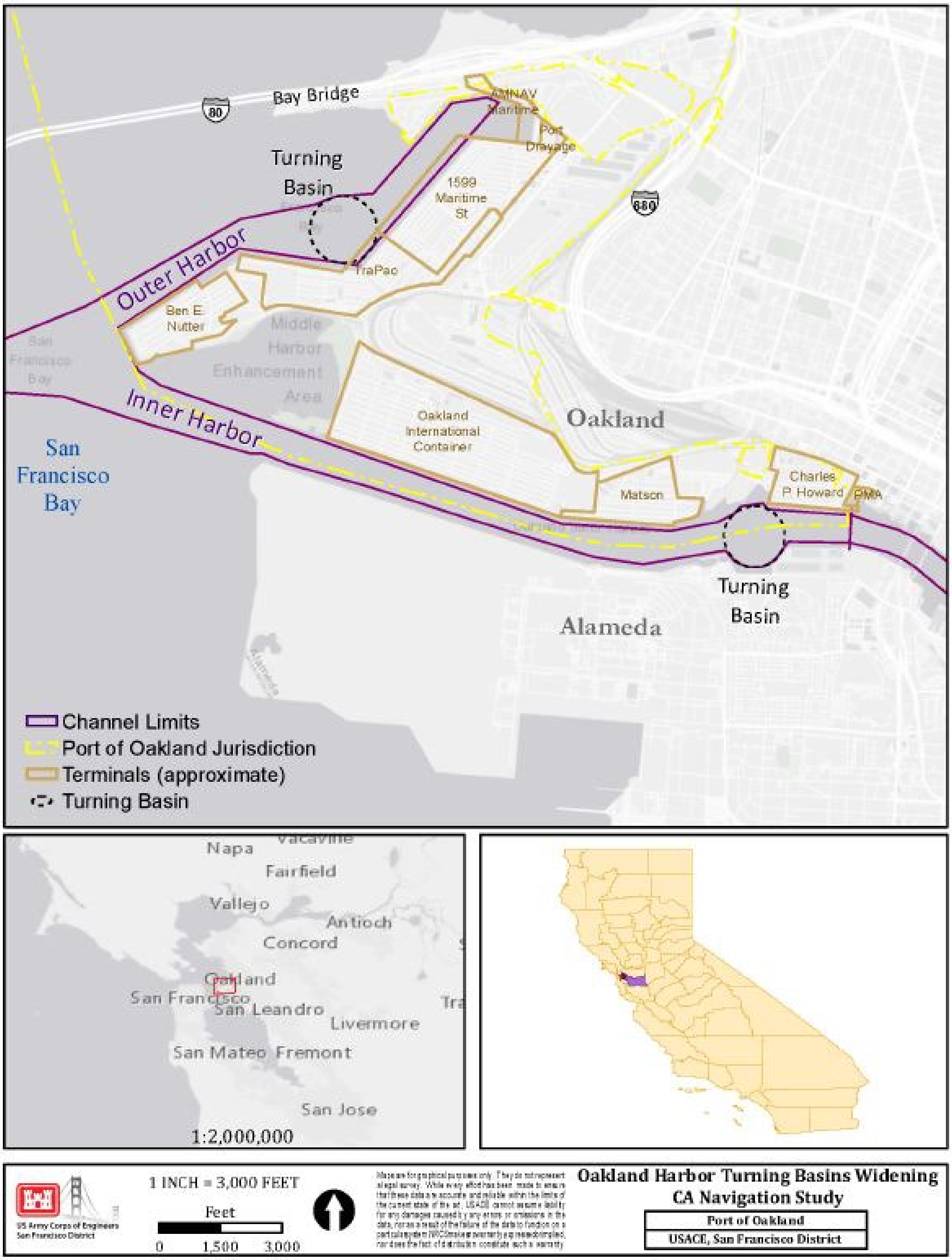


Figure 1: Study Area Location

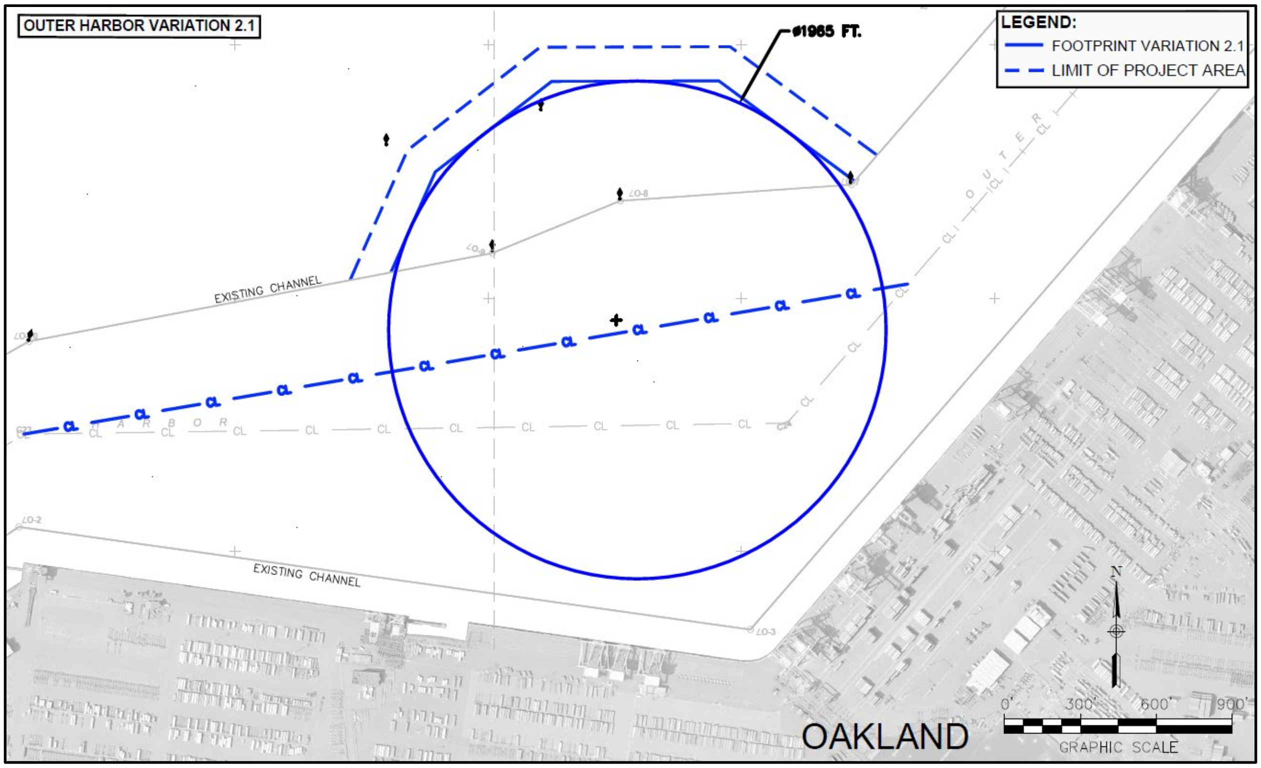


Figure 2: Outer Harbor Turning Basin Proposed Footprint

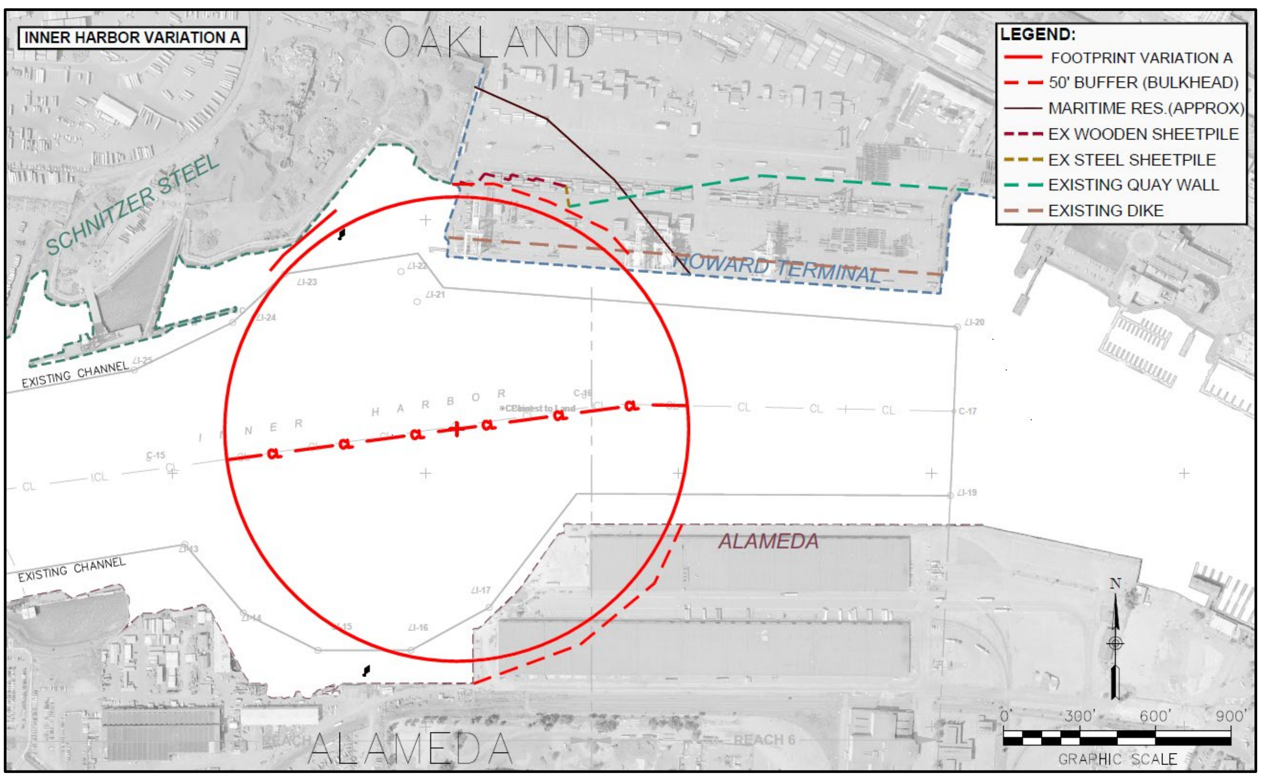


Figure 3: Inner Harbor Turning Basin Proposed Footprint

1.2. Datums

This Appendix relies on existing subsurface information taken from various consultant and agency reports, and as-built plans for existing facilities. The conversion factors presented in Table 1 were used to convert the reported elevations to NAVD88. All Elevations in this Appendix are reported relative to NAVD88 unless otherwise noted. Mean Low Low Water is approximately equal to NAVD88. These conversions are considered accurate enough for interpretation of subsurface data.

Table 1. Datum Conversions

Datum	Elevation (NAVD88)
MLLW	-0.2
NAVD 29	+ 2.7
Port of Oakland Datum (P.O.D.)	-0.5
City of Oakland Datum	+5.7

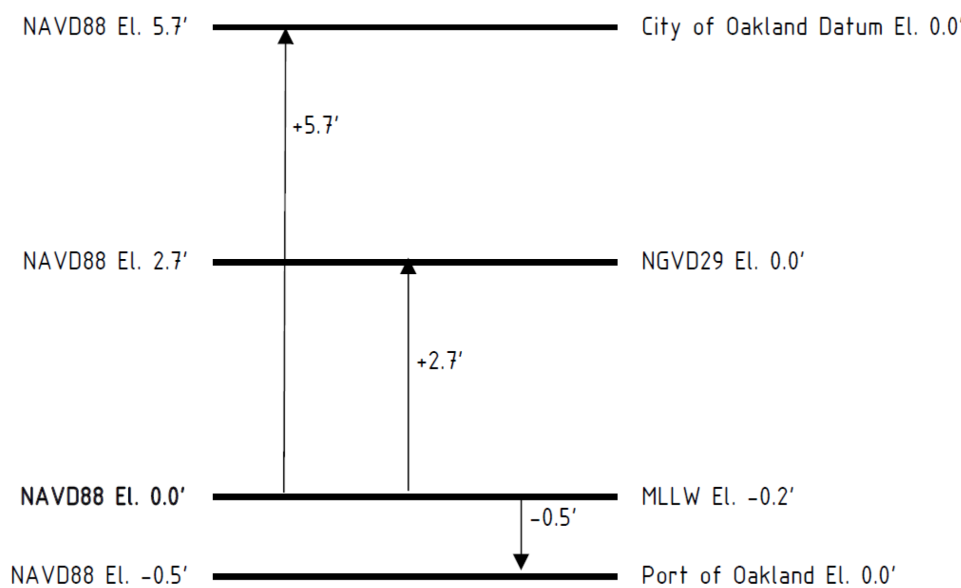


Figure 4: Datum Schematic

2. Project History

The first federal improvement of the Oakland harbor was authorized by the Rivers and Harbors Act adopted June 23, 1874. These improvements consisted of constructing two jetties to act as training walls to confine the flow of the San Antonio Estuary to scour a channel, the jetties were completed in 1894. The jetties no longer serve a navigational purpose and segments have been removed during subsequent improvements to the harbor. Significant change in the federally authorized channel have taken place in 1931, 1942, 1974-1975, and 2001-2010. In 1931, the Outer Harbor entrance was widened. The Outer Harbor was deepened to -35 feet and the turning basin was expanded in 1942. The deepening of the Inner Harbor to -35 feet was authorized in the Act of 1962 and completed in 1974. The authorized project for deepening the Entrance Channel, Outer Harbor and Inner Harbor channels to -42 feet was completed in 1998 and authorized by Section 202 of the Water Resources Development Act of 1986. The Inner and Outer Harbor were deepened to Elevation -50 feet between 2001 and 2010.

3. Geology

The Port of Oakland was constructed in a natural drainage channel, San Antonio Creek, which is located within the broad low-lying plain that borders the eastern shore of San Francisco Bay. The majority of the Port of Oakland, including the turning basin areas are located beyond the historic shoreline. Figure 5 shows the historic shoreline and former tidal flats (Radbruch, 1959). Materials beneath the bay plain consist of relatively thick deposits of unconsolidated marine sediments deposited during the Pleistocene and Holocene geologic time. Bedrock is on the order of 450 to 600 feet deep at the site.

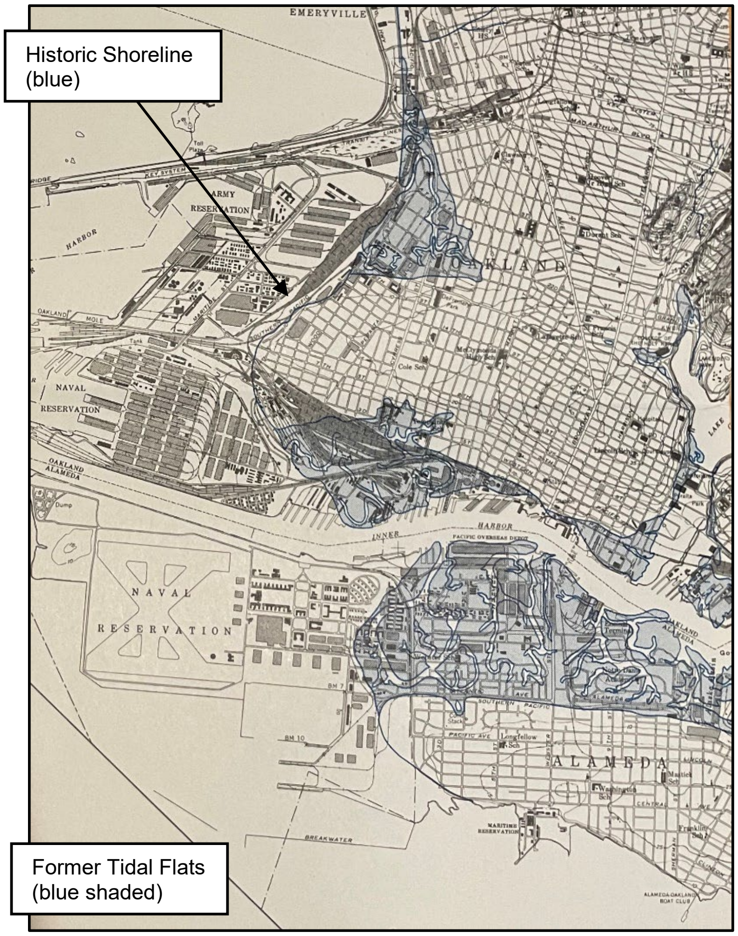


Figure 5: Former Shoreline and Tidal Flats

Helley and Graymer (1997) map the surface geology of the Port as artificial fill (af) over estuarine mud; see Figure 6. The soils immediately underlying the fill in former tidal flats consist of soft, compressible clays, known locally as Young Bay Mud (YBM). Young Bay Mud (YBM) is a soft, highly compressible marine clay that underlies much of the Port of Oakland. The YBM varies in the thickness across the site and may be deeper locally where it has filled eroded channels in the underlying formations. The YBM has been removed from the existing federal channel and turning basins. YBM and San Antonio Formation sands are exposed in the existing channel side slopes.

The San Antonio Formation consists of continental deposits, including the Merritt Sand Formation and the alluvial Posey Formation. The Merritt Sand Formation is mapped at the ground surface to the northeast of the Port. The Merritt Sands (Qm or Qms) are generally uniformly graded and medium dense to dense dune (aeolian) sand.

The Alameda Formation consists of interbedded Pleistocene sands and clays. The Alameda Formation was dissected and eroded prior to deposition of the Yerba Buena Formation, also known as Old Bay Mud or Old Bay Clay. The two units are often mapped together as layered sediments and have a combined thickness of several hundred feet. The proposed excavations are not anticipated to penetrate these formations. Figure 7 shows a geologic cross-section through the Inner Harbor

(Radbruch, 1957). The section shows that the YBM within the Oakland Estuary channel has been removed.

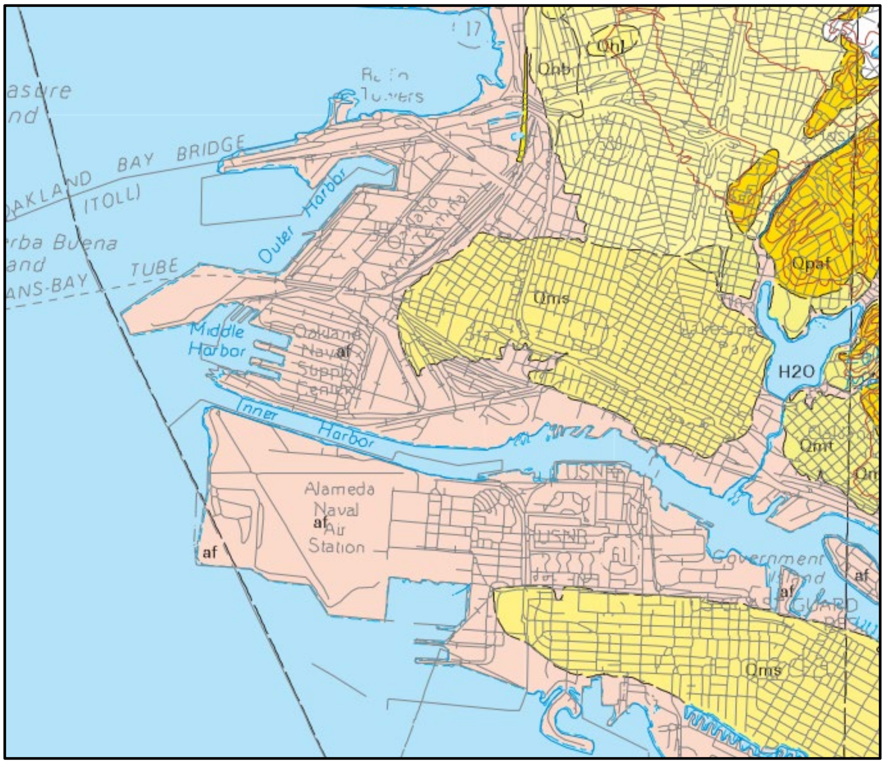


Figure 6: Surficial Geologic Map

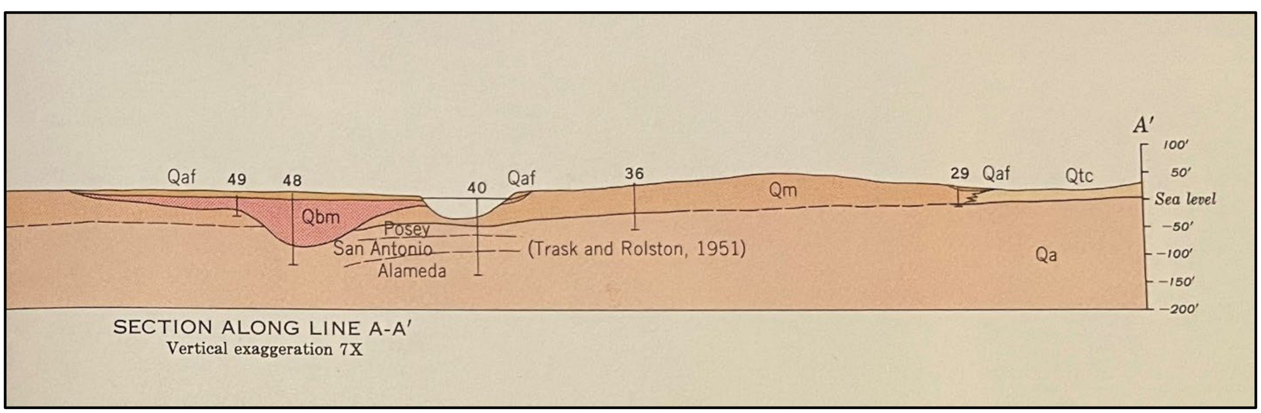


Figure 7: Geologic Cross-Section through the Inner Harbor

3.1. Seismicity

The San Francisco Bay area is recognized by geologists and seismologists as one of the most seismically active regions in the United States. Significant earthquakes occurring in the Bay area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system. A regional fault map is presented as Figure 8, illustrating the relative distances of the site to significant fault zones. The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The San Andreas Fault generated the great San Francisco earthquake of 1906 and the Loma Prieta earthquake of 1989 and passes 13 miles southwest of the site. The Hayward Fault is located approximately 4½ miles to the northeast.

Figure 8: Regional Active Faults

The Working Group on California Earthquake Probabilities developed estimates of future earthquakes in California. Their most recent report, the Uniform California Earthquake Rupture Forecast (2014), estimates that there is a 72% chance of a magnitude 6.7 or greater earthquake on one of the Bay Area faults between 2014 to 2044, and a 90% chance of a magnitude 6 or greater during the same time period (Field and WGCEP, 2015).

4. Outer Harbor

4.1. Existing Conditions

The Oakland Outer Harbor Turning Basin is located in the Outer Harbor Channel near Berths 25 through 30. The diameter of the turning basin is 1,650 feet. Figure 9 shows the current Outer Harbor Turning Basin (white circle) and the limit of the existing federal channel (white lines). The areas to the southwest of the white line are maintained to an Elevation of -50 ft by annual maintenance dredging. Figure 10 shows Cross-Section A-A' though the dredged slope.

Figure 9 also shows the proposed OHTB footprint (red circle), as well as, the top and toe of the proposed 3:1 (H:V) dredged slopes (red lines), locations of borings performed within or near the OHTB.

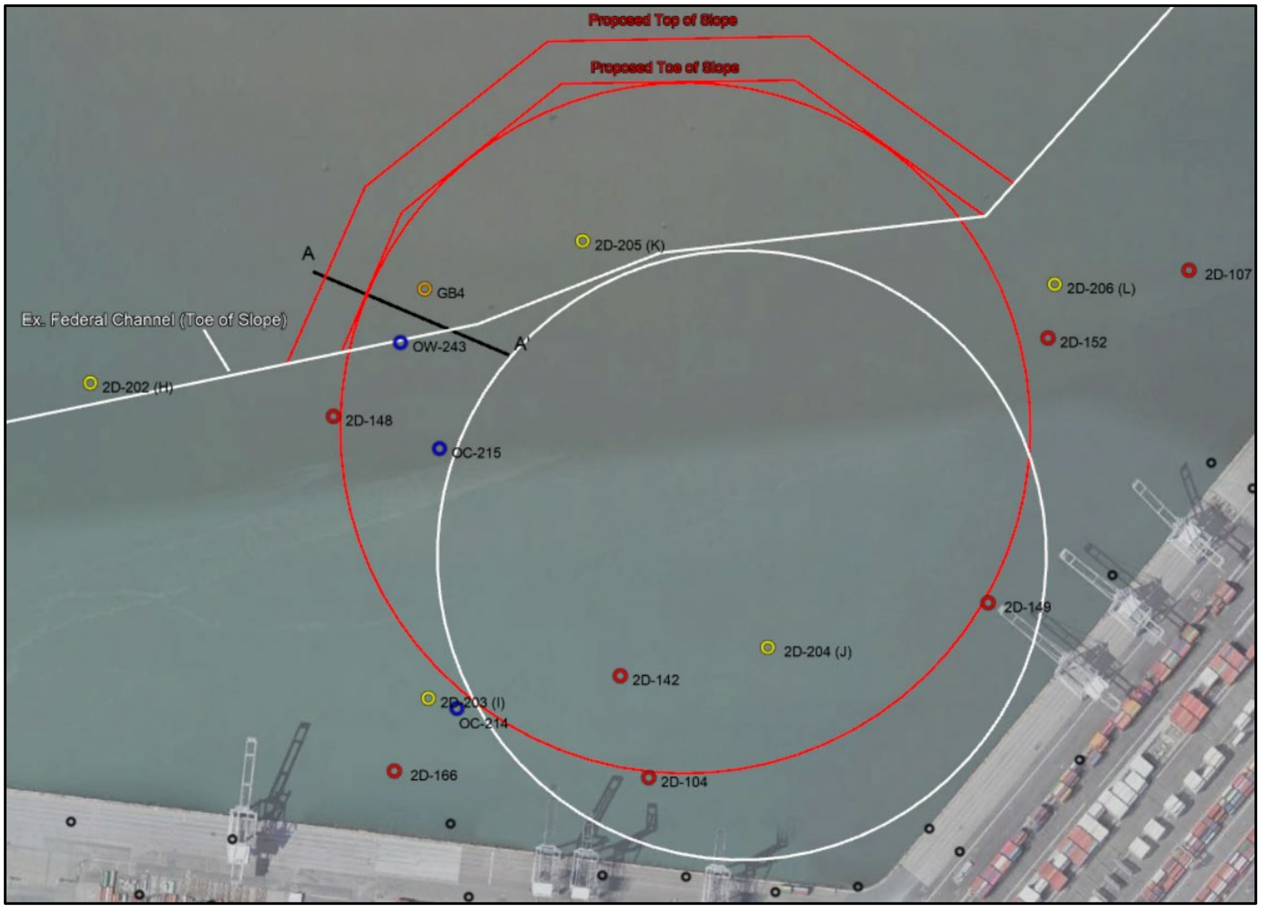


Figure 9: Outer Harbor

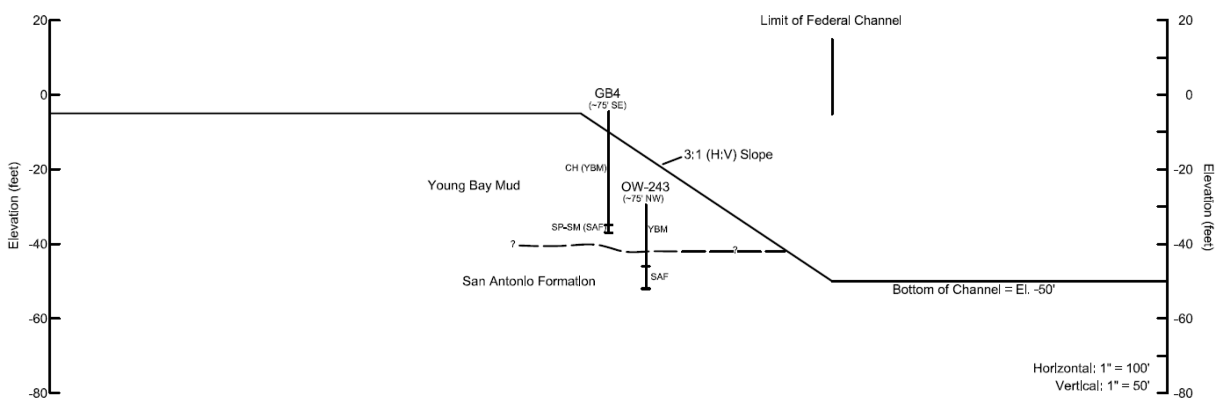


Figure 10: Outer Harbor Cross-Section A-A' (Existing)

Table 2 lists the borings with the study area and summarizes elevation of the contact between the YBM and San Antonio Formation, which ranges from Elevations -35 and -50 feet. The soils within the existing turning basin have been removed to Elevation -52 feet, meaning that the YBM has been removed from the OHTB. YBM is expected to be present in the excavation for the proposed turning basin and exposed in the permanent cut slopes.

Table 2. Outer Harbor Borings

Report	Boring #	Drill Date	Bottom of YBM Elevation (ft)[1]
Winzler & Kelly (1982)	2D-202 (H)	5/19/1982	> -47.5
	2D-203 (I)	5/18/1982	-44
	2D-204 (J)	5/19/1982	-43.5
	2D-205 (K)	5/18/1982	-44.5
	2D-206 (L)	5/20/1982	-47.5
USACE (1987)	2D-104	8/1973	-41.5
	2D-107	8/1973	-40.5
	2D-142	7/1974	> 46.6
	2D-148	7/1974	> -47.5
	2D-149	5/1975	-38.5
	2D-152	5/1975	-41
	2D-166	8/1977	> -40
EVS (1997)[2]	OC-214	8/15/1997	-48
	OC-215	8/15/1997	-50
	OW-243	8/5/1997	-46
SCI (1999)	***GB4***	9/23/1997	-35

[1]Elevations reported in NAVD88 / MLLW.
[2]Stratigraphy summarized in SCI (1999); boring logs not available.
Bold Italics included in Section 11 “Selected Borings”

As discussed in the previous section, there are two geologic units within the OHTB area: Young Bay Mud and San Antonio Formation sands. The YBM is a soft, highly compressible marine clay. Borings GB4 and OW-243, the closest borings to the proposed cut, encountered soft Fat Clay (CH) to Elevations -35 and -46 feet, respectively. Boring Lab test results show that the clays had dry densities of 38 to 45 pounds per cubic foot (pcf) and moisture contents of 100 to 123 percent. Below the YBM, the borings encountered very dense, poorly-graded sand with silt (SP-SM).

Approximately 1 to 2 feet of new material is deposited annually within the federal channel and turning basin. The most recent Operations and Maintenance Dredging Sampling and Analysis Report shows that the annual dredge material are typically silts and clays. (USACE, 2017).

4.2. Proposed Conditions

OHTB Variation 2 will require excavating material to the northwest of the existing turning basin. The existing turning basin slopes are inclined at 3:1 (H:V). No major slope failures have been observed along the existing federal channel. The conditions in the area to be excavated are expected to be similar to those encountered in borings GB4 and OW-243, as discussed in the previous section.

Figure 11 presents the proposed slope configuration. A 3:1 (H:V) slope was selected for preliminary design to match the existing slopes along the federal channel. Slope stability analyses were performed to evaluate the end-of-construction and a long-term stability of the cut slopes. Preliminary analyses indicate that the slopes would have a minimum factor of safety of 2.4. Figure 12 shows the analyzed cross-section and soil properties for the undrained stability case. The long-term stability of the slope was evaluated using effective stress parameters.

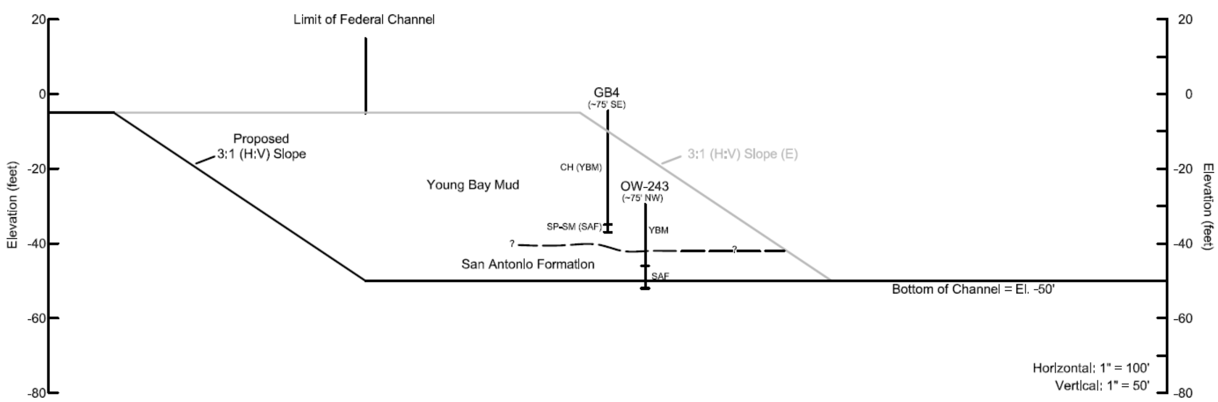
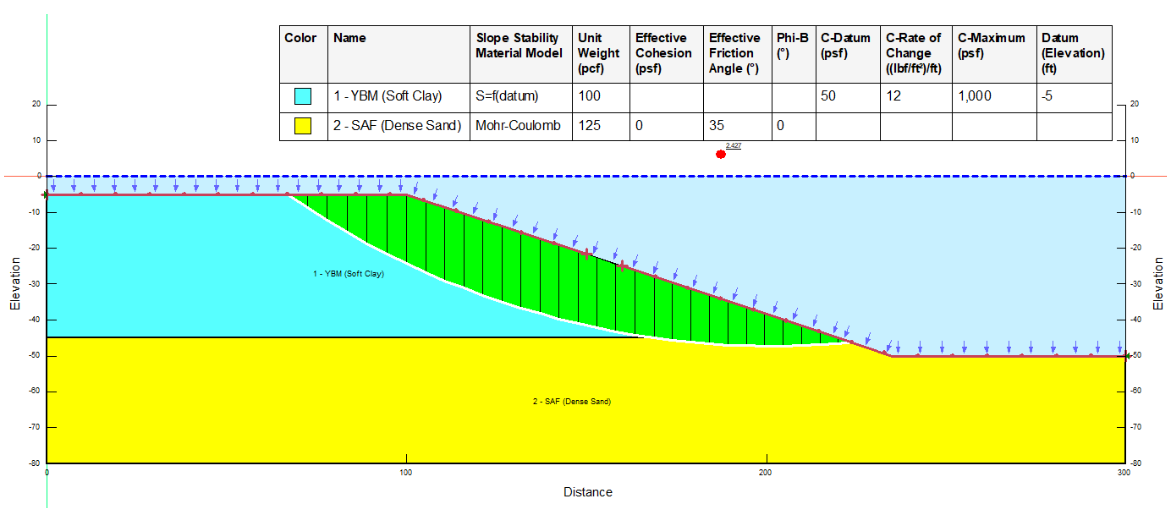


Figure 11: Outer Harbor Cross-Section A-A' (Proposed)



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	C-Datum (psf)	C-Rate of Change ((lbf/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)
	1 - YBM (Soft Clay)	S=f(datum)	100				50	12	1,000	-5
	2 - SAF (Dense Sand)	Mohr-Coulomb	125	0	35	0				

Figure 12: Slope Stability Analysis Results

4.3. Design Considerations

Slope stability analysis indicates that the slopes inclined at 3:1 (H:V) would have a long-term, static factor of safety of approximately 2.4 or greater. Slope reliability, seismic slope stability and deformation analyses may be performed during the design development phase of the Feasibility Study. It may be feasible to steepen the slopes to minimize cut volume. Additional geotechnical explorations should be performed during PED to confirm the soil conditions and design assumptions.

The San Antonio Formation sands are dense to very dense. A dredgeability analysis should be performed during design development, including review of dredging records from the -50 Foot Project.

5. Inner Harbor

The Oakland Inner Harbor turning basin is located approximately 18,000 ft to the east of the Oakland Harbor entrance. The diameter of the turning basin is 1,500 ft. It is maintained to an Elevation of -50 feet, by annual maintenance dredging. Figure 13 shows pertinent features.

- Existing federal channel (white)
- Existing bulkhead walls at Schnitzer and Alameda (yellow)
- Proposed Turning Basin (red)
- Proposed bulkhead walls at Schnitzer, Howard Terminal, and Alameda (blue)

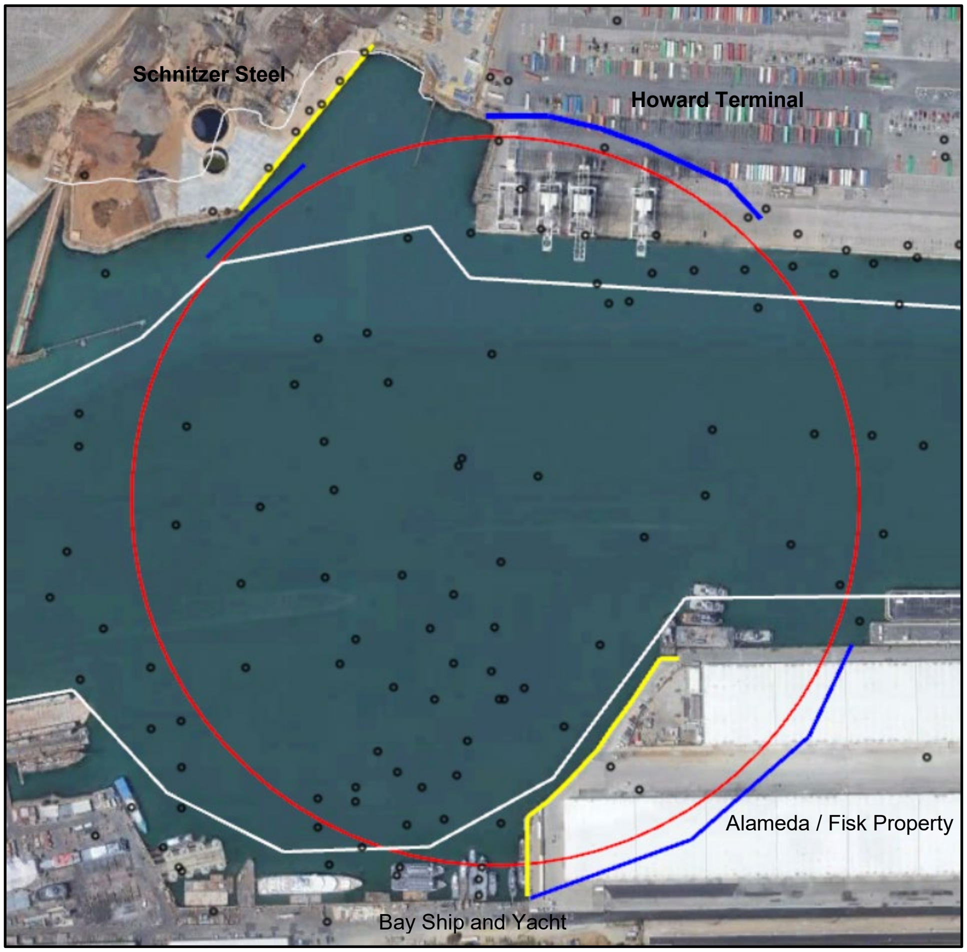


Figure 13: Inner Harbor

5.1. Existing Conditions

Areas within the federal channel are dredged to a minimum Elevation or -50 feet annually. Borings within the federal channel are shown on Figure 13. Similar to the Outer Harbor, the soils in the IHTB area consist of YBM over dense San Antonio Formation sands. Borings performed prior to dredging of the channel indicate that the bottom of the YBM generally ranged from Elevation -33 to -40 feet in the turning basin area. Much or all of the YBM within the federal channel has been removed by previous dredging projects.

Approximately 1 to 2 feet of new material is deposited annually within the federal channel and turning basin. The most recent Operations and Maintenance Dredging Sampling and Analysis Report shows that the annual dredge material are typically silts and clays. (USACE, 2017).

5.2. Proposed Conditions

The proposed improvements for each area (Howard Terminal, Schnitzer Steel, and Alameda) are discussed in Sections 6, 7, and 8, respectively.

6. Howard Terminal

Howard Terminal was constructed in 1980. There is an existing rock buttresses beneath the Howard Terminal Wharf. The TSP requires constructing a new bulkhead wall at Howard Terminal (Figure 14, blue).

6.1. Existing Conditions

Figure 14 shows the current configuration of Howard Terminal and available borings. Howard Terminal is a pile-supported wharf structure with a rock dike beneath. Figure 15 shows a typical cross-section through Howard Terminal based on the construction drawings. The footprint of the rock dike is represented by the yellow line on Figure 14.

The Woodward-Clyde Consultants (1979) Geotechnical Investigation report for the Howard Terminal recommends that all YBM be removed from beneath the rock dike and that the rock dike should be founded on the underlying dense sand. The typical section shows that the design bottom “elevation varies,” but is typically near Elevation -30 feet. The borings summarized in Table 2, as well as, the “Bottom of Bay Mud” contour map contained in 1979 Woodward-Clyde Report indicate that the bottom of YBM is typically shallower than Elevation -30 feet within the rock dike footprint, but may be as deep as Elevation -38 feet. Engeo (2019) Boring 1-B3 encountered 2 to 3 feet of YBM at the base of the dike, indicating that some YBM remains in place.

Figure 14: Howard Terminal

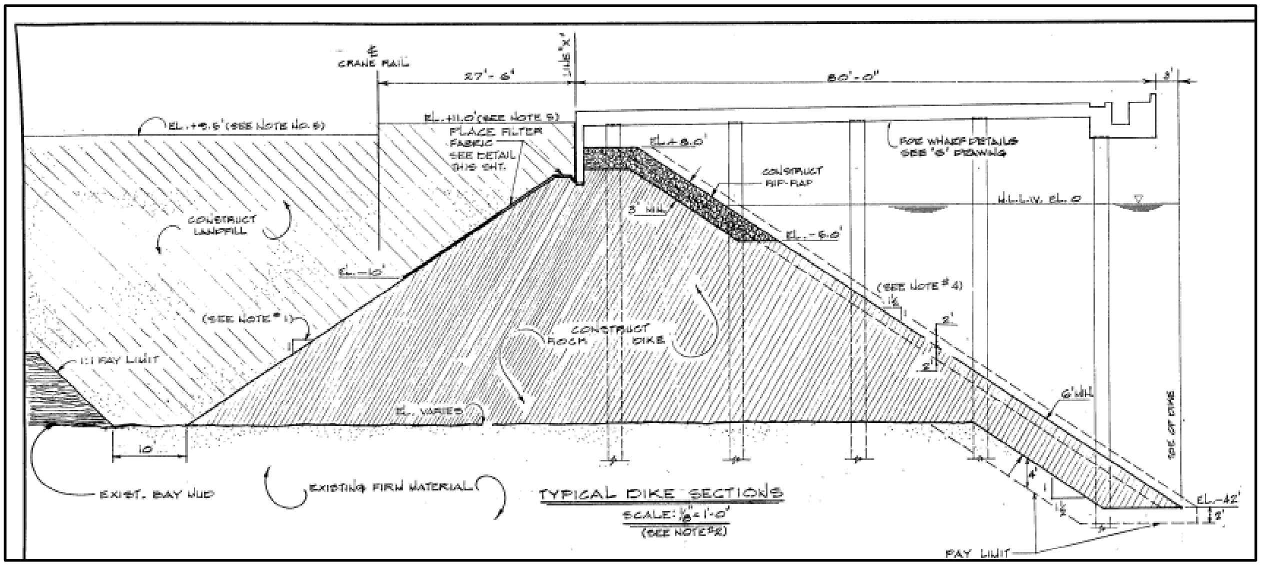


Figure 15: Typical Detail of Howard Terminal

Table 3. Howard Terminal Borings

Report	Boring #	Drill Date	Bottom of YBM Elevation (ft)[1]
USACE (1922)	22-67	1922	-33
	22-68	1922	-33
	22-69	1922	-33
USACE (1924)	24-1	1924	-26
	24-2	1924	-26.5
	24-3	1924	-26.5
	24-4	1924	-27.5
	24-5	1924	-27.5
	24-6	1924	-29
WCC (1979)	79-5	9/29/1978	-30
	79-6	10/2/1978	-23
	79-7	9/27/1978	-22
	79-8	6/28/1979	-29
	79-11	7/5/1979	-24
	79-12	7/6/1979	-22
	79-13	7/16/1979	-40
	79-16	8/29/1979	N/E, Fill
	79-17	8/29/1979	-6
SCI (1999)	GB27	9/16/1997	N/E
Engeo (2019)	***1-B3***	1/30/19	-41.5
	CPT-05	1/15/19	N/E[3]

[1]Elevations reported in NAVD88 / MLLW.
[2]Stratigraphy summarized in SCI (1999); boring logs not available.
[3]N/E – YBM Not Encountered

Bold Italics included in Section 11 "Selected Borings"

The rock buttress material is described in Woodward-Clyde (1979) as follows: "The rock used in the dike must possess both high strength and durability to be stable at 1½ to 1 slope against all future design loading conditions. In addition, the gradation of the rock should be such that the rock dike is porous enough not to allow any buildup of pore water pressures during seismically induced shaking. This latter requirement would infer that the rock sizes should be as large as possible with little to no fine particles. However, the subsequent construction of a wharf structure over the dike would entail installation of foundation piles through the dike. If the rock sizes in the dike were too large, it would not be practical to drive the piles through them. For this latter consideration, it was the consensus that if the rock size exceeded 12 inches, then there might be inordinate difficulties in pile installation operations. This consensus, therefore, determined the maximum rock size to be allowed in the dike section (as 12 inches) where piles will be installed. In rock dike areas where no piles will be installed in the future, larger rock sizes can be allowed."

The rock buttress material encountered in Engeo (2019) Boring 1-B3 is consistent with the Woodward Clyde recommendations; "Poorly graded gravel with clay (GP-GC), 1-inch to 2-inch diameter, subangular, very strong."

Samples of the material on the face of the slope were recently collected by a diver from the Port of Oakland collected hand samples. The material was generally 3- to 6-inch, sub-rounded to sub-angular cobbles.

Figure 16: Rock Dike Material Sampled by Diver

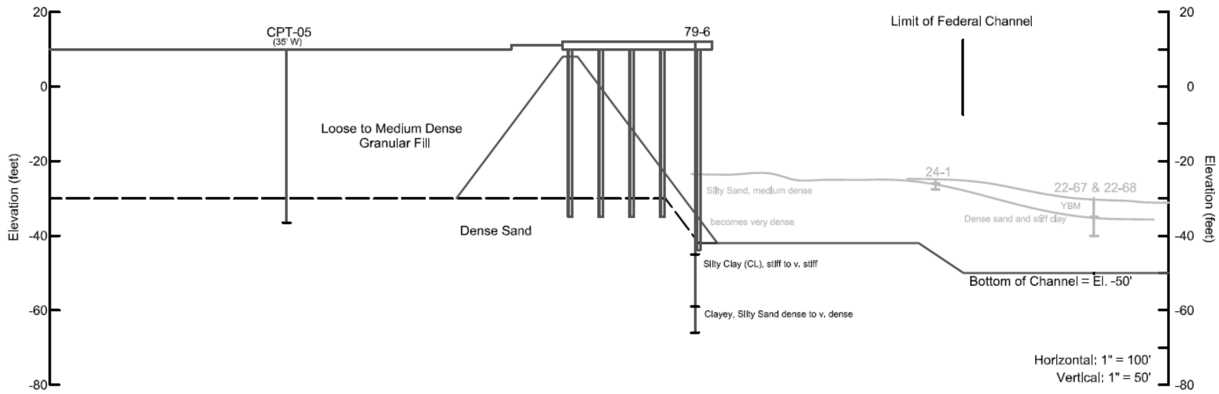


Figure 17: Howard Terminal Cross-Section B-B' (Existing)

Behind the rock buttress is a zone of artificial fill and was likely hydraulically placed. Woodward-Clyde (1979) report recommends that "fill to be placed under water consist of cohesionless fine to medium and medium to coarse grained sand, with maximum allowable fines content of less than 10 percent by weight." The material encountered in Engeo (2019) CPT-05 was consistent with this description; loose to medium dense sandy soil. Preliminary analysis of CPT-05 indicates that the fill could liquefy during a moderate to large earthquake.

Liquefaction was documented at Howard Terminal following the 1989 Loma Prieta Earthquake: "Liquefaction of the hydraulic fill caused appreciable settlements (max 30 cm) over large areas of the Howard and APL Terminals. Although pavement was damaged at the edges of the wharves and in the inboard container yards, there was no apparent damage to piles or adverse movements of the crane rails. (USGS PP 1551-B)" Further research for site-specific reports will be performed during the design development phase.

The wharf deck is founded on five rows of 24" concrete octagonal piles, driven through the buttress and founded in the underlying dense sand. The crane rail is supported on a row of 16" square concrete piles, battered in each direction.

6.2. Proposed Conditions

IHTB Variation 3 would require removal of a portion of the existing rock buttress beneath Howard Terminal and construction of a new bulkhead wall. At the feasibility level, the bulk head wall is assumed to be similar to the bulkhead wall that was constructed at the Fisc Property on the Alameda side of the channel for the -50-foot project. A detail of the Alameda bulkhead wall is presented in Figure 18. The wall employed vertical and battered piles. The design parameters and preliminary calculations are provided in the Structural Engineering Appendix. The wall should be designed to withstand seismic forces, including the added load of the liquefied fill behind the rock buttress. Depending on the final wall geometry and retained soils, stiffening the wall or densification (ground improvement) of a zone of fill may be required.

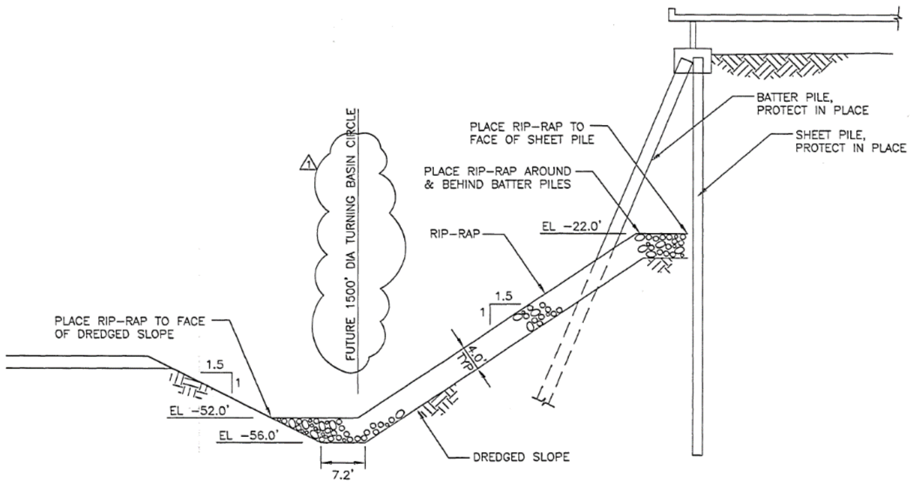


Figure 18: Conceptual Wall Detail

Figure 19 shows the proposed bulkhead through Cross-Section B-B'. Cross-Section B-B' is located between Station 13+50 and 14+00. The distance from the proposed bulkhead wall to the existing face of the wharf varies along the length of the wall.

Figure 21 shows the approximate wall location (blue) at six locations along the existing Howard Terminal Wharf. The wall will retain up to approximately 30 feet of loose to medium dense granular fill. The easternmost 100 to 150 feet, from approximately Station 17+00 to the end of the wall, will require pile driving through the rock dike.

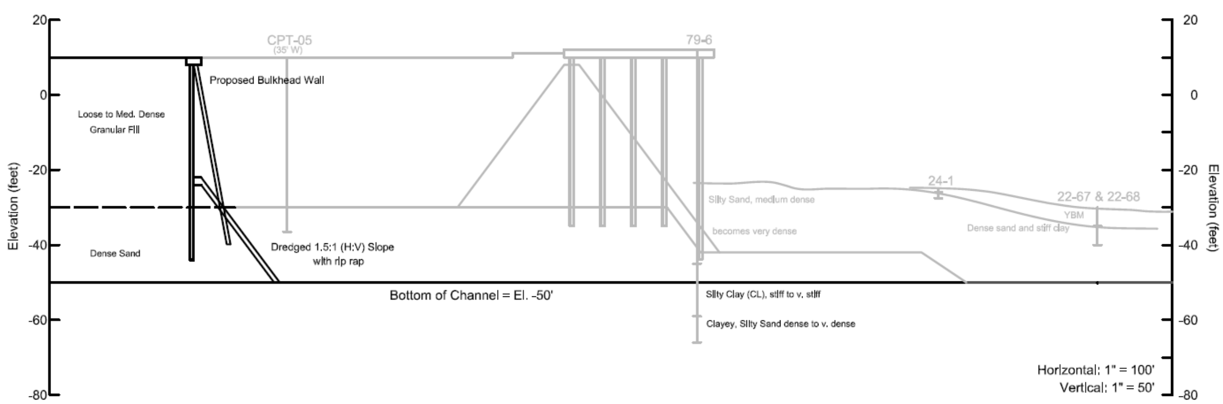


Figure 19: Howard Terminal Cross-Section B-B' (Proposed)

Figure 20: Proposed Bulkhead Wall Location at Various Cross-Sections

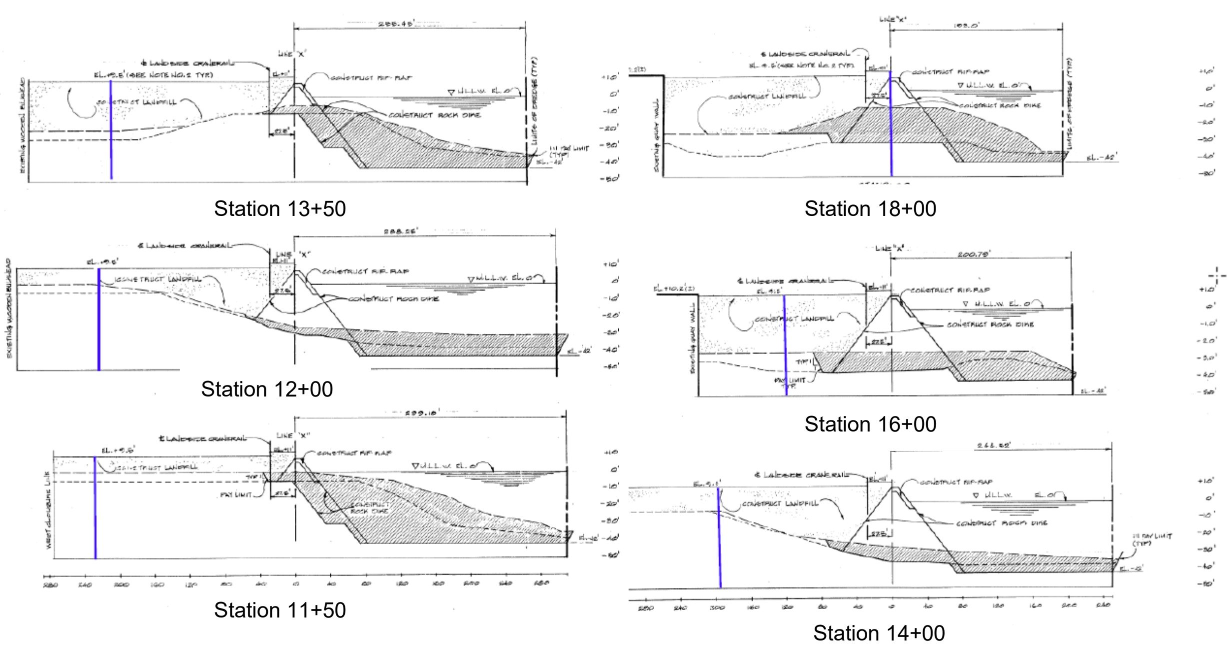


Figure 21: Proposed Bulkhead Wall Location at Various Stations

6.3. Design Considerations

The preliminary design employs vertical and battered piles. The final design may consider other design measures such as tie-backs and/or dead man anchors. The wall should be designed to withstand seismic forces, including the seismic lateral soil pressure and the load of the liquefied fill. Depending on the final wall geometry, densification (ground improvement) of a zone of fill may be required.

The project will require removal of the existing concrete pile foundations. Pile can be removed or cut below the proposed finished grades.

The existing rock buttress and dense sand may contribute to difficult pile driving. A preliminary pile driving analysis should be performed during design development.

The San Antonio Formation sands are dense to very dense. A dredgeability analysis should be performed during design development, including review of dredging records from the -50 Foot Project.

7. Schnitzer Steel

7.1. Existing Conditions

Most of the Schnitzer Steel property is built on filled land. The green line on Figure 22 shows the approximate 1956 shoreline. Subsequently, additional fill was placed in the later 1960's. The fill contains debris (wood, metal, etc). The existing bulkhead wall (yellow) was constructed circa 1973.

A photograph of the wall and plan are shown in Figures 24 and 25, respectively. The wall is constructed of steel H-piles at 9.3 feet on center, with horizontal steel "hatch covers" spanning between piles. The wall is also supported by 50-foot long, 1-3/4" tie rods and dead man anchors at 10 feet on center. There is a zone of compacted fill behind the wall.

Granular fill with varying amounts of debris (concrete, brick, wood, etc.) was encountered in each of the seven borings performed behind the Schnitzer Steel wall. Borings were generally performed t a depth of 20 feet or less. Deeper fill was encountered in MW-8 which is located within a historic slough. Boring SB-5 encountered YBM below the fill to approximately Elevation -6 feet. The boring did not fully penetrate the YBM layer into stiffer soils below; the elevation of the bottom of the YBM was not determined.

The existing federal channel and turning basin in front of the Schnitzer Steel wall have been excavated to Elevation -50 feet. The YBM within the limits of the federal channel has been removed as shown in borings OI86-1 and GB27.

Figure 22: Schnitzer Steel

Table 4. Schnitzer Steel Borings

Report	Boring #	Drill Date	Bottom of YBM Elevation (ft)[1]
GeoResource	***OI86-1***	10/23/1986	N/E, YBM removed
SCI (1999)	***GB27***	9/16/1997	N/E, YBM removed
Terraphase (2020)	SB-5	5/25/2016	N/E, YBM > El. -6 [3]
	SB-6	6/1/2016	N/E, Fill to > El. -2 [2]
	SB-7	6/1/2016	N/E, Fill to > El. -0.5 [2]
	MW-2	3/7/1991	N/E, Fill to > El.-4 [2]
	MW-8	4/26/2017	N/E, Fill to > El. -13 [2]
	MW-11	5/8/2019	N/E, Fill to > El. -8 [2]
	MW-12	5/9/2019	N/E, Fill > El. -8 [2]

[1]Elevations reported in NAVD88 / MLLW.
[2]Borings did not penetrate the bottom of the fill. Top of YBM Elevation not determined.
[3]Borings did not penetrate the bottom of the YBM. Bottom of YBM Elevation not determined.
Bold Italics included in Section 11 “Selected Borings”

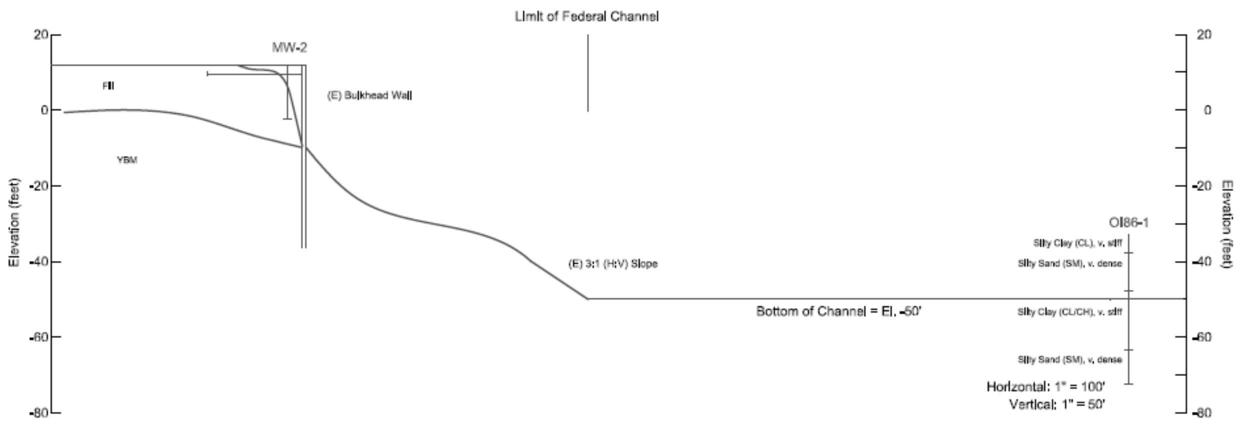


Figure 23: Schnitzer Steel Cross-Section C-C'(Existing)

Figure 24: Schnitzer Steel Wall

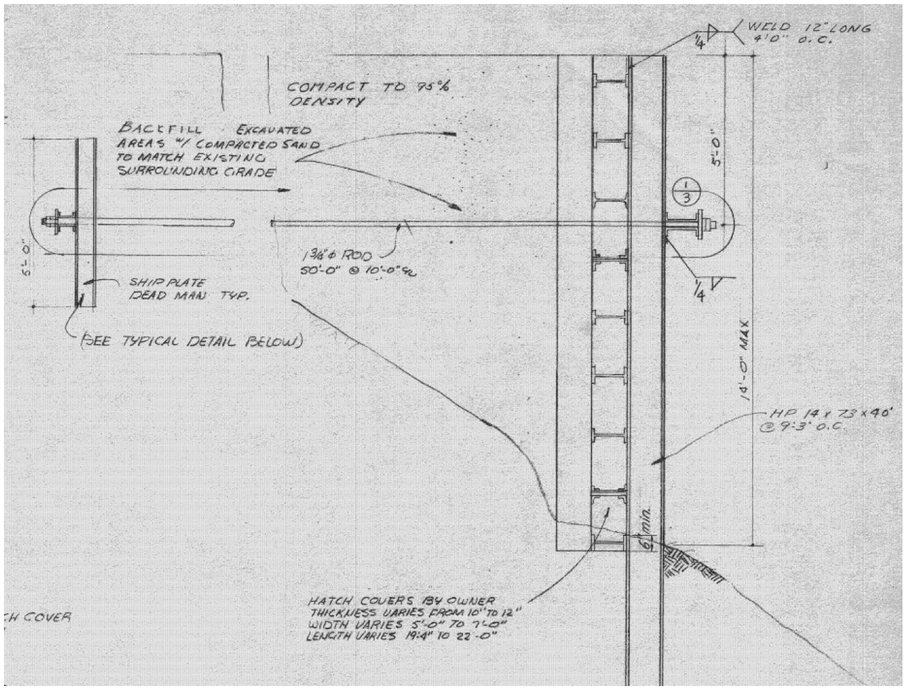


Figure 25: Schnitzer Wall Plan

7.2. Proposed Conditions

The TSP includes a below-grade, in-water wall in front of the Schnitzer Steel property in the northwestern portion of the Turning Basin. The proposed wall location is shown in blue on Figure 22. The wall will be approximately 300 to 400 feet long, and will be entirely submerged. The wall will likely be a concrete secant wall or driven pile structure. The wall will be offset 10 to 20 feet from the existing Schnitzer Steel wall in the direction of the turning basin, and will be designed so that soil removed as part of the turning basin project will not have any effects on the Schnitzer Steel wall. The top of the wall will be flush with the existing grade (mudline) at the base of the Schnitzer wall. The proposed wall will retain approximately 20 to 25 feet of soil that will be needed to create the necessary depth of the turning basin.

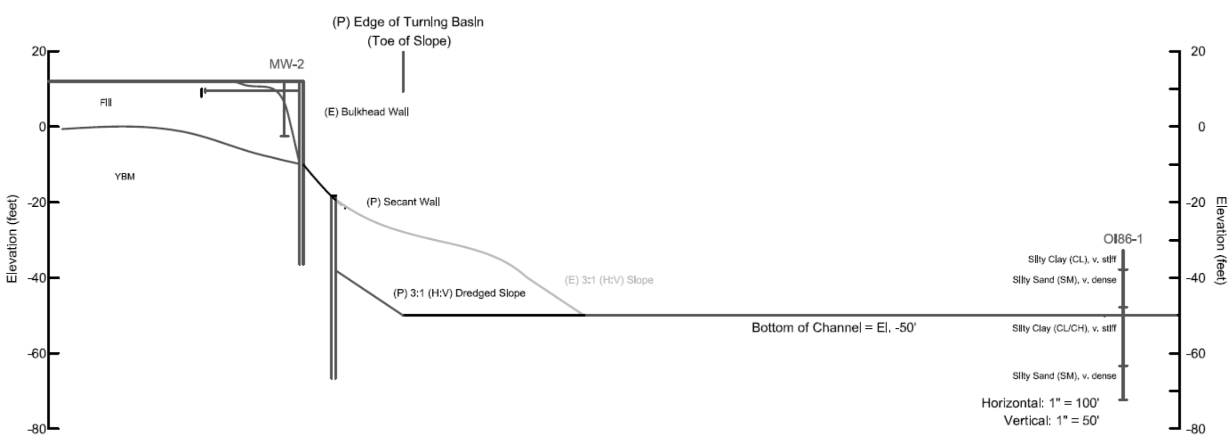


Figure 26: Schnitzer Steel Cross-Section C-C'(Proposed)

7.3. Design Considerations

Due to the previous use of the area, there is a potential for buried debris within the dredge area and proposed wall footprint. Geophysical and bathymetric surveys of the cove between Schnitzer Steel and Howard Terminal are planned during the Feasibility Study. The purpose is to detect buried objects that my conflict with the proposed wall construction.

8. Alameda

8.1. Existing Conditions

The existing bulkhead wall was constructed during the -50 foot project. The bulkhead wall is shown in yellow in Figure 27 is comprised of vertical and battered, concrete-filled steel piles. The wall is founded in dense sands and very stiff clays. There is a 1.5:1 (H:V) slope in front of the wall with rip rap rock slope protection. The area in front of the wall has been dredged to Elevation -50 feet.

The existing warehouse structures are founded on both concrete and timber piles bearing in the underlying dense sand.

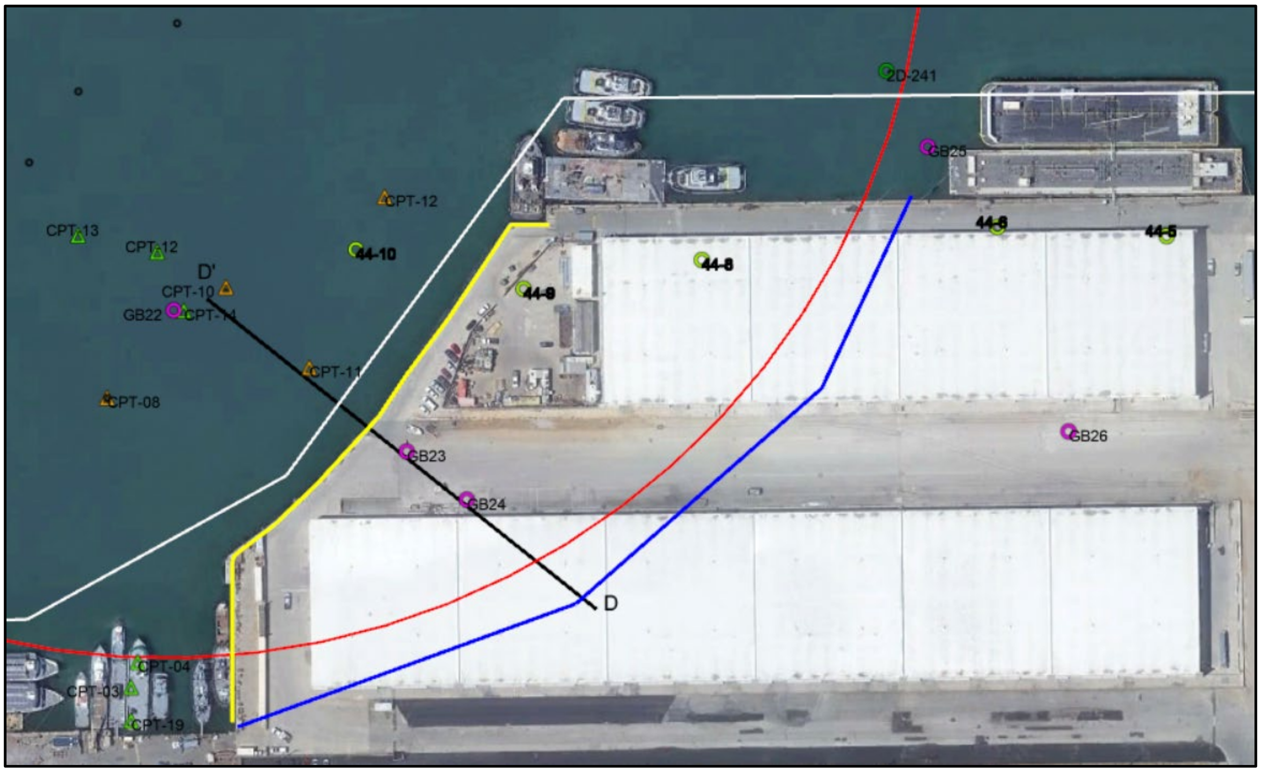


Figure 27: Alameda/Fisk Property

Table 5. Alameda Borings

Report	Boring #	Drill Date	Bottom of YBM Elevation (ft)[1]
USACE (1944)[2]	44-5	1944	-41.3
	44-6	1944	-34.1
	44-8	1944	-18.7
	44-9	1944	-18.0
	44-10	1944	-20.4
SCI (1999)	***GB22***	9/13/1997	N/E, YBM
	GB23	8/5/1997	-21.0
	GB24	8/5/1997	-15.0
	GB25	9/12/1997	-37.0
	GB26	8/7/1997	-38.0
Fugro (2003) [Green]	CPT-03	7/1/2003	-54.9
	CPT-04	7/1/2003	-52.7
	CPT-06	4/7/2003	-50.0
	CPT-12	4/8/2003	-54.1
	CPT-13	4/9/2003	-53.6
	CPT-14	4/10/2003	N/E, YBM
	CPT-19	7/1/2003	-26.6
Engeo (2007) [Orange]	CPT-08	3/20/2007	-53.4
	CPT-10	3/21/2007	-45.3
	CPT-11	3/21/2007	-45.1
	CPT-12	3/21/2007	-45.3

[1]Elevations reported in NAVD88 / MLLW.
[2]Boring logs not available; elevation of firm soil shown on plan.
Bold Italics included in Section 11 "Selected Borings"

9. Further Analysis and Design Development

The findings presented in this appendix are preliminary. Design will be further developed during the prior to the Agency Decision Milestone. The TSP for the Inner Harbor requires excavation at Howard Terminal and on private property on the Alameda side of the channel. Assumptions about the existing conditions and configuration of the slopes, wharf structures, and bulkhead walls in these areas were based on review of as-built plans and limited site reconnaissance. Existing conditions should be verified during the PED phase. Depending on the type of structural analysis required for design of the bulkhead walls, site-specific seismic hazard and site response analyses may be required.

Geophysical and bathymetric surveys of the cove between Schnitzer Steel and Howard Terminal are planned during the Feasibility Study. The purpose is to detect buried objects that my conflict with the proposed wall construction.

Additional geotechnical subsurface exploration should be performed during PED. A detailed plan will be developed a part of this feasibility study.

Chemical sampling of the soil in some areas may be required for disposal purposes. Disposal assumptions are discussed in the Channel Design Appendix.

10. References

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11. Selected Borings

Area	Report	Boring #
Outer Harbor	SCI (1999)	GB4
	SCI (1999)	79-6
	Fugro (2003) [Green]	
	Engeo (2007) [Orange]	
Howard Terminal	WCC (1979)	79-6
		79-7
		79-8
	Engeo (2019)	1-B3

Area	Report	Boring #
		CPT-05
Schnitzer Steel	GeoResource (1986)	OI86-1
	SCI (1999)	GB27
Alameda	SCI (1999)	GB22
		GB23
		GB24
		GB25
		GB26