COWI



ALAMEDA COUNTY, CALIFORNIA

PROJECT # PRWWF 24-002
SPECICATION # 32400202

Berkeley Water Transportation Pier Ferry Project

BASIS OF DESIGN – 60% SUBMITTAL

CITY OF BERKELEY

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1 SUMMARY

1.1 Project Scope

The following describes the Basis of Design (BoD) for the Berkeley Water Transportation Pier Ferry (BWTPF) Project, which is to partially replace the existing Municipal Pier in Berkeley, CA. The project consists of the following elements:

Waterside:

- A pile supported pier 22ft wide by 1080 ft long,
- A canopy structure along a portion of the pier
- A 15ft wide by 300 ft long breakwater
- Two ferry landings. The ferry landings consist of steel float, fenders, mooring hardware, aluminum boarding ramp system, guide piles, donut fenders, aluminum gangway, access platform and security gates systems. Note one ferry landing will be constructed initially. The second landing may be added at a future date.
- Electrical, mechanical, and fire protections systems on the pier and ferry landings.

Landside:

- Landside site improvements include renovation of the existing parking lot for ferry users; street improvements at Seawall Drive (with new pavement, safety striping for vehicle, bicycle, and pedestrian crossings, and stormwater bioswales to treat stormwater) and at University Ave (with new layover pullouts for public buses and shuttles, drop-off zone for shuttles, rideshares, and family vehicles safety striping for vehicles, bike lanes, and pedestrian crossings to the ferry terminal); and the renovation and expansion of the existing plaza at the pier abutment. Also included will be renovation of pedestrian pathways and amenities, including safety lighting, new restroom; erosion repair on east side of south cove parking lot, and landside electric vessel charging infrastructure.

1.2 Existing Conditions

1.2.1 Topographic Survey

Topographic survey of the site was conducted in August 2024 by Mountain Pacific Surveys.

1.2.2 Bathymetric and Lidar Survey

Bathymetry data for the site was produced by eTrac Engineering Inc, a Woolpert Company. For COWI. Survey data was collected on July 24, 25, 27 and 29, 2024. Plotted on August 22, 2024

1.2.3 Shore Protection

The existing rip rap slope at the pier ferry terminal was installed at an unknown date. Visual inspection conducted in 2024 by COWI confirmed the existing rip rap slope is in reasonable conditions. See report A272767 BWTPF Project – Shoreline Protection Review for details in Appendix D.

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1.2.4 Geotechnical Conditions

Project geotechnical findings and recommendations by ENGEO are provided in Appendix A.

1.3 Project Datum

The project datum is described as follows:

- Vertical data shall reference City of Berkeley MLLW (hereafter referred to as MLLW); Vertical Control Point # 13, Elevation 19.96'[1]
- Horizontal data shall reference NAD83, SPCS California Zone 03. Horizontal control Point # 13 2144146.431'N, 6036359.948'E

Conversions:

- EL 0.0 NAVD88 = +0.57 feet MLLW[2].

1.4 Water Levels

1.4.1 Tidal Datum

Water levels were retrieved from the Berkeley tidal gauge (ID 9414816), which is the closest station to the project site, and reference information provided by the National Oceanic and Atmospheric Administration (NOAA) benchmark sheet. Table 1 presents the water levels recorded at the station which form the basis for our project site. Tidal data is per NOAA averages based on the 1983-2001 tidal epoch.

Note: MLLW values below refer to City of Berkeley MLLW vertical datum, values differ from the NOAA MLLW. This project adopted the City of Berkeley datum and refers to it as "MLLW" throughout this document.

Datum	Value NAVD88 (ft)	Value MLLW (ft)	Description
HAT	+7.8	+8.4	Highest Astronomical Tide, on 12/23/2003 18:42
MHHW	+6.2	+6.8	Mean Higher-High Water
MHW	+5.6	+6.2	Mean High Water
MSL	+3.4	+4.0	Mean Sea Level
NAVD88	0.0	+0.6	North American Vertical Datum of 1988
MLLW	0.1	0.7	Mean Lower-Low Water
LAT	-2.1	-1. 6	Lowest Astronomical Tide, on 06/04/2008 14:06

Table 1-1 - Tidal reference levels at Berkeley, CA 9414816

1.4.2 Extreme Water Levels

A study done by AECOM (San Francisco Bay Tidal Datums and Extreme Tides Study) in 2016 updated and expanded the USACE 1984 study of tidal datums and extreme tides for San Francisco Bay. The study modelled and presented detailed extreme tide information for the entirety of the Bay shoreline. Table 1-2 reproduces the extreme tide elevation for point ID 519, coinciding with the Berkeley Pier.

[1] eTrac, 2024, Memo – Vertical Analysis Between NOAA MLLW and City of Berkeley MLLW

[2] eTrac, 2024, Memo – Vertical Analysis Between NOAA MLLW and City of Berkeley MLLW

Return Period (years)	Extreme Tide Elevation (ft – NAVD88)	Extreme Tide Elevation (ft – MLLW)
1	7.4	8.0
10	8.5	9.0
50	9.3	9.9
100	9.7	10.3

Table 1-2 - Extreme Tide Elevation – ID 519 (AECOM, 2016)

The extreme water level considered for this project is 10.3 ft MLLW, with a return period of 100 years (1% Annual Exceedance Probability)

1.4.3 Sea Level Rise

For the BWTPF project, a design life of 50 years has been established, placing its planning horizon around the year 2075.

The 2024 California Sea Level Rise Guidance integrates the advancements in sea level rise science, including refinements in climate models, improved ice-sheet dynamics understanding, and better regional projections, drawing from the most recent IPCC's Sixth Assessment Report (AR6) and the Special Report on the Ocean and Cryosphere in a Changing Climate (SROCC).

The projection for the San Francisco gauge has been conservatively selected with the Intermediate Scenario to define the Sea Level Rise Allowance (SLR), as shown in Table 1-3.

A SLR vulnerability assessment for the structures up to the end of the century and for higher emission scenarios (intermediate-high and high) is presented in Appendix D.

Condition	Station	Scenario	SLR Allowance (ft)
Design Life 2075	San Francisco	Intermediate	1.6

Table 1-3 - Projection FEMA/NOAA and California Coastal Commission Sea Level Rise Policy Guidance 2024 Update for San Francisco.

1.4.4 Design Still Water Level

The Still Water Level at the project location is shown in Table 1-4. This elevation excludes waves and waves impacts, but includes astronomical tides, storm surges and sea level rise for the design life of the project.

Parameter	Elevation (ft-NAVD88)	Elevation (ft-MLLW)
Extreme Water Level (1% AEP)	9.7	10.3
Sea Level Rise Allowance	1.6	1.6
Design Still Water Level	11.3	11.9

Table 1-4 Design Still Water Level

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2 GOVERNING CODES AND REFERENCE DOCUMENTS

2.1 Codes and Standards

The following codes, specifications, regulations and industry standards, where applicable, shall cover the main design and material for the structures and other related items:

Principal General Design Standard:

1. Accessibility Standard for Pedestrian Facilities in the Public Right-of-Way (PROWAG) 2024
2. ASCE 61, Seismic Design of Piers and Wharves, American Society of Civil Engineers, 2014 (ASCE 61-14)
3. ASCE 7, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, 2022 (ASCE 7-22).
4. ABS, American Bureau of Shipping and Affiliated Companies, Rules for Building and Classing Steel Barges, 2024
5. Alameda Countywide Clean Water Program; C.3 Stormwater Technical Guidance, May 19, 2024 (Version 8.2)
6. Alameda County Hydrology & Hydraulics Manual
7. Berkeley Stormwater Requirements Checklists
8. California Regional Water Quality Control Board San Francisco Bay Region Municipal Regional Stormwater NPDES Permit, Order No. R2-2022-0018 as amended by Order No. R2-2023-0019, NPDS Permit No. CAS612008 (May 11, 2022)
9. CBC, California Building Code, California Building Standards Commission, 2022 (2022 CBC Vol 1 & Vol 2)
10. Caltrans Standard Specifications 2025 Edition
11. Caltrans Standard Plans 2025 Edition
12. Layout and Design Guidelines for Marina Berthing Facilities, California Department of Boating and Waterways, Boating Facilities Division, July 2005
13. Unified Facilities Criteria, UFC 4-151-01 Design: Pier and Wharves
14. Unified Facilities Criteria, UFC 4-159-03 Design: Moorings, June 16, 2016

In situations where the above standard does not cover a particular design situation the applicable design practices and guidelines may include, but are not limited, to the following:

1. ACI 318-19, Building Code and Commentary for Structural Concrete, American Concrete Institute, 2019 (ACI-318-19)
2. ACI 315-18, Manual of Standard Practice for Detailing Reinforced Concrete
3. ACI 543, Design, Manufacture and Installation of Concrete Piles, American Concrete Institute, 2012 (ACI PRC 543R-12)

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4. AISC 360, Specification for Structural Steel Buildings, American Institute of Steel Construction, 14th edition, 2022 (AISC 360-22)
5. API Recommended Practice 2A-WSD, 22nd Edition
6. ASTM Standards, Various, American Society for Testing and Materials International (ASTM).
7. AWS D1.1, Structural Welding Code – Steel, American Welding Society, 2020 (AWS D1.1:2020)
8. AWS D1.2, Structural Welding Code – Aluminum, American Welding Society, 2014 (AWS D1.2:2014)
9. IBC, International Building Code, International Code Council, 2021 (IBC 2021) NDS, National Design Specification for Wood Construction ASD/LRFD, 2024 edition, and National Design Specification Supplement, Design Values for Wood Construction, 2024 edition (2024 NDS).
10. PIANC International Navigation Association A.D. 1885 Guideless for Design of Fender System (2002) WG 33.
11. PCI Design Handbook: Precast and Prestressed Concrete, 8th edition, Prestressed Concrete Institute, 2017
12. US Access Board Proposed Passenger Vessel Accessibility Guidelines, June 25, 2013

2.2 Reference Documents

The following documents contain reference information for the project.

1. Feasibility Study, Ferry Facility at Berkeley Municipal Pier, by GHD, Inc, Concord, Dated June 30, 2023.
2. Berkeley Water Transportation Pier Ferry Project, Condition Hydrographic Survey & Aerial Lidar Survey, by eTrac Inc., a Woolpert Company, Dated August 2024.
3. San Francisco Bay Tidal Datums and Extreme Tides Study, AECOM, 2016
4. State of California Sea Level Rise Guidance, 2024 Science and Policy Update

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3 GENERAL REQUIREMENTS

3.1 Function

3.1.1 Typical Function

Pier

The pier is expected to be accessible to the public 7 days a week. The pier will have a gate at the shoreside end of the pier to allow for closure of the pier afterhours. The pier is intended to be used by pedestrians and emergency vehicles.

Ferry Landing

The ferry landing(s) are expected to have vessel operations seven (7) days a week, but not 24 hours a day. The landing(s) will be serviced by vessels operated by WETA. See Appendix B for a summary of vessels that may operate at the landing. The ferry landing(s) will consist of a queuing area along the shore, a covered pier, an aluminum gangway, aluminum boarding system, steel float, guide piles to hold the float in place, and donut fenders. The landing(s) will extend parallel to the pier and provide boarding on each side of the float.

Americans with Disabilities Act (ADA) codes and standards must be applied in the design of the ferry landing's passenger boarding system (gangway, float, ramps, and platforms). The design will be in accordance with U.S. Access Board "Proposed Passenger Vessel Accessibility Guidelines", dated June 25, 2013.

3.1.2 Emergency Function

Pier and Ferry Landing(s) – Risk Category IV - Essential Facility

Since WETA is responsible for coordinating and providing ferry transportation in response to emergencies or disasters affecting the Bay Area transportation system, the landing(s) will be designed for use as a staging area for first responders and evacuation by ferry in the event of a catastrophe. For this reason, the ferry landing shall be designed as an essential facility, CBC Risk Category IV.

Breakwater – Risk Category 2 - Normal Facility

The breakwater is not required for post disaster emergency response and is designed as Risk Category 2 facility.

3.2 Location

The Project site (see Figure 3-1) is located in Berkeley Marina, West on the intersection of Seawall Drive and University Avenue.

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Figure 3-1 - Site Location and Project Footprint

3.3 Design Life

The planned design life for the project will be as follows:

- Pier/Breakwater (all components) = 50 years
- Ferry Landing (all components) = 50 years

The above design life assumes that a regular inspection and maintenance program is implemented to repair damage and deterioration.

3.4 Units

All drawings and calculations will be provided in English units as follows:

- Length: Feet and/or inches
- Force: Kips or pounds
- Time: Seconds and/or minutes/hour/days/months/years
- Temperature: Degrees Fahrenheit

Other units may be used if their English unit equivalents are also provided.

4 LOADS AND LOAD COMBINATIONS

4.1 Dead Loads (D)

Dead load is comprised of the self-weight of the structure (i.e., weight of permanent structure, reinforced concrete, structural steel and timber).

4.1.1 Unit Weights

The following unit weights shall be used in the calculations:

- Reinforced concrete: 150 lb/ft3 (normal weight)
- Reinforced concrete: 90-115 lb/ft3 (light weight)
- Structural steel: 490 lb/ft3
- Cast iron: 450 lb/ft3
- Aluminum alloys: 175 lb/ft3
- Timber (untreated): 45 lb/ft3
- Timber (treated): 50 lb/ft3
- Asphalt concrete: 150 lb/ft3
- Seawater: 64 lb/ft3

4.2 Live Loads (L)

Live loads are non-environmental loads on the structures, which are not permanently in place.

4.2.1 Pier

- Uniform (LU), 250 psf
- Concentrated (LC), vehicle loading AASHTO HL-93

The pier will be subject to AASHTO HL-93 (fire truck) loading. Truck wheel loads will be calculated in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Standard Specification for Highway Bridges, per Figure 4-1.

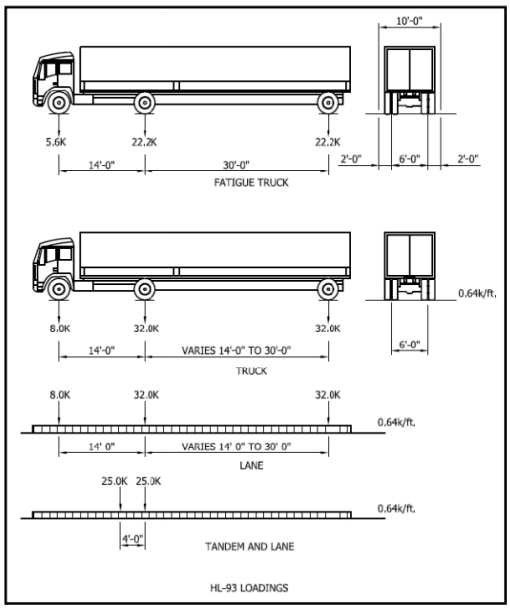


Figure 4-1 - AASHTO HL-93 Loads

4.2.2 Breakwater

- Uniform (LU), 250 psf
- Concentrated (LC), vehicle loading AASHTO H15-44

Breakwater will be subject to AASHTO H15-44 GVW 15 Tons (ambulance/maintenance truck) loading. Truck wheel loads will be calculated in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Standard Specification for Highway Bridges.

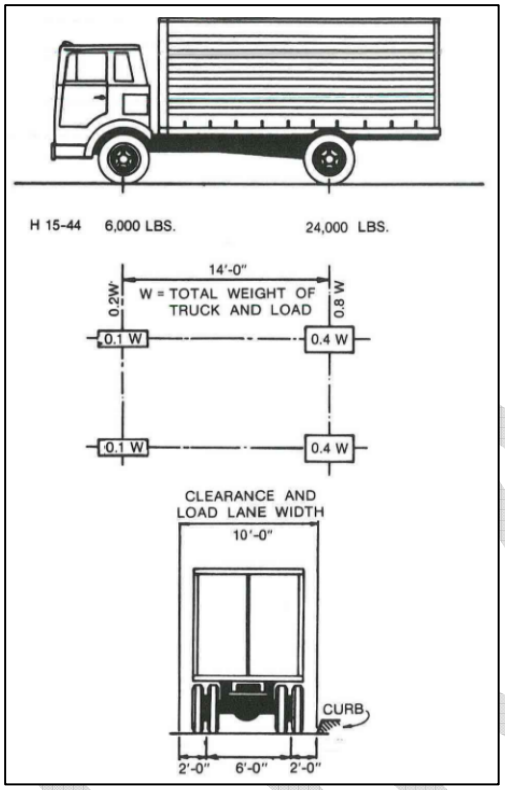


Figure 4-2 – AASHTO H15-44 Loads

4.2.3 Ferry Landing: Access Platform, Gangways, Walkway System, and Float

- Uniform Live Load (LU):
  - Access Platform Deck 250 psf
  - Gangway 100 psf
  - Float Pedestrian Walking Area 100psf
  - Float Deck
    - 40 psf for the structural design of the deck plating, beams, and connections of deck/ beams to frames, bulkheads, or walls
    - 0 psf (no uniform live load) applied to the deck for floating stability, draft, freeboard, heel, trim, sagging forces, hogging forces, or float torsion calculations

- Concentrated Live Load (LC)
  - Access Platform Deck 8000 lbs
  - All others 650 lbs

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For pier deck, LC is concentrated over an area 2.5 ft by 2.5 ft, applied at any point on the deck. For gangways, LC is concentrated over a 6" by 6" area applied at any point on the gangway. For float-mounted ramps and platforms, LC is concentrated over a 4.5" by 4.5" area applied at any point on the platforms. For floats, LC is concentrated over a 12" by 12" area applied at any point.

- Maximum deflection for gangways shall not exceed $L_{span}/360$.
- Marine Growth load of 5 psf shall be applied on submerged surfaces below MLLW

4.2.4 Gate/Railing

- Uniform (LU), horizontal or, 50 lb/ft
- Uniform (LU), vertical or, 100 lb/ft
- Concentrated (LC), horizontal or vertical 250 lbs

4.2.5 Stairs

- Uniform Live Load (LU) 100 psf
- Concentrated Live Load (LC), on one thread 300 lbs

4.2.6 Ladders

- Concentrated (LC), horizontal (rungs) 300 lbs
- Concentrated (LC), rails extending above platform (any direction) 100 lbs

4.2.7 Landside Area

- Uniform (LU), 250 psf
- Concentrated (LC), vehicle loading AASHTO HL-93

Landside area, including the ferry plaza and parking lot, will be subject to AASHTO HL-93 (fire truck) loading. Truck wheel loads will be calculated in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Standard Specification for Highway Bridges.

4.2.8 Plaza Railing

- Uniform (LU), horizontal or, 50 lb/ft
- Uniform (LU), vertical or, 100 lb/ft
- Concentrated (LC), horizontal or vertical 250 lbs

4.2.9 Utility Loads

The pier, access platform, gangway, float and fixed pier to be designed for weight of permanent utilities hung or placed on or under the deck.

4.3 Roof Live Load (Lr)

Loads pertain to canopy are designed in accordance with ASCE 7-22. Loads on the canopy are 59.45psf and act perpendicular to the canopy surface.

4.4 Buoyancy (Hydrostatic) Loads (B)

Hydrostatic pressures shall consist of uplift forces applied at the rate of 64 psf for every foot of submergence below water level.

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4.5 Berthing Loads (Be)

The energy (E_B) to be absorbed by fenders at berthing impact is referred to as the berthing energy. This energy will be determined using the kinetic energy method in accordance with PIANC WG33 Guidelines for Design of Fender Systems (2002) for the design vessel:

- $E_B = ½ \times W/g \times V_B^2 \times C_M \times C_B \times F_A$ where,
  E_B = Energy to be absorbed by fender system during berthing (k-ft)
  W = Displacement of a design vessel (kips)
  g = Acceleration of gravity = 32.17 ft/s^2
  V_B = Berthing velocity of the vessel (ft/sec)
  C_M = Added mass coefficient, for catamarans or single-hulls as applicable, per MOTEMS/ PIANC = 1.35 to 2.38
  $C_B = C_E C_G C_D C_C$
  C_E = Eccentricity coefficient per MOTEMS/ PIANC = 0.52 to 0.57
  C_G = Geometric coefficient ≡ 1.0
  $C_D = C_S$ = Deformation / softness coefficient ≡ 1.0
  C_C = Berth configuration coefficient ≡ 1.0
  F_A = Accidental factor for abnormal conditions = 1.0 for this project, which closely resembles existing projects

4.5.1 Vessel Berthing Design Conditions

- Vessel Speed = 1.5 ft/sec
- Incident Angle = 10 degrees
- Berthing Point on Vessel = L/4 (quarter point)

4.5.2 Donut Fender Berthing Design Conditions

- Vessel Speed = 1.2 ft/sec
- Incident Angle = 10 degrees
- Berthing Point on Vessel = L/4 (quarter point)

4.5.3 Design Vessels

Ferry Vessels

The ferry landing shall be designed for WETA's San Francisco Bay Ferry Fleet of vessels. See Appendix B for data pertaining to design ferry vessels characteristics (PENDING INFORMATION FROM WETA ON NEW ELECTRIC DESIGN VESSELS PARAMATERS).

4.6 Current Loads (C)

The design surface current speed is 1 knot.

The force, F_C, on the vertical faces of the float due to this current velocity is calculated per:

- $F_C = ½ \times C_D \times A \times \rho/g \times V^2$ where
  F_C = Static Transverse Current Force
  C_D = drag force coefficient
  A = area [ft^2] of the vertical face on which the current is acting
  ρ = 1.99 slugs/ft^3 for seawater
  g = 32.2 ft/sec^2
  V = current velocity

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4.7 Earth and Hydraulic Pressures on Retaining Structures (H)

Static earth pressures shall be considered in the design of the abutment along the shoreline, if applicable. Lateral earth pressures due to seismic forces shall be considered in the design of the retaining structures.

Hydraulic pressures due to water level differentials, resulting from tidal fluctuations and/or groundwater accumulations shall be considered in the design of retaining structures, if applicable.

4.8 Seismic Loads (EQ)

4.8.1 Pier and Breakwater

Seismic design of the pier and ferry will follow the displacement-based method described in ASCE 61-14. As the ferry landing pier is an "essential" facility that needs to be operational after a seismic event, the seismic performance criteria used for design is more stringent than the design classification "High" as described in ASCE 61-14 Section 2.2.1. As there is no clear guidance for essential facilities in ASCE 61-14, a more rigorous criteria were developed, see Table 4-1. The breakwater is designed as Risk Category Medium as defined in ASCE 61-14. Pseudo-Spectral Acceleration for the various earthquake are included in Table 4-3.

Operating Level Earthquake (OLE)	50% in 50 years (72-year return period)	No damage
Contingency Level Earthquake (CLE)	10% in 50 years (475-year return period)	Minimal Damage
Design Earthquake (DE)	Design Earthquake per ASCE 7	Minimal Damage
Maximum Considered Earthquake (MCE)	MCE Earthquake per ASCE 7	Controlled and Repairable Damage

Table 4-1 – Pier Performance Criteria (Risk Category Essential)

Operating Level Earthquake (OLE)	N/A	N/A
Contingency Level Earthquake (CLE)	20% in 50 years (224-year return period)	Controlled and Repairable Damage
Design Earthquake (DE)	DE per ASCE 7	Life Safety

Table 4-2 – Breakwater Performance Criteria (Risk Category Medium)

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PERIOD (seconds)	PSEUDO-SPECTRAL ACCELERATION (g)				
	OLE (72-year)	CLE (225-year)	CLE (475-year)	DE (ASCE7-05)	MCE (ASCE 7-05)
0.01	0.25	0.41	0.54	0.57	0.85
0.02	0.24	0.41	0.53	0.57	0.85
0.03	0.25	0.41	0.53	0.56	0.85
0.05	0.27	0.45	0.58	0.62	0.93
0.08	0.34	0.55	0.72	0.77	1.16
0.10	0.41	0.66	0.85	0.91	1.36
0.12	0.46	0.73	0.93	0.98	1.47
0.15	0.52	0.82	1.05	1.00	1.50
0.20	0.58	0.93	1.18	1.00	1.50
0.25	0.62	1.00	1.28	1.00	1.50
0.30	0.63	1.05	1.35	1.00	1.50
0.40	0.60	1.04	1.38	1.00	1.50
0.50	0.57	1.01	1.35	1.00	1.50
0.60	0.51	0.93	1.25	1.00	1.50
0.75	0.43	0.81	1.13	0.80	1.20
1.00	0.35	0.70	1.00	0.60	0.90
1.50	0.24	0.51	0.75	0.42	0.63
2.00	0.18	0.40	0.60	0.32	0.47
3.00	0.11	0.26	0.40	0.20	0.30
4.00	0.07	0.18	0.29	0.15	0.23
5.00	0.05	0.13	0.21	0.12	0.18
7.50	0.03	0.07	0.11	0.08	0.12
8.00	0.02	0.06	0.10	0.08	0.11
10.00	0.02	0.04	0.06	0.06	0.09

Table 4-3 – Pseudo-Spectral Accelerations

No Damage Stain Limits
Pier structure is to remain elastic.

Minimal Damage Strain Limits
Pier structure to meet "Minimal Damage" requirement per ASCE 61-14 Table 3-1

Table 3-1. Strain Limits for "Minimal Damage" per Section 2.4.3

		Hinge location		
Pile type	Component	Top of pile	In ground	Deep in ground (>10D_p)
Solid concrete pile	Concrete	$\varepsilon_c \le 0.005$	$\varepsilon_c \le 0.005$	$\varepsilon_c \le 0.008$
	Reinforcing steel	$\varepsilon_s \le 0.015$		
	Prestressing steel		$\varepsilon_p \le 0.015$	$\varepsilon_p \le 0.015$
Hollow concrete pile[a]	Concrete	$\varepsilon_c \le 0.004$	$\varepsilon_c \le 0.004$	$\varepsilon_c \le 0.004$
	Reinforcing steel	$\varepsilon_s \le 0.015$		
	Prestressing steel		$\varepsilon_p \le 0.015$	$\varepsilon_p \le 0.015$
Steel pipe pile	Steel pipe		$\varepsilon_s \le 0.010$	$\varepsilon_s \le 0.010$
	Concrete	$\varepsilon_c \le 0.010$		
	Reinforcing steel	$\varepsilon_s \le 0.015$		

[a]If the interior of the hollow pile is filled with concrete, all strain limits shall be the same as for solid piles.

Controlled and Repairable Stain Limits

Pier structure to meet "Controlled and Repairable Damage" requirement per ASCE 61-14 Table 3-2.

Table 3-2. Strain Limits for "Controlled and Repairable Damage" per Section 2.4.2

		Hinge location		
Pile type	Component	Top of pile	In ground	Deep in ground (>10D_p)
Solid concrete pile	Concrete	$\varepsilon_c \le 0.005 + 1.1\rho_s \le 0.025$	$\varepsilon_c \le 0.005 + 1.1\rho_s \le 0.008$	$\varepsilon_c \le 0.012$
	Reinforcing steel	$\varepsilon_s \le 0.6\varepsilon_{smd} \le 0.06$		
	Prestressing steel		$\varepsilon_p \le 0.025$	$\varepsilon_p \le 0.025$
Hollow concrete pile[a]	Concrete	$\varepsilon_c \le 0.006$	$\varepsilon_c \le 0.006$	$\varepsilon_c \le 0.006$
	Reinforcing steel	$\varepsilon_s \le 0.4\varepsilon_{smd} \le 0.04$		
	Prestressing steel		$\varepsilon_p \le 0.020$	$\varepsilon_p \le 0.025$
Steel pipe pile	Steel pipe		$\varepsilon_s \le 0.025$[b]	$\varepsilon_s \le 0.035$
	Concrete	$\varepsilon_c \le 0.025$		
	Reinforcing steel	$\varepsilon_s \le 0.6\varepsilon_{smd} \le 0.06$		

[a]If the interior of the hollow pile is filled with concrete, all strain limits shall be the same as for solid piles.
[b]If the steel pipe pile is infilled with concrete, a value of 0.035 may be used.

4.8.2 Kinematic Loading

Kinematic loads occur in piles or retaining structures when the soil embankment liquefies or slides due to slope instability during an earthquake, inducing bending moments in piles beneath the soil surface. The liquefaction potential of the existing soils with the proposed ground improvement are determined during the geotechnical investigation performed by ENGEO. If the soils are found to liquefy or develop critical slope failure surface, displacement profiles or equivalent soil pressures will be determined according to the recommendations in the geotechnical report. The pier shall be analyzed for kinematic loading on the piles and the material strains in the piles shall not exceed the strain limits.

The effects of kinematic loading shall be combined with the inertial seismic loading per ENGEO's recommendations as follows:

- 1.0 $E_{inertial}$ + 0.5 $E_{kinematic}$
- 1.0 $E_{kinematic}$ + 0.5 $E_{inertial}$

4.8.3 Pier Canopy Structure

The pier canopy is a non-building structure that is supported by another structure and shall be designed Per ASCE 7-22.

4.8.4 Ferry Landing

4.8.4.1 Gangway

Seismic load shall be determined as per ASCE 7-22, Chapter 15.

4.8.4.2 Float and Guide Piles

Earthquake loads are not considered in the design of these structures.

4.9 Wind Loads (W)

4.9.1 Extreme Wind Speed

Long term wind data is available for NOAA Station 9414750 Alameda, and 9414290 San Francisco. Short term data is also available for approximately 12 years for NOAA Station 9414847 Point Potrero.

Local wind data measurements for the Berkeley Marina (approximately five years) and Reef Light (less than two years).

The design parameters shown in *Table 4-4* derived from statistical analysis of the Alameda Station data.

Return Period (years)	Wind Speed (knots)	Wind Speed (mph)
10	35	40.3
50	39.8	45.8
100	42	48.3

Table 4-4 – Extreme Wind Conditions – Alameda Station

4.9.2 Wind Loads on Pier, Canopies, and other structures

Pier and canopies are classified as Open Structures and shall be designed to resist wind loads computed in accordance with ASCE 7-22.

- Basic Wind Speed (3-s gust) V 115 mph
  Basic wind speed is used for design of pier and canopy framing for strength and canopy cladding and its connections to steel framing and corresponds to approximately a 3% probability of exceedance in 50 years (Mean Return Interval = 1,700 years).

- Service Wind Speed V_{serv} 85 mph
  Service wind speed is used for drift limits in design of canopy steel framing and corresponds to approximately a 63% probability of exceedance in 50 years (Mean Return Interval = 50 years).

Wind Design Parameters:

- Risk Category IV
- Exposure Category D
- Topographic Factor, Kzt 1.0
- Directional Factor, Kd 0.85

Deflection Criteria for Canopy

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- Components and Cladding shall be designed for worst-case deflection resulting from pressures derived using basic wind speed

4.9.3 Wind Loads on Floats

Maximum three-second ultimate (LRFD) gust speed for mooring and berthing of ferries against float: 40 mph (Combined with other loads for design of mooring bitts, spring line lugs, guide piles, and guide pile collars.)

4.10 Wave Loads (Wa)

Numerical modelling of the wave conditions was performed using Spectral Wave (SW) module of the MIKE 21 modelling suite from DHI, Denmark. MIKE 21 SW is a third-generation time-dependent spectral wind-wave model based on unstructured meshes. It simulates the growth, propagation, decay, and transformation of wind generated waves and swells in offshore and coastal areas. The wave model was simulated for a continuous period of 27-years (January 1997-December 2023) and was used to extract wave parameters at the breakwater location.

Another numerical model was applied for modelling of the wave propagation into the terminal. MIKE 21 BW is a Boussinesq type non-linear wave model, which simulates in the time domain the properties of propagation of irregular, directional waves into the terminal considering all the important effects like shoaling, depth refraction, diffraction, bottom friction, non-linear wave-wave interaction, frequency and directional dispersion, partial and full reflection, and transmission through porous structures.

The results of the wave disturbance modeling were combined with the long-term wave conditions outside the breakwater (results from the SW model). This was done by establishing transfer functions, describing the transformation of wave heights from outside the breakwater to any given point sheltered by the breakwater (pier, plaza), and then combining the transfer functions with the long-term series of wave conditions. This provides a long-term time series of the wave heights at selected locations, for subsequent statistical analysis.

The design operational and extreme wave height were calculated using Weibull and Gumbel statistical distributions, both methods involve fitting the distribution parameters to historical wave height modeled data.

See Appendix D for wave modeling and extreme value analysis details.

4.10.1 Operational Wave Conditions

The operational condition represents the wave conditions that is expected to occur with a return period of one year. This condition is used to define the typical upper limit for safe and efficient operations, such as navigation, loading, and construction activities. *close *to Berth 2*

Table 4-5 shows the operation conditions at the breakwater and pier location.

Location	Operational Wave Conditions	
	Hs (ft)	Tp (s)
Breakwater	2.0	3.0
Pier*	1.4	3.0

**close to Berth 2*

Table 4-5 – Operational Wave Conditions - significant wave height (Hs) and Peak Period (Tp)

4.10.2 Extreme Wave Conditions

The extreme wave condition corresponds to the design sea state associated with a 100-year return period.

Table 4-6 shows the operation conditions at the breakwater, pier, and plaza location.

Location	Extreme Wave Conditions	
	Hs (ft)	Tp (s)
Breakwater	2.6	3.2
Pier	1.8-2.3	3.2
Plaza	2.4	3.2

Table 4-6 – Extreme Wave Conditions - significant wave height (Hs) and Peak Period (Tp)

4.10.3 Wake Waves

Provided that the site does not have any other port or terminal nearby, wakes should not have a significant effect on the ferry terminal operability. Moreover, because of the existing regulations regarding the maximum allowable navigation speed when ferries approach the landing, it is expected that shipping traffic will generate wakes up to 1 foot.

4.10.4 Sagging and Hogging

The ferry landing float must be checked for sagging and hogging in the long and short directions using a sinusoidal wave with trough to crest dimension equal to float depth but not more than six (6) feet and wavelength equal to float length (for the long direction) or float beam (for the short direction).

4.11 Mooring Line and Breasting Loads (M)

Mooring and breasting loads are to be calculated for the design vessel at the float under wave, wind, and current loading.

Ferry float fender lugs will be designed for 76 kips between any horizontal angle from 5 to 31° with respect to a line parallel to the long direction of the ferry float and 0 to 30° from horizontal.

4.12 Creep/Shrinkage/Thermal Loads (R+S+T)

4.12.1 Creep/Rib Shortening (R)

Creep (also referred to as rib shortening) is a material-specific internal load particular to prestressed concrete. The pier deck will not be prestressed and creep can be ignored. Creep of prestressed concrete piles is negligible and will be ignored.

4.12.2 Shrinkage (S)

Beams and decks, which are constructed from concrete components, are subject to forces resulting from shrinkage of concrete during the curing process. The new concrete decks will include temperature and shrinkage reinforcement as required by ACI 318-19.

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4.12.3 Thermal Loads (T)

The effects of thermal forces, resulting from fluctuations in temperature, will be considered. The temperature range for the proposed location shall be as follows, in accordance with the AASHTO Standard Specifications for Highway Bridges, 17th.

Material	Total Design Temperature Range (AASHTO Procedure B)
Steel	30 to 120°F
Concrete	40 to 115°F

Where thermal expansion and contraction causes frictional forces, the following friction coefficients shall be used:

Steel to steel:	0.30
Concrete to concrete	0.80
Steel to UHMW	0.20
Concrete to Steel	0.40
Timber to steel	0.50

4.13 Load Combinations

4.13.1 General

The ferry landing shall be proportioned to resist the load combinations represented in this section. Each component of the structure shall be designed to withstand all load conditions that are applicable. Components shall also be checked for serviceability (i.e., creep, fatigue and crack control as described in ACI 318) and for temporary loads. The following load conditions represent various combinations of loads and forces to which the facility may be subjected. Load combinations below are derived from the 2016 CBC and UFC 4-152-01.

The following load/force combinations are listed in tables and should be assembled as an equation as follows:

U_i or S_i = f_D (D) + f_L (LU or LC) + f_B (B) + f_{Be} (Be) + f_C (C) + …
f_H(H) + f_{EQ} (EQ) + f_W (W+Wa) + f_M (M) + f_{RST} (R+S+T)

U_i = Ultimate load combination
S_i = Service load combination
f_x = Load factor listed in tables below

The following load acronyms are applicable for the load combinations below.

D, DL	Total Dead Load
LU	Uniform Live Load
LC	Concentrated Live Load
Lo	Unreduced Uniform Roof Live Load
Lr	Reduced Uniform Roof Live Load
I	Impact Load (for LC only)
B	Buoyancy Loads
Be	Berthing Loads
C	Current
H	Horz. load due to lateral earth, ground water or pressure of bulk material.
E, EQ	Combined effect of horizontal and vertical earthquake loads
E_{h1}, E_{h2}	Earthquake actions in the respective principal horizontal direction

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k	50% of Peak Ground Acceleration (PGA)
k’	0.7k = (0.7)(0.5)PGA = 35% of PGA
W	Wind Load on Structure
Wa	Wave Load on Structure
M	Mooring/Breasting Load (Wind, wave, current of float/vessel)
R	Creep / rib shortening
S	Shrinkage
T	Temperature load

4.13.2 Load Resistance Factor Design

Load combinations and load factors used for load factor design are presented in Table 4-7. The following load conditions represent various combinations of loads and forces to which the facility may be subjected. Each component of the structure shall be designed to safely withstand all load conditions that are applicable. They shall also be checked for serviceability (i.e., creep, fatigue, and crack control as described in ACI-318, and temporary loads.

Table 4-7 - LRFD Load Combination

CBC Equation	**16-1**	**16-2**	**16-2**	**16-2**	**16-4**	**16-3**	**16-5**	**16-7**
UFC Equation	**U0**	**U1**	**U2**	**U3**	**U4**	**U5**	**U6**	**U7**
D[a]	1.4	1.2	1.2	1.2	1.2	1.2	1.0+k	1.0-k
LC+I or LU	–	1.6	–	1.6	–	1.6	0.1	–
B	1.4	1.2	1.2	1.2	1.2	1.2	1.2	0.9
Be	–	–	1.6[b]	–	–	–	–	–
C	–	–	1.2	1.2	1.2	1.2		
H[c]	–	1.6	1.6	1.6	1.6	1.6	1.0	1.0
EQ[d]	–	–	–	–	–	–	1.0	1.0
W+Wa	–	–	–	–	1.0	–	–	–
M	–	–	–	–	–	1.6	–	–
R+S+T	–	–	–	1.2	–	–	–	–

a. 0.9 (0.6 ASD) for checking members for min axial load and max moment.

b. Accidental Berthing: 1.2 support structure, 1.0 fender system components.

c. Where the effect of H resists the primary variable load effect, a load factor of 0.9 (0.6 ASD) shall be included with H where H is permanent and H shall be set to zero for all other conditions.

d. Effects of simultaneous seismic excitation in orthogonal horizontal directions shall be considered using eqs: $\pm 1.0\ E_{h1} \pm 0.3\ E_{h2}$ and $\pm 0.3\ E_{h1} \pm 1.0\ E_{h2}$

4.13.3 Allowable Stress Design Load Combinations

Where working stress design is used, structures and portions thereof shall resist the most critical effects resulting from the combination of loads below (see Table 4-8). The load combinations shall also be used for designing and checking vertical foundation capacity and long-term vertical deck loading, such as dead load deflection.

Table 4-8 - ASD Load Combinations

CBC Equation	**16-8**	**16-9**	**16-10**	**16-10**	**16-12**	**16-9**	**16-12**	**16-16**

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UFC Equation	S0	S1	S2	S3	S4	S5	S6	S7
D[a]	1.0	1.0	1.0	1.0	1.0	1.0	1.0+k’	1.0-k’
LC+I or LU	–	1.0	–	1.0	–	1.0	0.1	–
B	1.0	1.0	1.0	1.0	1.0	1.0	1.0	0.6
Be	–	–	1.0	–	–	–	–	–
C	–	–	1.0	1.0	1.0	1.0		
H[d]	–	1.0	1.0	1.0	1.0	1.0	1.0	1.0
EQ[e]	–	–	–	–	–	–	0.7	0.7
W+Wa	–	–	–	–	0.6	–	–	–
M	–	–	–	–	–	1.0	–	–
R+S+T	–	–	–	1.0	–	–	–	–

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5 OPERATIONAL REQUIREMENTS

5.1 Landside

- For pavement evaluation and design refer to the Pavement Design Memorandum included in the Appendix C.

5.2 Pier

- Top of Pier @ Landside End = +16.50 feet MLLW
- Top of Pier @ 100ft from Shore to Waterside End = +17.60 feet MLLW
- Concrete floor framing shall be limited to long term deflections of L/360. Long term deflection is the deflection due to the effects of creep and shrinkage after the attachment of nonstructural elements.
- Public gate- Pair of large swing gates (19'-0" wide x 10'-6" tall) that consist of square steel HSS member and flat plate infill pickets; galvanize finish. A personnel outswing gate is set within the south gate leaf for use during emergency access when the public gate is closed. The public gate is to always remain in the open position except for during an emergency when it will close and locked.
- Two Ferry gate entrances- 8'-6" tall by 4'-0" wide pair of swing gates and fence that consist of square steel HSS members and flat plate infill pickets; galvanize finish, setback 4 feet from edge of pier deck on concrete landings that lead to the gangways.
- Guardrails- 48" tall guardrail that consists of round HSS steel railing, shaped flat plate posts and solid round steel rods for in-fill pickets; maximum 4" gap in between pickets; galvanize finish. Hand railing with stainless steel finish contains LED handrail point lighting aimed at deck. This fixture will also be used for emergency egress lighting.
- Pier deck amenities:
  - Fish cleaning station- located at the far west end of the pier and will have one ADA compliant stainless-steel table, an ADA compliant stainless-steel sink and a standard stainless-steel sink for public use.
  - 60" wide metal bench bolted to pier deck. Type and quantity TBD.
  - Metal trash receptacle bolted to pier deck. Type and quantity TBD.
  - Swivel seats for fishing at the pier end termination bulb-out (3) areas. Type and quantity TBD.

5.3 Canopies

Pier will have a canopy structure along a portion of its length to provide protection from the elements for queuing passengers. Canopy structure shall allow for a fire truck to pass by without impediment. The cantilevered and curved pier canopy is 16' wide by 303' in length that covers half the width of the pier and both ferry entry gates at each end of the canopy. It has a shallow "S" shaped profile with the high point at the center of the pier and sloping down towards the north side of the deck supported by a single row of columns on low side 12" from the edge of the pier deck.

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- The canopy consists of a 7/8" thick x 24" wide translucent cellular polycarbonate panels secured to 2" x 3.5" aluminum curved frame mullion; clear anodized finish; supported by steel frame outriggers, round HSS purlins, beams and columns attached to the pier deck. The canopy color is blue on both the exterior and interior side. The canopy steel frame will be painted white high-performance paint suitable for marine type environment.
- The canopy will have LED linear downlights attached to the roof purlins, wall sconces on the columns and electronic passenger destination signage.

5.4 Breakwater

- Top of Breakwater = +17.60 feet MLLW
- Breakwater will consist of precast concrete panels and concrete bater piles

5.5 Gangway

- Gangway will be aluminum.
- Gangway will be ADA compliant per Access Board, "Proposed Accessibility Guidelines for Passenger Vessels", 2013)
    - The minimum clear width for a ramp is 102" including the handrails on each side.
    - The gangway shall be a minimum of 120 ft, See Appendix E (PENDING)
    - Gangway shall not exceed a slope of 1:12 for tides between MLLW and MHHW.

5.6 Ferry Landing

5.6.1 Steel Float

- The steel float will be 42-ft x 135-ft and will match WETA's Universal Charging Float (Ref. WETA Downtown SF Terminal Universal Charging Float (UFC) Project Drawings dated 4/24/2024). For additional information on UFC, refer to Section 10.
- Steel float is to have a design life of 50 years. The design life assumes regular maintenance of the steel float to replace anodes. During the design life of the steel float it is anticipated that the steel float will need to be removed from service and dry docked to repair/replace the coating. The frequency of the dry docking will be similar to other WETA steel floats currently in use.
- Zinc anodes shall be installed for the protection of the underwater hull area from corrosion. The anodes shall be installed in locations always submerged, and by bolting the ends of the steel straps to brackets attached to the hull steel. Epoxy coating shall be applied to exterior float bottom, sides, ends, and float interior area as specified in the project documents.
- Minimum Freeboard at Steel Float
    - The float shall have a nominal freeboard of 34 inches at the edge. Float to be checked for overtopping and stability with the proposed freeboard. The float shall have a fresh water ballasting system to adjust the freeboard to conform to Project requirements and adjust for trim. A nontoxic corrosion preventive admixture shall

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be used with the ballast water. A means of inspection to monitor each compartment of the float for leak detection will be provided.

5.6.2 Float Fenders

- Vertical Fenders shall extend a minimum of 7 feet above the deck of the float.
- Fenders type to be Arch Fender

5.6.3 Mooring Fittings

- Mooring fittings to be located to accommodate the full range of design vessels.
- Padeyes to be added to fender brackets.
- Mooring line guards to be included on fender brackets to prevent mooring lines from going under boarding ramps.

5.6.4 Ramps

- The boarding system ramps are to be aluminum.
- Ramps to be ADA compliant.
    - Maximum slope of fixed ramps 1:12.
    - Adjustable boarding ramps are to be designed to work for the range of design vessel boarding elevations while maintaining 1:12 slope.
    - Per ADA Rule 405.6 Rise: The rise for any ramp run shall be 30 inches maximum
    - The minimum clear width for a ramp is 48” including the handrails on each side.
- The boarding system to accommodate the vessel door spacing and height as listed in Appendix B.
- Fifty foot spacing is specified between fore and aft ramps to line up with the door spacing on the WETA boats.
- Face of boarding ramps are to be set back from the fender line 12". Taper boarding ramp vertical member at waterside end to avoid interference with vessels.
- Ramps main operating controls will be hardwired type. Each ramp will have its own control station and pendant and these devices will be coded to limit access only to trained ferry system operators.
- Brow ramps to be provided for each boarding ramp (3 total). WETA to provide details for brow ramps.

5.6.5 Steel Piles

- The float will be designed to have 36” diameter steel guide piles Although, calculated design forces are expected to be lower, each guide pile collar, at a minimum, will be designed to resist a minimum force of 100 kips in any horizontal direction acting through the center of the pile, transmitted through the collar to the float.

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- Steel piles to be coated and anodes for corrosion protection
- The steel piles will be designed for the following corrosion rates.
    - Atmospheric Zone 0.002 in per year – Top of Pile to 1 ft. above HAT
    - Splash Zone 0.006 in per year – 1 ft. above HAT to MLLW
    - Immersed Zone 0.004 in per year – MLLW to mudline
    - Soil Embedded Zone 0.001 in per year – mudline to pile tip

    - Corrosion rates based on California Department of Transportation (CalTrans) Corrosion Guidelines Version 3.2, May 2021.
    - The corrosion rates above are only applicable to the exterior surface of the steel pipe pile. The interior surface of the pile (soil plug side) will not be exposed to sufficient oxygen to support significant corrosion.
    - The piles are uncoated 10ft below the mudline.
    - The corrosion rates are applied for the unprotected years of steel corrosion.
    - Unified Facilities Criteria (UFC) 4-151-10, September 2001, Mod 1 September 2012, Table 5-1 "Period of Protection for Steel to be expected from Various Coating", the period of protection for a coal tar epoxy system with 15 to 20 mil thickness is between 10 and 20 years in a marine exposed environment. The pile protective coating is assumed to be effective for 15 years.
- Welds on steel pipe piles along the travel of the steel float to be grinded smooth prior to coating.
- Fiberglass cones to be installed on top of each pile

5.6.6 Donut Fenders

- Fender to be located to protect the corners of the float and provide a pivot point for vessels entering and leaving the berth.
- The fender shall be free to rotate around the supporting pile and move up and down along the pile to accommodate the tide.
- Geometry
    - Type: Foam-Filled Donut
    - Donut Fender Diameter: 5.75 to 6.1 feet
    - Minimum Energy Absorption: 3.1 kip-ft. per foot
    - Maximum Reaction Force: 11.9 kips per foot
    - Flat Side Freeboard Height: 10'-0"
    - Overall Flat Side Height: 15'-0"
    - Overall Height: 16 feet approximately
    - Pile Size: 36-inch diameter
    - Fender Color: Black

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6 MATERIAL PROPERTIES

6.1 Concrete

6.1.1 General

Concrete shall be in accordance with ACI 211.1 and ACI 301 capable of providing strength, durability, density and minimum chloride concentration. Design and Specifications shall have the goal to minimize concrete cracking.

Minimum Strengths for Concrete at 28 days

- General Site Concrete 4,000 psi
- Pier and Breakwater Structural Concrete
  - Deck 5,000 psi
  - Steel Pipe Plug 7,000 psi
- Prestress Concrete Piles 7,000 psi

6.1.2 Concrete Cover

- Submerged, in tidal or splash zone 3 in
- Cast against and exposed to earth 3 in
- Formed and exposed to earth or weather:
  - No. 6 through No. 18 bars 2 in
  - No 5. bar, W31 or D31 wire or smaller 1-½ in
- Above grade, formed, not exposed to earth or weather:
  - Slabs, walls, joists 1-½ in
  - Beams, columns 1-½ in

6.2 Reinforcing Steel

6.2.1 Mild Steel

- Deformed mild bars ASTM A615, Gr. 60
- Welded deformed mild bars ASTM A706, Gr. 60, Low Alloy
- Headed Welded deformed mild bars ASTM A706, Gr. 80, Low Alloy
- Spiral reinforcement ASTM A82, 70 ksi

6.2.2 Prestressing Strands

Prestressing strands shall conform to ASTM A416 and have an ultimate tensile strength of 270 ksi. The strands shall not be epoxy coated. Jacking force shall be determined in accordance with manufacturer's stressing procedures and losses shall be determined in accordance with PCI.

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6.3 Structural Steel

Structural steel shall meet the following minimum requirements unless noted otherwise:

- Float Guide & Donut Fender Pipe Piles ASTM A252 Gr. 3 modified for Fy=60 ksi
- Steel Structural Shapes: ASTM A992 or A572, Gr. 50
- Plates, Bars ASTM A992 or A572, Gr. 50
- Anchor Bolts ASTM F1554, Grade 55
- Threaded Rods ASTM A572, Grade 50
- High Strength Bolts ASTM F3125 Grade A325

All exposed steel shall be hot-dipped galvanized. Galvanizing process shall conform to ASTM A123 or ASTM A153 as applicable.

6.4 Welding

Welding of mild steel shall use E70XX Low Hydrogen electrodes and comply with AWS D1.1 unless noted otherwise. Aluminum filler metals and welding procedures shall comply with AWS D1.2.

6.5 Aluminum Alloy

All aluminum used in the fabrication shall be Alloy 6061-T6 or 6063-T6.

- Standard Structural Profiles ASTM B308
- Bars, Rods, Wires, Profiles and Tubes Hot Extruded or Similar Methods ASTM B221
- Structural Pipe and Tube ASTM B429

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7 ELECTRICAL

7.1 Governing Codes

Codes and Regulations will include the following:

- National Electrical Code (NEC)
- Title 8, California Administrative Code, Basic Electrical Regulations, Subchapter 5, Low Voltage and High-Voltage Safety Order
- Standards, Institute of Electrical and Electronic Engineers (IEEE)
- Standards, Underwriters Laboratories, Inc. (UL)
- Illumination Engineering Society (IES)
- Pacific Gas and Electric Company Greenbook
- National Fire Protection Association (NFPA)

7.2 Power Distribution

The existing power distribution system employs 12kV underground lines along University Avenue, utilizing PG&E step-down transformers for hotels, restaurants, and marina offices. The available power capacity is estimated at 400kW according to the GHD Feasibility Study Report. Coordination has been established with PG&E. Additional power will be required for the landside, pier, and breakwater development.

7.2.1 Landside/ Plaza/ Parking Lot:

A 4.4 MW large load application has been submitted to PG&E for ferry electrification. The preliminary design includes a 12kV PG&E duct bank along University Avenue. A 12kV service meter cabinet will be positioned at the plaza area, with a 12kV connection to medium voltage switchgear situated at the pier terminal to support electric ferry rapid charging.

For the new low voltage distribution related to general lighting and electrical vehicle (EV) charging systems, a utility selector switch will be established alongside a PG&E step-down transformer serving a main 1600A, 208/120V, 3-phase switchboard (MSB) in the Plaza area. Two 800A sub-switchboards, SWBD-A and SWBD-B, will be fed from the MSB. SWBD-A will serve the prefabricated restroom building, pier lighting, miscellaneous low voltage convenience receptacles, and the hotel load source selector switchboard on the ferry floats. SWBD-B will be located at the parking lot to cater for EV chargers and parking lighting as well. The number of EV chargers will correspond to 10% of the total parking stalls, with dual port chargers designed for every two parking stalls.

All power distribution will be executed underground and beneath the pier structure. Switchboards, panels, and all electrical equipment will be suitable for marine grade environments. All outlets will be GFI (Ground Fault Interrupter) type in a Marine grade enclosure. All feeders and branch circuits will have separate ground wires. All conduits near water shall be PVC Coated Galvanized Rigid Steel (GRS).

7.2.2 Pier/ Breakwater/ Float:

The pier and breakwater will have LED lighting. The Ferry Terminal low voltage power distribution will operate at 120/208V through cable feeders in conduits and be distributed as follows:

- 208V, 3-phase, 3-wire for all motor loads one horsepower and larger.
- 120V single-phase for exterior LED lighting fixtures.
- 208V, single-phase or 3-phase for special equipment.
  120V, single-phase for receptacle outlets and motors smaller than 3/4-horsepower, display lighting, and small equipment.

7.3 Lighting

New lighting will be designed for the University Ave, Plaza, parking lot, pier and ferry terminal. It is anticipated that additional lighting poles will be provided for the pathway along Seawall Drive. The lighting will be based on the IES recommended lighting level as in the table.

Illumination Subject Area	Recommended IESNA, Dark-Sky Assoc. Lighting Level (fc)	Required Uniformity Avg./Min. (fc/fc)	Dark Sky Policy Glare Mitigation Requirements	Energy Conservation Requirements
Roadways and Streets	0.6 Avg.	6/1	Full cut-off fixtures, install side shields and visors	Photocells
Ferry Terminal	20 Avg.	3/1	Illumination level to match fc/lx level	Photocells, dimming and timer control
Pier, Plaza and Parking Lot	2.0 Avg.	5/1	Full cut-off fixtures, fully equipped with shields, guards to seaside	Bi-level lighting plan, Photocells, and timers

The following Lighting Concepts will be followed:

- LED lighting for Canopy and Pier/Breakwater
- LED lighting for Gangway Walkway
- LED lighting for Rails
- Marine Signal Light with Solar Panel on the float
- Emergency path of egress lighting will be on central inverter.

7.4 Electric Vehicle Charging Stations

A 800A distribution switchboard shall be provided serving the EVSE throughout the parking lot. The master planning of the EVSE will include 20 dual port chargers for a total of 40 charging ports per CalGreen requirement. The EVSE distribution equipment must be installed as close to the proposed EVSE locations as possible. It is recommended that equipment clearances are evaluated prior to installation. All EVSE electrical distribution equipment installations must be in

accordance with NFPA 70 and local building codes and requirements. Additionally, bollards must be provided to prevent mechanical damage to the EV charging stations.

7.5 Security CCTV and Card Reader System

A security CCTV camera and card reader conduit system will be installed for the ferry terminal as well as the electronic signage display and public address speakers.

The security system will include:

- Card access control at the Ferry Security Gate.
- Clipper control panel and stations; quantity and locations to be verified with the city.
- CCTV camera requirements and locations to be verified with the city.

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8 FIRE PROTECTION

8.1 Fire Protection Design Criteria

8.1.1 Design Code & Criteria

The Fire Protection Systems shall be designed to meet the project requirements and to comply with the following codes and standards:

- California Building Code, 2022 Part 2, Title 24, CCR
- California Fire Code 2022 Part 9, Title 24, CCR. (2021 UFC and 2022 California)
- American Society of Testing of Material
- American National Standards Institute
- American Water Works Association
- Americans with Disabilities Act, Accessibility Guidelines for Buildings and Facilities
- American Welding Society
- American Society of Mechanical Engineers
- Occupational Safety and Health Association
- National Fire Protection Association
    - Section 14 - Standard for Installation of Standpipe and Hose Systems (2019)
    - Section 24 – Standard for the Installation of Private Fire Service Mains and Their Appurtenances (2022)
    - Section 307 – Standard for the Construction and Fire Protection of Marine Terminals, Piers, and Wharves (2021)
- Current applicable local, and State codes and owner standards, which includes, but not limited to; City of Berkeley regulations and amendments to the California Building Standards Code.

8.2 Utilities

8.2.1 Existing Utilities

Existing Site Utilities

The following existing utilities are on land side to serve the pier:

- Existing 8-inch-high pressure water main from the municipality (EBMUD) located under Seawall Drive near the project site. Adequate fluid pressure, and volume will be delivered from the 8-inch water main.

8.3 Fire Protection Water

8.3.1 Fire Water Supply

Fire Water Supply Evaluation and Design Approach

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All areas of the pier will be provided with a hydraulically designed firewater supply system to the fire hose cabinets and connectors on the pier. No fire pump is required. The fire water system will be supplied from a dry type fire water distribution system to the fire hose stations in lockable cabinets on the pier and located at maximum spaced intervals of 300 feet and 150 feet from structure dead end in accordance with NFPA 307 part 7.1. EBMUD has analyzed the water pressure at the nearby fire hydrant (#13615) on March 17, 2025. Static pressure was determined to be 86 psi and residual pressure at 750 gpm was determined to be 71 psi. For new Class I standpipe system requiring 100 psi by NFPA 14 at most remote pier hose cabinet, fire department will be connecting to new FDC and providing necessary additional pressure to the system so supplement water pressure from the 8-inch water main for firewater for the docks. The system will be designed in accordance with NFPA-14, 24 and 307 guidelines.

8.3.2 Function

Shoreside Fire Water

A new 6" diameter fire-water main line will be provided from new 6-inch fire water backflow preventer at shore side, with outlet connected to downstream dry pipe to the pier and ferry landings. A new fire hydrant and fire department connection will be provided within 30 feet of fire truck access nearest to the pier.

Underground 6-inch firewater pipe will be PE pipes with thrust blocks at joints. See technical specifications for proposed materials and joints.

Pier

The fire water pipe will be routed underneath the pier structure. The pipe will be supported and seismically restrained with 316 stainless steel rigid pipe clamps. The pipe will be located snug to underside of pier to avoid submergence including during tides. The pipe will feed three new fire hose cabinets along the pier and two new fire hose cabinets on the breakwater structure.

316 stainless steel grooved joint pipe and fittings will be used for the new fire water pipelines. Dielectric joint fittings at connection of dissimilar metals will be provided, and flexible hose will be used at pier and gangway joints. Flex connections will be able to provide enough flexibility for different tide levels and other required movements of the gangways.

Ferry Landing

Fire water will be piped onto the ferry landing float and under the boarding platform. One new fire hose cabinets will be provided at the ferry landing. The pipes will be supported and seismically restrained with 316 stainless steel strut attached to the structure and rigid pipe clamps.

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9 PLUMBING

9.1 Plumbing Design Criteria

9.1.1 Design Code & Criteria

The Plumbing Systems shall be designed to meet the project requirements and to comply with the following codes and standards:

- California Building Code, 2022 Part 2, Title 24, CCR
- California Plumbing Code 2010 Part 5, Title 24, CCR. (2021 UPC and 2022 California)
- American Society of Testing of Material
- American National Standards Institute
- American Water Works Association
- Americans with Disabilities Act, Accessibility Guidelines for Buildings and Facilities
- American Welding Society
- American Society of Mechanical Engineers
- Occupational Safety and Health Association
- Current applicable local, and State codes and owner standards, which includes, but not limited to; City of Berkeley regulations and amendments to the California Building Standards Code.

9.2 Utilities

9.2.1 Underground Utilities

Existing Site Utilities
The following existing utilities are on land side to serve the pier and restroom building domestic water:

- Existing 8-inch-high pressure water main from the municipality (EBMUD) located under Seawall Drive near the project site. Adequate fluid temperature, pressure, and volume will be delivered to equipment from the 8-inch water main.
- Existing 8-inch sewer main from the municipality (City of Berkeley) located under Seawall Drive near the project site.

9.3 Plumbing Systems

9.3.1 New Site Utility Work

Domestic Water
New 2-inch water supply pipeline will be connected to existing 8-inch-high pressure water main, and upstream and downstream of new water meter from the municipality (EBMUD). EBMUD has analyzed the water pressure at the nearby fire hydrant (#13615) on March 17, 2025. Static

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pressure was determined to be 86 psi and residual pressure at 750 gpm was determined to be 71 psi. A new 2-inch double-check valve reduced pressure zone domestic water backflow preventer will be provided in accordance with the City of Berkeley Public Works standards and connected to new upstream 2-inch water supply line and downstream 1 1/2-inch pier water supply line. New EBMUD standard water meter will be provided in new utility meter box on the new 2-inch water supply line. EBMUD will be coordinated with for activation of water service to the pier. The backflow preventer will be located at the new planter/vegetation area by Seawall Drive.

9.3.2 Plumbing to Structures

Pier

A new 1-1/2" diameter domestic cold-water line will be provided from the new domestic water backflow preventer at shore side to the pier and ferry landing.

The domestic water pipe will be routed along the side of the pier structure. The pipe will be supported and seismically restrained with 316 stainless steel rigid pipe clamps. The pipe will be located snug to the side of the pier deck to avoid submergence including during tides.

316 stainless NPT steel pipe and fittings will be used for the new domestic water pipelines. Dielectric joint fittings at connection of dissimilar metals will be provided, and NSF 61 compliant flexible hose will be used at pier-to-gangway, and gangway-to-ferry float joints. Flex connections will be able to provide enough flexibility for different tide levels and other required movements of the gangways.

Piping systems will be designed to provide reliable and continuous operation.

No sanitary waste and vent system will be provided for the pier or ferry landing.

Restroom Building

A new underground 1 1/2" diameter domestic cold-water line will be provided from new domestic water backflow preventer adjacent to the new restroom building at the landscaped area. New EBMUD standard water meter in new meter utility box will be provided on the new 1 1/2-inch water supply line. EBMUD will be coordinated with for activation of water service to the pier.

The domestic water pipe will be routed to the new prefabricated 3-stall restroom building and piped aboveground inside to connect to building water inlet connector.

NSF 61 compliant HDPE pipe and fittings will be used for the new underground domestic water pipeline. NSF 61 compliant 316 stainless steel NPT pipe and fittings will be used for the aboveground pipe connecting to the restroom building inlet connector.

New underground 4-inch sewer will be piped from the existing pier restroom building, sloped at minimum ¼"/foot to the existing sewer main.

Ferry Landing

1-inch domestic cold water will be piped to the ferry landing float under the gangway and around the boarding platform. Three hose bibs will be positioned around the boarding platform and

connected to the 1-inch water supply. The domestic water pipes will be supported and seismically restrained with 316 stainless steel channel strut attached to the structure and rigid pipe clamps.

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10 UNIVERSAL CHARGING FLOAT

10.1 Universal Charging Float Criteria

The WETA Universal Charging Floats (UCFs) contain power conversion, storage, and handling equipment to provide rapid charging of the WETA all-electric ferry vessels. The UCF is a barge-like structure affixed in its location via collars surrounding fixed marine pilings, allowing it to float with the tide. It contains power conversion equipment necessary to take 12kV alternating current input power from the shoreside utility, convert it to 1,000V direct current power which can then be transferred to a vessel moored alongside via four Megawatt Charging System cable connections. The UCF additionally includes 1,351 kWh of battery storage to load level electrical demand from the utility and provide charging redundancy in cases of short-term power disruptions from shore.

10.2 UCF Electrical Systems Overview:

The electrical systems of the UCF consist of switch gear, transformers, converters, batteries and chargers.

The primary electrical system architecture consists of the following from the shoreside grid to the vessel chargers:

- From the shoreside power grid is a shoreside power grid / utility supplied 12kV medium voltage input power feed.
- This power feed is connected to a medium voltage disconnect switch gear to isolate the UCF from the input power when desired / required. Standoff distances from the switchgear will exceed 5’ requirements.
- From the medium voltage disconnect switch gear, the 12kV input power is provided to two, 12kV primary, 690V + 690V secondary transformers. Each transformer will be contained in its own space internal to the UCF, containing no other equipment other than fire suppression systems and cabling. The doors will be interlocked to the disconnect switch gear to ensure that access cannot be obtained to the space while equipment is energized.
- Each transformer secondary winding feeds 690V AC power to an independent DC converter with its own input power isolation breaker.
- Each DC converter outputs 1000V DC power to two Megawatt Charging System (MCS) based vessel chargers, one on each side of the UCF. They are additionally connected to a dedicated battery bank, providing charging current to the batteries, or receiving charging current from the batteries and providing it to the vessel chargers. The 1000 V DC bus is ungrounded but will include ground fault detection to reduce risk associated with stray currents and satisfy the requirements of NFPA 70.
- The batteries are Echandia, LTO-chemistry lithium-ion batteries with a total capacity of 1351 kWh. These batteries have been selected due to the superior thermal runaway protection that LTO batteries provide in addition to their proven reliability and safety in the marine environment. Echandia batteries are designed for marine installations and along with their associated Battery Management Systems they comply with marine

regulations, have IEC 62619 certification and Classification Society approval under Det Norske Veritas. The batteries are proven in marine installations and mirror what is installed in the vessels that the UCFs will charge. The battery manufacturer is new to the US market and are UL 1973 listed but they have not yet sought UL 9540 and UL 9540A certification. Echandia has conducted extensive internal failure testing including thermal runaway, over-charge, seawater immersion, penetrative and large-scale fire testing, among others.

- The batteries will be monitored by Echandia's own battery management system, providing real time monitoring of temperatures, gas detection, State-of-Charge (SOC), and State-of-Health (SOH) to prevent thermal runaway, with direct input to ventilation and firefighting systems. The system is designed to limit risk for large scale battery failure, fire and explosion.
- Battery installation spacing meets manufacturer and marine requirements.

Additionally, the following auxiliary electrical equipment is incorporated into the design to provide hotel power and redundancy:

- Each 12kV primary, 690V + 690V secondary transformer has an input tap on one of the secondary windings for a backup generator connection. WETA envisions that 690V power would be provided from a WETA owned power generation barge. This barge would be brought to the UCF and electrically connected to maintain vessel charging capabilities when there is an extended shoreside power outage and ferry use is required for emergency response. In the case of Berkeley, the power generation barge would be moored near the UCF, either shoreside of or to the bay side of the UCF.
    - The twin secondary windings of the transformers act as isolation transformers and feed power to one DC converter each.
- The default power feed for house loads on the UCF is a dedicated 208V power feed from the shoreside power grid. There are two integrated power feeds from the batteries via the DC converters, installed to provide backup power in case of an outage.
    - The DC converters tied to the backup generator connections have inverters that provide an additional output of 600V AC, in addition to the 1000V DC standard output.
    - The 600V AC power is fed to independent 600V primary, 208V secondary transformers that provide power to the hotel source selector switch.
    - The hotel source selector switch has three 208V inputs; one from each 600V primary, 208V secondary transformer and one dedicated 208V power feed from the shoreside power grid.
    - The selector switch has motor operated breakers controlled by the Charging Control System.
- The hotel source selector switch has dedicated outputs to the two 208V panels feeding all hotel loads onboard the UCF as well as to a 208V primary, 230V secondary Y transformer feeding the single 230V panel providing power to the European-specification voltage equipment in the UCF.

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- The equipment fed from the 230V panel is the battery ventilation fans, the charging tower controls and the DC converter cooling skids.

The 12kV medium voltage disconnect switchgear and the 12kV primary transformers are designed to shoreside regulations and standards. The DC converters, batteries, and chargers are designed to marine regulations and standards.

10.3 UCF Mechanical Systems Overview

The Mechanical Systems of the UCF primarily consist of HVAC units, auxiliary ventilation fans, bilge pumps, cooling pumps, cooling skids and an automated mooring system including an HPU and winches.

- The medium voltage disconnect room has a bilge pump and ventilation which includes fire dampers and a fan.
- Each transformer room has a bilge pump and ventilation which includes fire dampers and a fan.
- The DC Grid room (housing the DC converters) has two bilge pumps and ventilation which includes fire dampers, fans and HVAC units.
- The pump room has a bilge pump, ventilation including fire dampers and fans, cooling skids for the DC converters, expansion tanks, a Hydraulic Pressure Unit and accumulator for the automated mooring system and three cooling pumps.
- The battery room has two bilge pumps and ventilation which includes fire dampers, fans and HVAC units.
    - The ventilation in the battery room will continuously ventilate the space at the USCG required 6 air exchanges per hour using non-sparking, explosion-proof fans.
    - The HVAC equipment will maintain the internal climate within manufacturer-specified climate conditions.
- Box coolers are installed on the charging end of the UCF.
- Winches are installed on deck, on both sides of the UCF for the automated mooring system.

All mechanical equipment is designed to marine standards and regulations.

10.4 UCF Fire Safety Overview

The battery space will be smoke tight and protected with A-60 structural fire protection on all bulkheads and the overhead deck.

All other spaces of the UCF will be smoke tight and protected with A-0 structure fire protection (bare steel bulkheads).

All spaces of the UCF will be monitored by a common Potter Electric ARC-100 addressable Fire Alarm and Control Panel using Potter Electric photoelectric smoke detectors and heat detectors.

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CCTV will be provided in the battery room with a direct feed to WETA's Central Bay Operations and Maintenance Facility.

10.5 UCF Fire Suppression Systems Overview

Each space onboard the UCF will be protected by a Stat-X, condensed aerosol fire suppression system controlled by the common Potter Electric ARC-100 FACP. The Stat-X system is UL-listed.

The battery space has two additional, fully redundant fire suppression systems.

In the case of a battery fire:

1. The initial response to any abnormal conditions noted by the Battery Management System (BMS) is to isolate the affected cells (at the string level) and maintain ventilation to cool and regain control of the cells.
    a. The batteries are continuously monitored by the BMS with respect to temperatures, gas detection, SOC, SOH, etc. to monitor health at any given time.
    b. If isolation and ventilation is successful, the system carries on in normal operation and can reintegrate the isolated string(s) once all monitored data returns to normal operating ranges.
2. The primary fire suppression system for the batteries is a FiFi4Marine Compressed Air Foaming System (CAFS) direct foam injection system, integrated into the Echandia batteries. This system has Classification Society approval from DNV, for use in extinguishing lithium-ion battery fires and has been tested in large-scale fire scenarios with approval from the Danish Maritime Authority and the Netherlands Competition Authority. FiFi4Marine is new to the US market and has not yet been tested to UL standards.
    a. The FiFi4Marine system includes a Li-Ion Tamer battery off-gas detection system to continuously monitor the batteries independent of the BMS.
    b. If the BMS is unable to rein-in the affected cells and the situation continues to deteriorate, the FiFi4Marine system will be activated automatically by its own gas and fire detection systems. This will occur the moment the FiFi4Marine system detects off-gassing and elevated cell temperature.
    c. Using compressed air and FiFi4Marines Bio4C premix, foam is produced and applied via solenoid valves and piping directly to the affected areas of the batteries creating a localized heat sink to cool the area of the battery.
3. The secondary fire suppression system for the batteries is the Stat-X condensed aerosol fire suppression system installed in the battery room. The Stat-X solution for the battery room is rated for suppression of lithium-ion battery fires.
4. The tertiary fire suppression system for the batteries is a freshwater deluge system from a shore connection. This system will utilize NFPA 750-listed nozzles and can be used to fully flood the battery room if required.

In the case of a fire in the battery room external to batteries:

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1. The primary fire suppression and detection system for a battery room fire external to the batteries is the Stat-X system.
2. The secondary fire suppression system for a fire in the battery room external to the batteries is the freshwater deluge system.

10.6 Space Access Overview

All spaces internal to the UCF will be accessible by trained personnel only and will have multiple access points for emergency egress.

All spaces will be arranged to allow safe movement throughout the space while conducting regular inspections. Where required for code compliance, adequate standoff distances have been accounted for.
Each space will have soft-patch access hatches for major equipment removal and replacement.

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APPENDIX A: GEOTECHNICAL REPORT & MEMORANDUMS

The geotechnical parameters shall be based on recommendations provided in the geotechnical reports and memorandums prepared by ENGEO including the following:

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BERKELEY WATER TRANSPORTATION PIER FERRY
BERKELEY, CALIFORNIA

GEOTECHNICAL REPORT

SUBMITTED TO
Mr. James Connolly
COWI
555 12th Street, Suite 1700
Oakland, CA 94601

PREPARED BY
ENGEO Incorporated

December 16, 2024
Latest Revision December 12, 2025

PROJECT NO.
25022.000.001

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ENGEO
Expect Excellence

Project No.
25022.000.001

December 16, 2024
Latest Revision December 12, 2025

Mr. James Connolly
COWI
555 12th Street, Suite 1700
Oakland, CA 94601

Subject: Berkeley Water Transportation Pier Ferry
Berkeley Marina
Berkeley, California

GEOTECHNICAL REPORT

Dear Mr. Connolly:

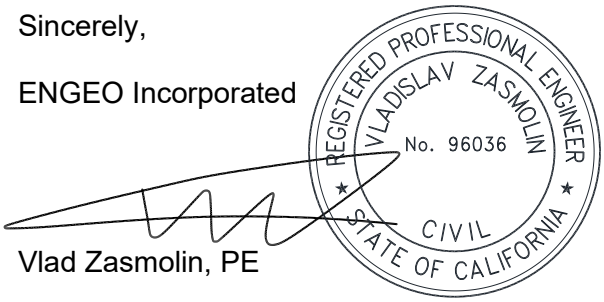
We prepared this geotechnical report for COWI as outlined in our agreement dated August 5, 2024. Our services comprised preparing a geotechnical report based on our review of previous geotechnical data, published geologic maps, historical aerial photographs, and conceptual plans for proposed development as well as a project-specific subsurface exploration and laboratory testing program.

From a geotechnical engineering viewpoint, the site is suitable for the proposed project, provided the geotechnical recommendations in this report are incorporated into the design and implemented during construction. The main geotechnical considerations for the planned project include the presence of non-engineered fill, soft compressible clay, liquefiable soil, seismically induced lateral spreading and slope instability, and shallow groundwater. We present our conclusions and recommendations related to the planned project in this report.

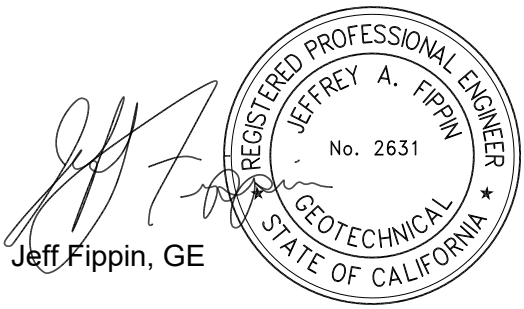
Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical testing and observation services during construction. If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated



Vlad Zasmolin, PE



Jeff Fippin, GE

vz/jaf/ca

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this geotechnical report for the Berkeley Water Transportation Pier Ferry project at the Berkeley Marina in Berkeley, California. Our scope of services is outlined in our agreement dated August 5, 2024, and includes the following.

- Review of published maps, previous reports, and historical information
- Review of aerial images
- Exploration of subsurface conditions
- Laboratory testing of soil samples
- Analysis of data
- Development of conclusions
- Preparation of this report

For our use, we received numerous as-built construction drawings and engineering reports. For the purposes of our report, we used the following select documents.

Construction Drawings

- The Golden Gate Ferry Company. 1926. General Pile Plan, Ferry Terminal, Berkeley, California. August 24, 1926. Sheet 1.
- City of Berkeley Department of Public Works. 1632. Yacht Harbor. Berkeley, California. August 4, 1936. Plan No. 1632.
- City of Berkeley Department of Public Works. 1952. University Avenue Extension to Berkeley Harbor, Berkeley, California. September 12, 1952. Plan 302-A-88.
- City of Berkeley Department of Public Works. 1965. Grading Plan for South Sailing Basin. Berkeley, California. March 1, 1965. Plan 4084.

Consultant Reports

- Earl and Wright Consulting Engineers. 1960. Berkeley Pier, Berkeley, California. June 6, 1960.
- A3GEO. 2013. South Cove Public Dock and Parking Lot Renovation Project Berkeley Marina, Berkeley, California. February 12, 2013. Project No. 1118-1A.
- IDA Structural Engineers. 2015. Berkeley Pier Load Evaluation, Berkeley, California. June 25, 2015. Project No. 13113.07.
- GHD. 2023. Feasibility Study Ferry Facility at Berkeley Municipal Pier, Berkeley, California. June 2022; Revised June 30, 2023. Project No. 11125268.

We prepared this report for the exclusive use of COWI and their consultants for the design of this project. If any changes are made in the character, design, or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to evaluate whether modifications are recommended. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 PROJECT LOCATION

The existing Berkeley Municipal Pier (Pier) is located at the Berkeley Marina (Marina) on the western edge of University Avenue as shown in Figure 1. The Marina is an irregularly shaped peninsula that consists of manmade fill situated on the eastern end of the San Francisco Bay. The proposed project includes reconstruction of 1,080 feet of the Pier extending west of the intersection of Seawall Drive and University Avenue. The associated Marina landside improvements will be constructed in the area between University Avenue, Seawall Drive, the Bay Trail, and former HS Lordships Parking Lot.

1.3 PROPOSED PROJECT

The Response to Qualifications (RFQ) document prepared by the City of Berkeley, dated December 15, 2023, outlines that the project will include demolishing and replacing 1,080 linear feet (lf) of the existing Municipal Berkeley Pier with a new “sword” shaped Pier and breakwater as shown in Exhibit 1.3-1.

EXHIBIT 1.3-1: Renderings of Proposed New Pier and Associated Marina Developments

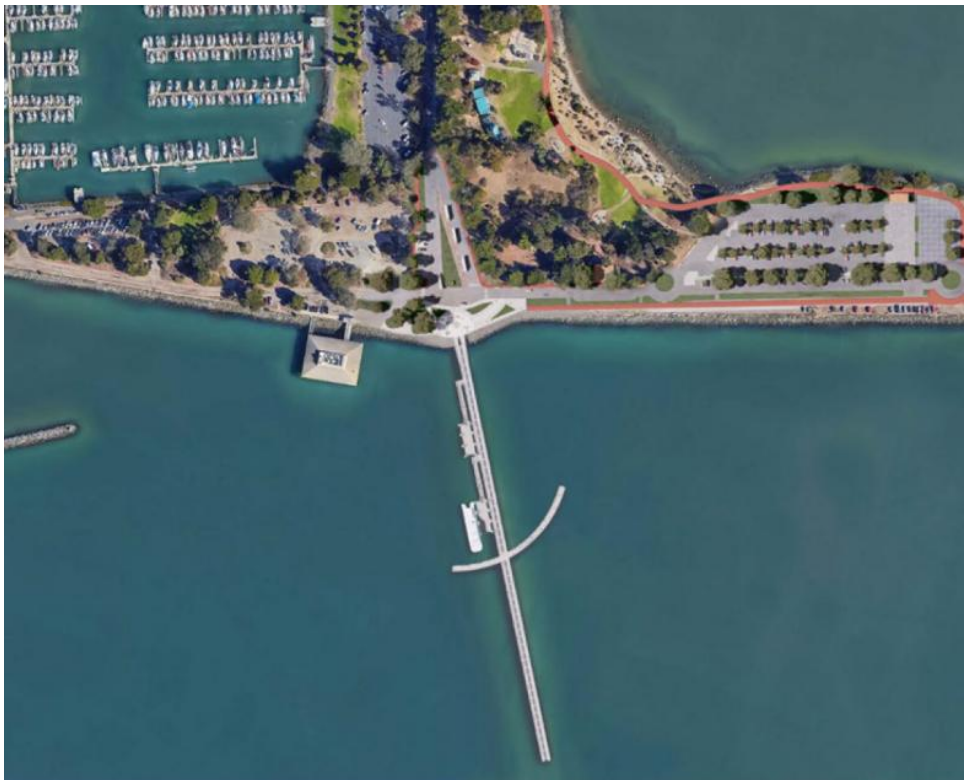


Exhibit 1.3-1 shows the planned offshore elements, which will include:

- A 580 lf ferry boarding pier segment extending westward from Seawall Drive. Two connecting gangways and berthing floats will be added on the northern end of the Pier to facilitate ferry access. The Pier and berthing floats will be supported by 24-inch plumb octagonal precast, prestressed concrete piles and 36-inch steel pipe piles, respectively.

- A 400 lf breakwater perpendicular to the pier providing wave and wind protection for the ferry boarding region. The breakwater will be constructed using 4-foot by 2-inch or 4-foot by 14-inch precast, prestressed concrete sheet piles laterally supported by 24-inch battered octagonal prestressed precast piles.
- A 500 lf recreational pier segment extending westward past the breakwater for recreational public access. This portion of the pier will be supported by 24-inch plumb octagonal precast, prestressed concrete piles.

The newly built pier will classify as an essential faciality and require ongoing access during and following seismic events and other disasters.

In addition, the project includes the redevelopment of landside lots adjacent to the Pier within the South Cove Sailing Basin. The associated improvements will include:

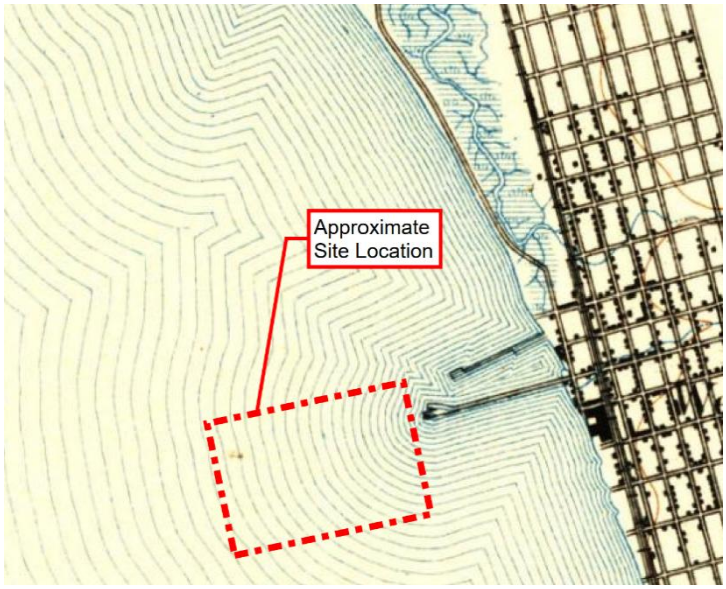
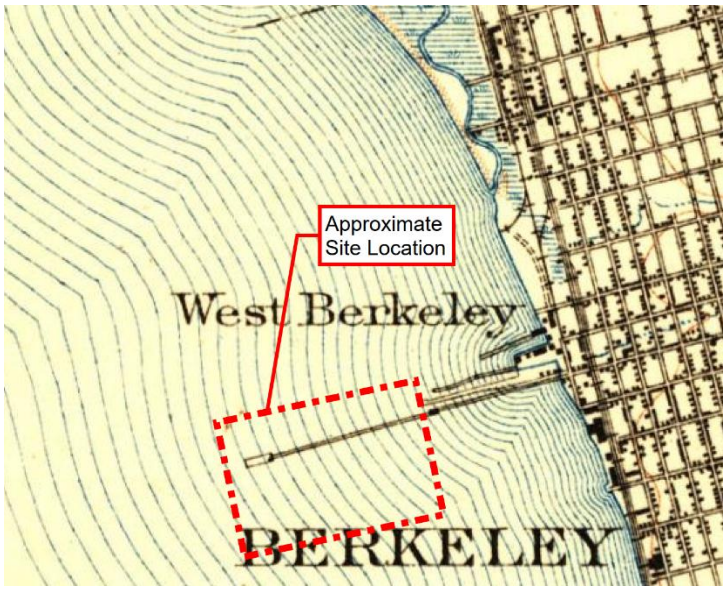
- Street, parking lot, and sidewalk renovations on Seawall Drive, University Avenue, HS Lordships Parking Lot, and Bay Trail.
- Grading of the pier entrance plaza area.
- Minor structures (restroom, light poles, canopies etc.).
- Renovation of sidewalks and pedestrian pathways.
- New underground utilities.

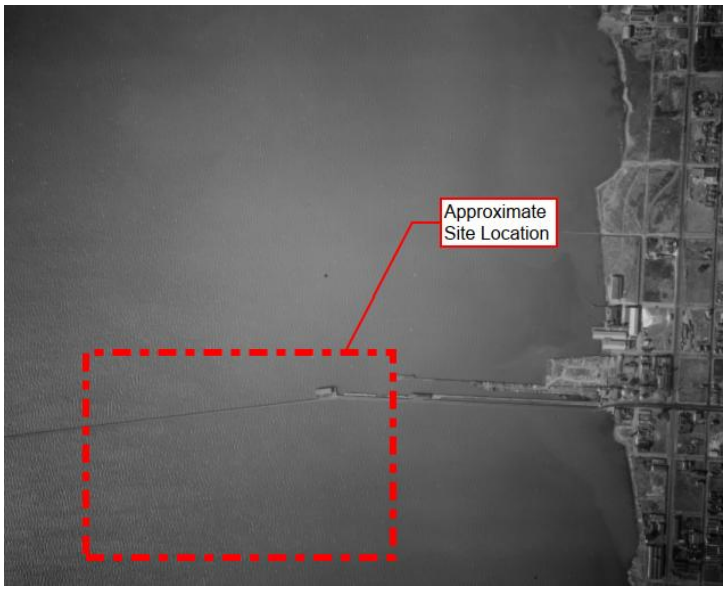
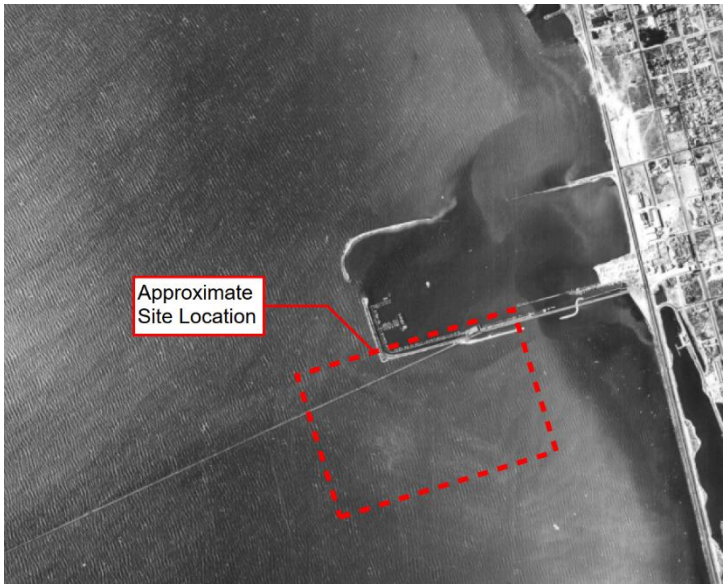
The elevation datum for this project is the City of Berkeley Mean Lower Low Water tidal datum (City of Berkeley, MLLW). The figures and data presented in this report reference elevations based on this datum. The City of Berkeley MLLW datum is equal to NAVD88 plus 0.07 foot. At this time, we understand the proposed finished grade at the pier entrance plaza will be Elevation 16½ feet. To achieve the proposed grade at this location, the site will need to be raised approximately 5½ feet. Grading at the rest of the site will be on the order of 1 to 2 feet.

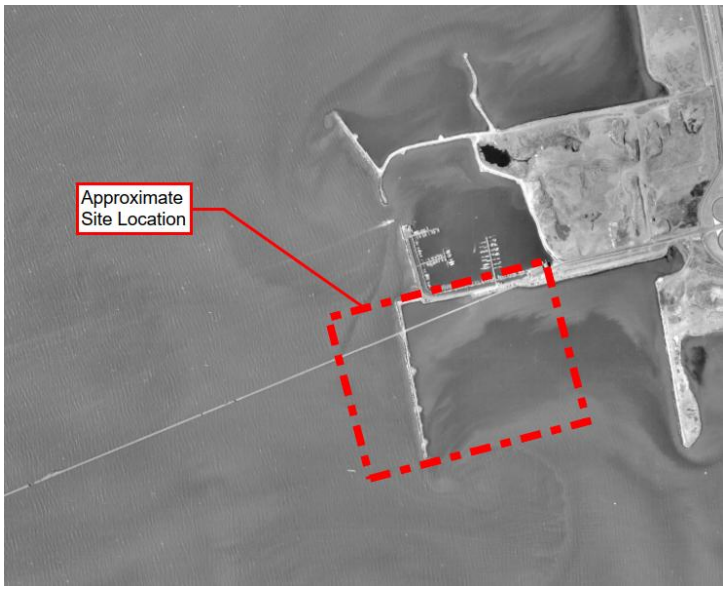
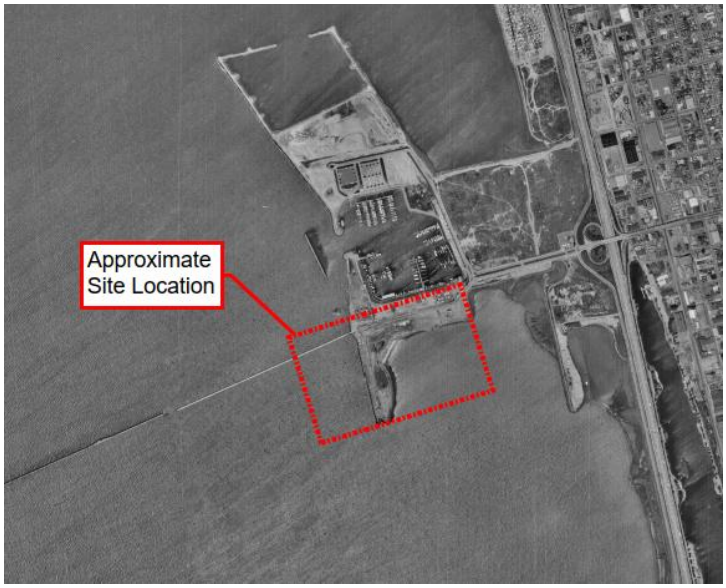
1.4 BERKELEY MARINA AND PIER HISTORY

The area encompassing the Berkeley Marina was historically part of the San Francisco Bay beyond the natural shoreline of Berkeley prior to development. The predevelopment shoreline in 1895 was located adjacent to the current day Interstate 80 (I-80). Table 1.4-1 presents a brief overview and timeline of development-related activities at the Berkeley Marina Site.

TABLE 1.4-1: Site History

 Source: U.S. Geologic Survey, 1895	1850s to early 1900s – The site was approximately 4,500 feet offshore the Berkeley shoreline and completely submerged. Adjacent to the site there were two wharves owned by The Standard Soap Company and Jacobs and Heywood Lumber yard extending 2,700 feet west of University Avenue and 2,000 feet west of Delaware Street, respectively. We presume both were used for bulk cargo shipping.
 Source: U.S. Geologic Survey, 1915	Early 1900s to 1926 – In 1915, it appears the Standard Soap Company pier was replaced with a municipal pier built by the City of Berkeley to serve as a commuter ferry and freight pier. The pier extended approximately 5,000 feet west of University Avenue.

 Source: USCB Historical Aerial Photograph, 1931	1926 to 1936 – In 1926, the Golden Gate Ferry Company constructed the existing Berkeley Pier by extending the municipal wharf. The Pier functioned as an automobile ferry service to San Francisco. It spanned 18,500 feet westward from the foot of University Avenue to the Northern end of Treasure Island; extending to water deep enough to accommodate a deeper ferry draft than the prior pier.
 Source: USCB Historical Aerial Photograph, 1939	1936 to 1957 – Ten years following the construction of the Berkeley Pier, the Bay Bridge was built, rendering ferry services from Berkeley to San Francisco obsolete. As such, in 1937, the ferry was shut down and the Pier was acquired by the City of Berkeley to operate as a recreational Pier. Furthermore, in 1936, the Berkeley Yacht Harbor was developed, including the construction of several small docking piers and three rock dikes acting as breakwaters.

 Source: USCB Historical Aerial Photograph, 1957	1957 to 1966 – In 1957, land reclamation began with the construction of perimeter dikes bordering current day McLaughlin Eastshore Park and Cesar Chavez Park. The areas became a landfill with soil and municipal waste being placed within the perimeter dikes. In addition, a rock breakwater was placed along the western perimeter of the current South Cove Sailing Basin shoreline. Around this time, the Pier appeared to show signs of significant deterioration with observable gaps in the decking (Earl and Wright, 1960).
 Source: Historical Aerial Photograph Viewer, 1968	1960s to 1970s – In 1966, the South Cove Sailing Basin area was filled using a combination of hydraulically placed dredge spoils from the surrounding San Francsico Bay and non-engineered general fill (City of Berkeley, 1967). The area was predominantly used for recreational purposes.



Source: Google Earth, 2002

1970s to Present – Following land reclamation, development in the area included commercial and office structures, restaurants, parks, and small piers. Additionally, in approximately 1991, the landfill located at McLaughlin Eastshore Park and Cezar Chavez Park was closed and capped with a clay liner. The Pier remained operational until 2015 when it was closed indefinitely following a structural assessment by IDA (2015).

1.5 PREVIOUS REPORTS

We reviewed past geotechnical reports that we and other geotechnical consultants prepared for projects in the Berkeley Marina and surrounding proximity. These reports include:

- ENGEO. 2018. Geotechnical Recommendations for Courtyard Redesign of DoubleTree Hotel. July 20, 2018. Project No. 15201.000.000.
- ENGEO. 2018. Brickyard Phase I Improvements. Geotechnical Exploration. Berkeley, California. June 11, 2018. Project No. 15007.000.000.
- ENGEO. 2016. Cezar Chavez Park. Berkeley, California. October 6, 2016. Project No. 12860.000.000.
- A3GEO. 2013. South Cove Public Dock and Parking Lot Renovation Project. Berkeley, California. February 12, 2013. Project No. 1118-1A.
- ENGEO. 2011. Lawrence Berkeley National Laboratory Second Campus; Golden Gate Field Site. Albany, California. July 28, 2011. Project No. 9289.000.000.

2.0 FINDINGS

2.1 GEOLOGY AND SEISMICITY

2.1.1 Regional Geologic Setting

The Site is located on the eastern margin of the San Francisco Bay, in the Coast Ranges geomorphic province of California. The Coast Ranges comprise a system of northwest-trending, fault-bounded mountain ranges, and intervening valleys that trend approximately parallel to the right-lateral transform boundary between the North American and Pacific Plates. The present geology of the Coast Ranges is the result of deformation and deposition along this tectonic boundary. Plate boundary fault movements are largely concentrated along the well-known fault

zones, which in the San Francisco Bay Area include the San Andreas, Hayward, and Calaveras Faults, as well as other lesser-order faults.

2.1.2 Regional Seismic Setting

The site is located in a seismically active area that contains numerous faults. Small earthquakes occur every year in the San Francisco Bay Area and larger earthquakes have been recorded and can be expected to occur in the future. Active faults are cataloged and mapped by the United Sates Geologic Survey (USGS) in the Quaternary Fault and Fold Database of the United States. An active fault is defined by the California Geologic Survey as one that experienced surface displacement within Holocene time (about the last, 11,700 years) (CGS, 2018). Figure 3 shows the approximate locations of known active faults, along with other Quaternary faults based on the USGS Quaternary Fault and Fold Database, as well as significant historical earthquakes recorded within the San Francisco Bay Area region.

To identify nearby faults that are capable of generating strong seismic ground shaking at the Site, we utilized the USGS Earthquake Hazard Toolbox and the 2018 National Seismic Hazard Model (NSHM) to perform a disaggregation of the seismic hazard at the peak ground acceleration (PGA) and at spectral periods up to 5 seconds for a return period of 2,475 years. The resulting faults are listed in Table 2.1.2-1.

TABLE 2.1.2-1: Faults Considered Capable of Producing Strong Ground Shaking at the Site*

SOURCE NAME	RUPTURE DISTANCE, R_{RUP} (mi)	MOMENT MAGNITUDE, M_W
Hayward (North) (2)	3.9	7.4
San Andreas (Peninsula) (13)	15.5	8.0
Hayward (North) (3)	4	7.1
Hayward (North) (1)	4.5	6.8
San Gregorio (North) (3)	17.6	7.8

*Based on USGS Earthquake Hazard Toolbox: NSHM Conterminous U.S. 2018

These results represent known fault sources contributing at least 1 percent to the seismic hazard at the site based on the disaggregation discussed above. The rupture distances (R_{RUP}) and mean moment magnitudes (M_W) listed are based on values assigned according to the 2018 NSHM, and the numbers in parentheses after the fault names correspond to fault subsections assigned by the NSHM. Note that the above fault table is not an exhaustive list and other faults in the region may generate seismic shaking at the project site.

2.1.3 Local Geologic Conditions Geology

Quaternary geology along the Bay margins is typically defined by periods of deposition and erosion correlated to worldwide glaciations, and their associated changes in sea level. Glacial episodes are associated with a fall in sea level, due to more of the Earth's water being stored in glaciers and ice caps. During the last glaciation between approximately 70,000 and 12,000 years ago, the sea level was as much as 300 feet below modern levels (Atwater, et al. 1977). Depositional environments during low stands of sea level are governed by eolian deposits along the shoreline, erosional processes, and high-energy stream channel deposits.

By contrast, interglacial periods are defined by higher sea levels and reduced ice cap cover. Soil deposited during these episodes typically consists of estuarine and low-energy shallow marine sediments allowed to settle to the sea floor.

During the Late Quaternary period, the San Francisco Bay experienced several episodes of sea level rise and fall related to these glaciations. According to Atwater, et al. (1977), samples recovered from foundation explorations at the Bay Bridge encountered marine estuary deposits that formed during the Sangamon interglacial period (100,000 years ago) and post-Wisconsin interglacial period (less than 10,000 years ago), during stands of higher sea level. These deposits are typical of Bay margin stratigraphy and are commonly referred to as Old Bay Clay (OBC) and Young Bay Mud (YBM), respectively.

The cyclical fluctuations in sea level have resulted in interbedded alluvial and marine soil deposits, each with distinctive colors. Alluvial deposits exhibit a yellowish-brown color because of their exposure to oxygen during terrestrial deposition, whereas marine deposits appear gray and olive because they did not oxidize while inundated. These alternating layers continue past the uppermost Old Bay Clay to form the Upper Alameda Formation.

As shown in Figure 4, mapping by Graymer (2000) indicates the site is capped by artificial fill placed east of the Berkeley shoreline. The fill is a combination of imported soil and dredge spoils from the San Francsico Bay that was placed between perimeter rock dikes. Our experience indicates that the quality and consistency of fill placed in this era can vary considerably. Some fill is quite dense, while other, especially older fill, was not compacted during placement is typically relatively loose.

Based on cross sections along the San Francisco Bay presented by Roger and Figures (1992) and our interpretation of the geologic units, the general stratigraphy of the site is summarized in Table 2.1.3-1 and discussed in detail in Section 2.5.

TABLE 2.1.3-1: Stratigraphic Units at the Berkeley Pier

UNIT NAME	SYMBOL	GEOLOGIC TIME PERIOD	DESCRIPTION
Artificial Fill	Qaf	Recent	Heterogeneous sandy and clayey fill mixed with debris
Young Bay Mud	Qybm	Holocene (Interglacial)	Soft to medium stiff marine silt and clay interbedded with marine sand
San Antonio Formation	Qal 1	Pleistocene (Wisconsin Glacial)	Stiff clay interbedded with layers of medium dense to dense sand.
Old Bay Clay	OBC	Pleistocene (Sangamon Interglacial)	Overconsolidated, stiff to very stiff marine clay with few sandy interbeds
Alameda Formation	Qal 2-4	Late Pleistocene (Glacial)	Very stiff to hard alluvial clay varying in sand content.
	Qobc 2-4	Late Pleistocene (Interglacial)	Overconsolidated, stiff to very stiff marine clay.

2.2 FIELD EXPLORATION

Prior to conducting our field exploration, we notified Underground Service Alert and retained the services of a private utility locator to clear the exploratory locations of existing utilities. Additionally, we obtained drilling and engineering permits from the City of Berkeley. The approximate exploration locations can be found in Figure 2.

2.2.1 Pavement and Subgrade Exploration

Prior to drilling, we retained the services of Penhall and West Coast Exploration to core through the existing pavement at select boring and cone penetration test (CPT) locations along University Avenue, Seawall Drive, Bay Trail, and the former HS Lordships parking lot. Coring was complete using an 8-inch-diameter, diamond-bit core drill. We identified pavement thickness, aggregate base thickness, and depth to any pavement fabric encountered. We delivered the pavement core samples to NCE's Point Richmond storage unit. Photographs of the asphalt cores are shown in Figure 5A through 5C.

We drilled Borings 1-C1 through 1-C8 after coring using a solid flight auger to a depth of 3½ to 6 feet below the ground surface (bgs) and collected bulk samples for lab testing. A summary of our in-situ measurements and laboratory testing is provided in Table 2.2.1-1. The logs of the borings are included in Appendix C.

TABLE 2.2.1-1- Summary from Pavement Exploration

LOCATION	THICKNESS OF ASPHALT CONCRETE (AC) (inches)	THICKNESS OF AGGREGATE BASE (AB) (inches)	DEPTH TO PAVEMENT FABRIC* (inches)	SUBGRADE LABORATORY TESTING		
				MOISTURE CONTENT (%)	PLASTICITY INDEX	R-VALUE
1-C1	12	12	4¼	12.2	25	10
1-C2	9¼	12	2½	-	-	-
1-C3	4¼	12	2	14.3	20	9
1-C4	2	8	None	10.9	17	-
1-C5	3	6½	None	-	-	-
1-C6	2½	9	None	-	18	17
1-C7/1-B4	10½	6	None	-	-	-
1-C8	4½	3	None	14.3	17	12
1-C9	4¼	-	2½	-	-	-
1-CPT1/1-B2	4¼	5	None	-	-	-
1-CPT2	4¼	6	2¼	17.2	22	-
1-CPT3	5	7	2¾	-	-	-
1-CPT4	2	9	None	-	-	-
1-B2	4¼	5	None	-	-	-
1-B3	5	7	2¼	-	24	9
1-B5	3½	7	None	-	-	-

* below ground surface

2.2.2 Landside Cone Penetration Tests

Between August 27, 2024, and August 29, 2024, we retained the services of ConeTec to perform CPTs to a maximum depth of 134 feet bgs (Elevation -125 feet), in general accordance with ASTM D5778. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). We encountered shallow refusal at 5 to 8 feet bgs in 1-CPT1, 1-CPT2, and 1-CPT3 due to the presence of debris and coarse gravel. We used a solid flight auger to drill out the impeding material and subsequently resumed the CPT. We performed shear-wave velocity measurements

in one of the CPTs in accordance with ASTM D7400. CPT logs are presented in Appendix A. Photographs of the operation can be seen in Exhibits 2.2.2-1a and 2.2.2-1b.

EXHIBIT 2.2.2-1a: CPT Drilling Operations at 1-CPT2



EXHIBIT 2.2.2-1b: CPT Drilling Operations at 1-CPT4



2.2.3 Landside Borings

Between August 28, 2024, and September 5, 2024, we retained the services of Pitcher Services LLC to advance borings up to a depth of 106½ feet bgs (Elevation -89 feet) using mud-rotary wash methods with a 3⅞-inch tricone drill bit. A representative of our firm observed the drilling and logged the subsurface conditions. At locations 1-B2 and 1-B5 we encountered difficult drilling conditions due to the presence of very dense coarse-grained gravel and cobbles that we presume are associated with a perimeter rock dike placed between 1952 to 1957. Photographs of the operations can be seen in Exhibits 2.2.3-1a and 2.2.3-1b.

We collected soil samples at frequent depth intervals using either a 3-inch outside-diameter (O.D.) California-type split-spoon sampler fitted with 6-inch-long brass liners, or a 2-inch O.D. standard penetration test (SPT) split-spoon sampler. We advanced the samplers with a 140-pound hammer with a 30-inch drop, employing an automatic-trip hammer system. We recorded the penetration of the sampler as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring log shows the number of blows required for the last foot of penetration, or the number of blows per depth of penetration for samples that met driving refusal. We did not convert the blow counts depicted on the boring log using any correction factors. In areas of soft fine-grained soil, we advanced thin-walled Shelby tubes, at select locations. The logs of the borings are included in Appendix C.

EXHIBIT 2.2.3-1a: Mud Rotary Drilling Operations at Location 1-B1



EXHIBIT 2.2.3-1b: Mud Rotary Drilling Operations at Location 1-B2



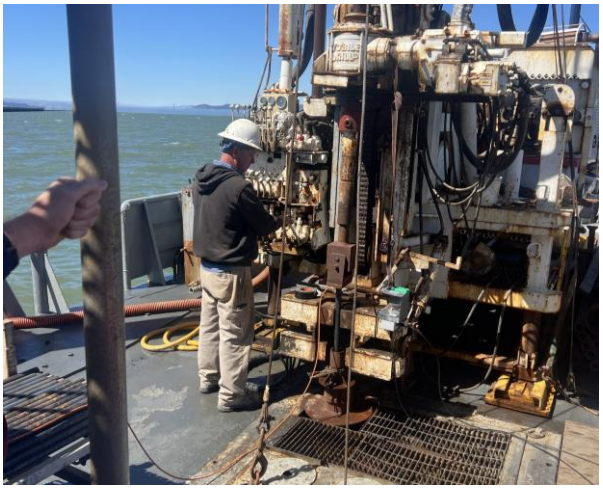
2.2.4 Offshore Exploration

Between September 4, 2024, and September 18, 2024, we retained the services of Gregg Drilling LLC to advance six mud-rotary borings and three CPTs up to a depth of 151½ feet below the mudline (Elevation -157½ feet) from the Quin Delta Research Vessel (Quin Delta). The drilling and sampling methodologies resemble those presented in Section 2.2.2 and 2.2.3. For CPTs, the Quin Delta has a decreased pushing capacity in contrast to a landside rig due to (1) longer unsupported rod length and (2) frictional losses of pushing capacity to the sides of casing. As such, we encountered CPT refusal in medium dense and dense sand, prompting us to switch to rotary wash drilling and sampling in these units. Upon encountering clay, we resumed the CPTs. The locations and elevations of the explorations are approximate and we estimated them by use of differential GPS onboard the Quin Delta and show them in Figure 2. The logs of the borings and CPTs are included in Appendices A and C, respectively.

EXHIBIT 2.2.4-1a: Quin Delta Research Vessel Stationed adjacent to the Berkeley Municipal Pier



EXHIBIT 2.2.4-1b: Drilling Operations on the Quin Delta Research Vessel



2.2.5 Sonic Borings

We mobilized a sonic drill rig on February 7, 2025, to evaluate the extents (vertical and lateral) of the perimeter rock dike. We drilled up to about 30 feet bgs (Elevation -20 feet) using a 6½-inch-diameter sonic core barrel. A representative of our firm observed the drilling and logged the subsurface conditions. Sonic drilling employs a high-frequency oscillator that generates resonant vibrations along the drill string while it cuts through the subgrade soil. The substantial energy transferred to the drill bit enables penetration through very dense soil. We retrieved bulk samples every 5 vertical feet during drilling, allowing for continuous characterization of the subsurface conditions. Exhibits 2.2.5-1 and 2.2.5-2 show photographs of our drilling operations. The logs of the borings are included in Appendix C.

EXHIBIT 2.2.5-1a: Sonic Drilling Operations at Location 2-B4



EXHIBIT 2.2.5-1b: Sonic Drilling Operations at Location 2-B7



2.3 LABORATORY TESTING

We performed laboratory testing on samples recovered during borehole drilling in accordance with Table 2.3-1 to determine various soil characteristics.

TABLE 2.3-1: Laboratory Testing

TEST	DESIGNATION	NUMBER OF TESTS PERFORMED
Natural Unit Weight and Moisture Content	ASTM D7263	18
Natural Moisture Content	ASTM D2216	21
Atterberg Limits	ASTM D4318	19
Grain Size Distribution	ASTM D6913	5
Passing No.200 Sieve	ASTM D1140	14
Consolidation – Constant Rate of Strain	ASTM D4186	3
Unconsolidated Undrained Triaxial Compression Test	ASTM D2850	48
Unconfined Undrained Triaxial Compression Test	ASTM D2166	3
Lab miniature Vane Shear	ASTM D4648	2
Corrosivity Testing (Redox, pH, Resistivity, Chloride, Sulfate, Sulfide)	ASTM D1498, D4972, G57, D4658M, D4327	2
R -value Testing	ASTM D2844	5

2.4 SURFACE CONDITIONS

The landside portion of the site is predominantly occupied by paved roads, parking lots, sidewalks, walking trails, and landscaped areas. Small one-story structures associated with Shorebird Park, Cal Sailing Club, Skates on The Bay Restaurant, and former HS Lordships Restaurant are located throughout the site boundary. Paved roads appear to be in poor condition along Seawall Drive and the former HS Lordships restaurant parking lot relative to those on University Avenue. In addition, the HS Lordships parking is bisected by a fence that appears to have previously bounded a construction staging area located on the southern portion of the parking lot until recently.

Based on our review of the landside topographic survey provided by NCE, the ground surface within the majority of the site boundary range between Elevation 12 feet and Elevation 19 feet with a generally gradual slope. However, there are outstanding topographic features on the site. The Pier entrance plaza is at a topographic low at Elevation 11½ feet that slopes up to Elevation 15 feet at approximately 7.5:1 (horizontal:vertical) inland. In addition, shorebird park contains manmade hills sloped at a 4:1 wedged between Seawall Drive and University Avenue with topographic highs up to Elevation 30 feet.

The existing Pier begins immediately to the west of the intersection of University Avenue and Seawall Drive. The Pier extends 3,000 feet to the west of the shoreline before the wooden decking terminates and only pile remnants remain. Based on our review of the Bathymetric and LiDAR survey completed by eTrac, the shoreline slopes at approximately 2:1 to 3:1 to the mudline at Elevation -6 feet. The mudline elevation remains relatively constant along the pier throughout the site extents.

2.5 SUBSURFACE CONDITIONS

Our exploration locations are presented in the Site Plan, Figure 2, and the specific stratigraphy for each CPT and boring is depicted on the exploration logs in Appendix A and Appendix C. The logs contain the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System. The logs graphically depict the subsurface conditions encountered at the time of the exploration. We present idealized subsurface cross-sections in Figure 6. The cross-sections are based on interpolation between surface explorations and are provided for reference only. Local variation should be expected.

2.5.1 Artificial Fill (Qaf)

We encountered up to 23 feet of artificial fill. Our research indicates this fill was placed on the South Cove Sailing Basin land during reclamation between 1966 to 1968. City of Berkeley (1965) indicates that the fill was placed directly on native marine sediment underlying the site. There is no documentation of engineering compaction and due the type of fill placement and era, it is unlikely the soil was compacted. In general, the fill we encountered consists of 4 to 10 feet of soft to stiff lean clay over loose to dense clean and clayey sand and gravel with varying amounts of debris.

2.5.1.1 Perimeter Rock Dike

During the geotechnical investigation that we performed between August 27, 2024, and September 18, 2024, we encountered dense to very dense sand and gravel material on the western shoreline of the Marina in exploration locations 1-B2, 1-CPT1, and 1-B5 at approximately 6 feet bgs. As shown in historical aerial photographs presented in Section 1.4, this material

appears to be associated with a rock breakwater placed on the western perimeter of the South Cove Sailing Basin between 1952 and 1957. Based on our historical aerial photograph review, review of construction plans, and exploration, we opine that this breakwater was used as a rock dike behind which fill was placed when land reclamation occurred. At Boring 1-B1, we encountered YBM immediately below the rock. At Boring 1-B5, the rock appears to be intermixed with fill and YBM at 16 feet bgs.

On February 7, 2025, we performed a supplemental geotechnical investigation that included seven sonic borings. We drilled the borings in an approximately parallel line from the pier entrance to the end of Seawall Drive to identify the extent of the rock dike at the proposed Pier Plaza. We encountered the rock dike in 2-B1 through 2-B4 beginning at approximately Elevation 5 feet and extending down to Elevation -9½ feet. The dike comprises light to dark gray well-graded gravel with sand, clayey gravel with sand, clayey sand with gravel, and silty sand with cobble up to 4½ inches in diameter. The clay we encountered with the gravel appears to be recent Bay deposits that accreted on top of the backside of the rock dike when it was in use as a breakwater. Immediately below the gravel, we encountered YBM in every exploration.

In 2-B5 and 2-B6, we encountered a dark gray silty gravel with sand at a similar depth as the top of the rock dike. However, in comparison to the dike, the fill contained finer gravel, fewer cobbles, seams of soft clay, and was easier to drill through. These borings were performed at least 10 feet beyond where aerial photographs indicate the backside of the rock dike was located. This layer was likely used to bridge YBM and accredited Bay sediment behind the dike to support fill placement. In 2-B7, the fill became more clay-like and similar in consistency to soil encountered at 1-B3.

2.5.2 Recent Bay Deposits/Young Bay Mud (Qybm)

Recent marine deposits along the bay margins often consist of soft fat clay deposited in a low-energy, estuarine environment during the current interglacial period, beginning approximately 8,000 years ago. This material is often referred to as Young Bay Mud (YBM). Locally, YBM can also contain thin, sandy interbeds. We encountered the top of YBM between Elevation 0 feet to Elevation -9½ feet throughout the site; however, its characteristics varied significantly landside and offshore.

We encountered two layers of offshore YBM. The upper layer consisted of a very soft gray fine-grained material with an abundant amount of shells directly below the mudline. Laboratory testing and CPT data indicate that this soil has undrained shear strengths of less than 100 pounds per square foot (psf). This unit is primarily recent sediments and for the purposes of this report, we refer to it as Recent Bay Deposits. Below the Recent Bay Deposits, we encountered soft to medium stiff YBM intermixed with sand.

The YBM we encountered in the landside explorations consisted of gray and black (with hues of blue and green) normally consolidated soft to medium-stiff lean and fat clay with abundant shells and minor organics. The YBM ranged in thickness between 4 to 16 feet, with the thinnest layer encountered below the rock dike and thicker deposits in the eastern and southern portions of the site. Triaxial strength and mini laboratory vane shear testing from samples of this YBM indicate shear strengths ranging from 300 to 750 psf. CPT tests corroborate the shear strength values, with undrained shear strength ranging from 500 to 800 psf, based on a cone factor (N_{kt}) of 14 (a conservative estimate). Additionally, we measured natural water contents in the unit varying between 27.6 to 86.7 percent; suggesting the presence of a fluctuating sand and shell content. The stiffer YBM on the landside is attributable to the overburden fill placed on the marine deposits

60 years ago. In addition, the landside YBM has experienced all primary consolidation resulting from land reclamation due to the time since fill placement.

2.5.3 San Antonio Formation (Qal 1)

The San Antonio Formation consists of estuarine and alluvial sediments lying between the OBC and YBM layers. The material was deposited in a complex ever-changing depositional environment ranging from alluvial fans to flood plains ranging old broad channels infilled with stiff sandy clay underlain by sandy channel fill.

The San Antonio Formation we encountered in our explorations consisted of yellowish brown, reddish brown, and reddish orange stiff clay with discontinuous lenses of medium dense to dense silt sand and clayey sand. Triaxial strength and mini laboratory vane shear testing of stiff clay samples collected in Modified California and Shelby Tube samplers indicate shear strengths ranging from 1,450 to 2,250 psf.

2.5.4 Old Bay Clay (OBC)

The San Antonio Formation is underlain by another interglacial marine deposit, colloquially named Old Bay Clay (OBC), and labelled as such in this report. This unit was deposited during the Sangamon interglacial period approximately 125,000 to 75,000 years ago. In general, OBC is typically described as moderately over-consolidated stiff clay with a characteristic greenish-blue hue, moderate plasticity and minor sand. Laboratory testing of one relatively undisturbed sample of OBC indicated an overconsolidation ratio (OCR) of approximately 1.5. Triaxial strength testing from samples collected in Modified California and Shelby Tube samplers indicate shear strengths ranging from 730 to 4,000 psf.

The OBC encountered in our borings consisted of gray, greenish gray and olive lean and fat clay with varying amounts of sand. In addition, there were lenses of gravel and sand interbedded in the OBC.

2.5.5 Upper Alameda Formation (OBC 2 to 4 and Qal 2 to 4)

Below the Sangamon Old Bay Clay lies the late Pleistocene Alameda Formation that was deposited 1,000,000 to 500,000 years ago. This formation has spanned a number of interglacial periods of sea level rise, as well as the intervening glacial ages. As such, much of the unit is highly interbedded marine bay mud and terrestrial alluvium deposits corresponding to the cyclic glaciation. These deposits share similar attributes to the younger OBC and San Antonio formation units encountered at shallower depth; however, they typically increase in shear strength due their age and depth.

2.6 GROUNDWATER CONDITIONS

We encountered groundwater in all of the sonic borings and one of our shallow borings during drilling. In addition, we performed pore pressure dissipation testing in our CPTs to approximate the groundwater levels. Table 2.6-1 summarizes our groundwater observations and interpretation of the pore pressure dissipation test. The groundwater encountered in 1-C7 was significantly shallower than other measured groundwater levels, indicating there may have been water perched on less permeable clay below.

TABLE 2.6-1: Groundwater Observations

SOURCE	APPROXIMATE DEPTH TO GROUNDWATER (feet)	ELEVATION (City of Berkeley MLLW)
1-C7	5	9
1-CPT2	13.4	5½
1-CPT3	11.4	3½
1-CPT4	11.1	5½
2-B1	7	4
2-B2	8	3
2-B3	8	3
2-B4	7	4
2-B5	13	3
2-B6	12	4
2-B7	13	3

The mean tide fluctuates between approximately Elevation 6.27 feet to 0 foot each day according to modelling completed by AECOM and documented in the San Francisco Bay Tidal Datums and Extreme Tides Study (2016). Due to the proximity of the site to the Bay, the groundwater level is likely heavily influenced by the tidal level. It is reasonable to assume, for construction planning, that the groundwater could be as shallow as Elevation 6.5 and fluctuating throughout the day.

The Ocean Protection Council (2024) presented sea level rise projections of 2.2 feet as an intermediate-high evaluation for the 2070-time horizon. Consequently, the mean sea level could be at approximately Elevation 5 feet owing to sea level rise within this timeframe. For the purposes of long-term design, including seismic evaluation, a groundwater table corresponding to the projected mean sea level is an appropriate assumption; while the groundwater could be shallower than this depth, due to fluctuating water table, the soil above the mean sea level is unlikely to be saturated which is a requirement of soil to be liquefiable.

Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, tidal influence, and other factors not evident at the time measurements were made. Perched groundwater may also result in locally shallower groundwater conditions.

3.0 DISCUSSIONS AND CONCLUSIONS

From a geotechnical viewpoint, the site is suitable for the proposed project, provided the geotechnical recommendations in this report are properly incorporated into the design plans and specifications. The geotechnical considerations for the Site are as follows.

- Non-engineered fill
- High seismic shaking and liquefiable soil
- Shoreline stability
- Soft compressible soil
- Shallow groundwater table

We discuss the above considerations, and the risks associated with these and other hazards with respect to the planned project in the following sections of this report.

3.1 NON-ENGINEERED FILL

As discussed in Section 2.5.1, we encountered up to 23 feet of artificial fill in the Marina. Common construction methods used during land reclamation activities in 1960s and prior did not employ engineering compaction during fill placement. Furthermore, our exploration encountered abundant debris within the fill. Therefore, the artificial fill at the site should be considered a non-engineered fill. The non-engineered fill is highly variable in soil type; the loose to medium dense clean sand and gravel that we encountered may exhibit a high potential for deformation during a seismic event. Our assessment of the liquefaction of saturated fill and the resulting potential post liquefaction reconsolidation settlement is discussed further in Section 3.5.3.

Additionally, the presence of non-engineered fill may lead to inconsistent bearing support and excessive foundation settlement of structures. We provide recommendations for fill improvement underneath building footprints in Section 5.2.

3.2 SHALLOW GROUNDWATER

Based on groundwater levels previously discussed in Section 2.6, we anticipate that the groundwater is likely tidally influenced. We also encountered shallow perched groundwater in 1-C7 at an elevation several feet above high tide. Shallow groundwater should be anticipated for excavations for underground construction that extend below about Elevation 6.5 feet and areas of shallower water perched on clay in the fill should also be anticipated. Regional dewatering operations should be avoided due to the presence of compressible soil below most of the site. Where necessary, dewatering should be limited to pumping from sumps within the trench, if possible. We recommend that the contractor constructing the subsurface utilities plan to perform some localized potholing prior to construction to assess if the groundwater will affect their improvements. Moreover, we provide recommendations in Section 5.3 to mitigate excessively wet ground at the base of excavations, if encountered.

3.3 CONSOLIDATION SETTLEMENT OF YOUNG BAY MUD

We encountered soft to medium stiff highly compressible YBM deposits below the artificial fill at the site. The thickness of the YBM that we encountered varied between 4 to 16 feet throughout the project improvement area. These deposits are normally consolidated or lightly over consolidated and will experience consolidation and creep settlement when subjected to new loading. Our study of historical aerials photographs indicates that the reclamation of the site was conducted more than 50 years ago. Primary consolidation settlement due to reclamation has completed, though long-term creep (secondary consolidation) of the site may be ongoing. Future loading from placement of fill or structural loads from buildings and site improvements will trigger additional long-term settlement. The amount of settlement depends on the proposed loads, thickness of the YBM, and the previous load experienced by the deposit. Based on our conversations with you, generally grades will be raised between 1 to 5½ feet within the site.

To estimate potential consolidation settlement under planned fill loads we performed consolidation analysis using the Settle 3D software program. Settle 3D is a three-dimensional analytical program that is primarily used for consolidation settlement analysis. We selected material properties, including unit weights, stress history, compression indices, coefficient of consolidation, etc., based on laboratory tests data, field measurements and our considerable experience with YBM in the San Francsico Bay Area. We provide an estimate of long-term consolidation in Table 3.3-1

TABLE 3.3-1: Summary of Consolidation Analyses

CIVIL FILL (feet)	SETTLEMENT (inches)
1 to 2	< 2
3 to 5½	2½ to 5

If mitigation measures are not implemented, we anticipate that approximately 75 percent of the settlement will occur within 3 months of fill placement and consolidation could continue for up to 5 years, slowing with time. The YBM will also experience creep; this settlement will occur for decades and will be approximately 1 inch over a 50-year time period. We provide mitigation recommendations for long-term settlement in settlement sensitive areas in Section 5.9.

3.4 SEISMIC INPUT FOR ANALYSES

3.4.1 Shear-Wave Velocity

As described in Section 2.2.1, we measured the shear-wave velocity (V_S) to a depth of 100 feet bgs during performance of 1-CPT1. We additionally considered the CPT-Vs correlation by Robertson (2009) to estimate the V_S of offshore bay deposits and a portion of the deeper Alameda Formation. Based on measured and correlated Vs Profiles, we classified the site as Site Class D in accordance with Chapter 20 of ASCE 7-16. While there are minor differences in the landside and offshore stratigraphy, it has a nominal impact on the time-averaged shear-wave velocity over the top 100 feet or 30 meters (V_{S30}).

3.4.2 Pier Structure Seismic Design Criteria

We developed seismic-design response spectra for the pier structure in accordance with the guidelines presented in the 2014 ASCE/COPRI 61 Standard: Seismic Design of Piers and Wharves (ASCE 61-14) with following amendments.

1. Based on our discussions with you, we understand the Ferry Pier will be designed as an “Essential” facility and the breakwater as a “Moderate” facility; ASCE 61-14 does not consider performance criteria for an Essential facility, so COWI developed performance criteria for this classification based on prior project experience. ASCE 61-14 incorporates, by reference, the seismic site classification and site-specific ground motion procedures described in the 2005 ASCE/SEI 7 Standard (ASCE 7-05).

2. In our evaluation, we found that the Contingency Level Earthquake (CLE) response spectra generally exceeded the Design Earthquake (DE) response spectrum in our period range of interests, creating incoherence in the seismic performance requirements presented in the code. Based on discussions on this topic, the team we opted to use a site-specific MCE response spectrum to assess the Design Earthquake performance levels.

The seismic hazard and performance levels that we considered are listed in Table 3.4.2-1.

TABLE 3.4.2-1: Seismic Hazard and Performance Levels

PERFORMANCE LEVEL	SEISMIC HAZARD LEVEL FOR "ESSENTIAL"	SEISMIC HAZARD LEVEL FOR "MODERATE"
Operating Level Earthquake (OLE)	50 percent in 50 years (72-year return period)	N/A
Contingency Level Earthquake (CLE)	10 percent in 50 years (475-year return period)	20 percent in 50 years (225-year return period)
Design Earthquake (DE)	Site Specific MCE Earthquake per ASCE 7*	Site Specific MCE Earthquake per ASCE 7*

*Amended from ASCE 61-14

It should be noted that the Design Earthquake (DE) is defined as two-thirds of the Maximum Considered Earthquake (MCE) which is defined in ASCE 7.

3.4.3 Pier Structure Site-Specific Seismic-Hazard Analysis

We performed a site-specific seismic-hazard analysis in accordance with ASCE 61-14 and ASCE 7-05 to develop response spectra for the seismic hazard and performance levels discussed in Section 3.4.2. The procedure for performing this analysis is discussed in Appendix F. We present the site-specific OLE, CLE, DE, and MCE response spectra in Table 3.4.3-1.

TABLE 3.4.3-1: Site-Specific OLE, CLE DE, and MCE Response Spectra

PERIOD (seconds)	PSEUDO-SPECTRAL ACCELERATION (g)				
	OLE (72-year)	CLE (225-year)	CLE (475-year)	DE (ASCE7-05)	MCE (ASCE 7-05)
0.01	0.25	0.41	0.54	0.57	0.85
0.02	0.24	0.41	0.53	0.57	0.85
0.03	0.25	0.41	0.53	0.56	0.85
0.05	0.27	0.45	0.58	0.62	0.93
0.08	0.34	0.55	0.72	0.77	1.16
0.10	0.41	0.66	0.85	0.91	1.36
0.12	0.46	0.73	0.93	0.98	1.47
0.15	0.52	0.82	1.05	1.00	1.50
0.20	0.58	0.93	1.18	1.00	1.50
0.25	0.62	1.00	1.28	1.00	1.50
0.30	0.63	1.05	1.35	1.00	1.50
0.40	0.60	1.04	1.38	1.00	1.50
0.50	0.57	1.01	1.35	1.00	1.50
0.60	0.51	0.93	1.25	1.00	1.50
0.75	0.43	0.81	1.13	0.80	1.20
1.00	0.35	0.70	1.00	0.60	0.90
1.50	0.24	0.51	0.75	0.42	0.63
2.00	0.18	0.40	0.60	0.32	0.47
3.00	0.11	0.26	0.40	0.20	0.30
4.00	0.07	0.18	0.29	0.15	0.23
5.00	0.05	0.13	0.21	0.12	0.18
7.50	0.03	0.07	0.11	0.08	0.12
8.00	0.02	0.06	0.10	0.08	0.11
10.00	0.02	0.04	0.06	0.06	0.09

We estimated the site-specific MCE peak ground acceleration (PGA) to be 0.85 g. The PGA is a median component (RotD50) value and probabilistically controlled.

3.4.4 2022 CBC Seismic Design Parameters

For landside improvements and structures, the 2022 CBC seismic parameters should be used for design. The 2022 CBC utilizes seismic design criteria established in the ASCE/SEI Standard "Minimum Design Loads and Associated Criteria for Buildings and Other Structures," (ASCE 7-16). The 2022 CBC requires that a classification of Site Class F be assigned to any site that is considered liquefiable. However, based on our experience with structures of this size, we anticipate the structural period will be less than 0.5 second, which allows classification as Site Class D per the exception in Section 20.3.1 of ASCE 7-16.

ASCE 7-16 requires a site-specific seismic-hazard analysis for Site Class D sites with a mapped S_1 value greater than or equal to 0.2; however, Section 11.4.8 of ASCE 7-16 and Supplement No. 3 provide an exception to this requirement. A site-specific seismic-hazard analysis is not required when the value of the parameter S_{M1} determined by Equation 11.4-2 and shown in Table 3.4.4-1 is increased by 50 percent for developing the mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response, calculating S_{D1}, and evaluating C_s in accordance with Chapter 12 of ASCE 7-16.

In Table 3.4.4-1 below, we provide the CBC seismic parameters based on the United States Geological Survey's (USGS') Seismic Design Maps for your use. When using this table, considerations should be given to exceptions in Section 11.4.8 of ASCE 7-16, as described previously.

TABLE 3.4.4-1: 2022 CBC Seismic Design Parameters Latitude: 37.8629 Longitude: -122.3176

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.72
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.65
Site Coefficient, F_a	1
Site Coefficient, F_v	1.7*
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.72
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.11*
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.15
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.74*

*The parameters above should only be used for calculation of T_s, determination of Seismic Design Category, and, when taking the exceptions under Items 1 and 2 of ASCE 7-16 Section 11.4.8. (Supplement Number 3 https://ascelibrary.org/doi/epdf/10.1061/9780784414248.sup3).

We recommend that we collaborate with your design team to further evaluate the effects of implementing the exception in the structural design and identify if there is a need for performing a site-specific seismic-hazard analysis. We can prepare a proposal for a site-specific seismic-hazard analysis, if requested.

3.5 SEISMIC HAZARDS

3.5.1 Ground Rupture

The site is not located within a State of California Earthquake Fault Hazard Zone and no known faults cross it; therefore, ground rupture is unlikely.

3.5.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Area could cause considerable ground shaking at the Site, similar to that which has occurred in the past. To mitigate the shaking effects, structures should be designed using sound engineering judgment and the guidelines provided in ASCE 61-14, as a minimum. Seismic design provisions of current codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead and live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage, but with some non-structural damage, and (3) resist major earthquakes without collapse but with some structural, as well as non-structural damage. Conformance to the code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.5.3 Soil Liquefaction

As shown in Figure 7, the site is mapped in a State of California Seismic Hazard Zone (CGS, 2006) for areas that may be susceptible to liquefaction. Soil considered most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sand below the groundwater table. Empirical evidence indicates that loose silty sand is also potentially liquefiable. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess pore-water pressures to develop. If excess pore-water pressures exceed the effective confining stress from the overlying soil, the sand is said to have liquefied, and if the liquefied sand consolidates or vents to the surface, ground settlement and surface deformation may occur.

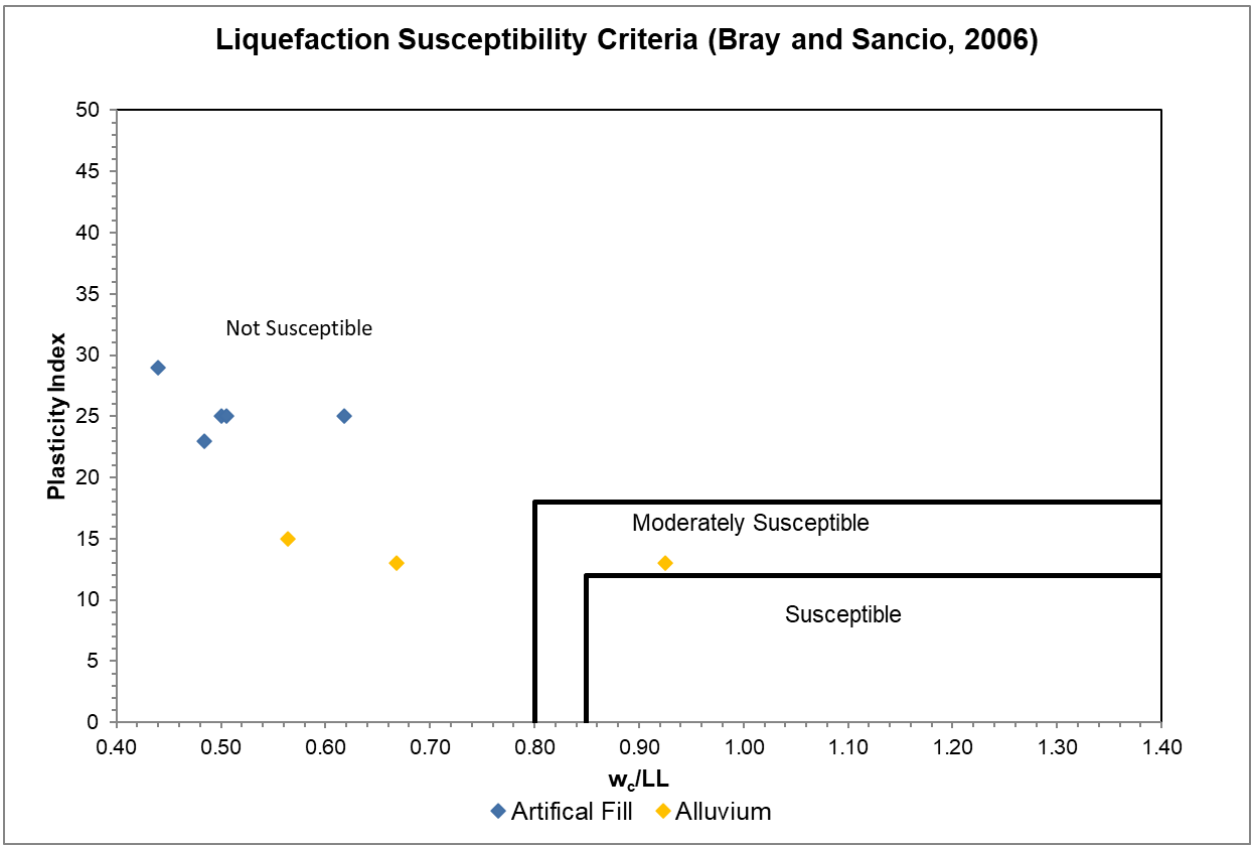
3.5.3.1 Liquefaction Susceptibility Screening of Soil Samples.

To identify an appropriate soil behavior type index (I_c) cutoff for the subsequent liquefaction analysis in Section 3.5.3.2, we sampled potential transitional soil (soil that appears to be on the border between clay-like and sand-like) that may pose a risk of liquefaction in our borings. We identified transitional material at various depths in the artificial fill and between Elevation -10 feet and -20 feet in the San Antonio Formation. We considered the criteria presented by Bray and Sancio (2006) to assess the potential for liquefaction triggering based on the on the index testing on the collected samples.

Bray and Sancio observed that soil with a plasticity index (PI) less than 12 and a water content (w_c) to liquid limit (LL) ratio of more than 0.85 are susceptible to liquefaction/cyclic softening. Soil with a PI greater than 18 and/or w_c/LL less than 0.8 were deemed to be not susceptible to liquefaction because they are too plastic and/or their water contents are too low.

We plotted this data on Exhibit 3.5.3.1-1. An additional sample in 1-B2 at Elevation -18.5 feet yielded a PI of 22; while we did not perform a moisture content test for the sample, it would plot in the "Not Susceptible" region based on the PI. Based on these results, we opine that the soil logged as sandy lean clay and clayey sand in the at this site is not susceptible to liquefaction.

EXHIBIT 3.5.3.1-1: Liquefaction Susceptibility Testing



3.5.3.2 CPT-Based Liquefaction Analysis

The data presented in Section 3.5.3.1 as 'Not Susceptible' to liquefaction generally has an I_c of 2.5 or greater. Based on this screening, we established an I_c cutoff value of 2.5 when performing our liquefaction analysis; this means that soil with an I_c above 2.5 is excluded from liquefaction triggering in our calculations.

We performed liquefaction analyses on the landside CPTs based on analysis methods by Idriss and Boulanger (2014) to estimate potential liquefaction triggering and volumetric liquefaction settlement. We estimated the cyclic stress ratio (CSR) for a geometric mean peak ground acceleration (PGA) of 0.85 based on the site-specific seismic hazard analysis for the MCE and a Moment Magnitude (M_w) 8.1 based on a San Andreas Fault rupture scenario. As discussed in Section 2.6, we assumed a groundwater table at Elevation 5 feet. Our results indicate there are discontinuous layers of liquefiable soil within the artificial fill. Soil in the San Antonio formation may experience a nominal amount of liquefaction.

We estimated settlement of liquefiable soil in each of our CPTs based on the method by Idriss and Boulanger (2014). Our analysis indicates that the ground could experience settlement up to the amounts shown in Table 3.5.3.2-1.

TABLE 3.5.3.2-1: Estimated Liquefaction Induced Volumetric Settlement.

CPT	VOLUMETRIC LIQUEFACTION INDUCED SETTLEMENT (inches)
1-CPT1	2½
1-CPT2	1¾
1-CPT3	1
1-CPT4	Nominal

We pushed 1-CPT1 through the rock dike; it took us several attempts to complete this CPT as we encountered shallow refusal and relocated the CPT several feet each time. The eventual breakthrough likely indicated a weaker zone within the dike. As such, we do not believe liquefaction in 1-CPT1 is persistent throughout the plaza area.

3.5.3.3 Liquefaction Ejecta

In addition to the previously described liquefaction analysis, we also evaluated the capping effect of the non-liquefiable soil above the liquefied layer. Based on empirical evidence, liquefaction-induced ejecta can occur if the pore-water pressure generated within the liquefied strata exerts a force sufficient to break through the overlying soil and vent to the surface resulting in sand boils or fissures. The loss of subsurface material beneath the building footprint could result in undermining the foundation, localized reduction in bearing capacity, and unpredictable settlements.

Hutabarat and Bray (2022) present a method for evaluating the severity of liquefaction-induced ejecta based on CPT data. The method considers the liquefaction ejecta demand from the liquefiable soil and the resistance to ejecta provided by the non-liquefiable soil above. We performed an analysis using this method assuming that 5 feet of engineered fill will be placed above the existing ground surface at the Pier entrance plaza and 2 feet in other regions of the site, to achieve planned finished grades. Based on our analysis, the site has a liquefaction-induced ejecta severity risk of “nominal” in 1-CPT1,1-CPT3, and 1-CPT4; and “moderate” in 1-CPT2. The moderate risk of liquefaction ejecta in 1-CPT2 is due to a clean sand layer situated between Elevation 8 feet to Elevation -2 feet. We opine that the lack of lateral continuity of this sand throughout the site and presence of moderately plastic lean clay and clayey sand capping the upper 9 feet throughout the site will inhibit significant amounts of ejecta.

TABLE 3.5.3.3-1: Estimated Liquefaction Induced Ejecta Severity

CPT	EJECTA SEVERITY
1-CPT1	Nominal
1-CPT2	Moderate
1-CPT3	Nominal
1-CPT4	Nominal

3.5.3.4 Lateral Spreading and Seismic Slope Stability

Lateral spreading is a seismically induced failure within a soil zone (typically due to liquefaction) that causes the overlying soil mass to move toward a free face or down a gentle slope. We distinguish this phenomenon from seismic slope stability, which is a failure of soft soil due to an imparted seismic loading. The effects of both pose a risk to lateral movement on the western end of the shoreline, at the pier entrance plaza, due to the presence of non-engineered fill and YBM. We performed a seismic slope deformation analysis and present our methodology and results in Section 3.6.

3.6 SHORELINE DEFORMATION ANALYSIS STABILITY

3.6.1 Geometry and Idealized Soil Profiles

We developed a geologic cross section using exploration data from our field exploration, regional geologic maps and the LiDAR and bathymetric survey data provided by eTrac. We also reviewed historical construction documents regarding Pier construction and filling of the South Cove Sailing Basin. Since we did not have as-built documents outlining the existing rock dike along Seawall Drive, we estimated the extents by evaluating our sonic boring exploration, reviewing the South Cove Sailing Basin grading plans (City of Berkeley, 1965), and georeferencing historical aerial imagery to the current shoreline. We performed our analysis based on the idealized stratigraphy of Cross-Section A-A' shown in Figure 2 and Figure 6.

3.6.2 Soil Strength Parameters

Prior to performing slope stability analyses, we evaluated the strength of the soil profile based on laboratory testing, CPT data, and various field correlations.

Laboratory testing of the YBM showed significant variation in water content and undrained shear strength. Consequently, we relied on continuous CPT data, using an N_{kt} of 14, to estimate shear strength values and corroborated our estimated values with existing literature. We also recognize that the seismic deformations in the YBM are likely to occur parallel with the layer; representative of a direct simple shear test (DSS), in contrast to CPT and UU failure modes. As such, we reduced our CPT undrained shear strength estimates by 20 percent. We assigned a static undrained shear strength ratio (vertical strength ratio) of 0.27 to the YBM.

Additionally, we modelled YBM as having a dynamic shear strength 30 percent greater than the corresponding static shear strength. We based this increase on studies showing that the undrained shear strength of YBM experiences a roughly 10 percent increase per log cycle increase in the rate of shear strain (Serna et al. 2019) under shear strains less than 3 percent.

As previously presented, our liquefaction analysis of the artificial fill indicates that approximately half of it may experience liquefaction or cyclic softening in discontinuous layers. We estimate that the liquefied fill will have a residual vertical strength ratio of 0.1 during a seismic event, while clayey non-liquefiable soil will retain a shear strength of 500psf. We averaged the material strengths to develop our estimate of shear strength.

The strength parameters that we used in our analyses are summarized in Table 3.6.2-1.

TABLE 3.6.2-1 Slope Stability Analysis Material Properties

SOIL LAYER	UNIT WEIGHT, γ (pcf)	COHESION STATIC (SEISMIC), c (psf)	FRICTION ANGLE, ϕ (deg)
Sandy Fill	120	-	30
Clayey Fill	120	500	-
Liquefied Fill	120	-/290	-
Qybm under Dike	95	540 (700)	-
Qybm	95	415 (540)	-
Offshore Qybm	80	100	-
Rock Dike/Riprap	135	-	45
DSM	135	4,320	-
Qal 1	125	1,700	-
Qobc 1	120	1,900	-
Qal 2	125	2,750	-
Qobc 2	120	3,000	-
Qal 3	125	3,000	-
Qobc 3	120	2,000	-
Qal 4	125	4,000	-

3.6.3 Method of Analysis

We performed a simplified deformation analysis using the computer program SLIDE (Version 9.035), which is a limit equilibrium program that allows the user various search routines to locate the minimum factor of safety and critical slip surface. We used circular and non-circular searching methods and the Spencer's Method of Slices (Spencer, 1973) for our analyses. We used a design groundwater level of Elevation 5 feet, corresponding to mean sea level and spectral ordinates corresponding to CLE and MCE response spectra.

In evaluating the stability of slopes at the site under seismic conditions, we used the pseudo-static method of analysis. The pseudo-static method models the effects of transient earthquake loading on a potential slide mass by using an equivalent sustained horizontal force determined as the product of a seismic coefficient and the weight of the potential slide mass. We used a staged pseudo-static strength for the calculation of effective stresses; this method estimates the strength of the soil based on effective stresses in a static condition then applies the external pseudo-static horizontal loading in a second step to estimate slope stability.

To estimate potential displacement of the shoreline, we used the methodologies from the National Cooperative Highway Research Program (NCHRP) Report 611 (2008) and the Bray and Macedo (2019) Simplified Procedure for Estimating Seismic Slope Displacement. These methods require estimation of the yield coefficient (k_y) which is the pseudo-static coefficient that results in a factor of safety of 1.0. The k_y and other characteristics about the seismic spectra are then used to estimate a lateral deformation. The value of estimated displacement resulting from these methods is a mean value of displacement based on a regression analysis and as such should be considered an index of potential displacement and not an exact amount of displacement. The following table shows our estimate of potential seismic deformation of the shoreline of Section A-A' based on both of these methods. We show the results of this analysis in Appendix G-1.

TABLE 3.6.3-1: Estimate of Potential Seismic Deformation

CASE	FACTOR OF SAFETY		YIELD ACCELERATION (g)		WALL PRESSURES (ksf)		POTENTIAL SEISMIC SLOPE DISPLACEMENT (inches)	
	Circular	Block	Circular	Block	CLE	MCE	CLE	MCE
Existing – Static	2.5	3.6	-	-	-	-	-	-
Existing – Psuedostatic	-	-	0.26	0.23	-	-	4 to 8	13 to 21

We assume that these magnitudes of deformation are unacceptable for accessibility to the pier as well as kinematic loading on the pier foundation elements at the shoreline. We developed two alternative mitigation systems to reduce displacements.

1. Buttressing the shoreline with Deep Soil Mix (DSM) Ground Improvement: DSM is a method of in situ ground improvement where soil is mechanically mixed with a cementitious binder to significantly increase its strength. The DSM would be accomplished to create a series of overlapping vertical columns forming shear panels oriented perpendicular to the shoreline.
2. Sheet Pile Wall – this wall would be oriented parallel to the shoreline and extend deep enough into the stable soil below the site to restrain the soil from moving laterally in an earthquake.

We discuss ground improvement methods further in Section 4.

We evaluated the minimum DSM dimensions and strength using the same pseudo-static analysis as previously presented and the shear strength shown in Table 3.6.2-1. The DSM shear strength is based on an assumed compressive strength of DSM of 200 pounds per square inch (psi) and an area replacement ratio of 0.3; we assumed that the shear strength is approximately one-half the compressive strength. Based on experience, we assumed a displacement of no more than 6 inches was acceptable for shoreline stability which results in a k_h of 0.33g to 0.37g for the MCE event. We then evaluated slope displacements behind (to the east) and in front (to the west) of the DSM; modifying the depth and width of the DSM until achieving a factor of safety of at least 1.0. We show the results of this analysis in Appendix G-2. While our results indicate 5 to 7 inches of displacement during the MCE; the values are based on simplified pseudo-static procedures and should be interpreted as an index of movement rather than prescriptive values. As such, we opine that the displacement is consistent with a criteria of approximately ½ foot of displacement, and the project will be able to maintain ingress and egress to the pier following an MCE level event.

To develop active and earthquake loads on the sheet pile wall, we performed a limit equilibrium analysis using a force to represent the bulkhead capacity; this approach is discussed in NCHRP 611 and is referred to as the GLE method. We applied k_h values ranging between 0.45g to 0.54g, which is consistent with 2 inches of wall deflection for an MCE event. We changed the value of the externally applied force until we calculated a factor of safety of at least 1. We show the results of this analysis in Appendix G-3.

Our experience and more rigorous numerical modeling we performed of similar sites indicates that the lateral movement may have a shape similar to the failure surfaces shown in Appendix F and have deformations that are larger at the shoreline and decrease with distance inland. However, the pseudo-static method of analysis is not able to accurately capture this deformation distribution, and it often indicates low factors of safety farther away from the shoreline due to the increased mass above the slide plane. Based on our judgement and our work on many other

bayfront sites in the area, we expect that the deformation during a seismic event will be limited to approximately 135 feet from the shoreline.

3.6.4 Results of Slope Deformation Analysis

Table 3.6.4-1 shows the results of our analyses of the two mitigations evaluated. The results show a range since we evaluated the displacement using two methods and considered both circular and non-circular surfaces.

TABLE 3.6.4-1 Results of Displacement Analyses

CASE	FACTOR OF SAFETY		YIELD ACCELERATION (g)		WALL EQUIVALENT FLUID PRESSURES (pcf)		POTENTIAL SEISMIC SLOPE DISPLACEMENT (inches)	
	Circular	Block	Circular	Block	CLE	MCE	CLE	MCE
Improved DSM (Front)	-	-	0.52	0.36	-	-	Nominal to 3	5 to 7
Improved DSM (Behind)	-	-	0.55	0.46	-	-	Nominal to 2	2 to 3
Sheet Pile Wall	-	-	-	-	170	340	2	2

For the evaluation of DSM, we estimate the location of the rock dike based on exploration, historical aerial photographs review, and review of construction plans. The presence of rock will make performing DSM difficult and more expensive as the equipment is not compatible with material of cobble size or greater. We assumed DSM would be performed to avoid the rock dike. There may not be enough space to perform the DSM mitigation in front of the rock dike; the limited space assumed results in a lower performance than the behind rock option. The estimated range of displacement presented in Table 3.6.4-1 includes slope displacement both in front and behind the area to receive DSM.

The values of pressure shown for the sheet pile wall option are the active plus earthquake pressure down to the bottom of the YBM at Elevation -15 feet. The magnitude of the seismic-induced pressure and height of cantilever of the sheet piles may necessitate either addition of tiebacks or bracing piles to provide additional support.

3.7 SOIL CORROSION POTENTIAL

We performed corrosion tests on two samples of soil from our borings. We transported the samples under the chain-of-custody to CERCO Analytical to evaluate the corrosion potential of the subsurface materials towards ferrous metals and concrete. The tests included minimum resistivity, pH, chloride content, and sulfate content. Test results are included in Appendix D. A summary of the results is presented below.

TABLE 3.7-1: Corrosivity Test Results (CERCO, 2024)

SAMPLE LOCATION	DEPTH (FEET)	REDOX (MV) [a]	PH[b]	MINIMUM RESISTIVITY[c] (OHMS-CM)	CHLORIDE[d] (MG/KG)	SULFATE[e] (MG/KG)	CORROSIVITY
1-B2	4.5-5	270	6.86	360	570	97	Severely Corrosive
1-B3	5.5-6	200	8	760	N.D.	17	Corrosive

a. ASTM D1498
b. ASTM D4972
c. ASTM G57
d. ASTM D4327

Based on the resistivity measurements and chloride ion concentration from our collected samples, we classified the soil as severely corrosive to buried metal and concrete. YBM and marine environments are known to be corrosive to metal and concrete. The site conditions present a risk to embedded pipes, walls, concrete structures, and piles. Sacrificial steel thickness or cathodic protection should be employed for steel elements and special consideration should be given to the concrete mix design. We recommend a corrosion consultant be retained to evaluate the specific corrosion recommendations for the project.

4.0 CONCEPTUAL SHORELINE STABILIZATION MITIGATIONS

Based on our discussion with you, to achieve the desired seismic performance for the pier we recommend implementation of a mitigation of the effects of seismic slope instability. This section provides additional detail regarding proposed shoreline stabilization presented in Section 3.6. We considered a range of stabilization methods as potential mitigation alternatives and recommend the use of DSM ground improvement and/or tied-back retaining walls.

4.1 DSM

DSM will increase the strength of the YBM, significantly reducing the potential for soil movement within and behind the improvement area. The DSM buttress would likely consist of several shear panels oriented perpendicular to the shoreline and Seawall Drive that are created by overlapping vertical DSM columns that will likely be between 3 and 5 feet in diameter. Various specialty DSM contractors utilize different equipment, and the exact size of the DSM columns, spacing of DSM columns, size of the panels, and spacing of the panels will depend on the contractor selected. The DSM will need to extend to the bottom of the YBM and approximately 1 diameter into the alluvium below. We show our recommendation of the lateral extent of the DSM in Figure 8; the length of the area to receive DSM is approximately 110 lf to protect the Pier and 260 lf to protect the plaza region. The soil directly below the rock dike cannot efficiently be improved using DSM due to the very dense nature and the presence of large cobbles in the rock dike.

Our experience indicates that the DSM panels should have a minimum depth to width ratio of 1:¾; additional design evaluation should be performed to verify that the panels are wide enough to not experience plunging of the toe or tension in the trailing elements if this mitigation is advanced. An additional benefit of the DSM buttress is that it generates spoils that amount to approximately 40 to 50 percent of the volume mixed; once they are allowed to cure, the spoils can be used as engineered fill at the site with enhanced engineering properties.

The location of any existing utilities that will remain within the planned DSM footprint will need to be accommodated in the final DSM design. They may need to be relocated or the DSM designed to have an opening around existing utility lines. All utilities within 25 feet of DSM should be located from potholing to reduce the risk of damage from DSM construction activities. Ideally new utilities can be kept in the fill above the top of the DSM. If new utilities need to extend into the DSM depth, they should be backfilled with controlled density fill with a shear strength at least equal to the DSM.

4.2 RETAINING WALL

A reinforced cast-in-place concrete (on a footing or deep foundation), soldier pile with shotcrete facing, or reinforced sheet pile wall may be used as an alternative to or in conjunction with the DSM for shoreline stability if it is designed to accommodate additional seismic and soil loads. To accomplish this, the wall should be installed with tie backs or braces and extended below the YBM. Refusal or bending of sheet or solider piles is expected if they are driven. As such, pile locations should pre-drilled or spudded prior to installation and backfilled over top to meet the appropriate grade.

Table 3.6.4-1 shows our estimated range of pressures for active and earthquake loading assuming a 2-inch deformation is tolerable for the wall. For the purpose of design, we recommend using an equivalent fluid pressure of 170 pounds per cubic foot (pcf) for a CLE-level event and 340 pcf for a MCE-level event (averages between selected methods). Figure 9 shows the distribution of these pressures as well as our recommendations for active pressure below the potential deformation surface and passive resistance for sheet pile design.

5.0 EARTHWORK RECOMMENDATIONS

As used in this report, relative compaction refers to the in-place dry unit weight of soil expressed as a percentage of the maximum dry unit weight of the same soil, as determined by the ASTM D1557 laboratory compaction test procedure, latest edition. Compacted soil is not acceptable if it is unstable; it should exhibit only minimal flexing or pumping, as observed by our field representative. The term “moisture condition” refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry.

Structural areas are defined as regions where constructed elements transfer loads to the subgrade, such as pavement and buildings. These elements rely on an unyielding subsurface to maintain their integrity.

5.1 GENERAL DEMOLITION AND SITE CLEARING

After demolition of the Pier and associated improvements, the Site should be cleared of all obstructions, including existing foundations and debris. Any existing underground utilities that are not to remain in service after construction within the site should be identified and removed entirely, including pipes and their backfill. Depressions resulting from the removal of underground obstructions extending below the proposed finished grades should be cleared and backfilled with suitable material compacted to the recommendations presented in Section 5.5.

5.2 NON-ENGINEERED FILL

As described previously, we identified the presence of non-engineered fill. The planned structures other than the Pier will generally be lightly loaded. We recommend the existing fill be removed and recompacted to a depth of 3 feet within the building pad and extend a minimum of 5 feet laterally beyond the footprint of proposed minor structures.

5.3 OVER-OPTIMUM SOIL MOISTURE CONDITIONS AND UTILITY EXCAVATION

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. Additionally, it is common to encounter overly wet soil in the subgrade of existing pavement areas when they are reconstructed. Wet soil should also be anticipated during utility excavation and other underground construction due to shallow groundwater. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather,
2. Mixing with drier materials,
3. Mixing with a lime, lime-fly ash, or cement product; or
4. Overexcavating unstable soil and replacing with a geotextile stabilization fabric such as Mirafi 500x placed below the layer Class 2 Aggregate Base.

We should be allowed to evaluate Options 3 and 4 prior to implementation.

Due to the sandy and loose artificial fill and shallow groundwater at the site, shoring will likely be required for any excavation in an area that cannot be sloped. Temporary sheet piles or a shield or continuous hydraulic skeleton shoring should be anticipated for utility excavations that extend below a depth of about 5 feet.

5.4 ACCEPTABLE FILL

From a geotechnical perspective, the existing fill at the site soil is suitable for use as engineered fill, provided they are processed to remove concentrations of YBM, organic material, any shells, debris, and particles greater than 6 inches in maximum dimension. Unsuitable materials and debris and particles larger than 6 inches should be removed from the project site. If a soil mixing ground improvement is implemented, the cement-rich spoils generated from ground improvement may also be used as fill. The use of spoils as fill will cause adverse effects on planted vegetation within the grading area.

Imported fill material should meet the above requirements and have a plasticity index less than 25 and at least 20 percent passing the No. 200 sieve. Import materials should be submitted to us for approval prior to delivery to the site. The contractor should allow us to sample, and test proposed imported fill materials at least 72 hours prior to delivery to the Site.

5.5 FILL COMPACTION

5.5.1 Grading in Structural Areas

Areas to receive fill should be excavated to a firm unyielding surface, scarified to a depth of 8 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. All fill should be placed in loose lifts that do not exceed 12 inches or the depth of penetration of the compaction equipment used, whichever is less. We recommend the following compaction and moisture content requirements for the placement and compaction of engineered fill.

TABLE 5.5.1-1: Fill Compaction and Moisture Content Recommendations

FILL LOCATION	MINIMUM RELATIVE COMPACTION (%)	MINIMUM MOISTURE CONTENT (percentage points above optimum moisture content)
General Fill	90	2
Pavement Subgrade (top 12 inches)	95	0
Caltrans Class 2 Aggregate Base	95	0

Pavement subgrade soil should be in a stable, non-pumping condition at the time aggregate base is placed and compacted. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor, and our representative.

5.5.2 Underground Utility Backfill

Project consultants involved in utility design should specify pipe bedding materials. We recommend that utility trench backfilling be done under our observation. Trench backfill in structural areas should be placed and compacted in accordance with Table 5.5.1-1. If uniformly graded gravel is used, we recommend that it be encapsulated in 6-ounce filter fabric.

Jetting of backfill is not an acceptable means of compaction. Controlled density fill is also suitable for pipe zone and trench zone backfill.

5.5.3 Landscape Fill

We recommend processing, placing, and compacting fill in landscaped areas in accordance with the “General Fill” material in Table 5.5.1-1, except it should be compacted to at least 85 percent relative compaction.

5.6 CUTOFF CURBS

Overly wet pavement subgrade or aggregate base can cause premature failure or increased maintenance of the pavement section. This condition often occurs where landscape areas directly abut and drain toward streets. Cutoff barriers are not required by City of Berkeley standards, but they can be implemented to reduce the impacts of landscaping directly adjacent to pavement. If desired to install cutoff barriers, they should be considered where pavement areas lie downslope of any landscape areas that are to be irrigated, and should extend to a depth of at least 6 inches below the base rock layer. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture barriers.

5.7 SITE SURFACE DRAINAGE

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finished grades be sloped away from buildings and pavements to the maximum extent practical. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent for pervious surfaces within 10 feet of foundations. Where lot lines or surface improvements restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. We also recommend infiltration be restricted to prevent introducing collected runoff to subgrade with low permeability and to limit excessive sheet flow. As a minimum, we recommend the following.

1. Roof downspouts should discharge into closed conduits that are directed away from foundations to appropriate drainage devices.
2. Water should not be allowed to pond near foundations, pavements, or exterior flatwork.

5.8 STORMWATER BIORETENTION AREAS

Infiltration testing was not included in our scope as part of this geotechnical exploration. Based on Site soil and the groundwater table encountered, infiltration rate of 0.05 inches/hour may be possible at the ground surface; however, the presence of the shallow groundwater should be considered in design of infiltration.

If bioretention areas are planned at grade, we recommend that, when practical, they be placed a minimum of 5 feet away from the building and other structural Site improvements, such as streets, retaining walls, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural site improvements can either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or
2. Incorporate filter material compacted in accordance with Section 5.5.1 and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, site improvements located adjacent to bioretention areas that are underlain by base rock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement. Where adjacent site improvements include streets steeper than 3 percent or design elements that will experience lateral loads (such as from impact or traffic), additional design considerations may be required. In addition, although not recommended, if trees are to be planted within bioretention areas, HDPE Tree Boxes that extend below the bottom of the bioretention system should be installed to reduce potential impact to subdrain systems that may be part of the bioretention area design. For this condition, the waterproofing system should be connected to the HPDE Tree Box with a waterproof seal.

5.9 CONSOLIDATION MITIGATION

If the amount of potential settlement discussed in Section 3.3 is inconsistent with anticipated long-term performance, it can be mitigated by placing lightweight fill, pre-loading the area with compacted soil before construction, or a combination of both. We provide a general description

of both methods below and specific recommendations pertaining to settlement mitigation at the Pier Plaza area in Appendix J.

5.9.1 Lightweight Cellular Concrete

The most cost-effective type of lightweight fill for this site appears to be cellular concrete. Cellular concrete is a mixture of cement, water, and a proprietary foaming agent. The proportion of foaming agent added can reduce the weight of the cellular concrete to as little as 17 pounds per cubic foot (pcf). For this project, we assume that cellular concrete of 30 pcf will be used.

Cellular concrete will weigh approximately one quarter the weight of soil fill. If a “net zero” pressure balance is required, a foot of existing fill can be removed and replaced with cellular concrete for each 3 feet of cellular concrete placed above existing grade. Where soil, aggregate base, asphalt concrete or Portland Cement concrete are included in the fill section, at least 1½ feet of existing fill should be removed and replaced with cellular concrete for each foot of these materials placed.

Cellular concrete is lighter than water, where cellular concrete is placed the area should be pumped dry and kept dry until the cellular concrete is completely placed and cured. To address long-term buoyancy and considerations for sea level rise, we recommend considering permeable cellular concrete which will experience very little, if any buoyancy effects.

Cellular concrete, while robust and an appropriately strong backfill material, is subject to erosion from both tracked and rubber tired equipment. We recommend that cellular concrete be protected with a layer of aggregate or soil once placement is complete to reduce the risk of generating concrete dust from surface erosion caused by equipment traffic.

5.9.2 Surcharging

A surcharge program would involve temporarily blanketing the settlement sensitive area with temporary fill until the YBM is practically complete with settlement under the weight of the surcharge. Subsequently, the surcharge may be cut to final subgrade and construction of settlement sensitive infrastructure can be built. The settlement during the surcharge should be monitored with settlement plates placed at the base of the fill. We estimate that surcharge will take on the order of 2 to 3 months; however, the actual time before surcharge removal will depend on the results of the monitoring. The surcharge can be removed once approximately 75 percent of the settlement under the surcharge load is achieved.

6.0 FOUNDATION RECOMMENDATIONS

6.1 DRIVEN PILES SUPPORTING PIER

Based on our discussion with you, we understand that the proposed pier will have a finished deck at approximately Elevation 16 feet and include the construction of the following piles.

- 24-inch plumb octagonal precast prestressed concrete piles to support the pier
- 24-inch battered octagonal precast prestressed concrete piles to support the breakwater
- 4-foot-by-14-inch or 4-foot-by-12-inch concrete sheetpile breakwater
- 36-inch-diameter plumb steel pipe piles for berthing floats
- 24-inch-diameter steel pipe piles for concrete abutment

During installation, pile driving should be continuous, as interruptions for extended periods of time may allow the pile to set up and result in harder pile driving resistance to reach design tip elevations. High driving resistance may be encountered as the pile advances through the alluvial deposits where dense sands were encountered. The landside piles may require spudding or predrilling through the rock dike. Jetting may be necessary to drive piles through dense sand layers. If piles for pier support are jetted for installation, they should be driven at least an additional 10 feet.

The contractor should submit a pile driving program for our review and approval. At a minimum, the pile driving submittal should include an analysis of the pile driving system to be used by the contractor. This analysis should indicate the hammer type and an approximate blow count that would represent refusal conditions for the soil type, piles, and driving system.

To develop estimates of vertical pile lateral capacity and lateral pile performance, we used an idealized soil profile with soil properties matching those used in our slope stability analysis in Section 3.6. We present the generalized stratigraphy used in analysis for offshore piles in Table 6.1-1 and for piles driven through the rock dike in Table 6.1-2.

TABLE 6.1-1: Generalized Offshore Pile Stratigraphy and Soil Properties

GEOLOGIC UNIT	ELEVATION TOP (feet)	ELEVATION BOTTOM (feet)	UNIT WEIGHT, γ (pcf)	COHESION, c (psf)	FRICTION ANGLE, ϕ (deg)
Offshore Qybm	-6	-13	80	100	-
Qal 1 (Clay)	-13	-19	125	1700	-
Qal 1 (Sand)	-19	-23	125	-	34
Qal 1 (Clay)	-23	-29	125	1700	-
Qobc 1	-29	-86	120	1900	-
Qal 2	-86	-99	125	2750	-
Qobc 2	-99	-104	120	3000	-
Qal 3	-104	-112	125	3000	-
Qobc 3	-112	-133	120	2000	-
Qal 4	-133	-152	125	4000	-

TABLE 6.1-2: Generalized Landward Pile Stratigraphy and Soil Properties

GEOLOGIC UNIT	ELEVATION TOP (feet)	ELEVATION BOTTOM (feet)	UNIT WEIGHT, γ (pcf)	COHESION, c (psf)	FRICTION ANGLE, ϕ (deg)
Rock Dike	Varies	-9	135	-	40
Qybm	-9	-14	95		
Qal 1 (Clay)	-14	-19	125	1700	-
Qal 1 (Sand)	-19	-23	125	-	34
Qal 1 (Clay)	-23	-29	125	1700	-
Qobc 1	-29	-86	120	1900	-
Qal 2	-86	-99	125	2750	-
Qobc 2	-99	-104	120	3000	-
Qal 3	-104	-112	125	3000	-
Qobc 3	-112	-133	120	2000	-
Qal 4	-133	-152	125	4000	-

6.1.1 Vertical Capacity

We developed estimates of the ultimate compression and tension capacity for the octagonal piles and sheet pile walls and present them in Figure 10 and Figure 11, respectively. During pile installation, we recommend performing high-strain dynamic testing using a pile driving analyzer (PDA) to verify the pile capacities and monitor driving stresses.

TABLE 6.1.1-1: Recommended Factor of Safety for ASD and Resistance Factor for LRFD Pile Design

DESIGN METHOD	LOAD CONDITION/LIMIT STATE	PILE CAPACITY COMPONENT	FACTOR OF SAFETY/RESISTANCE FACTOR
ASD (Factor of Safety)	Static	Compression and Tension	2.0
	Seismic		1.5
Load Resistance Factor Desing* (Resistance Factor, ϕ)	Strength Limit	Compression and Tension	0.7
	Extreme Limit	Compression	1.0
		Tension	0.8

*The values of resistance factor assume PDA testing is performed to confirm ultimate capacity

Due to the substantial tip area of the breakwater, the strain necessary to develop end bearing is considerably incompatible with the strain required to develop skin friction. Therefore, for the concrete sheet pile wall, we derived capacity solely from skin friction.

We also developed t-z curves (skin friction load-deflection springs) for the battered octagonal piles and sheet piles to represent the mobilization of the soil side friction from static loading (settlement from initial loading of the pile not including the elastic settlement of the soil from overall loading) using the computer program Apile v2023.10.4 by Ensoft Inc. We developed a spring at the top and bottom of each soil layer and every 5 feet through the Old Bay Clay. In addition, we developed q-w curves (end bearing load-deflection springs) at select depths for the battered concrete octagonal pile, where partial mobilization of tip resistance is anticipated. For evaluation of mobilized side friction of piles at depths between the spring depths, linear interpolation may be performed. A summary of the t-z and q-w springs may be found in Appendix F.

6.1.2 Lateral Capacity

For the purposes of estimating soil reaction to lateral loads, we developed lateral soil-structure load-deflection curves (p-y springs). We developed the springs for octagonal and steel pipe piles using the computer software LPile v2022.12.01 by Ensoft Inc. For the sheet pile wall, we used the NCHRP 611 (2008) method to develop p_w-y springs. We developed a spring at the top and bottom of each soil layer and every 5 feet through the OBC, until the spring stiffness stopped increasing with depth. For evaluation of soil resistance to lateral pile loads at depths between the spring depths, linear interpolation may be performed. We recommend using the best estimate spring models to evaluate kinematic pile loading due to slope deformation.

We utilized the Matlock model for Soft Clay for the YBM, the Stiff Clay without Free Water model by Reese for the stiff alluvial clay and OBC; the Reese sand model for interbedded sand in the San Antonio Formation and the rock dike. P-y springs can be found in Appendix G.

6.2 FOUNDATION FOR MINOR STRUCTURES

In this section we provide recommendations for foundations of lightly loaded, non-occupied, minor ancillary structures such as restroom facilities, landscape arbors, seating facilities, and monuments.

6.2.1 Structurally Reinforced Mat Foundation

Structures may be supported on a structurally reinforced mat foundation designed to impose an average allowable bearing pressure of at most 750 pounds per square foot (psf) for dead-plus live loads. Areas of concentrated loading may be designed using a maximum allowable bearing capacity of 1,000 psf for dead plus live loading. These values may be increased by one-third for transient loads such as wind or seismic.

6.2.2 Spread Footings

Structures can alternatively be supported on continuous and isolated spread footings with a maximum allowable bearing capacity of 1,000 psf. The footings should be designed with minimum footing dimensions in Table 6.2.2-1

TABLE 6.2.2-1: Typical Footing Dimensions

FOOTING TYPE	*MINIMUM DEPTH (inches)	MINIMUM WIDTH (inches)
Isolated	24	24
Continuous	24	12

* below lowest adjacent pad grade

The minimum footing depths shown above are taken from lowest adjacent pad grade.

The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. Footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

6.2.2.1 Slabs-On-Grade/Interior Concrete Floor Slabs

We recommend the following minimum design for slab-on-grade floors.

1. A minimum concrete thickness of 5 inches.
2. Minimum steel reinforcing of No. 3 rebar on 18-inch centers each way placed within the middle third of the slab to help control the width of shrinkage cracking that inherently occurs as concrete cures.

6.2.2.2 Waterstop

If a two-pour system is used for footings and slab, the cold joint between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent finish exterior grade. If this is not done, then we recommend the addition of a waterstop between the two pours to reduce moisture penetration through the cold joint and migration under the slab. Use of a monolithic pour would eliminate the need for the waterstop.

6.2.3 Shallow Foundation Settlements

Provided our report recommendations are followed; we estimate total static foundation settlements of up to 1 inch with a similar amount of differential settlement over a lateral distance of 50 feet. Due to liquefaction in the artificial fill, we expect differential settlement will be on the order of 1½ inches over a lateral distance of 50 feet.

6.2.4 Shallow Foundation Lateral Resistance

Lateral loads may be resisted by friction along the base and by passive resistance along the sides of foundations. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend the following ultimate values for design.

- Passive Lateral Resistance: 250 pcf
- Coefficient of Friction: 0.30

The values above are unfactored and an appropriate factor of safety should be implemented based on the load combination considered and the method of analysis. These two resistances can be combined; however, due to incompatibility of strain to peak value, one of the two values should be reduced by half if they are combined.

Passive lateral pressure should not be used for footings on or above slopes.

6.2.5 Slab Moisture Vapor Protection

When structures are constructed with concrete slab-on-grade floors or structural mat foundations, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

- A vapor retarder membrane should be placed directly beneath the slab or mat. The vapor retarder membrane should be sealed at all seams and pipe penetrations. Vapor retarders should conform to Class A vapor retarder in accordance with ASTM E1745-97 “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs.”
- For non-structural slab-on-grade floors, a 4-inch-thick layer of ¾-inch clean crushed rock should be placed below the vapor retarder membrane to act as a capillary break.
- Concrete should have a concrete water-cement ratio of no more than 0.50.
- Inspection and testing should be performed during concrete placement to check that the proper concrete and water-cement ratio are used.
- Slabs should be moist cured for a minimum of 3 days or other equivalent curing should be specified by the structural engineer.

6.2.6 Drilled Piers

Appropriate structures may be supported on drilled, cast-in-place, straight-shaft friction piers. The following design criteria should be incorporated into the structural design for the proposed light pole foundation. The piers should have a minimum diameter of 12 inches and extend to a depth of at least 6 feet below the existing ground surface. Piers should be designed for an allowable skin friction of 250 psf plus the weight the pier for combined dead-plus-live loads with a one-third increase allowed for either transient wind or seismic loading.

Resistance to uplift loads is developed in friction along the pier shafts. We recommend that an allowable uplift frictional resistance of 200 psf be used for tension resistance.

Lateral loads exerted on drilled piers and may be resisted by a passive resistance based on an equivalent fluid pressure of 250 pounds per cubic foot acting against the projected area of the individual pier shafts below grade.

The bottoms of pier excavations should be dry, reasonably clean, and free of loose soil before reinforcing steel is installed and concrete is placed. We recommend that the excavation of piers be performed under our direct observation to establish that the piers are founded in suitable materials and constructed in accordance with the recommendations presented in this letter.

Due to the potential for caving, each shaft may need to be cased. If groundwater is encountered, remove it from excavations prior to concrete placement. If groundwater cannot be removed from excavations prior to concrete placement, then we recommend that concrete be placed by tremie pipe. The concrete should be tremied to the bottom of the hole keeping the tremie pipe below the surface of the concrete to avoid entrapment of water in the concrete. As concrete is poured, water is displaced out of the hole.

7.0 RETAINING WALLS

7.1 ABOVE-GRADE RETAINING WALLS

Unrestrained walls constructed on the Site on level and sloped foregrounds should be designed for active lateral fluid pressure as provided below.

TABLE 7.0-1: Active Earth Pressure (Drained)

BACKFILL SLOPE CONDITION	ACTIVE PRESSURE (pcf)
Level	40
3:1	45
2:1	50

Passive pressures acting on foundations and shear keys may be assumed as 250 pcf, provided that the area in front of the retaining wall is level for a distance of at least 10 feet or three times the depth of foundation and keyway, whichever is greater. The upper 1 foot of soil should be excluded from passive pressure computations unless it is confined by pavement or a concrete slab. The friction factor for sliding resistance may be assumed as 0.30. On a preliminary basis, the retaining wall footings may be planned using an allowable bearing pressure of 1,000 psf on firm recompacted pads. The footings should be at least 24 inches below lowest adjacent grades.

The above lateral earth pressures assume sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended. Damp-proofing of the walls should be included in areas where wall moisture would be problematic.

Under seismic conditions, the active incremental seismic force along the wall should be added to the static active pressure and can be calculated as follows.

$$\Delta P = 10 \times H^2$$

Where H is the design height of the wall (in feet) and ΔP is the active incremental seismic force in pounds per foot length of wall. Since seismic loading requires soil movement, evaluation of the seismic case should consist of adding the seismic increment to the active soil pressure for all wall types. The resultant seismic force should be applied at $^1/_3$H from the base of the wall, indicative of a triangular pressure distribution.

7.1.1 Retaining Wall Drainage

Either graded rock drains or geosynthetic drainage composites should be constructed behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives.

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-2.02F) placed directly behind the wall, or
2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Envelop rock in a minimum 6-ounce, non-woven geotextile filter fabric.

For both types of rock drains:

1. The rock drain should be placed directly behind the walls of the structure.
2. The rock drains should extend from the wall base to within 12 inches of the top of the wall.
3. A minimum of 4-inch-diameter perforated pipe (glued joints and end caps) should be placed at the base of the wall, inside the rock drain and fabric, with perforations placed down.
4. The pipe should be placed at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

We should review and approve geosynthetic composite drainage systems prior to use.

7.1.2 Backfill

Backfill behind the retaining walls should be placed and compacted in accordance with Section 5.5 as general site fill. Light compaction equipment should be used within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

7.2 PIER ABUTMENT AND MECHANICALLY STABILIZED EARTH WALL

Fill placed at the pier plaza will be retained by both the pier abutment and a mechanically stabilized earth (MSE) seawall (Retaining Wall) with a precast panel facing. We provide design recommendations for the pier abutment in Appendix K and our complete design submittal for the MSE Retaining wall in Appendix L.

8.0 CONSTRUCTION MONITORING

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to:

1. Review the final grading and foundation plans and specifications prior to construction to evaluate whether our recommendations have been implemented, and to provide additional or modified recommendations, as needed. This also allows us to check if any changes have occurred in the nature, design, or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.

   Perform construction monitoring to check the validity of the assumptions we made to prepare this report. Earthwork and ground improvement operations should be performed under the observation of our representative to check that the Site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fill has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is important.
   If we are not retained to perform the services described above, then we are not responsible for any party's interpretation of our report (and subsequent addenda, letters, and verbal discussions).

9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements of Berkeley Water Transportation Pier project located in Berkeley, California. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The preliminary conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strive to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is provided, express or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data is representative of the actual subsurface conditions across the Site. Considering possible underground variability of soil, rock, stockpiled material, and groundwater,

additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, we should be notified immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration. In addition, our geotechnical exploration did not include work to assess the existence of possible hazardous materials. If any hazardous materials are encountered during construction, notify the proper regulatory officials immediately.

This document must not be subject to unauthorized reuse, that is, reusing without our written authorization. Such authorization is essential because it requires us to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications, or other changes to our documents. Therefore, we must be engaged to prepare the necessary clarifications, adjustments, modifications, or other changes before construction activities commence or further activity proceeds. If our scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, we cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies, or other changes necessary to reflect changed field or other conditions.

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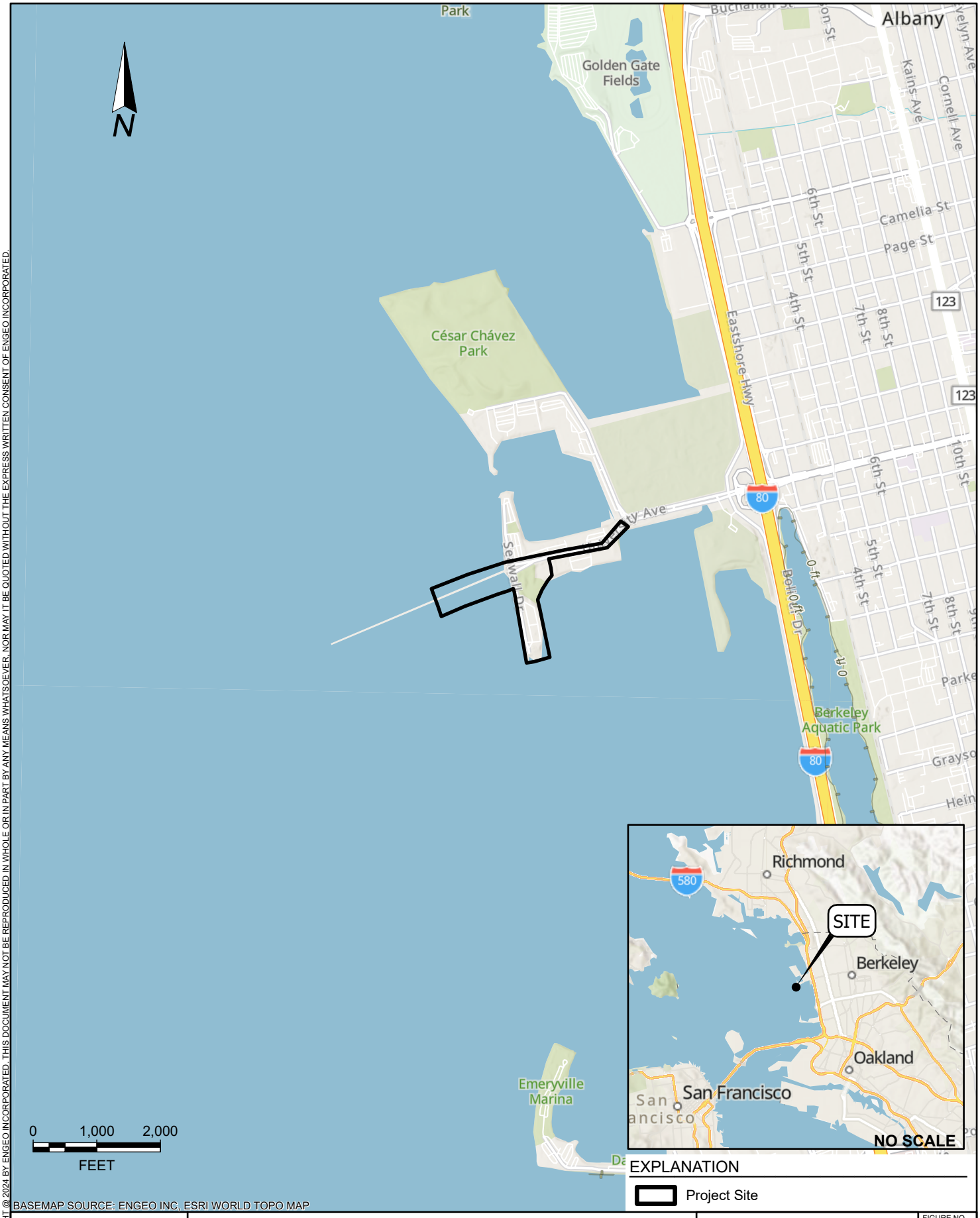
U.S. Geological Survey, San Francisco [map], 1:62500, Topographic Quadrangle Map, San Francisco, California. 1915.

UCSB Aerial Photography Library. FrameFinder



FIGURES

FIGURE 1: Vicinity Map
FIGURE 2: Site Plan
FIGURE 3: Regional Faulting and Seismicity Map
FIGURE 4: Geologic Map
FIGURES 5A THROUGH 5C: Asphalt Core Photographs
FIGURE 6: Cross Sections A-A' and B-B'
FIGURE 7: Seismic Hazard Zone Map
FIGURE 8: Ground Improvement and Retaining Wall Extents
FIGURE 9: Lateral Pressure Diagram for Retaining Wall
FIGURE 10: Ultimate Pile Compression Capacity
FIGURE 11: Ultimate Pile Tension Capacity



BASEMAP SOURCE: ENGEO INC, ESRI WORLD TOPO MAP

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VICINITY MAP

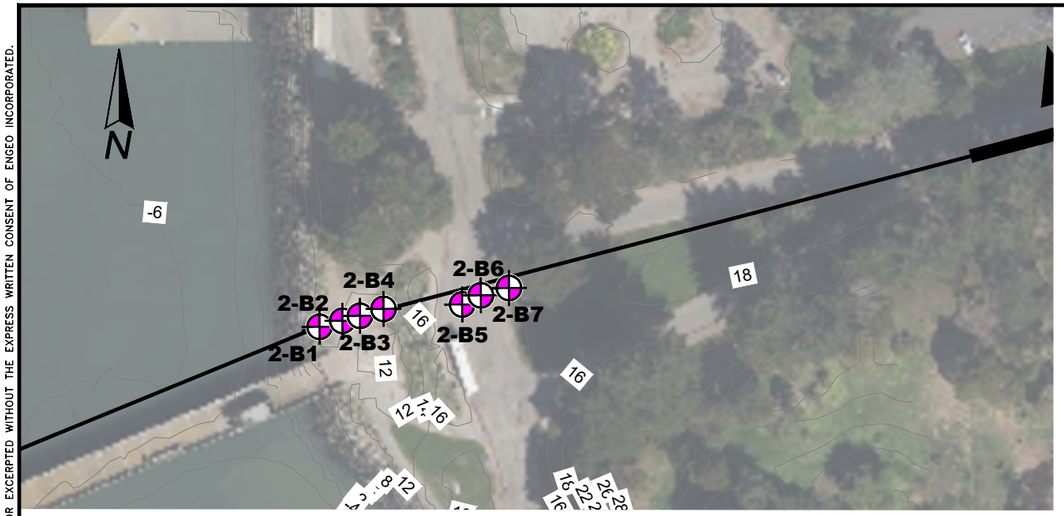
BERKELEY WATER TRANSPORTATION PIER FERRY

BERKELEY, CALIFORNIA

PROJECT NO. :	25022.000.001		FIGURE NO. 1
SCALE:	AS SHOWN		
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INSET 1: SONIC BORINGS

SCALE = 1" 100'

SEE INSET 1: SONIC BORINGS

1-C1
1-C2
1-C3
1-CPT2
A'
1-B3
1-CPT1
1-B2
1-B1
1OS-B1
1OS-CPT1
1OS-CPT2
B
1OS-B2
1OS-B3
1-C4
A
1OS-B5
1OS-B4
B'
1OS-B6
1OS-CPT3
1-CPT3
1-C5
1-CPT4
1-C6
1-B4
1-C9
1-C7
1-C8
1-B5

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- 1-BX — Boring (ENGEO, 2024)
- 1OS-BX — Offshore Boring (ENGEO, 2024)
- 1-CX — Cores/Shallow Boring (ENGEO, 2024)
- 2-BX — Sonic Boring (ENGEO, 2025)
- 1-CPTX — Cone Penetration Test (ENGEO, 2024)
- 1OS-CPTX — Offshore Cone Penetration Test (ENGEO, 2024)

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE AND LIDAR

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FEET

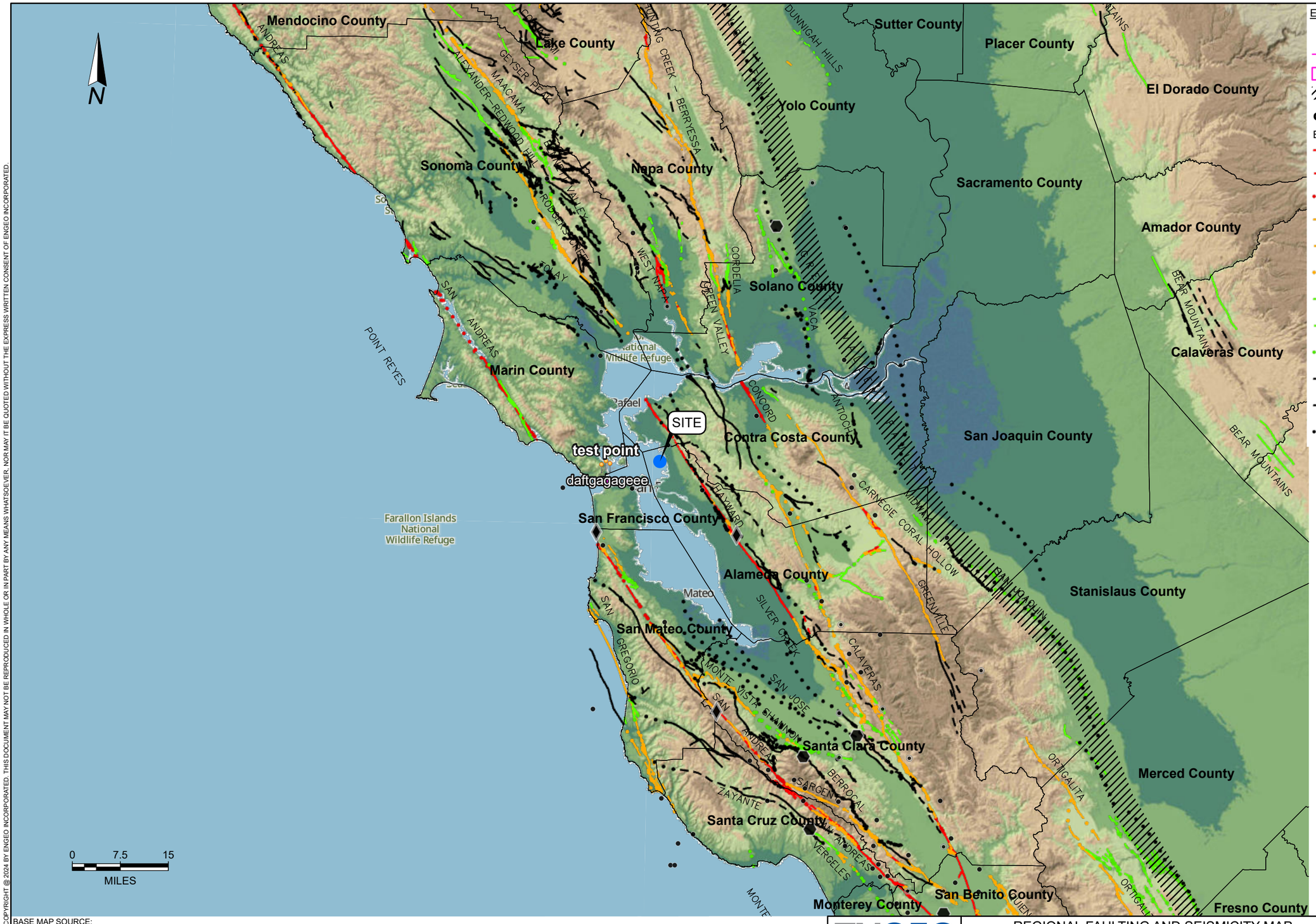
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SITE PLAN
BERKELEY WATER TRANSPORTATION PIER FERRY
BERKELEY, CALIFORNIA

PROJECT NO.: 25022.000.001	FIGURE NO. 2
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EXPLANATION

- Markup Points
- Markups Lines
- Markup Shapes
- Historic Blind Thrust Fault Zone

QUATERNARY FAULTS 2020

Based on time of most recent surface deformation

- Historic (< 150 years), well constrained location
- Historic (< 150 years), moderately constrained location
- Historic (< 150 years), inferred location
- Latest Quaternary (<15,000 years), well constrained location
- Latest Quaternary (<15,000 years), moderately constrained location
- Latest Quaternary (<15,000 years), inferred location
- Late Quaternary (< 130,000 years), well constrained location
- Late Quaternary (< 130,000 years), moderately contrained location
- Late Quaternary (< 130,000 years), inferred location
- Undifferentiated Quaternary (< 1.6 million years), well constrained location
- Undifferentiated Quaternary (< 1.6 million years), moderately constrained location
- Undifferentiated Quaternary (< 1.6 million years), inferred location

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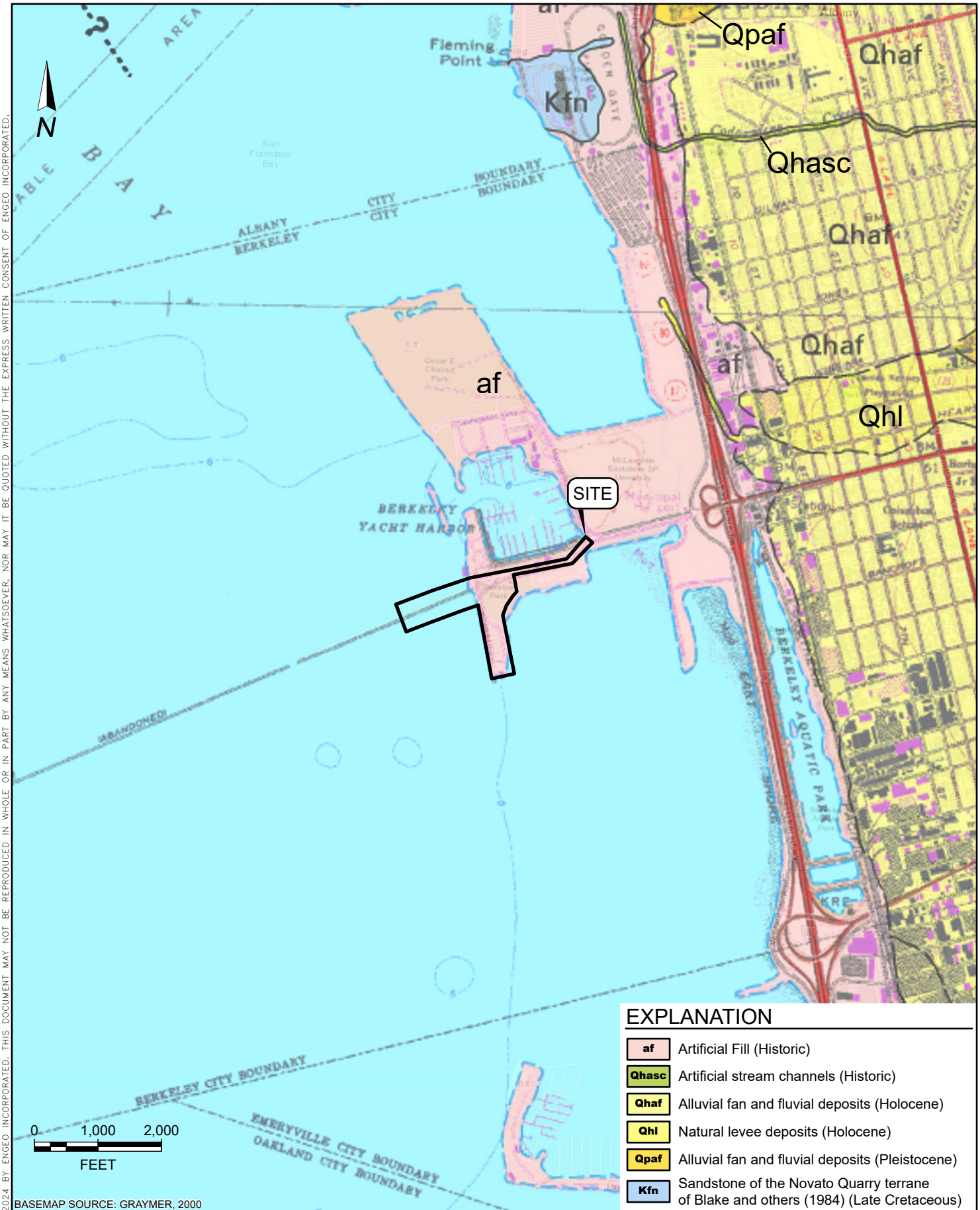
BASE MAP SOURCE:
ENGEO
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2020
C.G.S. HISTORIC EARTHQUAKE DATABASE

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REGIONAL FAULTING AND SEISMICITY MAP
BERKELEY WATER TRANSPORTATION PIER FERRY
BERKELEY, CALIFORNIA

PROJECT NO. : 25022.000.001		FIGURE NO. 3
SCALE: AS SHOWN		
DRAWN BY: VZ	CHECKED BY: JF	

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EXPLANATION

af	Artificial Fill (Historic)
Qhasc	Artificial stream channels (Historic)
Qhaf	Alluvial fan and fluvial deposits (Holocene)
Qhl	Natural levee deposits (Holocene)
Qpaf	Alluvial fan and fluvial deposits (Pleistocene)
Kfn	Sandstone of the Novato Quarry terrane of Blake and others (1984) (Late Cretaceous)

BASEMAP SOURCE: GRAYMER, 2000

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GEOLOGIC MAP

BERKELEY WATER TRANSPORTATION PIER FERRY

BERKELEY, CALIFORNIA

PROJECT NO. :	25022.000.001
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FIGURE NO. 4

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LAYOUT: 03. GEOLOGIC 8.5 X 11 PORTRAIT USER: NCLOUGH

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1-B2



1-B3



1-B5



1-C1



1-C2



1-C3

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PROJECT NUMBER: **25022.000.001**		FIGURE NO. **5A**
SCALE: **NO SCALE**		
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1-C4



1-C5



1-C6



1-C7



1-C8



1-C9

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BERKELEY, CALIFORNIA

PROJECT NUMBER: **25022.000.001**		FIGURE NO. **5B**
SCALE: **NO SCALE**		
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1-CPT2



1-CPT3



1-CPT4

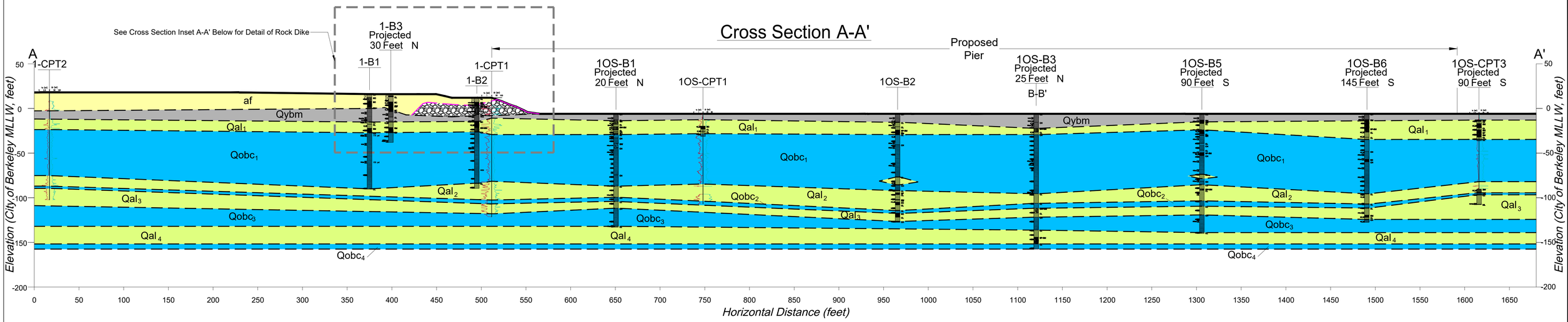
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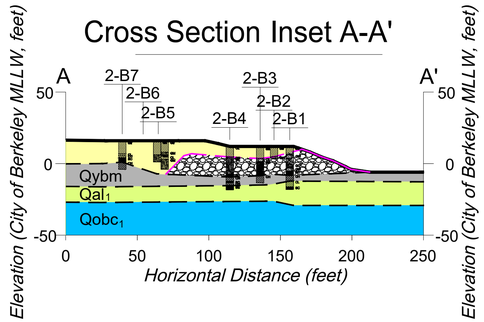
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BERKELEY, CALIFORNIA

PROJECT NUMBER: **25022.000.001**		FIGURE NO. **5C**
SCALE: **NO SCALE**		
DRAWN BY: **NWC**	CHECKED BY: **JF**	

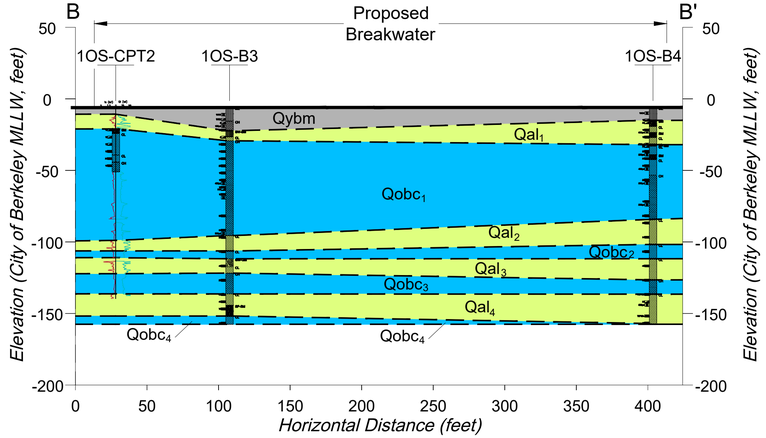
Cross Section A-A'



Cross Section Inset A-A'



Cross Section B-B'



Legend

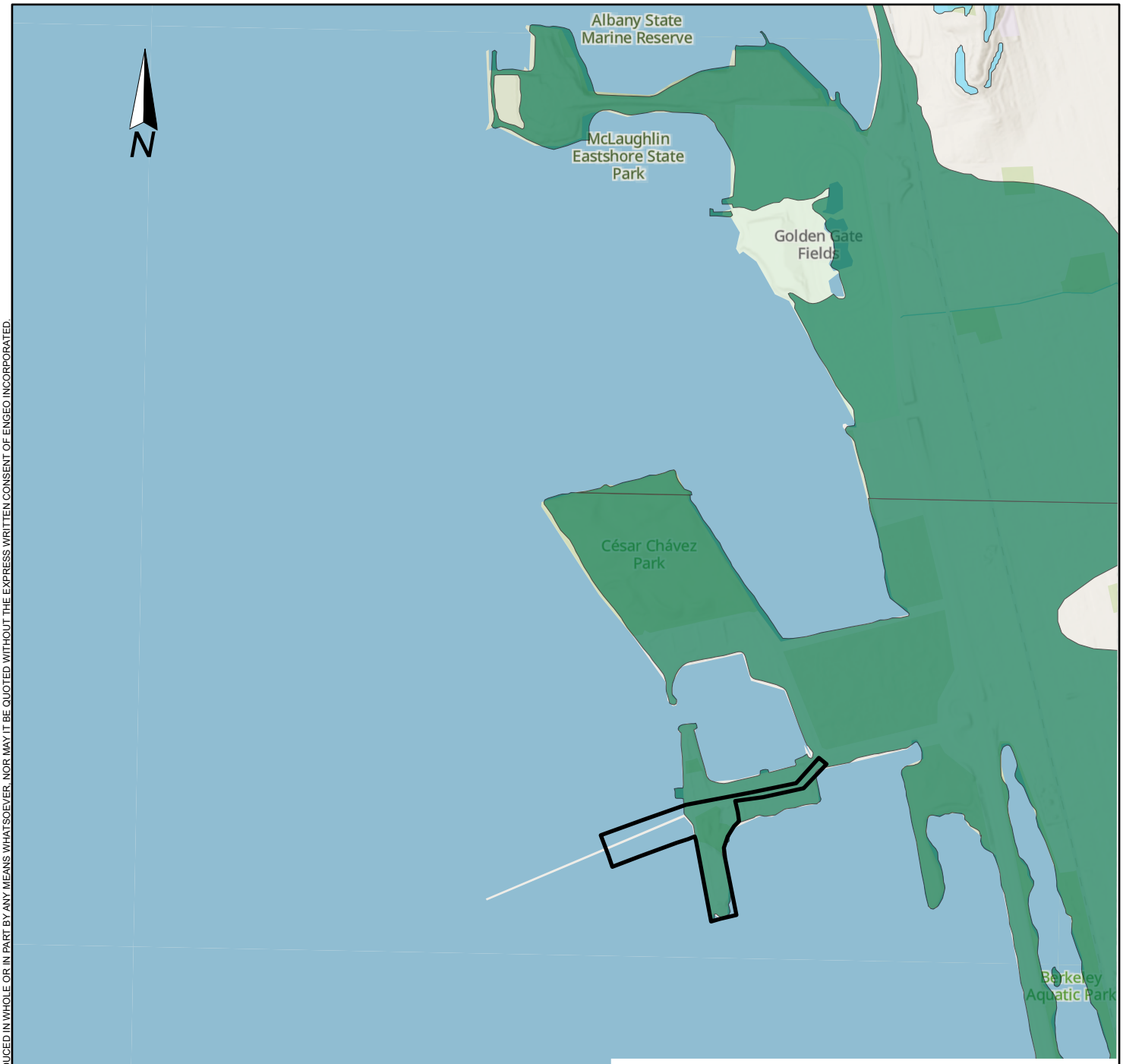
af	Artificial Fill
Qybm	Young Bay Mud (Holocene)
Qal	Alluvium (Pleistocene and/or Holocene), Number Indicates Layer Order
Qobc	Old Bay Clay/Old Bay Deposits (Pleistocene), Number Indicates Layer Order
1-BX	Boring (ENGEO, 2024)
1OS-BX	Off-Shore Boring (ENGEO, 2024)
1-CPTX	Cone Penetration Test (ENGEO, 2024)
1OS-CPTX	Off Shore Cone Penetration Test (ENGEO, 2024)
2-BX	Boring (ENGEO, 2025)

———	Existing Ground Surface
– – – – – ?– –? –	Geologic Contact, dashed where approximate queried where inferred
	Rock Dike

Disclaimer: Cross Section Is For Illustration Purposes Only. The Transition Between Materials May Be Abrupt Or Gradual. Variations Should Be Expected.

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ENGEO Expect Excellence	CROSS SECTIONS A-A' AND B-B' BERKELEY WATER TRANSPORTATION PIER FERRY BERKELEY, CALIFORNIA	PROJECT NO.: 25022.000.001 SCALE: AS SHOWN DRAWN BY: LL / CHECKED BY: JF	FIGURE NO. 6



EXPLANATION

Project Site

Liquefaction Zones
Areas Where The Historical Occurrence Of Liquefaction, Or Local Geological, Geotechnical And Groundwater Conditions Indicate A Potential For Permanent Ground Displacements Such That Mitigation As Defined In Public Resources Code Section 2693(C) Would Be Required

Earthquake-Induced Landslide Zones
Areas Where The Previous Occurrence Of Landslide Movement, Or Local Topographic, Geological, Geotechnical And Subsurface Water Conditions Indicate A Potential For Permanent Ground Displacements Such That Mitigation As Defined In Public Resources Code Section 2693(C) Would Be Required.

BASEMAP SOURCE: ESRI MAPPING SERVICE
CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY

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SEISMIC HAZARD ZONE MAP

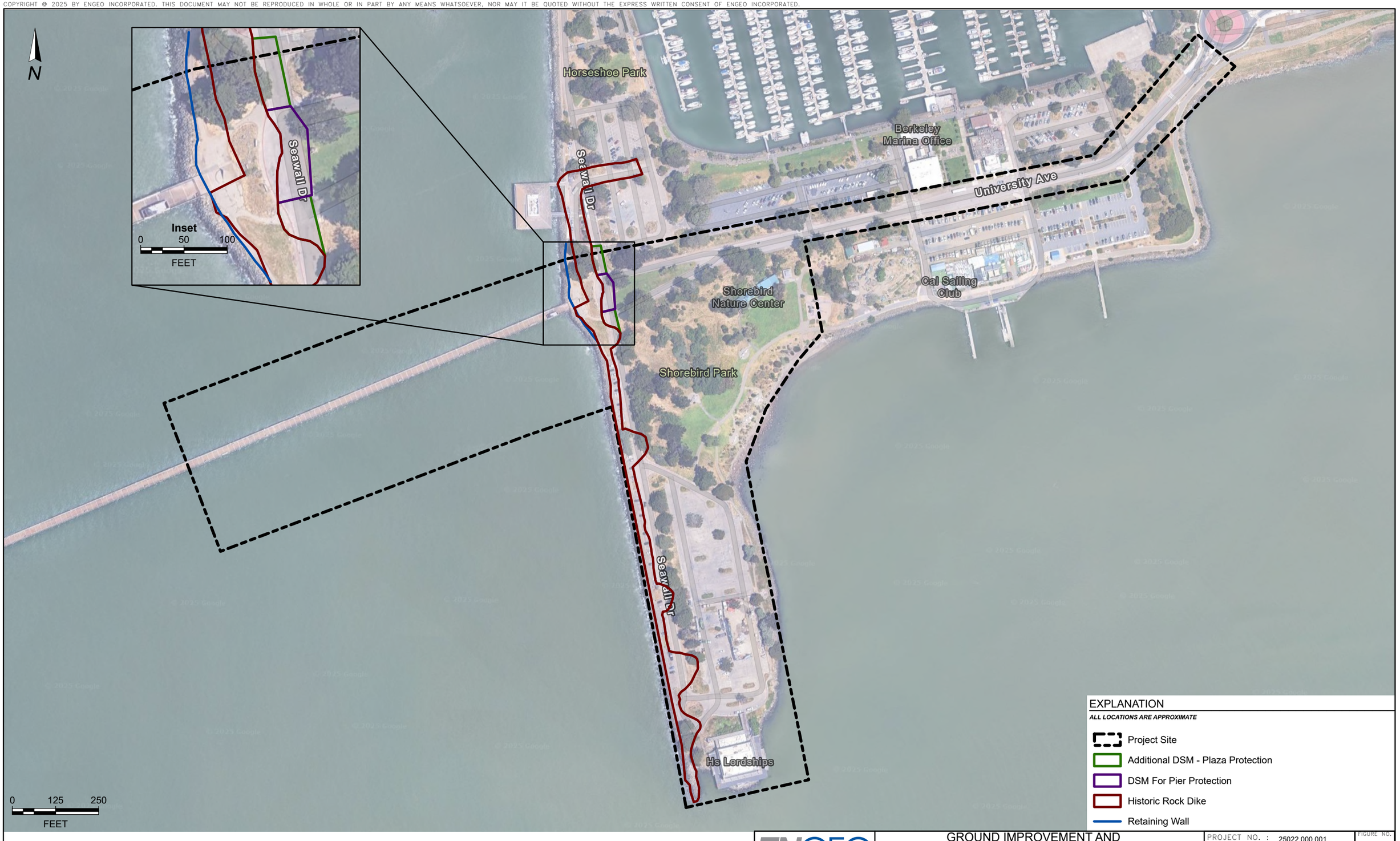
BERKELEY WATER TRANSPORTATION PIER FERRY

BERKELEY, CALIFORNIA

PROJECT NO. : 25022.000.001		FIGURE NO. 7
SCALE: AS SHOWN		
DRAWN BY: VZ	CHECKED BY: JF	

ORIGINAL FIGURE PRINTED IN COLOR

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EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- Project Site
- Additional DSM - Plaza Protection
- DSM For Pier Protection
- Historic Rock Dike
- Retaining Wall

BASEMAP SOURCE: GOOGLE EARTH MAPPING SERVICE 2022

ENGEO Expect Excellence

GROUND IMPROVEMENT AND RETAINING WALL EXTENTS

BERKELEY WATER TRANSPORTATION PIER FERRY
BERKELEY, CALIFORNIA

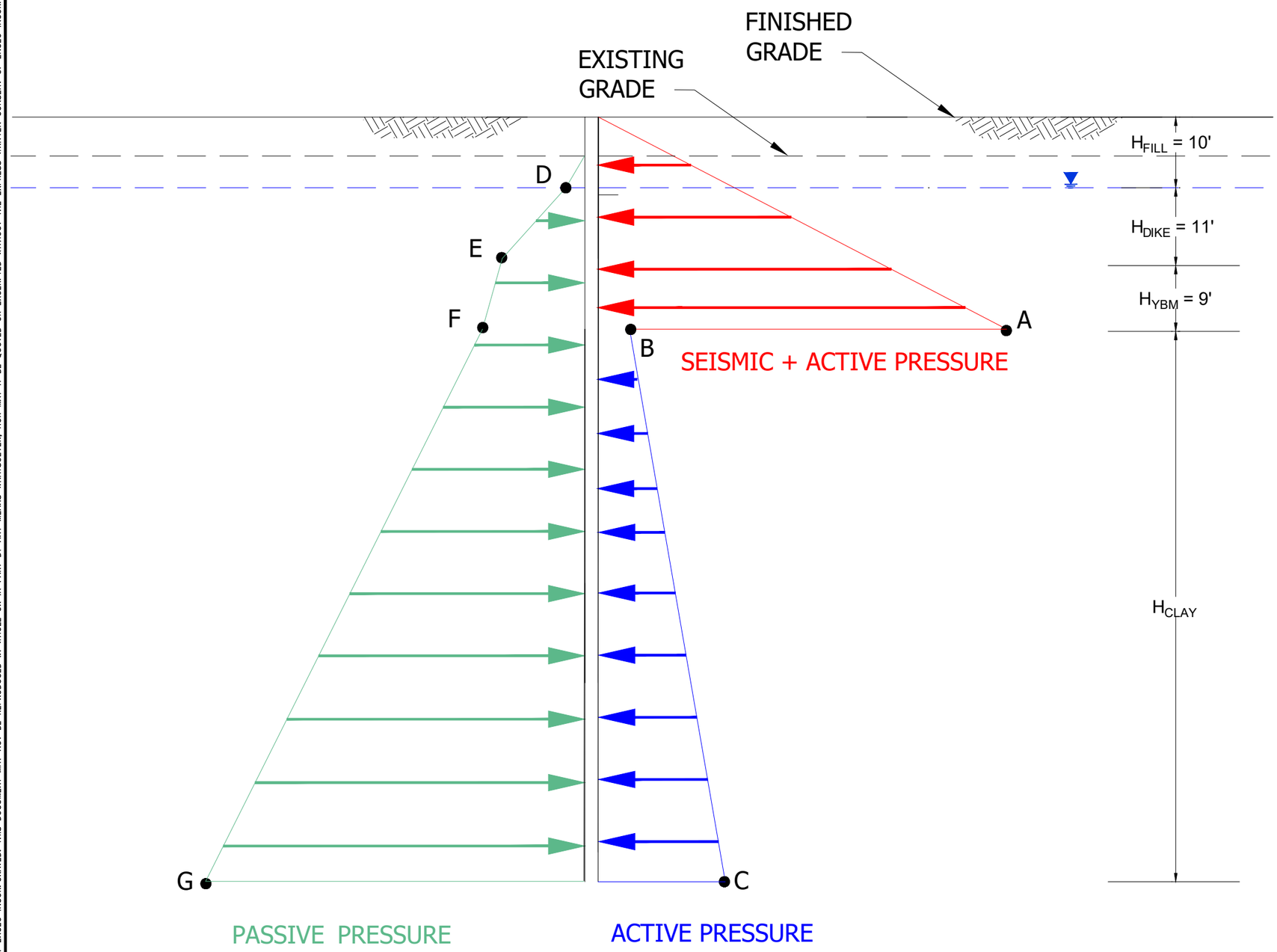
PROJECT NO. :	25022.000.001
SCALE:	AS SHOWN
DRAWN BY: NWC	CHECKED BY: JF

FIGURE NO. 8

PATH: G:\DRAFTING\PROJECTS_24000 TO 25999\25022\GEX\25022000001_BERKELEYWATERTRANSPORTATIONPIERFERRY – GEX.APRX
LAYOUT: 08. EXTENTS OF GROUND IMPROVEMENT 11 X 17 LANDSCAPE USER: NLAMOTTEKERR

ORIGINAL FIGURE PRINTED IN COLOR

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NOTES:

1. LATERAL EARTH PRESSURES SHOWN ARE FOR SEISMIC LOADING CONDITIONS ON THE RESTRAINED WALL.
2. PRESSURE DIAGRAM CONSIDERS HYDROSTATIC CONDITIONS, AND DOES NOT INCLUDE ADDITIONAL INFLUENCE OF CONFINED OR PARTIALLY CONFINED AQUIFERS THAT MAY BE PRESENT ON-SITE.
3. DEPTH TO GROUNDWATER IS 10 FEET BELOW FINISHED GRADE.
4. PASSIVE PRESSURE OF FILL AND DIKE IS DERIVED ASSUMING SLOPED CONDITIONS ALONG THE SHORELINE.

LATERAL EARTH PRESSURE PARAMETERS			
Layer	SEISMIC + ACTIVE EFPs [CLE/MCE] (pcf)	ACTIVE EFP_a (pcf)	PASSIVE EFP_p (pcf)
Fill	170 / 340	N/A	95
Dike	170 / 340	N/A	145
Young Bay Mud (YBM)	170 / 340	N/A	55
Clay	N/A	30	140

PRESSURE POINT CALCULATION (PSF)

Point A: $\sigma_A = 30 * EFP_s$

Point B: $\sigma_B = 850$

Point C: $\sigma_c = \sigma_B + (H_{CLAY} * EFP_{a,clay})$

Point D: $\sigma_D = 475$

Point E: $\sigma_E = 2000$

Point F: $\sigma_F = 2500$

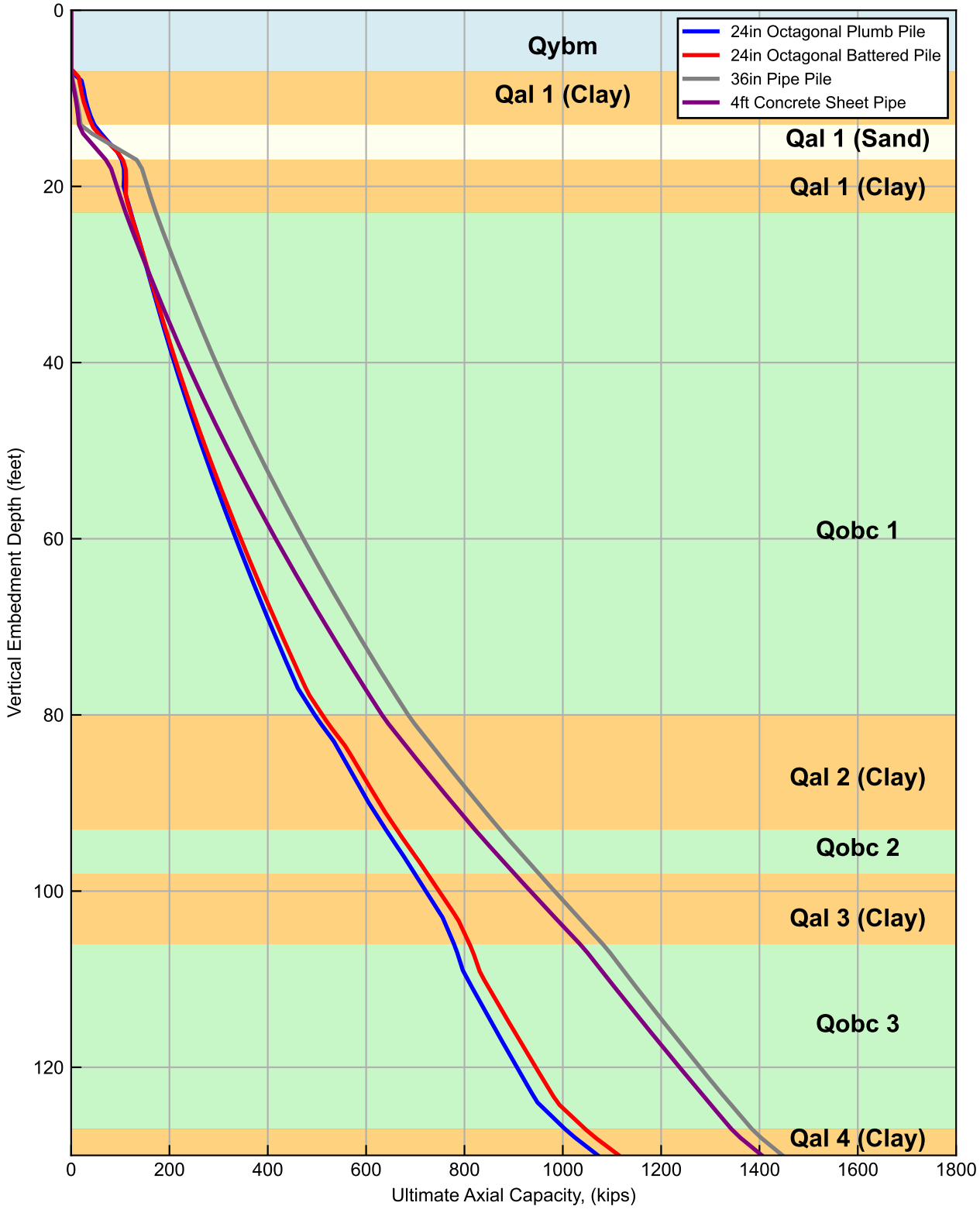
Point G: $\sigma_G = \sigma_F + [(H_{CLAY}) * EFP_{p,clay})]$

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LATERAL PRESSURE DIAGRAM
FOR RETAINING WALL
BERKELEY WATER TRANSPORTATION PIER FERRY
BERKELEY, CALIFORNIA

PROJECT NO.: 25022.000.001
SCALE: NO SCALE
DRAWN BY: VZ
CHECKED BY: JF
FIGURE NO. 9

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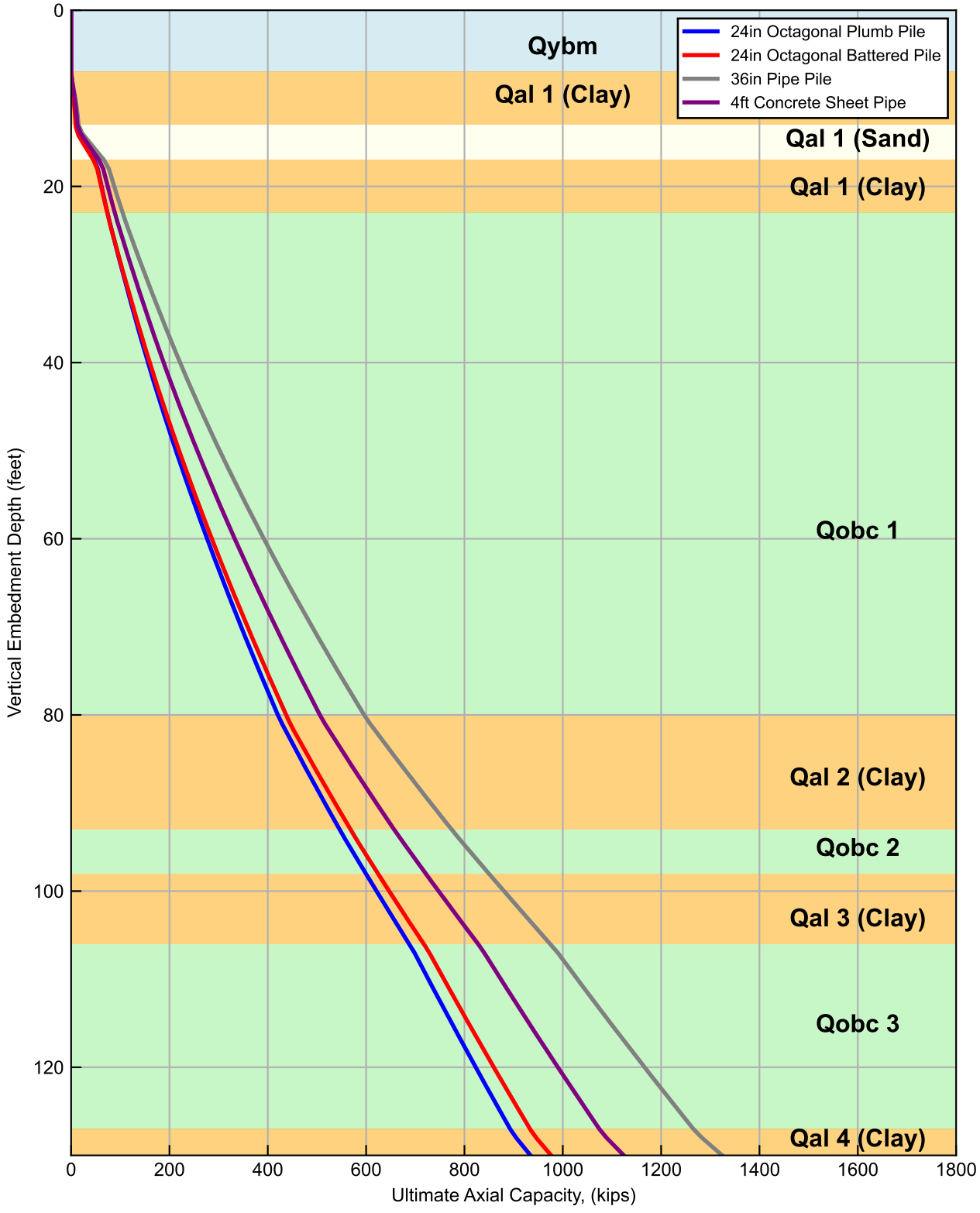
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Ultimate Pile Compression Capacity
Berkeley Water Transportation Pier Ferry
BERKELEY, CALIFORNIA

PROJECT NO:	25022.000.001
SCALE:	AS SHOWN
DRAWN BY:	VZ
CHECKED BY:	JF

FIGURE NO. 10

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ENGEO Expect Excellence

Ultimate Pile Tension Capacity
Berkeley Water Transportation Pier Ferry
BERKELEY, CALIFORNIA

PROJECT NO:	25022.000.001
SCALE:	AS SHOWN
DRAWN BY:	VZ
CHECKED BY:	JF

FIGURE NO. 11



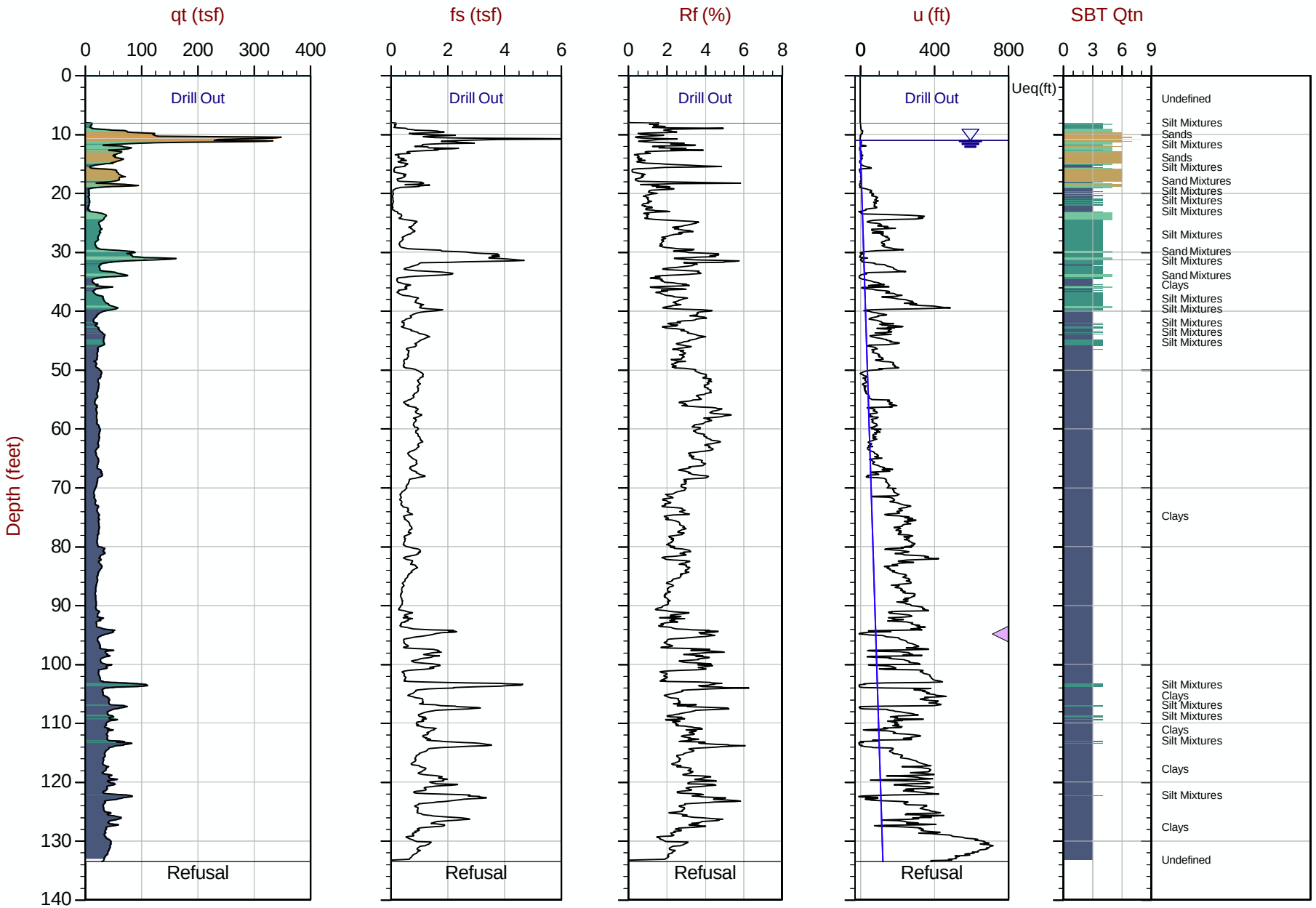
APPENDIX A

CONE PENETRATION TEST LOGS

ENGEO, Inc.

Job No: 24-56-28254
Date: 2024-08-27 15:22
Site: Berkeley Marina

Sounding: 1-CPT01
Cone: 813:T1500F15U35 Area=15 cm²



Max Depth: 40.700 m / 133.53 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-28254_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4190831m E: 560023m

Overplot Item: Ueq, Assumed Ueq, Dissipation, Ueq achieved, Dissipation, Ueq not achieved, Dissipation, Ueq assumed, Hydrostatic Line

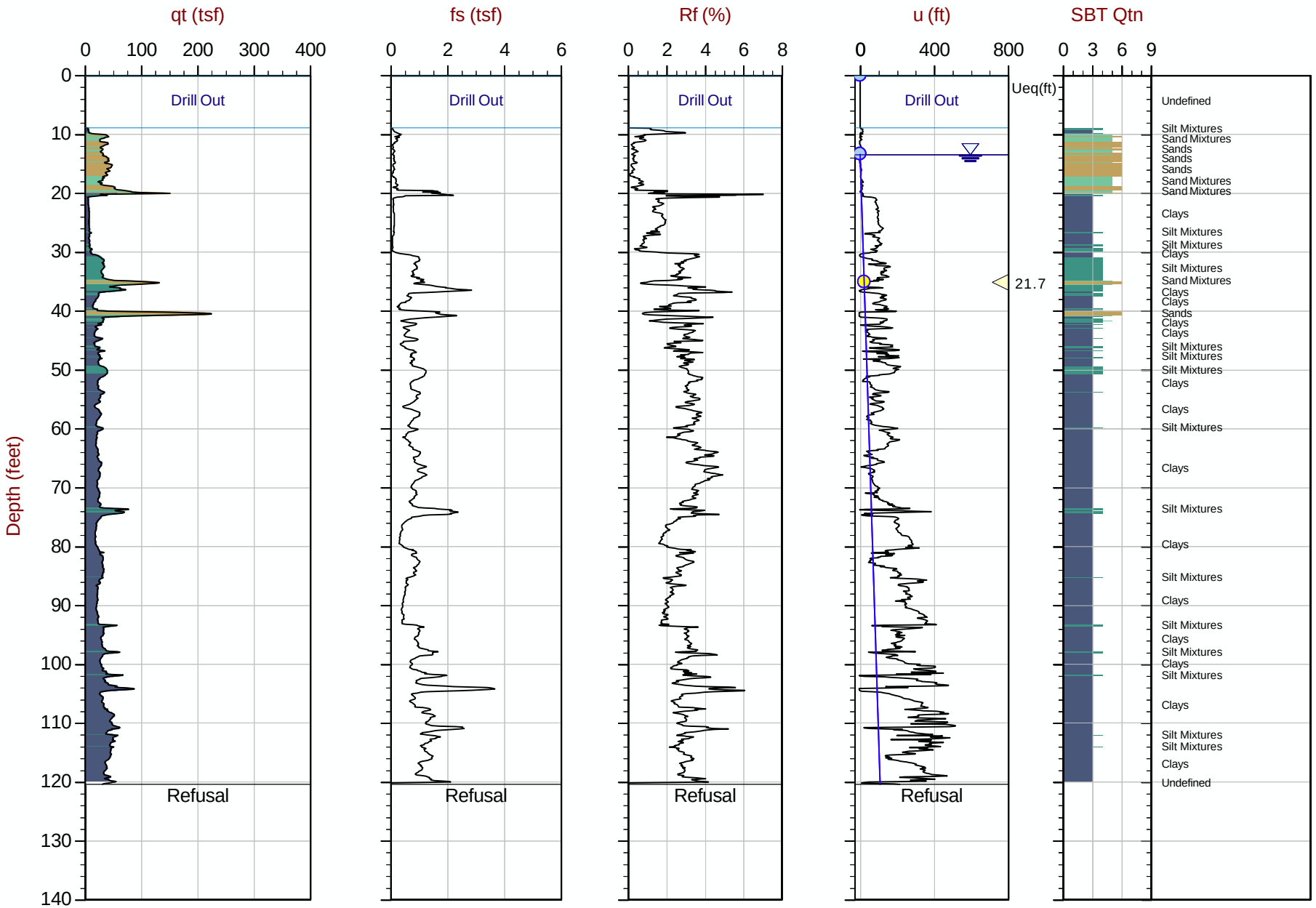
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO, Inc.

Job No: 24-56-28254
Date: 2024-08-27 13:42
Site: Berkeley Marina

Sounding: 1-CPT02
Cone: 813:T1500F15U35 Area=15 cm²



Max Depth: 36.700 m / 120.41 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-28254_CP02B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4190873m E: 560133m

Overplot Item: Ueq | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Dissipation, Ueq assumed | Hydrostatic Line

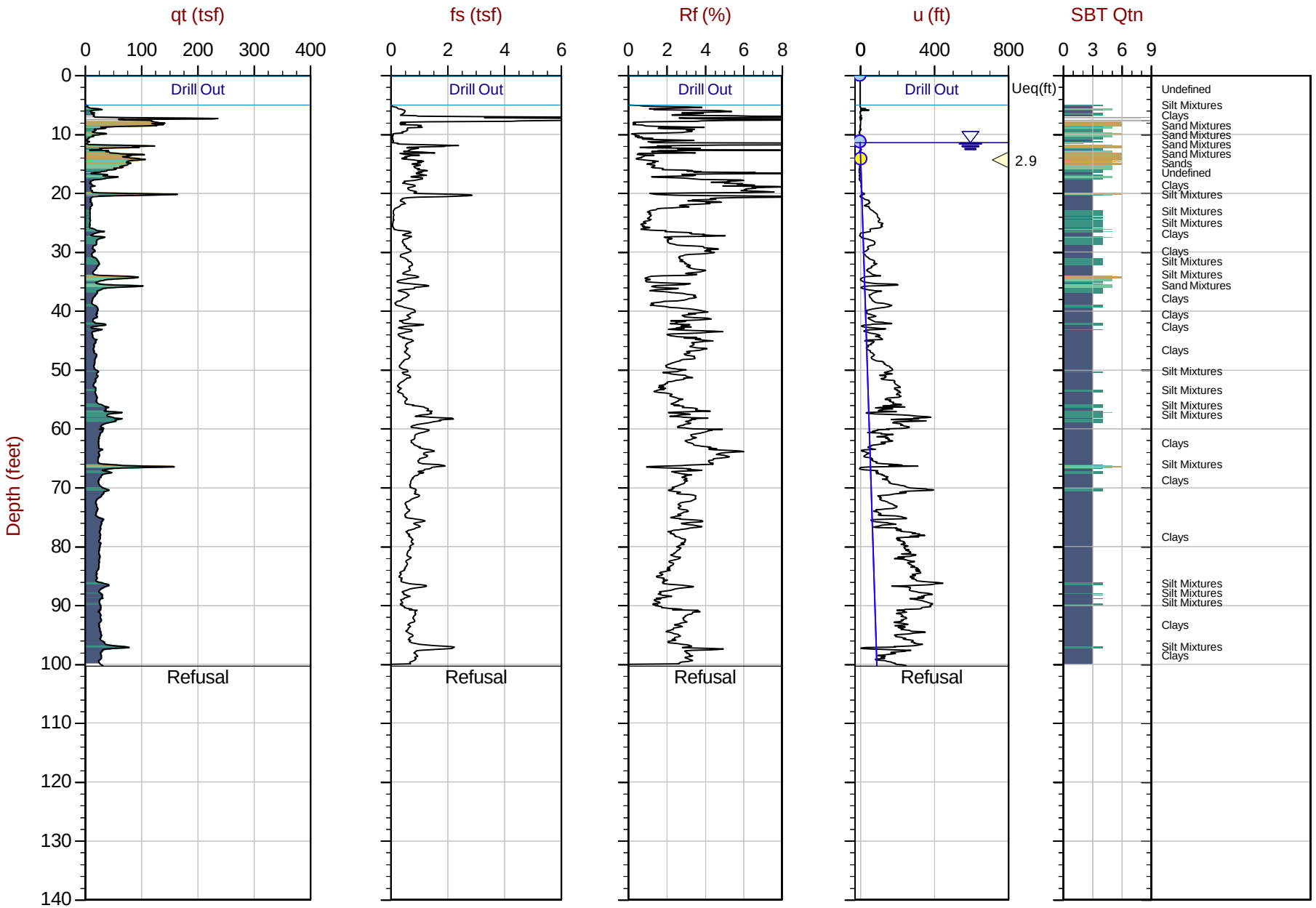
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO, Inc.

Job No: 24-56-28254
Date: 2024-08-27 09:00
Site: Berkeley Marina

Sounding: 1-CPT03
Cone: 813:T1500F15U35 Area=15 cm²



Max Depth: 30.575 m / 100.31 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-28254_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4190667m E: 560090m

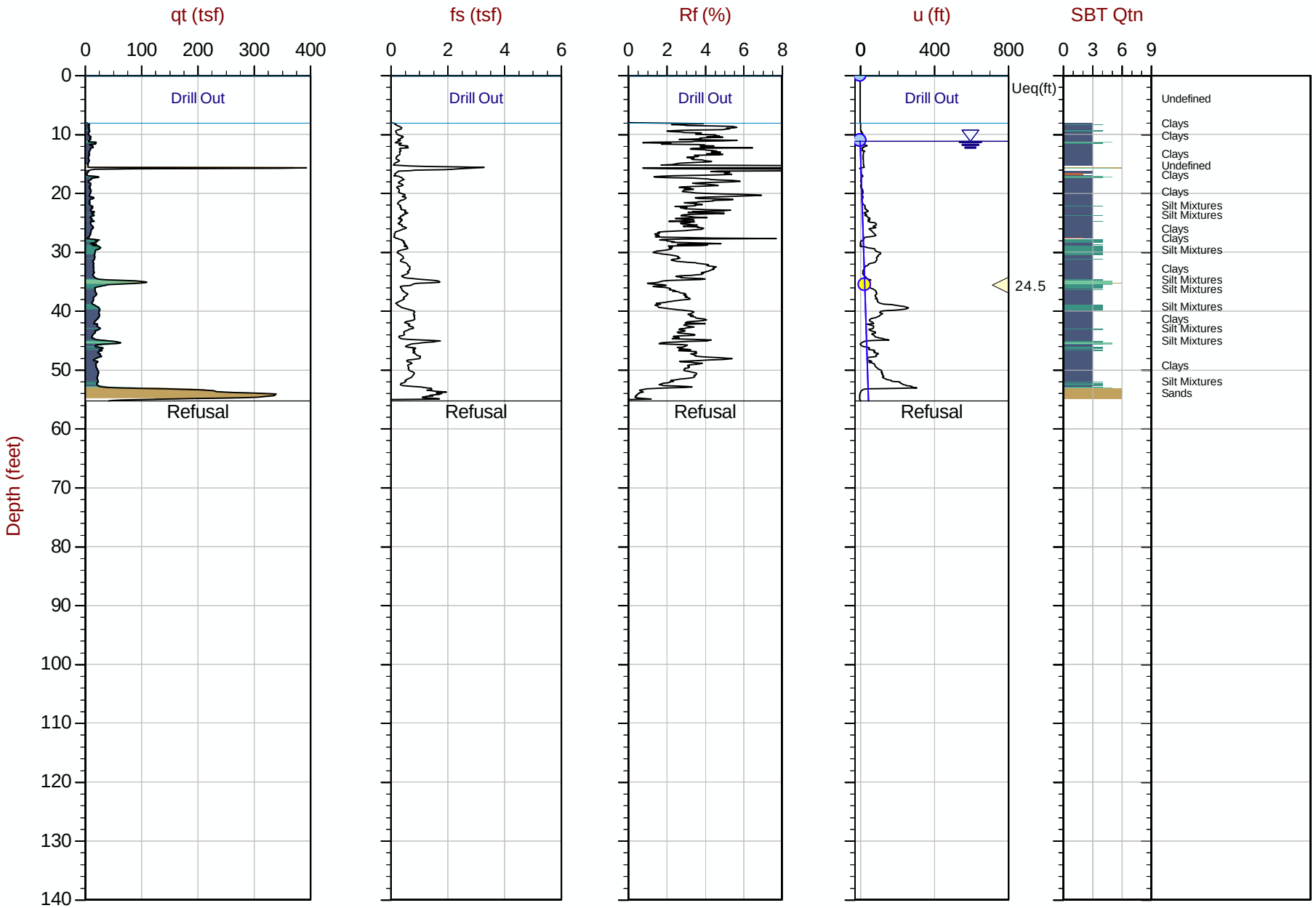
Overplot Item: Ueq, Assumed Ueq, Dissipation, Ueq achieved, Dissipation, Ueq not achieved, Dissipation, Ueq assumed, Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ENGEO, Inc.

Job No: 24-56-28254
Date: 2024-08-29 10:54
Site: Berkeley Marina

Sounding: 1-CPT4
Cone: 813:T1500F15U35 Area=15 cm²



Max Depth: 16.850 m / 55.28 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-28254_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4190842m E: 560337m

Overplot Item: Ueq, Assumed Ueq, Dissipation, Ueq achieved, Dissipation, Ueq not achieved, Dissipation, Ueq assumed, Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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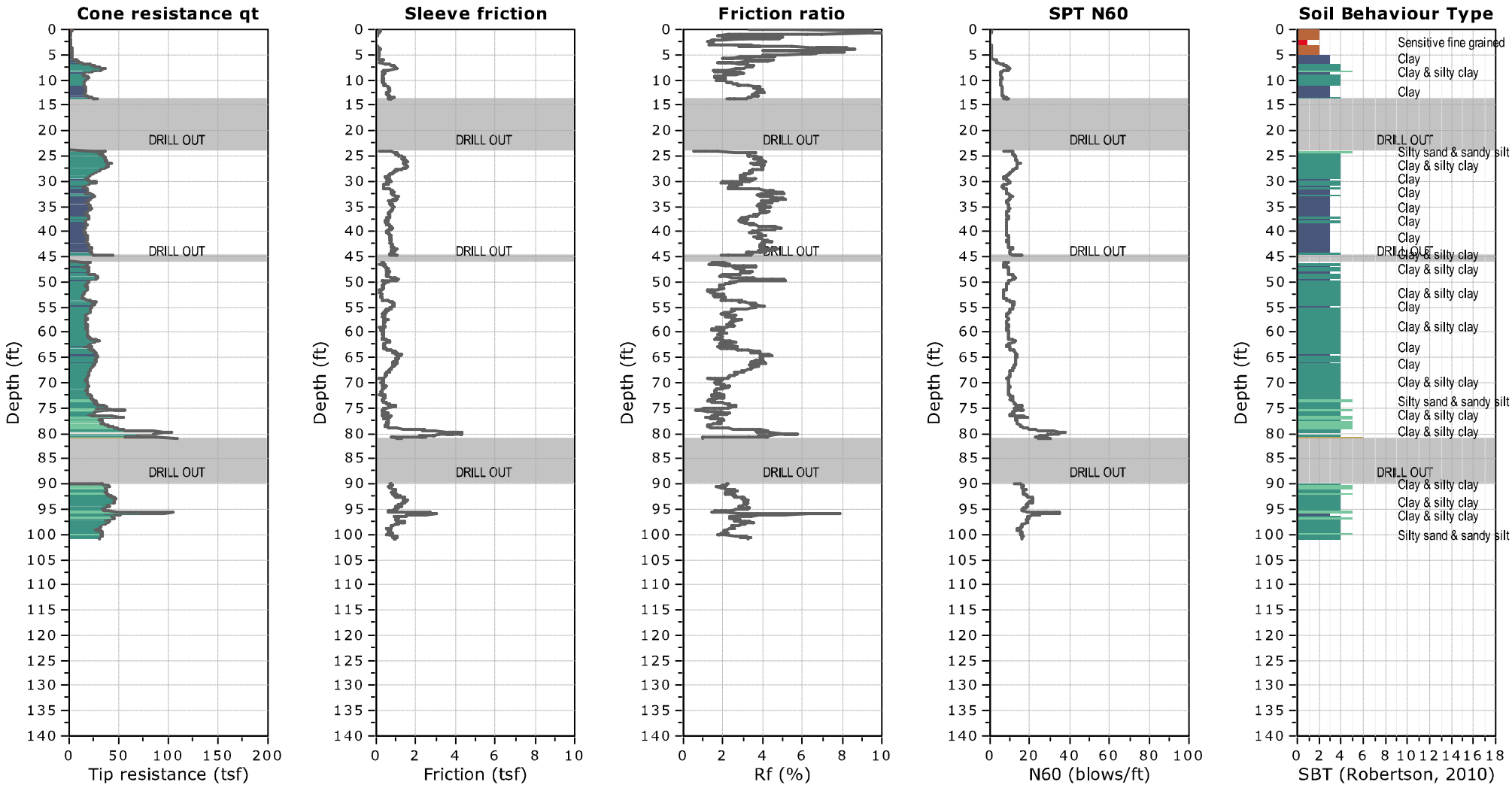
CPT: 1OS-CPT1

CLIENT: ENGEO

SITE: BERKELEY PIER, BERKELEY, CA

FIELD REP: Vlad Z
Cone ID: 231229

Total depth: 100.92 ft, Date: 9/09/2024



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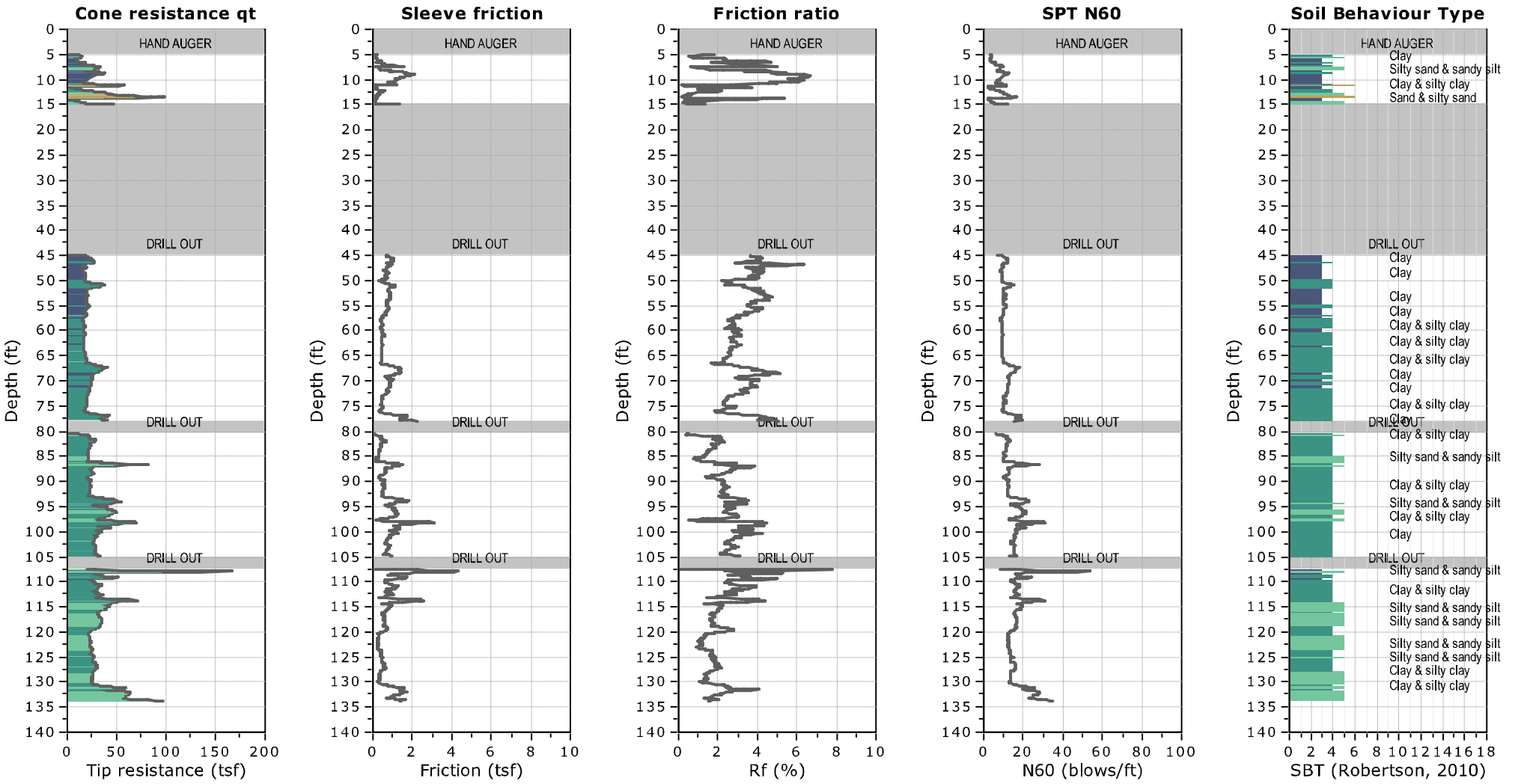
CPT: 1OS-CPT2

CLIENT: ENGEO

SITE: BERKELEY PIER, BERKELEY, CA

FIELD REP: Vlad Z
Cone ID: 231229

Total depth: 133.66 ft, Date: 9/05/2024



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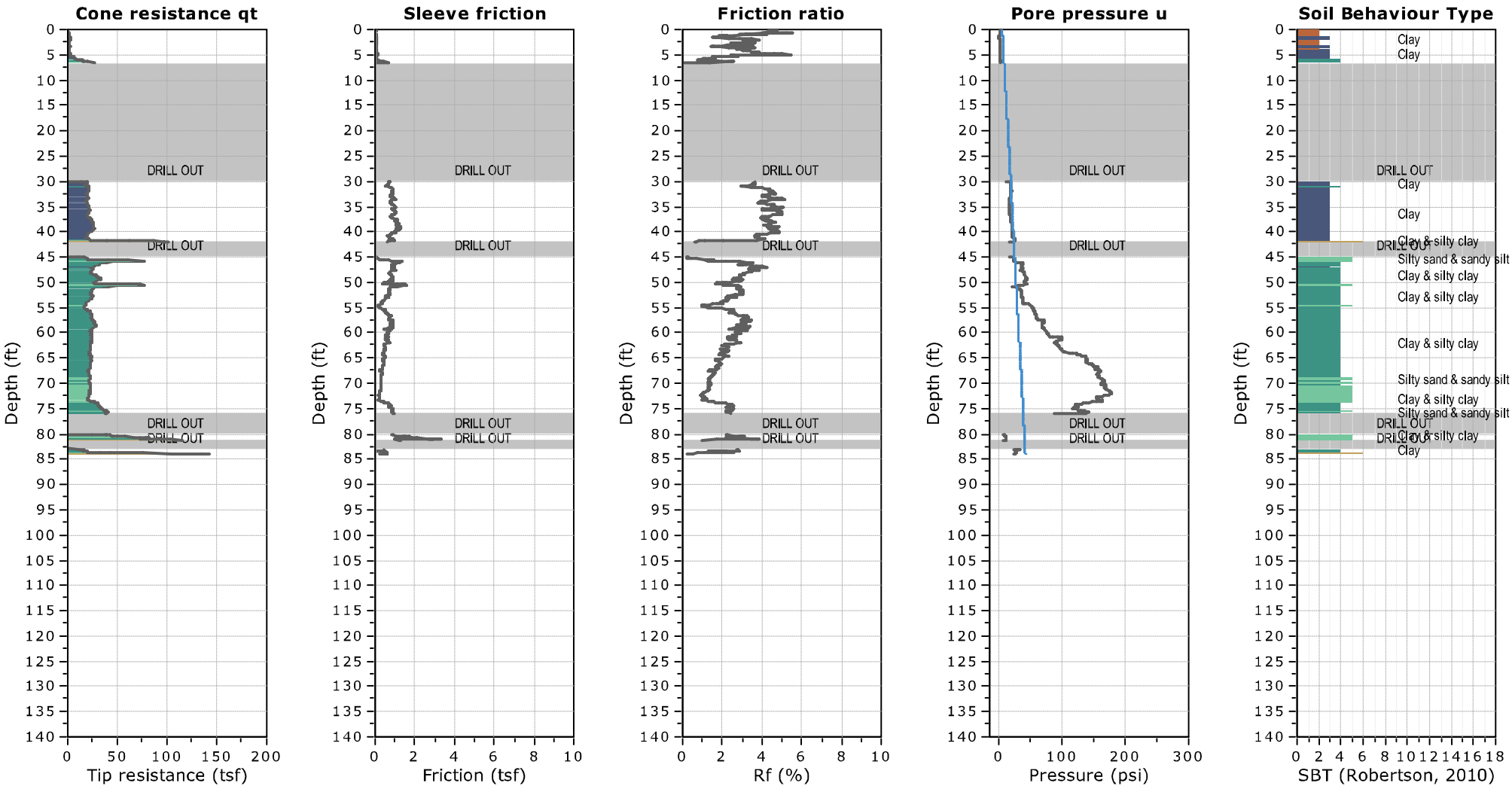
CPT: 1OS-CPT3

CLIENT: ENGEO

SITE: BERKELEY PIER, BERKELEY, CA

FIELD REP: Vlad Z
Cone ID: 231229

Total depth: 83.99 ft, Date: 9/10/2024

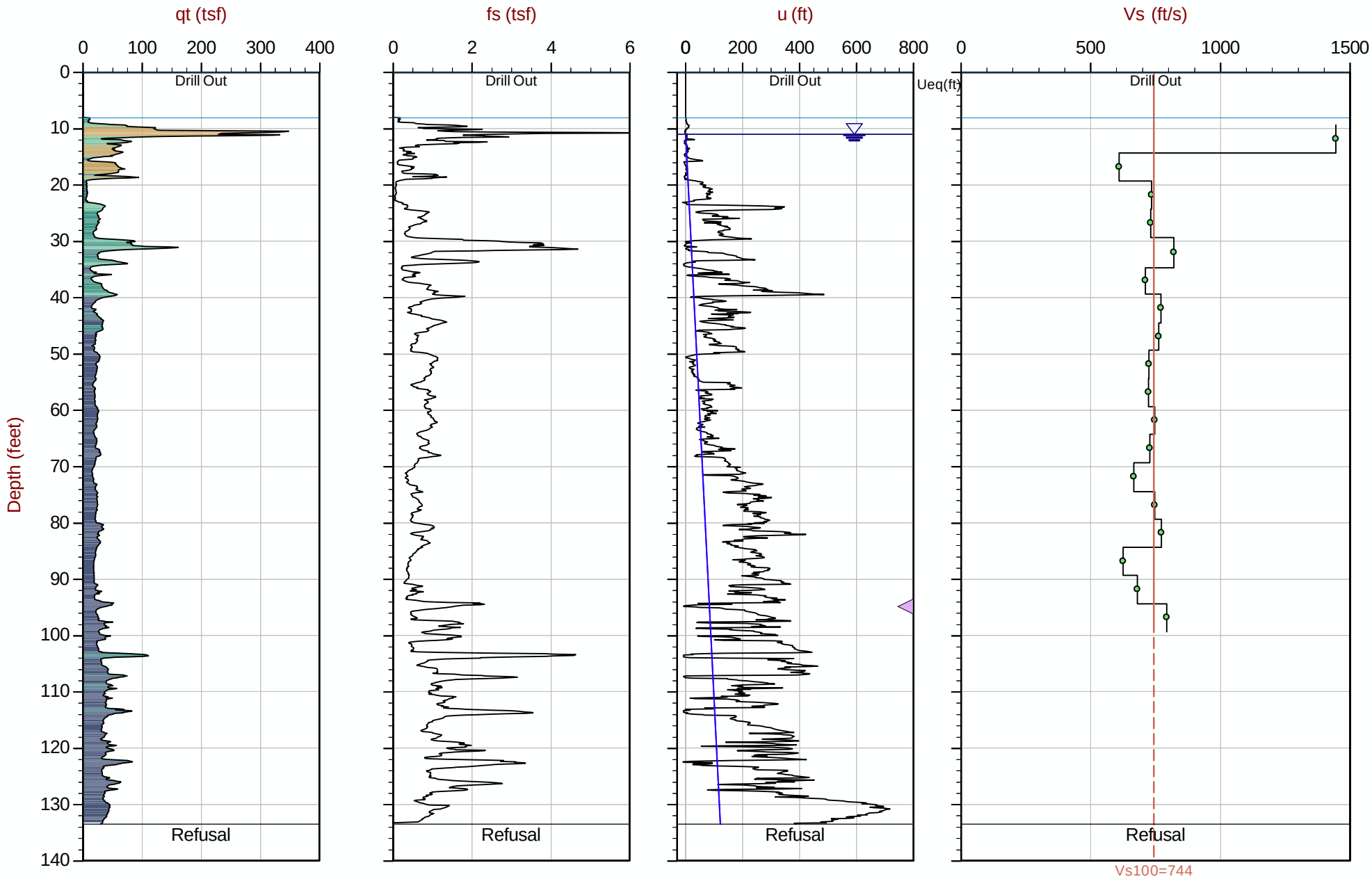


CONETEC

ENGEO, Inc.

Job No: 24-56-28254
Date: 2024-08-27 15:22
Site: Berkeley Marina

Sounding: 1-CPT-01
Cone: 813:T1500F15U35 Area=15 cm²



Max Depth: 40.700 m / 133.53 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-56-28254_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4190831m E: 560023m

Overplot Item: Ueq | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Dissipation, Ueq assumed | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



APPENDIX B

CONE PENETRATION LIQUEFACTION CALCULATION

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

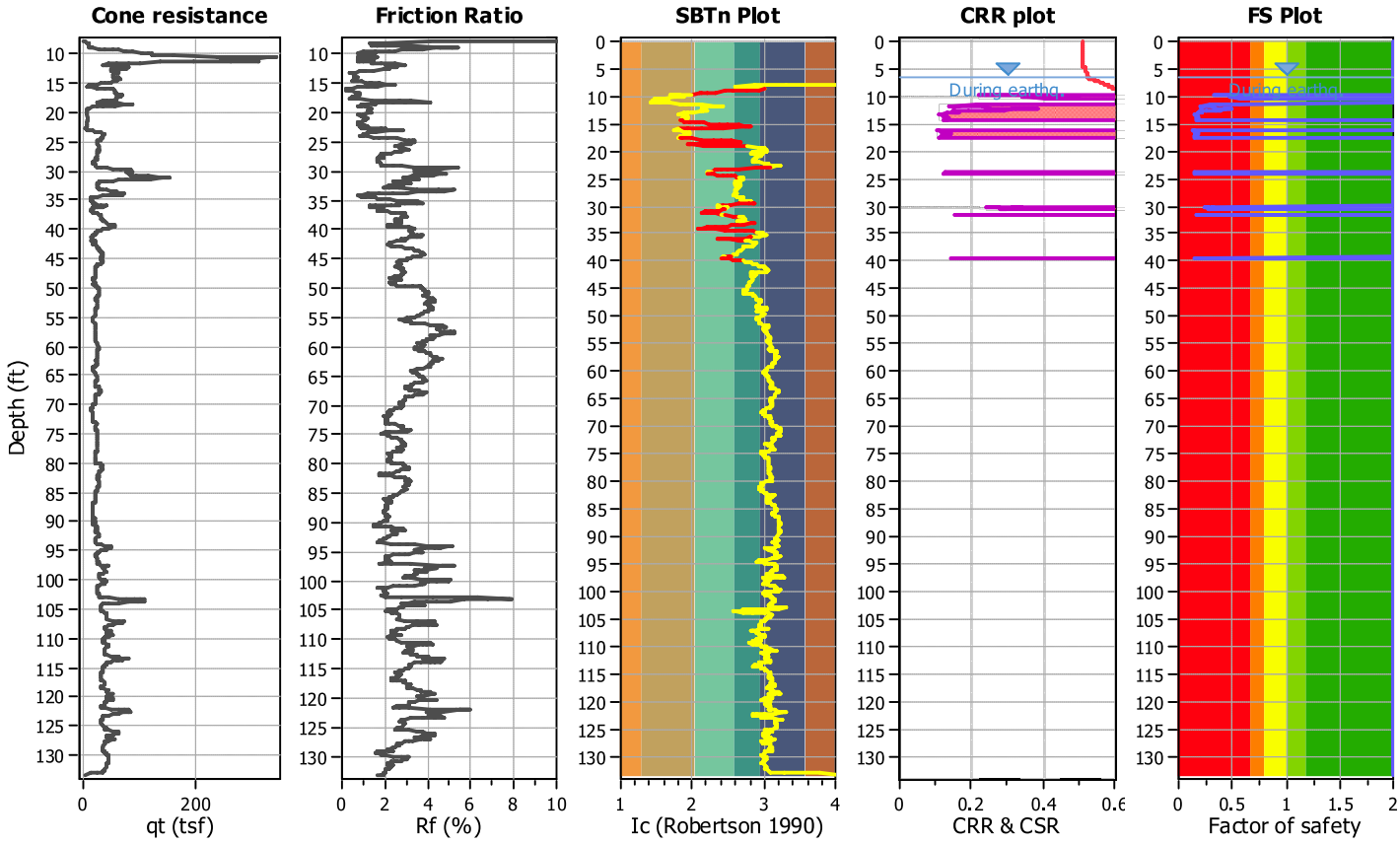
LIQUEFACTION ANALYSIS REPORT

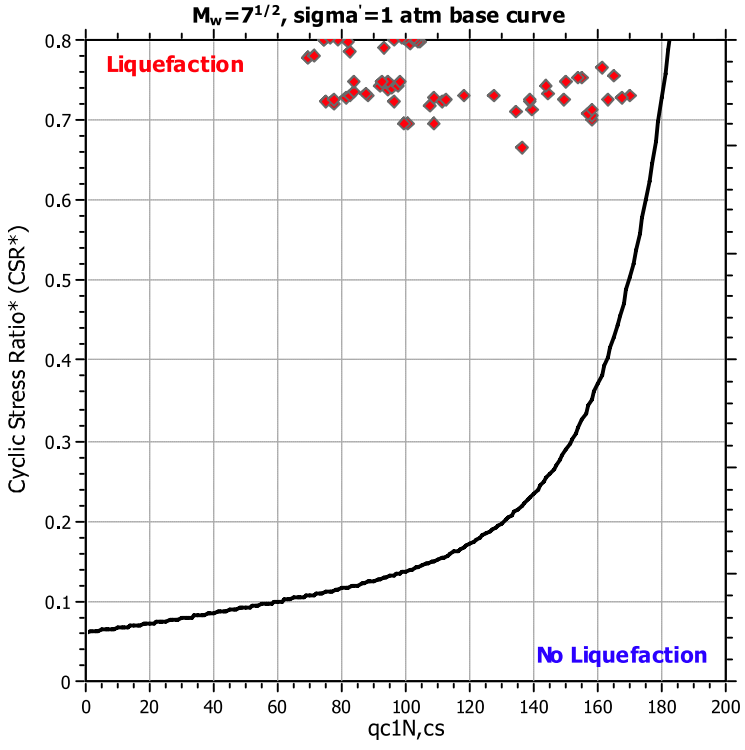
Project title : **Location :**

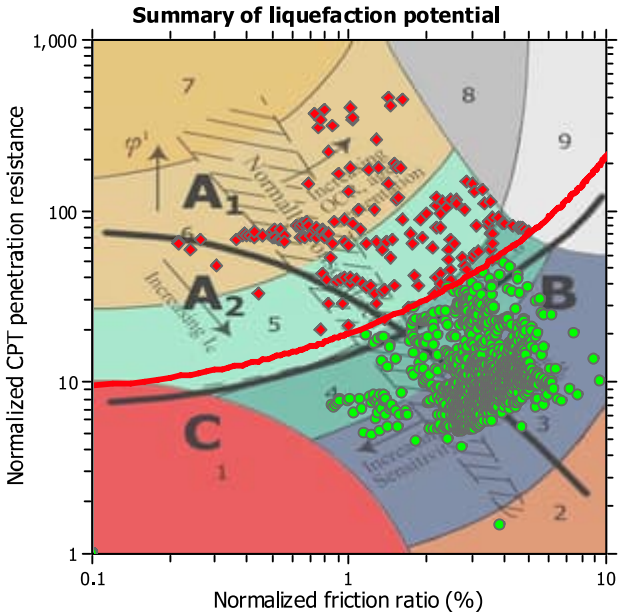
CPT file : 1-CPT1

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	6.60 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	6.60 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.50	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.85	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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http://www.geologismiki.gr

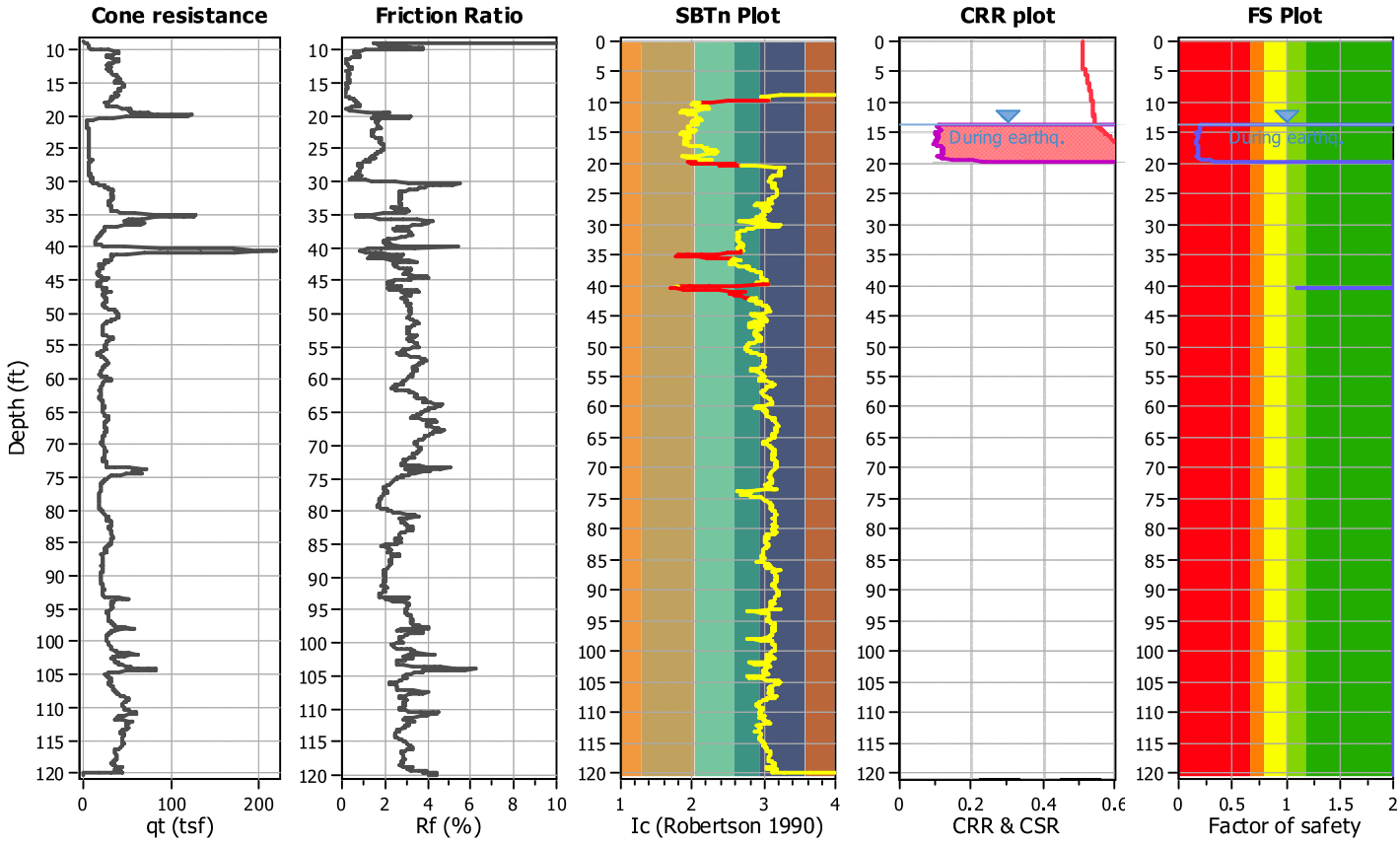
LIQUEFACTION ANALYSIS REPORT

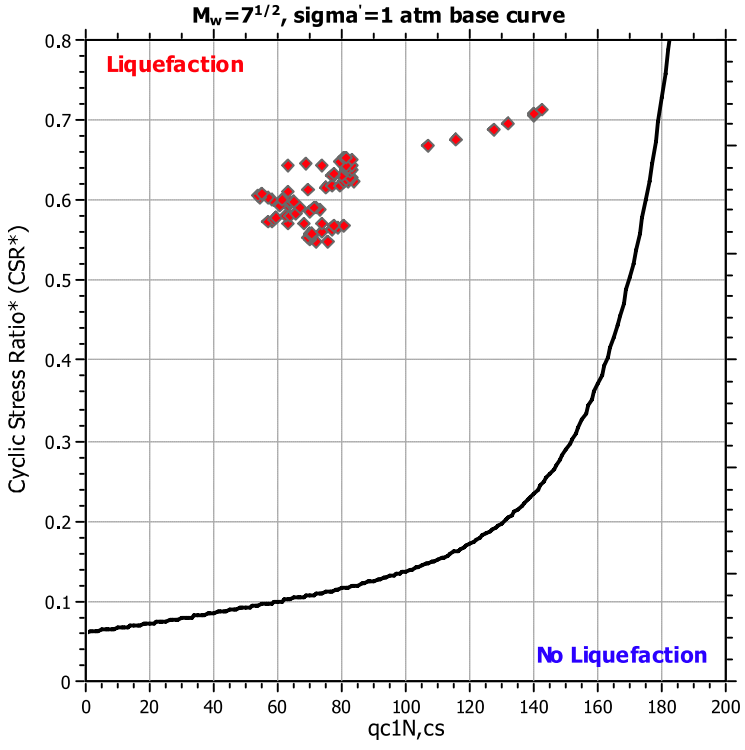
Project title : **Location :**

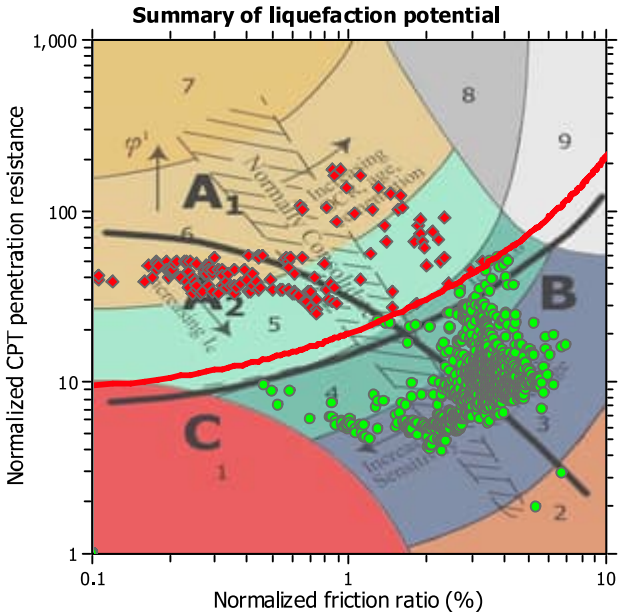
CPT file : 1-CPT2

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	13.70 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	13.70 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.50	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.85	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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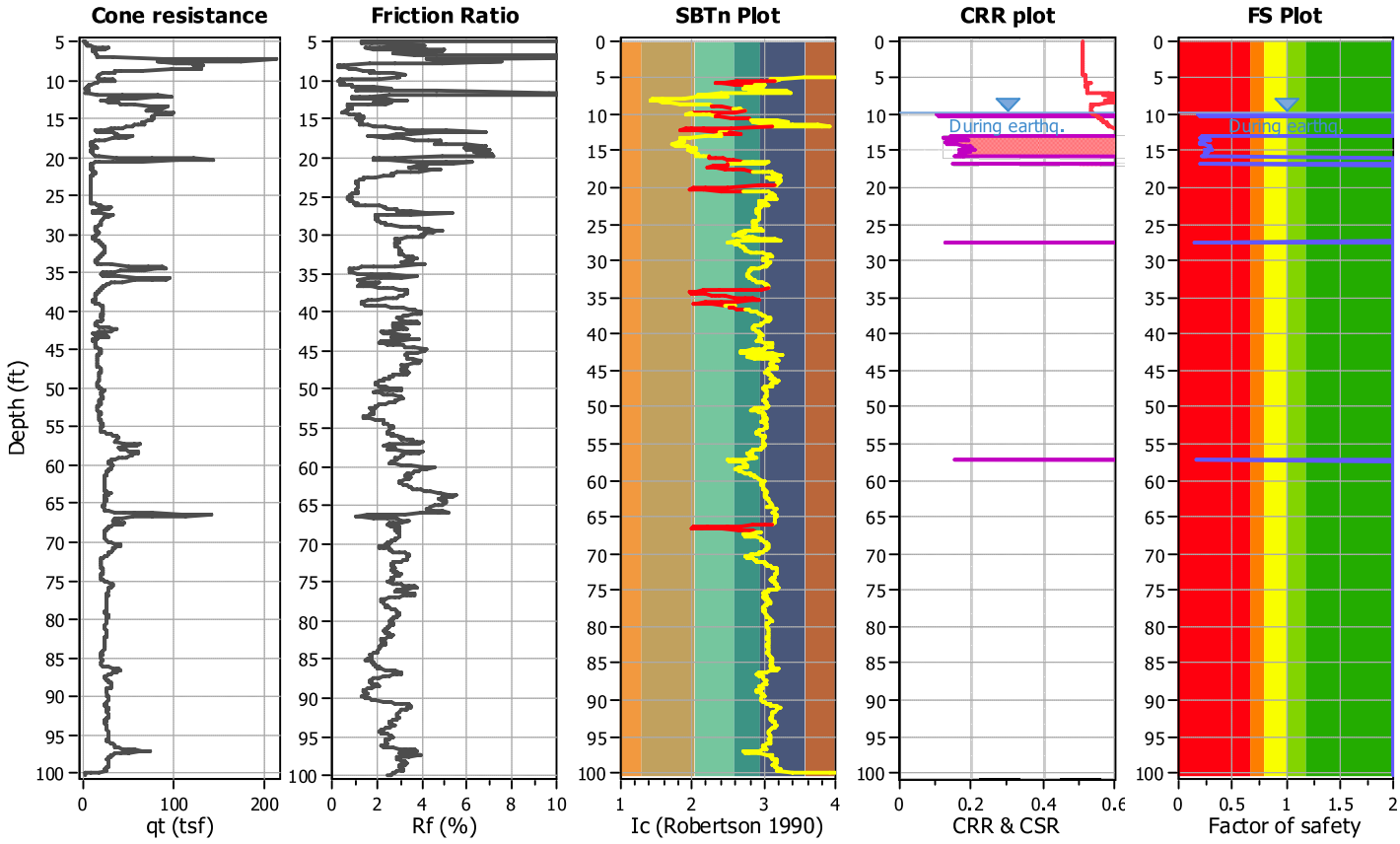
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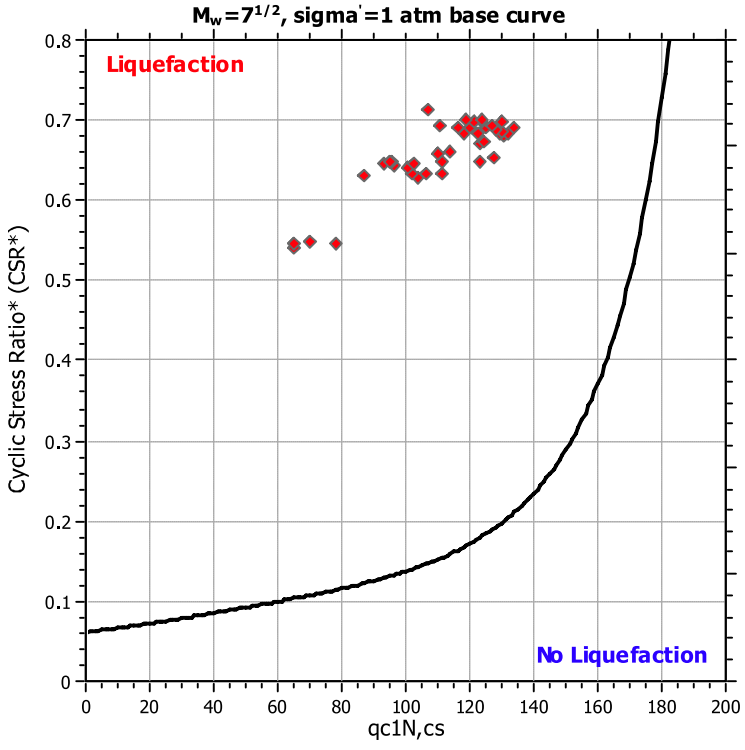
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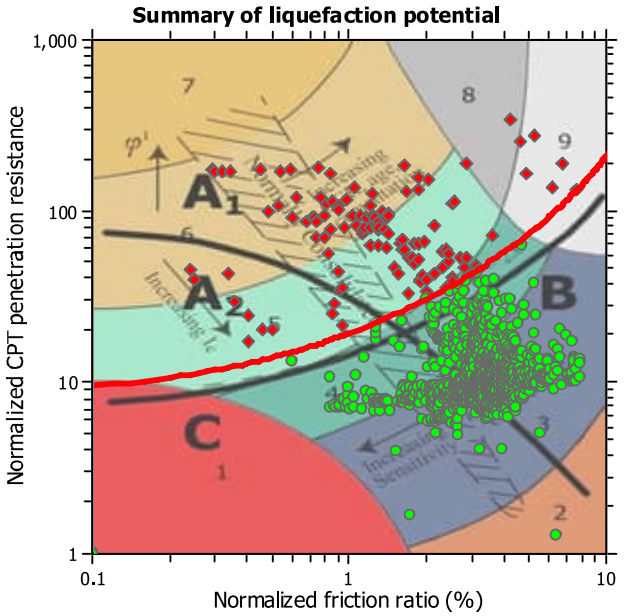
CPT file : 1-CPT3

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	9.80 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	9.80 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.50	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.85	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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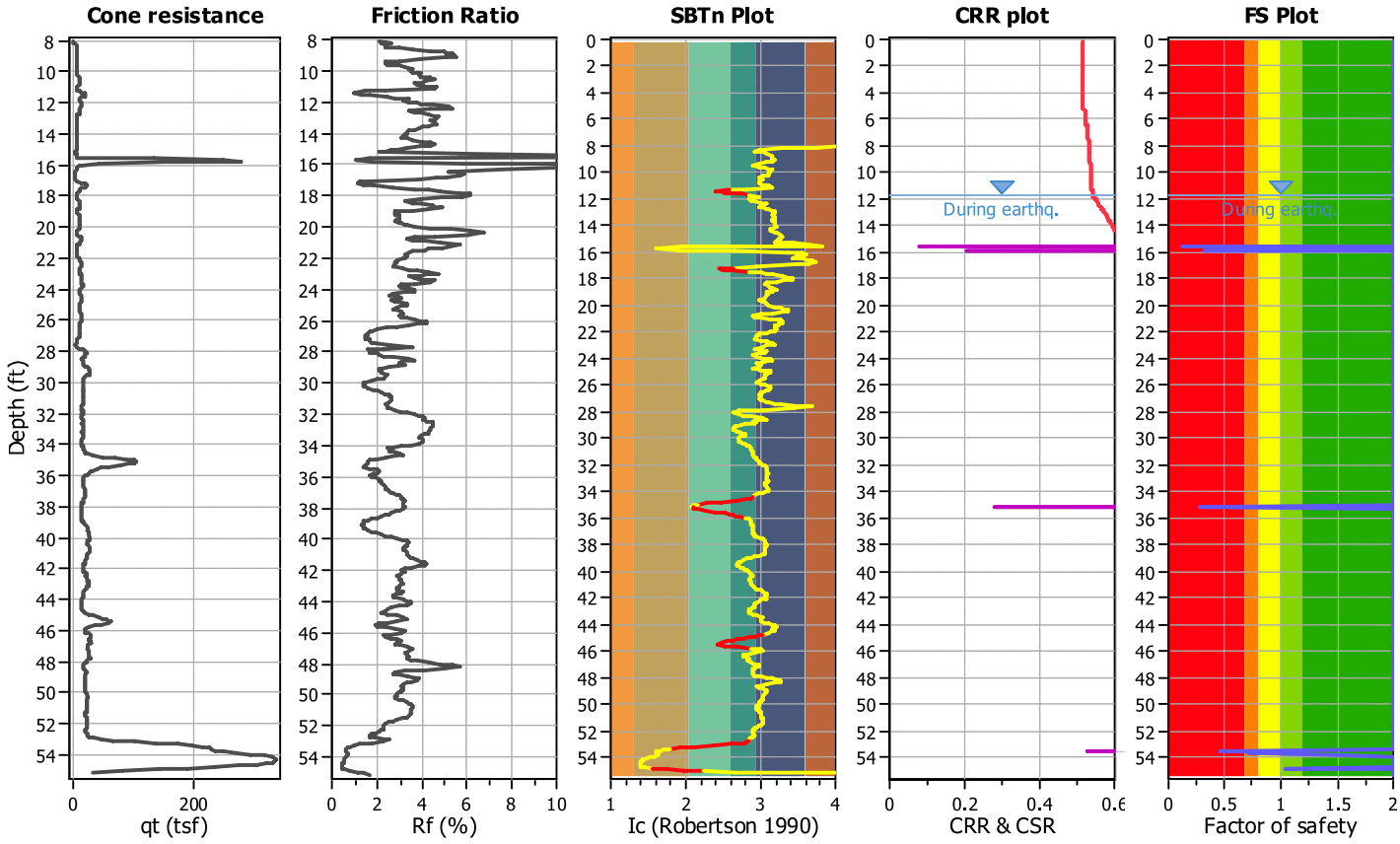
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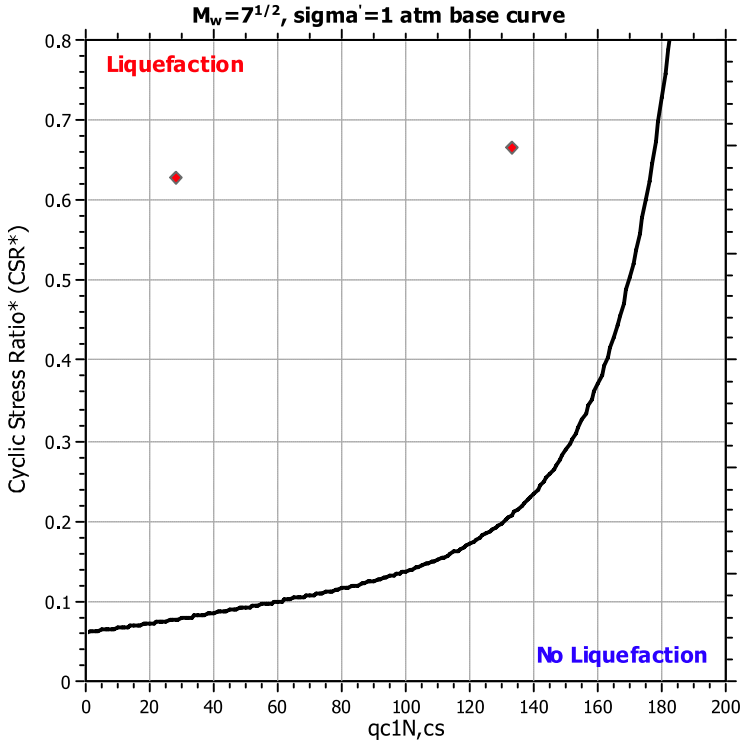
Project title : **Location :**

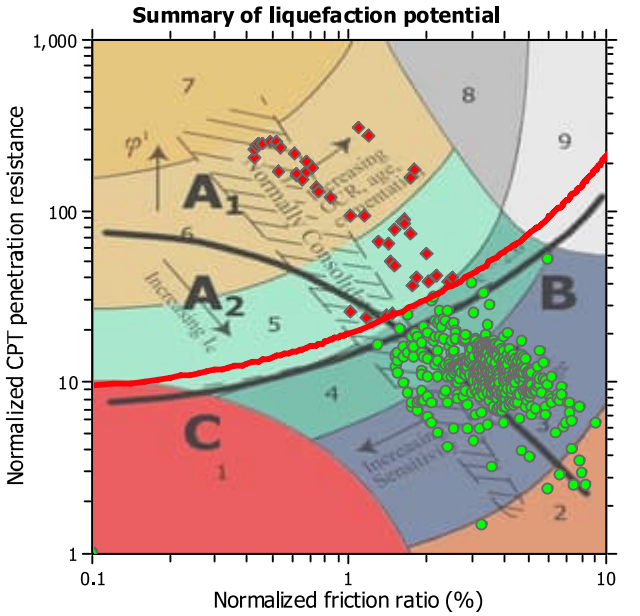
CPT file : 1-CPT4

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	11.70 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	11.70 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.50	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.85	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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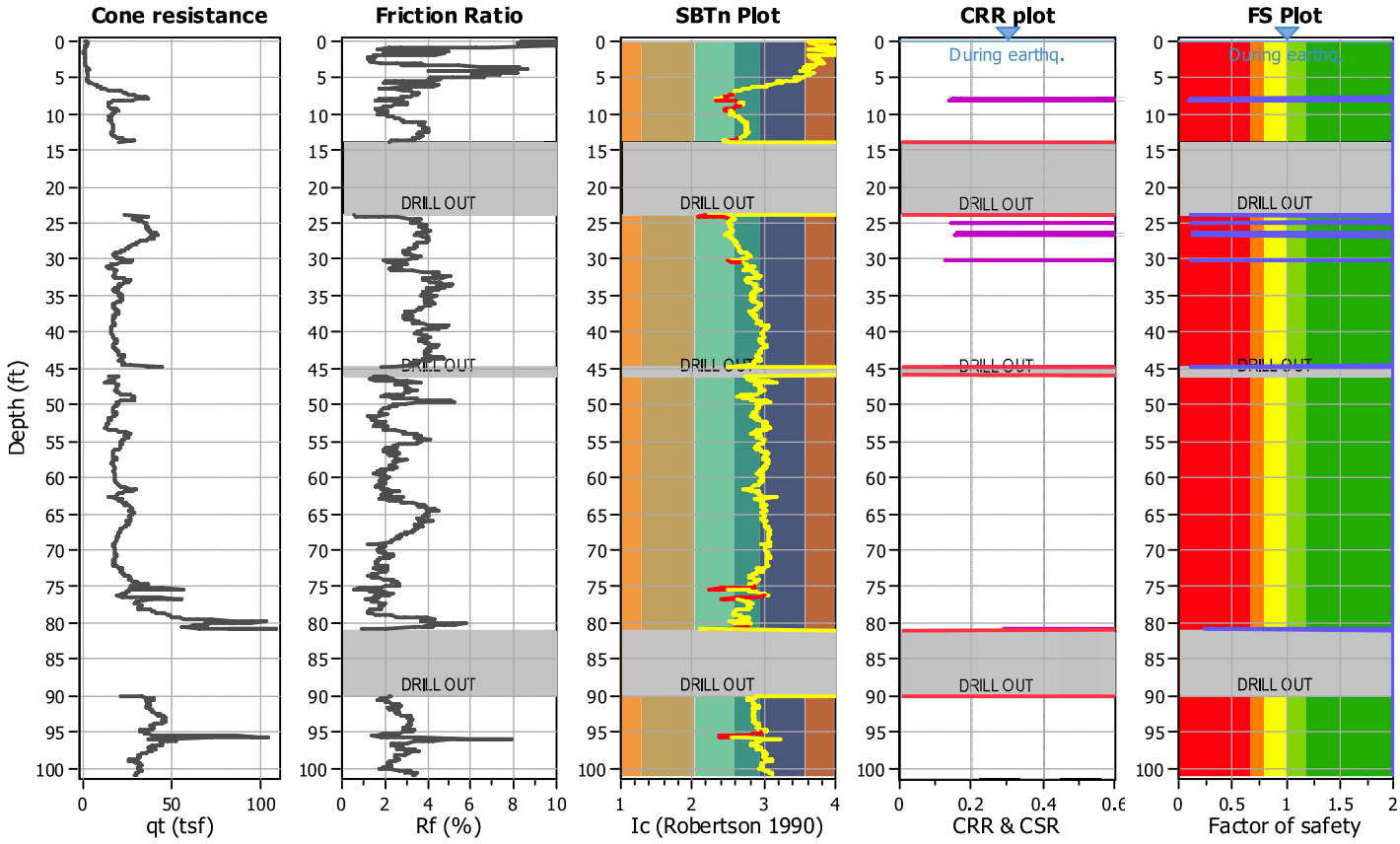
LIQUEFACTION ANALYSIS REPORT

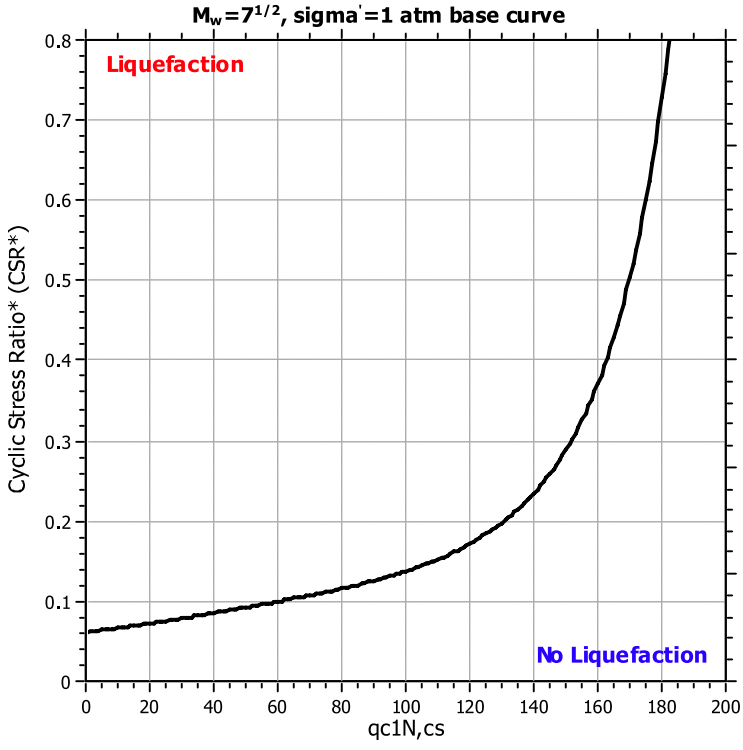
Project title : **Location :**

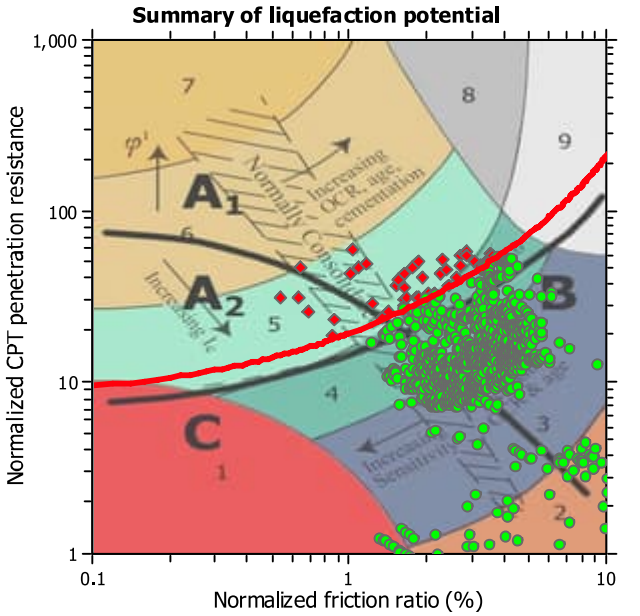
CPT file : 1OS-CPT1

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.50	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.85	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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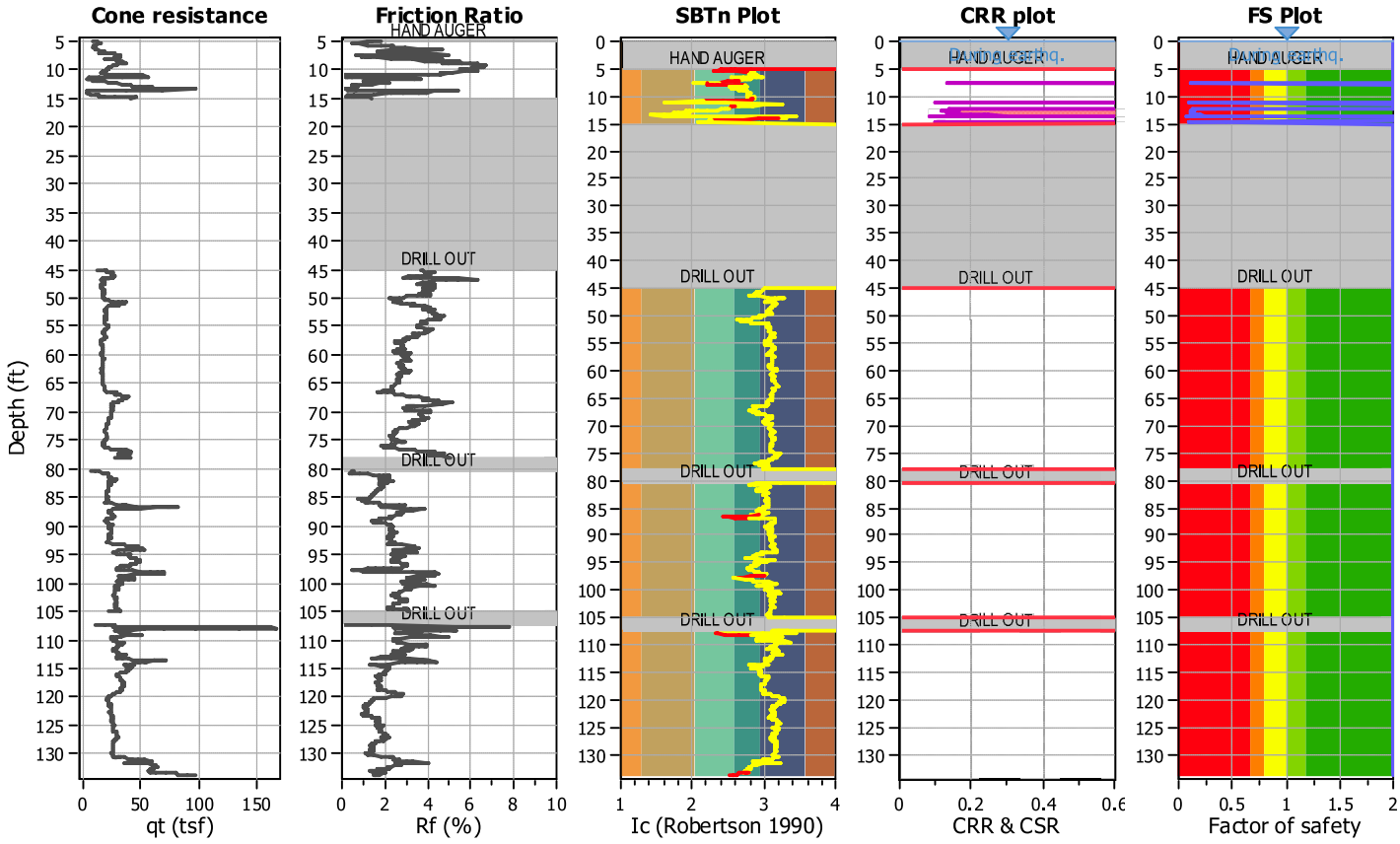
LIQUEFACTION ANALYSIS REPORT

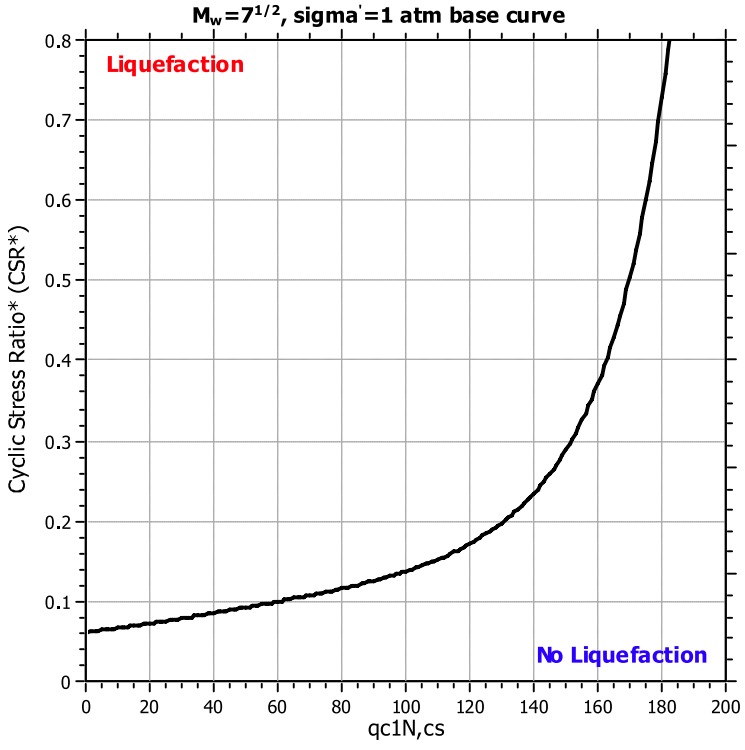
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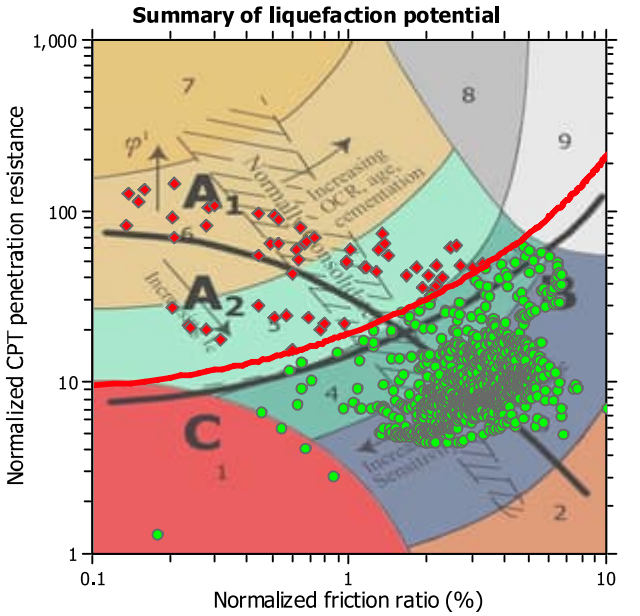
CPT file : 1OS-CPT2

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.50	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.85	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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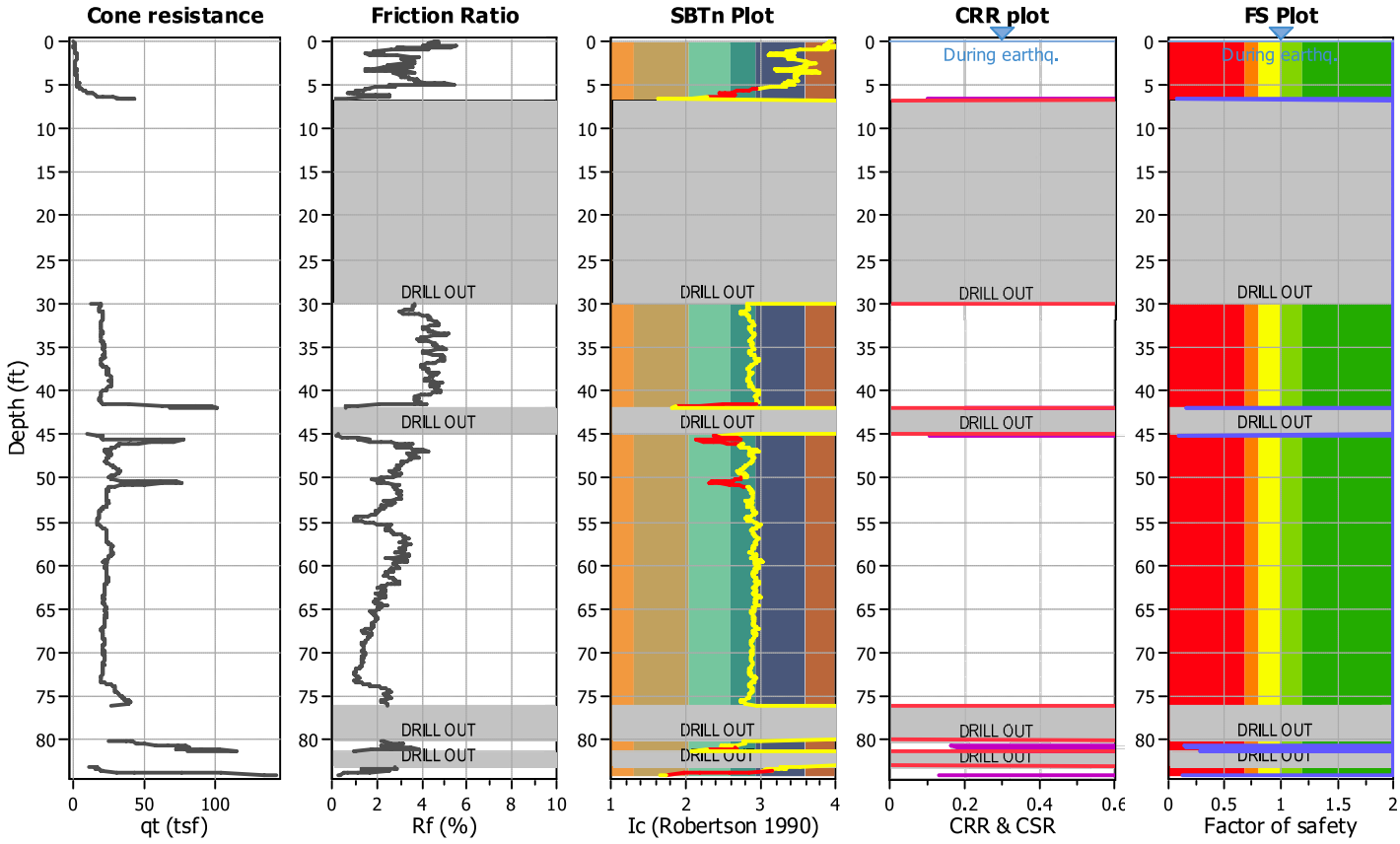
LIQUEFACTION ANALYSIS REPORT

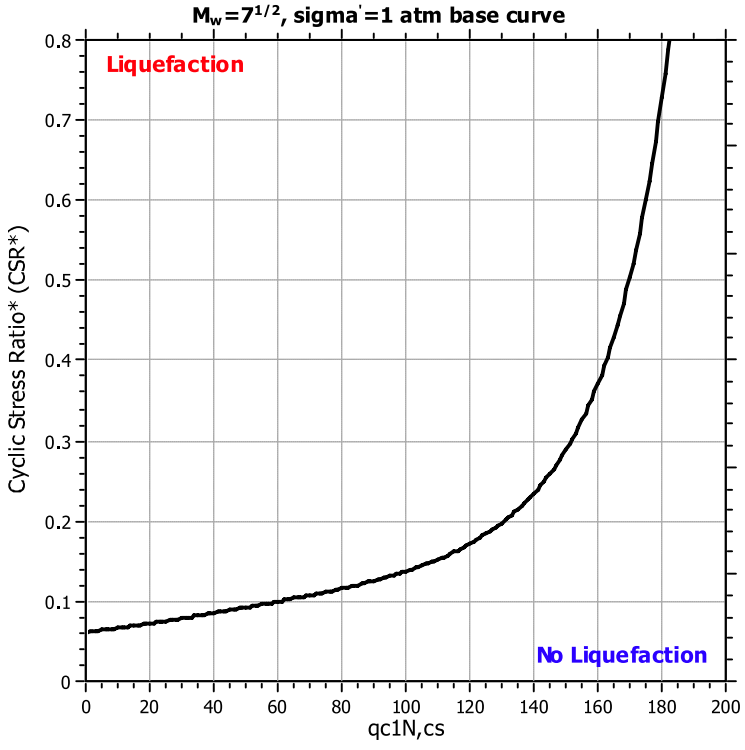
Project title : **Location :**

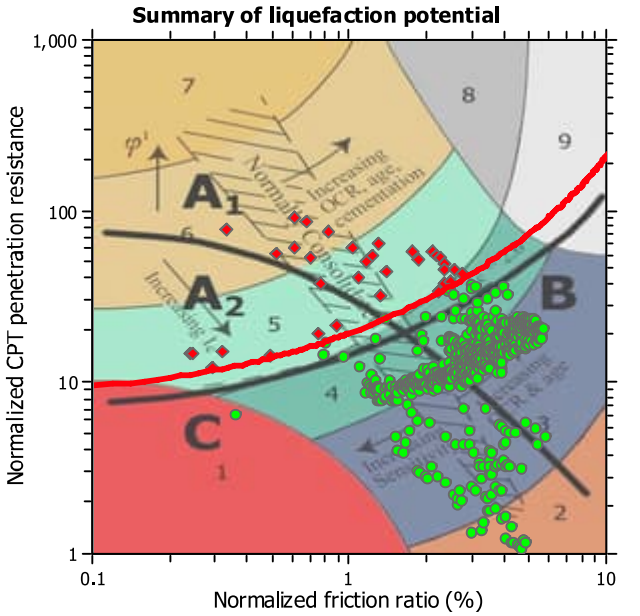
CPT file : 1OS-CPT3

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	8.10	Ic cut-off value:	2.50	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.85	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry



APPENDIX C

KEY TO BORING LOGS
BORING LOGS

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity CL - Inorganic clay with low to medium plasticity OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity CH - Fat clay with high plasticity OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE					CLEAR SQUARE SIEVE OPENINGS			
200	40	10	4	3/4 "	3"	12"		
SILTS AND CLAYS	SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE			

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Dames and Moore Piston
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

LINE TYPES

- ———— Solid - Layer Break
- – – – – – Dashed - Gradational or approximate layer break

GROUNDWATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

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SOIL BORING 1-B1

LATITUDE: 37.8629 LONGITUDE: -122.3171

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/04/2024
HOLE DEPTH: 106.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 17 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0	15		**SANDY LEAN CLAY WITH GRAVEL** (**CL**), dark yellowish brown, dry, ~30% fine-grained sand, ~15% subangular fine to coarse gravel, rootlets, wood [FILL]											
5	10		**CLAYEY GRAVEL** (**GC**), dark reddish brown, medium dense, dry, angular fine to coarse gravel, some fine- to coarse-grained sand, rootlets [FILL]			31	35	24	12	7.3	110.5		408	
10	5		**CLAYEY SAND WITH GRAVEL** (**SC**), dark yellowish brown, loose, moist, fine-grained sand, some subangular fine gravel [FILL]			12	45	29	45	19.8	94.7		299.2	
15	0		**FAT CLAY** (**CH**), bluish black, soft, wet, shells [YOUNG BAY MUD]											

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SOIL BORING 1-B1

LATITUDE: 37.8629 LONGITUDE: -122.3171

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/04/2024
HOLE DEPTH: 106.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 17 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
	-5					WOH								
25														
	-10		very soft			50psi								
						WOH								
30														
						WOH								
	-15		**SANDY LEAN CLAY** (**CL**), olive gray, medium stiff, wet, ~30% fine-grained sand [ALLUVIUM]			400psi								
			SANDY LEAN CLAY (**CL**), light yellowish brown, medium stiff, moist, ~45% fine-grained sand [ALLUVIUM]											
35						9								
	-20													
			SILTY SAND (**SM**), light yellowish brown, medium dense, moist, fine- to medium-grained sand, iron oxide staining [ALLUVIUM]											

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SOIL BORING 1-B1

LATITUDE: 37.8629 LONGITUDE: -122.3171

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/04/2024
HOLE DEPTH: 106.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 17 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						20			16					
	-25													
			FAT CLAY (**CH**), olive gray, stiff, moist [OLD BAY CLAY]											
45														
	-30					175psi 350psi								
50														
	-35													
55			Light gray, very stiff, 15% fine-grained sand			WOH				22.7 20.7	102.0 103.8		1986	
	-40													

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SOIL BORING 1-B1

LATITUDE: 37.8629 LONGITUDE: -122.3171

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/04/2024
HOLE DEPTH: 106.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 17 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
	-45													
65														
	-50													
70			Dark greenish gray, no sand			14								
	-55													
75			Stiff			150psi								
	-60													

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SOIL BORING 1-B1

LATITUDE: 37.8629 LONGITUDE: -122.3171

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/04/2024
HOLE DEPTH: 106.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 17 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
85	-65													
90	-70		Grayish green, very stiff, shells			16				23.2	99.6		458.3	
95	-75													
	-80													

SOIL BORING 1-B1

LATITUDE: 37.8629 LONGITUDE: -122.3171

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/04/2024
HOLE DEPTH: 106.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 17 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
	-85													
105			Stiff, ~15% fine-grained sand, shells			22								
			ALLUVIUM LEAN CLAY (**CL**), yellowish brown, stiff, moist, ~15% fine-grained sand [ALLUVIUM]											

End of boring at approximately 106½ feet below ground surface. Groundwater was not measured due to drilling method.

SOIL BORING 1-B2

LATITUDE: 37.8630 LONGITUDE: -122.3176

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/03/2024
HOLE DEPTH: 101.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 11.6 ft (MLLW)

LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			ASPHALT (**AC**), 4¼-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 5-inch Aggregate Base											
	10		**LEAN CLAY WITH SAND** (**CL**), light yellowish brown, moist, ~15% fine- to coarse-grained sand, trace fine gravel [FILL]											
			~25% fine to coarse gravel											
5			**CLAYEY SAND** (**SC**), light yellowish brown, medium dense, wet, fine- to medium-grained sand [FILL]			41	42	25	44	21.0	107.8		662	
	5		**CLAYEY SAND WITH GRAVEL** (**SC**), dark gray, very dense, moist, fine- to coarse-grained sand, some fine to coarse angular gravel [ROCK DIKE]			65			18	15.6				
			Wet				38	18	13	13.6				
10			**WELL-GRADED SAND WITH SILT AND GRAVEL** (**SW-SM**), dark gray, dense, moist, fine- to coarse-grained sand, some angular fine to coarse gravel [ROCK DIKE]			29			5	8.2				
	0													
15			wet hard drilling			81			2	14.9				
	-5													
			FAT CLAY (**CH**), bluish black, soft, wet, shells [YOUNG BAY MUD]											

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SOIL BORING 1-B2

LATITUDE: 37.8630 LONGITUDE: -122.3176

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/03/2024
HOLE DEPTH: 101.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 11.6 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
	-10					WOH				27.6	96.8		724	
										27.3	97.0			
25														
	-15		**CLAYEY SAND** (**SC**), light yellowish brown, very stiff, wet, fine- to coarse-grained sand [ALLUVIUM]			WOH 300psi								
30			Pale olive to brown, moist, trace subrounded fine gravel			13	35	22	44					
	-20													
			WELL-GRADED SAND WITH SILT (**SW-SM**), brown, medium dense, wet, fine- to medium-grained sand, trace subrounded fine gravel, ~10% fines [ALLUVIUM]											
35						16								
	-25		Dark gray							25.8	100.1			
			FAT CLAY WITH SAND (**CH**), olive gray to grayish green, very stiff, moist, ~15% fine-grained sand [OLD BAY CLAY]											

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SOIL BORING 1-B2

LATITUDE: 37.8630 LONGITUDE: -122.3176

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/03/2024
HOLE DEPTH: 101.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 11.6 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			Olive			32				21.4	107.0			
45	-30		Gray to olive gray, trace fine-grained sand and subrounded fine gravel			31				23.2	103.9		2959	
50	-35		Medium stiff, no sand or gravel			26								
55	-40		Olive, stiff, wet			WOH 50psi								
	-45													

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SOIL BORING 1-B2

LATITUDE: 37.8630 LONGITUDE: -122.3176

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/03/2024
HOLE DEPTH: 101.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 11.6 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			Gray to olive gray, moist, trace subrounded fine gravel			22								
	-50													
65														
	-55													
70			Greenish gray, medium stiff, no sand or gravel, brown mottling			14				35.6	84.3			
	-60													
75														
	-65													

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SOIL BORING 1-B2

LATITUDE: 37.8630 LONGITUDE: -122.3176

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/03/2024
HOLE DEPTH: 101.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 11.6 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			Olive gray to greenish gray, stiff, wet, shells			WOH								
	-70													
85														
	-75													
90			~15% fine-grained sand, brown mottling			4							1056	
	-80													
95														
	-85		**SANDY LEAN CLAY** (**CL**), brown, very stiff, moist to wet, ~40% fine-grained sand [ALLUVIUM]											

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SOIL BORING 1-B2

LATITUDE: 37.8630 LONGITUDE: -122.3176

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/03/2024
HOLE DEPTH: 101.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 11.6 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						29								

End of boring at approximately 101½ feet below ground surface. Groundwater was not measured due to drilling method.

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SOIL BORING 1-B3

LATITUDE: 37.8632 LONGITUDE: -122.3173

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 08/29/2024
HOLE DEPTH: 52.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 15 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sample Graphic	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blows/Foot	Atterberg Limits (LL-PL-PI)	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Shear Strength (psf)	Compressive Strength (psf)	Pocket Penetrometer (tsf)	Torvane (psf)
			ASPHALT (**AC**), 5-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 7-inch Aggregate Base											
			LEAN CLAY WITH SAND (**CL**), brown, medium stiff, moist, ~20% fine-grained sand, ~10% angular fine gravel [FILL]				41-17-24			17.20				
			reddish orange mottling, dark gray and pale olive staining											
5	10					8							2	1000
			Light yellowish brown, very stiff, with trace fine-grained sand				38-13-25		23.5	100.2	784			
			CLAYEY SAND (**SC**), light yellowish brown, loose, moist, fine- to coarse-grained sand, maganese staining [FILL]			8		17						
10	5		Dark gray, wet, some fine- to coarse- grained sand and angular fine gravel, brick fragments, olive and yellowish brown staining			4			22.1					
			WELL-GRADED GRAVEL WITH SILT AND SAND (**GW**), dark gray, loose, wet, angular fine gravel, ~10% fine- to coarse-grained sand, ~10% fines [FILL]											
			CLAYEY SAND (**SC**), gray, loose, moist, fine- to coarse-grained sand, ~45% fines, yellow-brown staining [FILL]											
15	0					6	39-14-25		19.7					
			CLAYEY SAND WITH GRAVEL (**SC**), gray, soft, moist, ~40% fines, ~15% angular fine gravel [FILL]											
			FAT CLAY (**CH**), dark gray, very stiff, wet, organic odor [YOUNG BAY MUD]											

ENGEO
Expect Excellence

SOIL BORING 1-B3

LATITUDE: 37.8632 LONGITUDE: -122.3173

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 08/29/2024
HOLE DEPTH: 52.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 15 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sample Graphic	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blows/Foot	Atterberg Limits (LL-PL-PI)	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Shear Strength (psf)	Compressive Strength (psf)	Pocket Penetrometer (tsf)	Torvane (psf)
						WOH			74.4 86.7	56.0 49.2	300		2.5	
25	-10		**SANDY LEAN CLAY** (**CL**), gray, medium stiff, moist, ~35% fine-grained sand [YOUNG BAY MUD]			3	32-19-13		29.6		498		0.5	200
30	-15		**SANDY LEAN CLAY** (**CL**), yellowish brown, very stiff, moist, ~30% fine-grained sand, ~10% subangular fine gravel [ALLUVIUM]			11							3.5	
35	-20		Shells			55							1.5	
			SILTY SAND (**SM**), yellowish brown, dense, moist, fine- to coarse-grained sand, ~20% fines [ALLUVIUM]											

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Expect Excellence

SOIL BORING 1-B3

LATITUDE: 37.8632 LONGITUDE: -122.3173

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 08/29/2024
HOLE DEPTH: 52.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 15 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sample Graphic	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blows/Foot	Atterberg Limits (LL-PL-PI)	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Shear Strength (psf)	Compressive Strength (psf)	Pocket Penetrometer (tsf)	Torvane (psf)
			LEAN CLAY (**CL**), olive brown, very stiff, moist, trace fine-grained sand [ALLUVIUM]			16							2	500
45	-30					25							2	1000
50	-35		~35% fine-grained sand			WOH 400psi							3.5	

End of boring at approximately 52½ feet below ground surface. Groundwater was not measured due to drilling method.

ENGEO
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SOIL BORING 1-B4

LATITUDE: 37.86184 LONGITUDE: -122.31905

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 52.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 14.2 ft (MLLW)
LOGGED BY / REVIEWED BY: QP / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			ASPHALT (**AC**), 10½-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 6-inch Aggregate base											
			SANDY LEAN CLAY WITH GRAVEL (**CL**), dark bluish gray, soft, moist, ~30% fine- to coarse-grained sand, ~25% subangular fine gravel, yellowish red staining [FILL]											
	10					4								
5														
	5		brown, stiff, ~20% fine- to coarse-grained sand, less gravel			WOH	45	23		21.8	105.1		506	
10														
	0		~25% fine- to coarse-grained sand, ~10% subangular fine gravel			5								
15														
	-5		**FAT CLAY** (**CH**), bluish black, medium stiff, wet, ~10% fine- to coarse-grained sand, organic odor [YOUNG BAY MUD]											

ENGEO
Expect Excellence

SOIL BORING 1-B4

LATITUDE: 37.86184 LONGITUDE: -122.31905

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 52.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 14.2 ft (MLLW)
LOGGED BY / REVIEWED BY: QP / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						WOH								
			CLAYEY GRAVEL WITH SAND (**GC**), reddish brown, medium dense, moist, angular fine gravel, some fine- to coarse-grained sand [ALLUVIUM]			600psi			40					
						22								
	-10													
25			**LEAN CLAY** (**CL**), reddish brown, very stiff, moist, trace fine-grained sand [ALLUVIUM]											
			light yellowish brown											
	-15					100psi				33.3	89.7		781	
30														
	-20													
35			yellowish red			12								
	-25													

ENGEO
Expect Excellence

SOIL BORING 1-B4

LATITUDE: 37.86184 LONGITUDE: -122.31905

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 52.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 14.2 ft (MLLW)
LOGGED BY / REVIEWED BY: QP / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
45	-30		~10% fine to medium grained sand			24								
			SILT WITH SAND (**ML**), reddish brown, very stiff, moist, ~25% fine-grained sand [ALLUVIUM]											
50	-35					50psi								
			LEAN CLAY WITH SAND (**CL**), reddish brown, very stiff, moist, ~20% medium-grained sand, trace subrounded fine gravel [ALLUVIUM]			50psi WOH							2596	
			End of boring at approximately 52½ feet below ground surface. Groundwater was not measured due to drilling method.											
55	-40													
	-45													

ENGEO
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SOIL BORING 1-B5

LATITUDE: 37.8594 LONGITUDE: -122.3163

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 51.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 15 ft

LOGGED BY / REVIEWED BY: QP / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			ASPHALT (**AC**), 3½-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 7-inch Aggregate Base											
			SANDY LEAN CLAY (**CL**), dark brown, moist, ~20% fine- to coarse-grained sand, yellowish red mottling [FILL]											
5	10					50/3"			39	16.2 15.7	104.8 105.3			
			CLAYEY GRAVEL WITH SAND (**GC**), gray, very dense, moist, angular [ROCK DIKE]											
10	5													
15	0		**GRAVELLY LEAN CLAY WITH SAND** (**CL**), bluish black to reddish brown, medium dense, moist [ROCK DIKE]			21			54	21.60 9.80				

ENGEO
Expect Excellence

SOIL BORING 1-B5

LATITUDE: 37.8594 LONGITUDE: -122.3163

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 51.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 15 ft

LOGGED BY / REVIEWED BY: QP / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						30								
			FAT CLAY (**CH**), bluish black, very soft, wet, trace organics and shells [YOUNG BAY MUD]			1								
25	-10									19.4	112.2		312	
						WOH								
30	-15					100psi								
			LEAN CLAY (**CL**), yellowish red, yellowish red, moist [ALLUVIUM]											
35	-20					200psi				31.1	89.1		1630	

ENGEO
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SOIL BORING 1-B5

LATITUDE: 37.8594 LONGITUDE: -122.3163

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 51.5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 15 ft

LOGGED BY / REVIEWED BY: QP / JF
DRILLING CONTRACTOR: Pitcher Services LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			Reddish brown, hard, ~30% fine-grained sand, bluish gray mottling			48								
			End of boring at approximately 41½ feet below ground surface. Groundwater was not measured due to drilling method.											
45	-30													
50	-35													
55	-40													

SOIL BORING 1-C1

LATITUDE: 37.8647 LONGITUDE: -122.3114

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/28/2024
HOLE DEPTH: 6 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: 12.5 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: West Coast exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			ASPHALT (**AC**), 12-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 12-inch Aggregate Base											
	10		**SANDY LEAN CLAY** (**CL**), dark gray, moist, ~30% fine- to coarse-grained sand, ~15% subangular fine gravel [FILL]				44	25		12.2				
5														

End of boring at approximately 6 feet below ground surface. Groundwater was not encountered during drilling.

SOIL BORING 1-C2

LATITUDE: 37.8641 LONGITUDE: -122.3121

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/28/2024
HOLE DEPTH: 5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: 13 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0			**ASPHALT** (**AC**), 9¼-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 12-inch Aggregate Base											
			SANDY LEAN CLAY (**CL**), dark gray, moist, ~40% fine- to coarse-grained sand, ~5% fine gravel [FILL]											
10														
5			End of boring at approximately 5 feet below ground surface. Groundwater was not encountered during drilling.											

SOIL BORING 1-C3

LATITUDE: 37.8637 LONGITUDE: -122.3135

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/28/2024
HOLE DEPTH: 6 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: 14 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			ASPHALT (**AC**), 4¼-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 12-inch Aggregate Base											
			SANDY LEAN CLAY (**CL**), yellowish brown, moist, ~35% fine- to coarse-grained sand, ~5% subangular fine gravel [FILL]				36	20		14.3				
5	10		**LEAN CLAY WITH SAND** (**CL**), dark gray, moist, ~15% fine-grained sand [FILL]											

End of boring at approximately 6 feet below ground surface. Groundwater was not encountered during drilling.

ENGEO
Expect Excellence

SOIL BORING 1-C4

LATITUDE: 37.8627 LONGITUDE: -122.3153

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 08/28/2024
HOLE DEPTH: 3.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: 14.8 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			ASPHALT (**AC**), 2-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 8-inch Aggregate Base											
			GRAVELLY LEAN CLAY WITH SAND (**CL**), brown, moist, ~20% subangular fine gravel, ~15% fine- to medium-grained sand [FILL]				33	17		10.9				
			POORLY GRADED SAND (**SP**), dark gray, dry, angular coarse-grained sand [FILL]											

End of boring at approximately 3½ feet below ground surface. Groundwater was not encountered during drilling.

ENGEO
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SOIL BORING 1-C5

LATITUDE: 37.8615 LONGITUDE: -122.3163

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 08/28/2024
HOLE DEPTH: 4 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: 17.4 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			3-inch Asphalt Concrete											
			6½-inch Aggregate Base											
			SILTY SAND WITH GRAVEL (**SM**), yellowish brown, dry to moist, ~20% fines, ~20% fine gravel [FILL]											
	15													

End of boring at approximately 4 feet below ground surface. Groundwater was not encountered during drilling.

ENGEO
Expect Excellence

SOIL BORING 1-C6

LATITUDE: 37.8610 LONGITUDE: -122.3160

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 08/28/2024
HOLE DEPTH: 5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: 16.5 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0	15		**ASPHALT** (**AC**), 2½-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 9-inch Aggregate Base											
			SANDY LEAN CLAY (**CL**), yellow-brown, moist, ~30% fine-grained sand, ~5% subangular fine gravel [FILL]				38	18						
5			**LEAN CLAY WITH SAND** (**CL**), olive brown, moist, ~30% fine-grained sand [FILL]											

End of boring at approximately 5 feet below ground surface. Groundwater was not encountered during drilling.

ENGEO
Expect Excellence

SOIL BORING 1-C7

LATITUDE: 37.8607 LONGITUDE: -122.3164

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 08/28/2024
HOLE DEPTH: 5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: 14.2 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0			**ASPHALT** (**AC**), 10½-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 6-inch Aggregate Base											
			LEAN CLAY (**CL**), dark brown, moist, ~30% fine-grained sand, ~15% subangular fine gravel [FILL]											
	10													
5			End of boring at approximately 5 feet below ground surface. Groundwater was encountered at 5 feet below ground surface.											

SOIL BORING 1-C8

LATITUDE: 37.86297 LONGITUDE: -122.31758

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 08/28/2024
HOLE DEPTH: 6 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: 13.3 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0			**ASPHALT** (**AC**), 4½-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 3-inch Aggregate Base											
			GRAVELLY LEAN CLAY WITH SAND (**CL**), bluish black, moist, ~20% fine- to coarse-grained sand [FILL]				39	17		14.3				
	10													
5			End of boring at approximately 5 feet below ground surface. Groundwater was not encountered during drilling.											

SOIL BORING 1-CPT2

LATITUDE: 37.8632 LONGITUDE: -122.3163

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/29/2024
HOLE DEPTH: 5 ft
HOLE DIAMETER: 4 in
SURFACE ELEV.: 18.7 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: ConeTec
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0			**ASPHALT** (**AC**), 4¼-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 6-inch Aggregate Base											
			SANDY LEAN CLAY (**CL**), yellowish brown, moist, ~30% fine- to medium-grained sand [FILL]				40	22		17.2				
			Brick											
	15		**CLAYEY GRAVEL** (**GC**), dark gray, moist, ~20% fines, ~10% fine- to coarse-grained sand [FILL]											
5			End of boring at approximately 5 feet below ground surface. Groundwater was not encountered during drilling.											

ENGEO
Expect Excellence

SOIL BORING 1-CPT3

LATITUDE: 37.86297 LONGITUDE: -122.31758

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/29/2024
HOLE DEPTH: 5 ft
HOLE DIAMETER: 4.5 in
SURFACE ELEV.: 14.8 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: ConeTec
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0			**ASPHALT** (**AC**), 5" Asphalt Concrete											
			AGGREGATE BASE (**AB**), 7" Aggregate Base											
			CLAYEY GRAVEL WITH SAND (**GC**), dark brown, moist to wet, ~30% fines, ~20% fine- to medium grained sand [FILL]											
			WELL-GRADED SAND WITH CLAY AND GRAVEL (**SW**), dark gray, moist, ~20% fines, ~30% subangular fine gravel [FILL]											
5	10		End of boring at approximately 5 feet below ground surface. Groundwater was not encountered during drilling.											

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Expect Excellence

SOIL BORING 1-CPT4

LATITUDE: 37.8630 LONGITUDE: -122.3176

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/29/2024
HOLE DEPTH: 6 ft
HOLE DIAMETER: 4.5 in
SURFACE ELEV.: 16.7 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: ConeTec
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0	15		**ASPHALT** (**AC**), 2-inch Asphalt Concrete **AGGREGATE BASE** (**AB**), 9-inch Aggregate Base **SANDY LEAN CLAY** (**CL**), yellowish brown, moist, ~30% fine- to medium-grained sand [FILL]											
5														

End of boring at approximately 6 feet below ground surface. Groundwater was not encountered during drilling.

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SOIL BORING 1OS-B1

LATITUDE: 37.86286 LONGITUDE: -122.31808

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 127 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			FAT CLAY (**CH**), bluish black, very soft, wet, shells [YOUNG BAY MUD]											
	-10													
5						WOH				22.1	108.8		747.2	
			LEAN CLAY (**CL**), greenish gray, medium stiff, moist [YOUNG BAY MUD]											
	-15		**CLAYEY GRAVEL WITH SAND** (**GC**), yellowish brown, medium dense, moist, subrounded fine gravel, some fine-grained sand [ALLUVIUM]			21	33	15	48	18.3				
10						17								
			LEAN CLAY (**CL**), yellowish brown, medium dense, moist, ~45% fine-grained sand [ALLUVIUM]			WOH	35	18					2245	
	-20													
15			**WELL-GRADED SAND WITH SILT** (**SW-SM**), yellowish brown to dark gray, medium dense, wet, fine- to medium-grained sand [ALLUVIUM]			24			9.8	19.4	113.1			
						24								
	-25		**LEAN CLAY** (**CL**), yellowish brown, stiff, moist, ~20% fine-grained sand [ALLUVIUM]											

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SOIL BORING 1OS-B1

LATITUDE: 37.86286 LONGITUDE: -122.31808

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 127 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			FAT CLAY (**CH**), dark gray, stiff, moist [OLD BAY CLAY]			16				32.9	92.5			
			SILTY SAND (**SM**), dark gray, medium dense, moist, ~20% fines [OLD BAY CLAY]			11								
	-30		**SANDY LEAN CLAY** (**CL**), greenish gray, very stiff, moist, ~30% fine-grained sand [OLD BAY CLAY]											
25						34								
			FAT CLAY (**CH**), dark greenish gray, very stiff, moist, light gray mottling [OLD BAY CLAY]			26								
	-35													
30			no mottling			18								
			WELL-GRADED GRAVEL WITH SAND (**GW**), dark gray, dense, wet, subrounded fine gravel, ~15% fine- to medium-grained sand, shells [OLD BAY DEPOSITS]											
	-40													
35						500psi								
			FAT CLAY (**CH**), dark gray, stiff, moist, ~25% fine-grained sand [OLD BAY CLAY]											
	-45													

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SOIL BORING 1OS-B1

LATITUDE: 37.86286 LONGITUDE: -122.31808

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 127 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)

LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						16								
						400psi								
	-50													
45						22				30.4	94.4			
	-55													
50			~15% fine-grained sand			16								
	-60													
55			Medium stiff											
	-65													

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SOIL BORING 1OS-B1

LATITUDE: 37.86286 LONGITUDE: -122.31808

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 127 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			Bluish gray, stiff, wet, shells			9								
	-70													
65										43.1	78.6		1655	
	-75		**FAT CLAY** (**CH**), pale olive, very stiff, moist, fine-grained sand, ~15%, reddish yellow mottling [OLD BAY CLAY]											
70						26								
	-80													
75			Stiff			12								
	-85													

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SOIL BORING 1OS-B1

LATITUDE: 37.86286 LONGITUDE: -122.31808

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 127 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			LEAN CLAY (**CL**), yellowish red, very stiff, moist [ALLUVIUM]			300psi								
	-90													
85			Brown, sand lense @ 85'			27								
	-95													
90			very stiff			200psi				23.6	102.6		2590	
	-100		**FAT CLAY WITH SAND** (**CH**), light olive gray, very stiff, moist, ~10% fine-grained sand [OLD BAY CLAY]			34				47.8	86.7			
95														
	-105		**SANDY LEAN CLAY** (**CL**), brown, very stiff, moist, ~35% fine- to coarse-grained sand [ALLUVIUM]											

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SOIL BORING 1OS-B1

LATITUDE: 37.86286 LONGITUDE: -122.31808

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 127 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						35				20.2	108.6			
	-110													
105														
			FAT CLAY WITH SAND (**CH**), bluish gray, medium stiff to very stiff, moist, ~20% fine-grained sand, reddish brown mottling [OLD BAY CLAY]											
	-115													
110						37				27.5	96.40		3300	
	-120													
115														
	-125													

SOIL BORING 1OS-B1

LATITUDE: 37.86286 LONGITUDE: -122.31808

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 127 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: AC / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			Dark Gray, stiff, no sand			30								
	-130													
125			**LEAN CLAY** (**CL**), brown, very stiff, moist [ALLUVIUM]			400psi							4166	

End of boring at approximately 127 feet below ground surface.

ENGEO
Expect Excellence

SOIL BORING 1OS-B2

LATITUDE: 37.86246 LONGITUDE: -122.31908

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/18/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			FAT CLAY (**CH**), dark gray, very soft, wet, shells [YOUNG BAY MUD]											
	-10													
5						WOH								
						2								
	-15													
10			**SANDY LEAN CLAY** (**CL**), gray, soft, moist, ~30% fine-grained sand [YOUNG BAY MUD] Stiff, pale olive staining, reddish orange mottling shells			7								
						200psi				18.7	111.5		1482	
	-20													
15			**LEAN CLAY** (**CL**), light yellowish brown, very stiff, moist, ~30% fine-grained sand, maganese staining [ALLUVIUM]			19				17.9	114.0		1875	
	-25		**POORLY GRADED SAND WITH SILT** (**SP-SM**), olive, medium dense, wet, fine-grained sand, ~5% fines [ALLUVIUM]											

SOIL BORING 1OS-B2

LATITUDE: 37.86246 LONGITUDE: -122.31908

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/18/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)

LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			LEAN CLAY (**CL**), reddish orange, very stiff, moist, trace fine-grained sand [ALLUVIUM]			15								
			SILTY SAND (**SM**), light yellowish brown, medium dense, wet, ~20% fines [ALLUVIUM]											
	-30		**SANDY LEAN CLAY** (**CL**), olive-brown, stiff, moist, ~30% fine- to medium-grained sand [ALLUVIUM]											
25			grades yellowish brown			8								
	-35													
30			Olive-brown, very stiff, ~20% fine-grained sand			10								
						200psi								
	-40									23.0	102.5		1799	
35			**FAT CLAY WITH SAND** (**CH**), olive, very stiff, moist, ~20% fine-grained sand [OLD BAY CLAY]			18				30.4	100.0			
						300psi								
	-45													

ENGEO
Expect Excellence

SOIL BORING 1OS-B2

LATITUDE: 37.86246 LONGITUDE: -122.31908

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/18/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
45	-50		Gray, wet, trace fine-grained sand, yellowish brown mottling			19								
50	-55													
55	-60		Greenish gray			24								
	-65					400psi				29.0	94.4		2345	

ENGEO
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SOIL BORING 1OS-B2

LATITUDE: 37.86246 LONGITUDE: -122.31908

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/18/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
65	-70		Reddish orange stains			13								
70	-75		**LEAN CLAY** (**CL**), yellowish brown, very stiff, moist, ~10% fine- to medium-grained sand [ALLUVIUM]											
75	-80					12				32.5	100.0			
	-85					5								

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SOIL BORING 1OS-B2

LATITUDE: 37.86246 LONGITUDE: -122.31908

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/18/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			FAT CLAY (**CH**), olive, stiff to very stiff, moist, trace fine-grained sand, iron oxide staining [OLD BAY CLAY]			13								
			Bluish gray, dark gray staining											
			Olive, no staining											
	-90													
85			Bluish gray			WOH								
						300psi								
			LEAN CLAY (**CL**), yellowish brown, very stiff, moist, trace fine-grained sand [ALLUVIUM]											
	-95		**POORLY GRADED SAND** (**SP**), olive brown, medium dense, wet, medium- to coarse-grained sand, shells [ALLUVIUM]											
90			**LEAN CLAY** (**CL**), reddish orange, very stiff, moist [ALLUVIUM]			19				31.7	92.5		1815	
	-100													
95						40								
			SILTY SAND (**SM**), yellowish brown, dense, moist, ~30% fines [ALLUVIUM]											
			SANDY LEAN CLAY (**CL**), yellowish brown, hard, moist, ~30% fine- to medium-grained sand [ALLUVIUM]											
			Light yellowish brown, dense, wet											
	-105		Yellowish brown, hard, moist, reddish orange, olive, and white staining			44								
			CLAYEY SAND (**SC**), yellowish brown, dense, wet, fine- to coarse-grained sand, sandstone fragments											

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SOIL BORING 1OS-B2

LATITUDE: 37.86246 LONGITUDE: -122.31908

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/18/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			fine- to coarse-grained sand, standstone fragments, reddish orange staining [ALLUVIUM]											
			LEAN CLAY WITH SAND (**CL**), yellowish brown, very stiff, moist, ~15% fine-grained sand [ALLUVIUM]											
	-110													
105						38								
	-115		**LEAN CLAY WITH SAND** (**CL**), olive, very stiff, moist, ~25% fine- to medium-grained sand [OLD BAY CLAY]											
110						37								
			CLAYEY SAND (**SC**), yellowish brown, wet, fine- to coarse-grained sand, ~40% fines, trace subangular fine gravel [ALLUVIUM]											
	-120													
115														
			LEAN CLAY (**CL**), yellowish brown, ~10% fine-grained sand, maganese staining [ALLUVIUM]											
	-125													

SOIL BORING 1OS-B2

LATITUDE: 37.86246 LONGITUDE: -122.31908

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/18/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0			**FAT CLAY** (**CH**), gray, very stiff, moist, trace fine-grained sand [OLD BAY CLAY]			21								

End of boring at approximately 121.5 feet below ground surface. Groundwater was not measured due to drilling method.

SOIL BORING 1OS-B3

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/16/2024
HOLE DEPTH: 151.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			SILT (**ML**), dark gray, very soft, wet, shells [YOUNG BAY MUD]											
	-10													
5						WOH								
			Greenish black, soft, ~40% medium-grained sand			WOH								
	-15													
10			**FAT CLAY** (**CH**), greenish gray, stiff, wet [YOUNG BAY MUD]			8								
						100psi				20.4	109.7		1653	
	-20													
15						35								
			WELL-GRADED SAND WITH SILT (**SW-SM**), greenish gray, medium dense, wet, medium-grained sand, ~10% fines [ALLUVIUM]											
	-25													

SOIL BORING 1OS-B3

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/16/2024
HOLE DEPTH: 151.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)

LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			LEAN CLAY WITH SAND (**CL**), yellowish brown, stiff, wet, ~15% fine-grained sand [ALLUVIUM]			8								
	-30		**FAT CLAY** (**CH**), olive gray, hard, moist, ~10% fine-grained sand [OLD BAY CLAY]											
25						32				24.8	102.3		4033	
	-35													
30			Very stiff			19								
	-40													
35			Greenish gray			23				23.4	104.5		2304	
	-45													

ENGEO
Expect Excellence

SOIL BORING 1OS-B3

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/16/2024
HOLE DEPTH: 151.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			Olive to olive gray, very stiff,~10% fine-grained sand			26								
	-50													
45			stiff			19								
	-55													
50			greenish gray, vertical sand vein			17								
						400psi								
	-60													
55						18								
			sand lense			N/A								
	-65													

ENGEO
Expect Excellence

SOIL BORING 1OS-B3

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/16/2024
HOLE DEPTH: 151.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						12								
	-70									44.6	76.5		1551	
65														
	-75													
70														
	-80													
75			reddish yellow mottling, very stiff, ~25% fine-grained sand			30								
	-85													

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SOIL BORING 1OS-B3

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/16/2024
HOLE DEPTH: 151.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
	-90													
85			Olive, ~30% fine-grained sand			34								
						200psi				25.3	99.0		3108	
	-95													
90			**SANDY LEAN CLAY** (**CL**), yellowish brown, stiff, moist, maganese staining [ALLUVIUM]			23								
			olive gray											
	-100													
95														
	-105													

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SOIL BORING 1OS-B3

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/16/2024
HOLE DEPTH: 151.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			GRAVELLY LEAN CLAY WITH SAND (**CL**), grayish green, hard, moist, ~30% rounded fine gravel, ~10% medium-grained sand [OLD BAY CLAY]			47				22.7	106.4		3401	
	-110													
105														
			WELL-GRADED GRAVEL WITH SILT (**GW-GM**), pale olive, dense, moist, rounded fine gravel, ~25% fine-grained sand, ~10% fines [ALLUVIUM]											
	-115													
110						40								
			LEAN CLAY WITH SAND (**CL**), yellowish brown, very stiff, moist, ~25% fine-grained sand, iron oxide mottling [ALLUVIUM]											
	-120													
115														
			FAT CLAY (**CH**), greenish gray, stiff, moist [OLD BAY CLAY]											
	-125													

SOIL BORING 1OS-B3

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/16/2024
HOLE DEPTH: 151.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)

LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						23				39.8	81.4		1289.4	
	-130													
125														
	-135													
130			**SANDY LEAN CLAY** (**CL**), olive, very stiff to hard, moist, ~25% fine-grained sand [OLD BAY CLAY]			53								
	-140													
135			very stiff, reddish yellow mottling			48				19.1	112.0		2374	
	-145		**POORLY GRADED SAND WITH SILT** (**SP-SM**), yellowish brown, very dense, moist, fine- to medium-grained sand [ALLUVIUM]											

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Expect Excellence

SOIL BORING 1OS-B3

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/16/2024
HOLE DEPTH: 151.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						78								
	-150													
145			**SANDY LEAN CLAY** (**CL**), greenish gray, very stiff, moist, ~45% medium-grained sand, iron oxide mottling [OLD BAY CLAY]											
	-155													
150						30								

End of boring at approximately 151½ feet below ground surface. Groundwater was not measured due to drilling method.

SOIL BORING 1OS-B4

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/13/2024
HOLE DEPTH: 151.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			SANDY SILT (**ML**), gray, very soft, wet, shells [YOUNG BAY MUD]			WOH				38.2	83.3		69	
5	-10													
	-15		**SILTY SAND** (**SM**), olive, medium dense, moist, fine-grained sand, ~20% fines [YOUNG BAY MUD]			12								
10			more clayey			24				15.50				
	-20		**LEAN CLAY WITH SAND** (**CL**), yellowish brown, stiff, moist, ~20% fine-grained sand, pale olive staining [ALLUVIUM]											
15			hard			16				26.0	99.8			
			SILTY SAND (**SM**), yellowish brown, medium dense, wet, fine-grained sand, ~30% fines [ALLUVIUM]			500psi								
	-25													

ENGEO
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SOIL BORING 1OS-B4

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/13/2024
HOLE DEPTH: 151.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			LEAN CLAY (**CL**), yellowish brown, hard, moist, iron oxide staining [ALLUVIUM]			19								
	-30													
25			**SAND SILT** (**ML**), gray, moist, ~40% fine-grained sand [OLD BAY CLAY]			500psi				24.7				
			POORLY GRADED SAND WITH SILT (**SP-SM**), olive, medium dense, wet, fine- to medium-grained sand, ~10% fines, pale olive staining [OLD BAY CLAY]			20								
	-35		**LEAN CLAY WITH SAND** (**CL**), olive, very stiff, moist, ~15% fine-grained sand, pale olive staining [OLD BAY CLAY]											
30			reddish orange staining			22				25.3	100.7			
			SILTY SAND (**SM**), olive gray, medium dense, wet, fine-grained sand, ~15% fines [OLD BAY CLAY]			200psi								
	-40													
35						29								
			LEAN CLAY (**CL**), olive, very stiff, moist, ~10% fine-grained sand, iron oxide staining [OLD BAY CLAY]											
	-45													

SOIL BORING 1OS-B4

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/13/2024
HOLE DEPTH: 151.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)

LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						32								
						400psi				29.6	92.0		1477	
	-50													
45														
			FAT CLAY (**CH**), greenish gray, very stiff, moist, trace fine-grained sand, reddish orange and pale olive staining [OLD BAY CLAY]											
	-55													
50						26								
						200psi								
	-60									28.6	94.7		3005	
55														
	-65													

SOIL BORING 1OS-B4

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/13/2024
HOLE DEPTH: 151.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)

LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
	-70													
65			Olive staining			19								
			Shells			200psi								
	-75													
70			Hard											
	-80													
75			Very stiff, iron oxide mottling			23				42.6	80.0		738	
	-85		**SANDY LEAN CLAY** (**CL**), yellowish brown, hard, moist, ~30% fine-grained sand [ALLUVIUM]											

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SOIL BORING 1OS-B4

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/13/2024
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HOLE DIAMETER: 6 in
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LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			iron oxide staining ~15% fine-grained sand			30								
	-90													
85														
	-95													
90			~30% fine- to coarse-grained sand, 10% subangular fine gravel			34								
	-100													
95			**LEAN CLAY WITH SAND** (**CL**), olive, very stiff to hard, moist, ~20% fine-grained sand [OLD BAY CLAY]											
	-105													

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SOIL BORING 1OS-B4

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/13/2024
HOLE DEPTH: 151.5 ft
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SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			Hard, trace fine-grained sand, manganese staining, iron oxide and olive staining			38								
			~20% fine-grained sand											
	-110													
105														
			LEAN CLAY (**CL**), olive, very stiff to hard, moist, ~20% fine-grained sand [OLD BAY CLAY]											
	-115													
110						48				27.9	96.4			
	-120													
115														
	-125													

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SOIL BORING 1OS-B4

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/13/2024
HOLE DEPTH: 151.5 ft
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SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			FAT CLAY (**CH**), gray, very stiff, moist, trace fine-grained sand [OLD BAY CLAY]			30								
	-130													
125														
	-135		iron oxide mottling											
130			**LEAN CLAY WITH SAND** (**CL**), yellowish brown, hard, moist, ~25% fine-grained sand [ALLUVIUM]			75								
	-140													
135						26								
						500psi				19.2	110.8			
	-145													

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SOIL BORING 1OS-B4

LATITUDE: 37.86237 LONGITUDE: -122.31962

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/13/2024
HOLE DEPTH: 151.5 ft
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DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
145	-150		olive and iron oxide mottling			72								
150	-155					53								

End of boring at approximately 151½ feet below ground surface. Groundwater was not measured due to drilling method.

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SOIL BORING 1OS-B5

LATITUDE: 37.86187 LONGITUDE: -122.31995

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/12/2024
HOLE DEPTH: 134 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			SANDY SILT (**ML**), gray, very soft, wet, ~30% fine-grained sand [YOUNG BAY MUD]											
	-10					WOH				35.1	78.3		91	
5														
			SILTY SAND (**SM**), gray, medium dense, wet, fine-grained sand, ~40% fines [YOUNG BAY MUD]											
	-15					200psi								
			SILTY SAND (**SM**), olive brown to yellow brown, medium dense, wet, fine-grained sand, 25% fines [ALLUVIUM]											
10														
			LEAN CLAY (**CL**), yellowish brown, stiff to very stiff, moist, ~10% fine-grained sand, maganese and pale olive staining [ALLUVIUM]											
						11								
	-20													
15														
						300psi								
	-25		**SILTY SAND** (**SM**), olive brown, medium dense, moist, ~20% fines, shells, iron oxide staining [ALLUVIUM]											
						14			31	23.0				

SOIL BORING 1OS-B5

LATITUDE: 37.86187 LONGITUDE: -122.31995

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/12/2024
HOLE DEPTH: 134 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						14			31	23.0				
			SANDY LEAN CLAY (**CL**), yellowish brown, very stiff, moist, ~30% fine-grained sand [ALLUVIUM]			28								
	-30		**SILTY SAND** (**SM**), yellowish brown, medium dense, moist, fine-grained sand, ~20% fines [ALLUVIUM]											
25			**FAT CLAY WITH SAND** (**CH**), olive gray, stiff to very stiff, moist, ~20% fine-grained sand [OLD BAY CLAY]			31								
			Olive, stiff, shells											
	-35					400psi				25.9	96.8		1986	
30														
			SANDY LEAN CLAY (**CL**), olive-brown, stiff, moist, ~35% fine-grained sand [ALLUVIUM]			23								
	-40													
35			**FAT CLAY** (**CH**), gray, very stiff, moist, ~10% fine-grained sand, maganese staining [OLD BAY CLAY]											
										26.2	99.2		1405	
						15								
	-45		no sand											

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SOIL BORING 1OS-B5

LATITUDE: 37.86187 LONGITUDE: -122.31995

Berkeley Water Transportation Pier Ferry
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DATE DRILLED: 09/12/2024
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SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			SANDY LEAN CLAY (**CL**), olive, meidum stiff, wet, ~40% fine- to medium-grained sand [OLD BAY CLAY]											
	-50		**POORLY GRADED SAND WITH SILT** (**SP-SM**), olive, dense, wet, ~10% fines [OLD BAY CLAY]			500psi								
45						35								
			FAT CLAY (**CH**), greenish gray, very stiff, moist, maganese staining [OLD BAY CLAY]			26								
	-55													
50														
	-60					300psi								
55														
	-65													

SOIL BORING 1OS-B5

LATITUDE: 37.86187 LONGITUDE: -122.31995

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/12/2024
HOLE DEPTH: 134 ft
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SURFACE ELEV.: -6 ft (MLLW)

LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
	-70					100psi				47.1	74.3		1483	
65														
	-75		shells **SANDY LEAN CLAY** (**CL**), yellowish brown, very stiff, moist, ~30% fine- to medium-grained sand [ALLUVIUM] no sand, iron oxide mottling			29								
70														
	-80		olive-brown and yellow brown mottling			25				40.2	80.7			
75														
			FAT CLAY (**CH**), olive, stiff, moist, yellowish brown mottling [OLD BAY CLAY]			300psi								
	-85		**LEAN CLAY** (**CL**), yellowish brown, stiff, moist,											

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SOIL BORING 1OS-B5

LATITUDE: 37.86187 LONGITUDE: -122.31995

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/12/2024
HOLE DEPTH: 134 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			~10% fine-grained sand, maganese staining [ALLUVIUM]											
			wet			38								
	-90		**WELL-GRADED SAND WITH SILT** (**SW-SM**), yellowish brown, moist, fine- to medium-grained sand, ~10% fines [ALLUVIUM]											
85			**LEAN CLAY WITH SAND** (**CL**), yellowish brown, very stiff, moist, ~15% fine-grained sand, maganese staining [ALLUVIUM]											
						30								
	-95													
90										19.8	105.9		3813	
	-100													
95														
			SILT WITH SAND (**ML**), olive, hard, moist, ~20% fine-grained sand, sandstone fragments [OLD BA CLAY]			59								
	-105													

SOIL BORING 1OS-B5

LATITUDE: 37.86187 LONGITUDE: -122.31995

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/12/2024
HOLE DEPTH: 134 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			LEAN CLAY (**CL**), yellowish brown, very stiff, moist, trace fine-grained sand, trace sand [ALLUVIUM]											
	-110													
105														
						53				20.5	108.8		2649	
	-115		~20% fine-grained sand											
110														
			FAT CLAY (**CH**), greenish gray, very stiff, moist [OLD BAY CLAY]											
	-120													
115														
			yellowish brown staining			25								
	-125		gray, no staining											
						200psi								

ENGEO
Expect Excellence

SOIL BORING 1OS-B5

LATITUDE: 37.86187 LONGITUDE: -122.31995

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/12/2024
HOLE DEPTH: 134 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						200psi				46.0	74.9		2246	
	-130													
125														
	-135													
130														
	-140		**LEAN CLAY WITH SAND** (**CL**), yellowish brown, very stiff, moist, ~20% fine-grained sand [ALLUVIUM]			35								

End of boring at approximately 134 feet below ground surface. Groundwater was not measured due to drilling method.

ENGEO
Expect Excellence

SOIL BORING 1OS-B6

LATITUDE: 37.86152 LONGITUDE: -122.3205

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/11/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0			**FAT CLAY WITH SAND** (**CH**), gray, very soft, wet, ~25% fine-grained sand, shells [YOUNG BAY MUD]			WOH								
	-10													
5						WOH								
	-15		**SILTY SAND** (**SM**), gray, loose, wet, fine-grained sand, ~40% fines [YOUNG BAY MUD]											
10			Olive gray, medium dense			9								
	-20													
15						47			8.1	17.6	117.3			
			POORLY GRADED SAND WITH SILT (**SP-SM**), olive brown, dense, wet, fine-grained sand, ~10% fines [YOUNG BAY MUD]											
	-25													

ENGEO
Expect Excellence

SOIL BORING 1OS-B6

LATITUDE: 37.86152 LONGITUDE: -122.3205

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/11/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			SANDY LEAN CLAY (**CL**), olive brown to yellowish brown, very stiff, moist, ~35% fine-grained sand, maganese and iron oxide staining [ALLUVIUM]			24								
			2" sand lense											
	-30													
25			yellowish brown			20	28	14						
			2" sand lense											
			LEAN CLAY (**CL**), yellowish brown, very stiff, moist, ~10% fine-grained sand, maganese staining [ALLUVIUM]											
	-35													
30						31								
	-40													
35						19				28.2	96.0			
	-45													

ENGEO
Expect Excellence

SOIL BORING 1OS-B6

LATITUDE: 37.86152 LONGITUDE: -122.3205

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/11/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			FAT CLAY WITH SAND (**CH**), olive gray, stiff, moist, ~15% fine-grained sand [OLD BAY CLAY]			21								
						17								
	-50													
45			trace sand			200psi								
	-55													
50			**CLAYEY SAND** (**SC**), gray, medium dense, wet, fine-grained sand, ~20% fines [OLD BAY DEPOSITS]			500psi								
	-60		**FAT CLAY** (**CH**), greenish gray, very stiff, moist [OLD BAY CLAY]											
55						29				20.9	107.8			
	-65													

ENGEO
Expect Excellence

SOIL BORING 1OS-B6

LATITUDE: 37.86152 LONGITUDE: -122.3205

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/11/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			Shells			300psi								
	-70													
65														
	-75													
70						200psi				44.5	77.2		2036	
	-80													
75			Olive-gray, iron staining			23								
	-85													

SOIL BORING 1OS-B6

LATITUDE: 37.86152 LONGITUDE: -122.3205

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/11/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)

LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			Pale olive staining			36								
	-90													
85			Fragments of sandstone and claystone, reddish orange staining			29								
	-95													
90			**LEAN CLAY** (**CL**), yellowish brown, stiff, moist, maganese staining, trace sands [ALLUVIUM]			48								
						14								
	-100													
95														
	-105													

ENGEO
Expect Excellence

SOIL BORING 1OS-B6

LATITUDE: 37.86152 LONGITUDE: -122.3205

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/11/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			trace angular gravel			500psi								
			SANDY LEAN CLAY (**CL**), olive, stiff, wet, ~25% medium- to coarse-grained sand, yellowish brown staining [OLD BAY CLAY]											
	-110													
105			**LEAN CLAY** (**CL**), yellowish brown, very stiff, moist, ~5% fine-grained sand, claystone fragments, reddish orange and pale olive staining [ALLUVIUM]			28								
			sand lense											
	-115													
110						400psi								
	-120		**SILTY SAND** (**SM**), yellowish brown, dense, wet, fine-grained sand, ~25% fines [ALLUVIUM]											
115						26								
			SANDY LEAN CLAY (**CL**), yellowish brown, very stiff, moist, ~35% fine-grained sand, iron oxide and maganese staining [ALLUVIUM]											
	-125		**FAT CLAY** (**CH**), greenish gray, stiff, moist, iron oxide staining [OLD BAY CLAY]											

SOIL BORING 1OS-B6

LATITUDE: 37.86152 LONGITUDE: -122.3205

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/11/2024
HOLE DEPTH: 121.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						13								

End of boring at approximately 121½ feet below ground surface.

SOIL BORING 1OS-CPT1

LATITUDE: 37.86271 LONGITUDE: -122.31836

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/09/2024
HOLE DEPTH: 101 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			Pushed CPT											
5	-10													
10	-15													
15	-20		**CLAYEY SAND** (**SC**), yellowish brown, medium dense, wet, fine- to medium-grained sand, maganese staining [ALLUVIUM]			12	28	13	45.1	18.70				
			LEAN CLAY (**CL**), yellowish brown, medium stiff, wet, maganese staining [ALLUVIUM]											
	-25					6								

ENGEO
Expect Excellence

SOIL BORING 1OS-CPT1

LATITUDE: 37.86271 LONGITUDE: -122.31836

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/09/2024
HOLE DEPTH: 101 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			LEAN CLAY WITH SAND (**CL**), olive brown to olive gray, stiff, moist, ~15% fine-grained sand [OLD BAY CLAY]			6								
			Greenish gray, very stiff			34								
	-30		Pushed CPT											

SOIL BORING 1OS-CPT2

LATITUDE: 37.8626 LONGITUDE: -122.31967

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 134 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			Pushed CPT											
	-10													
5														
	-15													
10														
	-20													
15			**SILTY SAND** (**SM**), yellowish brown, medium dense, wet, ~35% fines [ALLUVIUM]			17								
	-25		**LEAN CLAY WITH SAND** (**CL**), yellowish brown, very stiff, wet, ~30% fine-grained sand [ALLUVIUM]											

ENGEO
Expect Excellence

SOIL BORING 1OS-CPT2

LATITUDE: 37.8626 LONGITUDE: -122.31967

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 134 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						17								
	-30													
25			~15% fine-grained sand			12								
	-35													
30						18				28.1	96.4			
	-40		**LEAN CLAY WITH SAND** (**CL**), olive-brown, hard, moist, ~15% fine-grained sand [ALLUVIUM]											
35						23								
	-45		**FAT CLAY** (**CH**), greenish gray, stiff, moist [OLD BAY CLAY]											

ENGEO
Expect Excellence

SOIL BORING 1OS-CPT2

LATITUDE: 37.8626 LONGITUDE: -122.31967

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/05/2024
HOLE DEPTH: 134 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
						17								
	-50													
-45			Pushed CPT											

ENGEO
Expect Excellence

SOIL BORING 1OS-CPT3

LATITUDE: 37.86153 LONGITUDE: -122.32103

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/10/2024
HOLE DEPTH: 101.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			CPT											
	-10		**SILTY SAND** (**SM**), gray, loose, wet, fine-grained sand, ~30% fines, shells [YOUNG BAY MUD]											
5						31								
	-15		**SANDY LEAN CLAY** (**CL**), olive brown, very stiff, moist, ~40% fine-grained sand [ALLUVIUM]											
10			**SILTY SAND** (**SM**), olive brown, medium dense, wet, fines, reddish orange mottling, gray staining [ALLUVIUM]											
						26			26	16.1				
	-20													
15			**SANDY LEAN CLAY** (**CL**), yellowish brown, stiff, moist, ~35% fine-grained sand [ALLUVIUM]											
						27								
	-25		**SILTY SAND** (**SM**), yellowish brown, medium dense, moist, fine-grained sand, ~30% fines [ALLUVIUM]											

ENGEO
Expect Excellence

SOIL BORING 1OS-CPT3

LATITUDE: 37.86153 LONGITUDE: -122.32103

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/10/2024
HOLE DEPTH: 101.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			POORLY GRADED SAND (**SP**), yellowish brown, medium dense, wet, fine-grained sand, trace fines [ALLUVIUM]											
			SILTY SAND (**SM**), yellowish brown, dense, moist, fine-grained sand, ~25% fines [ALLUVIUM]			56								
	-30													
25			**LEAN CLAY WITH SAND** (**CL**), yellowish brown, very stiff, moist, ~25% fine-grained sand, reddish orange staining [ALLUVIUM]											
						23								
	-35		**LEAN CLAY WITH SAND** (**CL**), olive-gray, very stiff, moist, ~15% fine-grained sand [OLD BAY CLAY]											
30			CPT											
	-40													
35														
	-45													

ENGEO
Expect Excellence

SOIL BORING 1OS-CPT3

LATITUDE: 37.86153 LONGITUDE: -122.32103

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/10/2024
HOLE DEPTH: 101.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
45	-50													
50	-55													
55	-60													
	-65													

ENGEO
Expect Excellence

SOIL BORING 1OS-CPT3

LATITUDE: 37.86153 LONGITUDE: -122.32103

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/10/2024
HOLE DEPTH: 101.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
65	-70													
70	-75													
75	-80													
	-85													

ENGEO
Expect Excellence

SOIL BORING 1OS-CPT3

LATITUDE: 37.86153 LONGITUDE: -122.32103

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/10/2024
HOLE DEPTH: 101.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
	-90		**WELL-GRADED SAND** (**SW**), bluish gray, medium dense to dense, wet, medium- to coarse-grained sand, ~10% subrounded fine gravel [OLD BAY CLAY]											
85						29				18.5				
			claystone fragments											
	-95		**FAT CLAY WITH SAND** (**CH**), greenish gray, very stiff, moist [OLD BAY CLAY]											
90														
			LEAN CLAY WITH SAND (**CL**), yellowish brown, very stiff to hard, moist, ~15% fine- to medium-grained sand [ALLUVIUM]			44								
	-100													
95														
	-105													

ENGEO
Expect Excellence

SOIL BORING 1OS-CPT3

LATITUDE: 37.86153 LONGITUDE: -122.32103

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 09/10/2024
HOLE DEPTH: 101.5 ft
HOLE DIAMETER: 6 in
SURFACE ELEV.: -6 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0			**SANDY SILT** (**ML**), yellowish brown, hard, moist, reddish orange staining [ALLUVIUM]			78				16.8				
			End of boring at approximately 101½ feet below ground surface. Groundwater was not measured due to drilling method.											

SOIL BORING 2-B1

LATITUDE: 37.862953 LONGITUDE: -122.317585

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 02/07/2025
HOLE DEPTH: 30 ft
HOLE DIAMETER: 6.5 in
SURFACE ELEV.: 11 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ /
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Sonic
HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			ASPHALT (**AC**), 3-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 3-inch Aggregate Base											
	10		**GRAVELLY LEAN CLAY WITH SAND** (**CL**), light yellowish brown, very stiff, moist, ≈25% subangular to subrounded fine to coarse gravel, ≈15% fine- to medium-grained sand [FILL]											
5			brick fragments											
	5		grades more gravel											
			WELL-GRADED GRAVEL WITH SAND (**GW**), light yellowish brown, dry, subangular to subrounded fine to coarse gravel, ≈35% fine- to coarse-grained sand, rebar piece [ROCK DIKE]											
10	0		**CLAYEY GRAVEL WITH SAND** (**GC**), gray to pale olive, wet, subangular to subrounded fine to coarse gravel, ≈30% fine- to coarse-grained sand, ≈25% fines [ROCK DIKE]											
15	-5		**WELL-GRADED GRAVEL WITH SAND** (**GW**), light gray, dry, subangular to subrounded fine to coarse gravel, ≈30% fine- to coarse-grained sand [ROCK DIKE]											
			CLAYEY SAND WITH GRAVEL (**SC**), dark gray, dry to moist, fine- to coarse-grained sand, ≈30% subrounded fine to coarse gravel, ≈20% fines [ROCK DIKE]											
			6-inch seam of dry, coarse rounded gravel, up to 4-inch diameter clasts											

ENGEO
Expect Excellence

SOIL BORING 2-B1

LATITUDE: 37.862953 LONGITUDE: -122.317585

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 02/07/2025 LOGGED BY / REVIEWED BY: VZ /
HOLE DEPTH: 30 ft DRILLING CONTRACTOR: Gregg Drilling LLC
HOLE DIAMETER: 6.5 in DRILLING METHOD: Sonic
SURFACE ELEV.: 11 ft (MLLW) HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			FAT CLAY WITH SAND (**CH**), gray, soft to medium stiff, moist, ≈25% shells, ≈20% fine-grained sand [YOUNG BAY MUD]											
	-10													
25	-15		**LEAN CLAY WITH SAND** (**CL**), light yellowish brown, hard, moist, ≈15% fine- to medium-grained sand, maganese staining [ALLUVIUM]											
			CLAYEY SAND (**SC**), light yellowish brown, moist, fine-grained sand, ≈40% fines [ALLUVIUM]											
30			End of boring at approximately 30 feet below ground surface. Groundwater was encountered at 7 feet below ground surface.											

SOIL BORING 2-B2

LATITUDE: 37.862961 LONGITUDE: -122.317568

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 02/08/2025
HOLE DEPTH: 8 ft
HOLE DIAMETER: 6.5 in
SURFACE ELEV.: 11.5 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Sonic
HAMMER TYPE: N/A

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			ASPHALT (**AC**), 3-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 3-inch Aggregate Base											
	10		**LEAN CLAY WITH SAND** (**CL**), light yellowish brown, stiff, wet, ≈20% fine- to medium-grained sand, ≈10% subangular fine to coarse gravel [FILL]											
5														
	5		**WELL-GRADED GRAVEL WITH SAND** (**GW**), light gray, moist, fine to coarse gravel, ≈35% fine- to coarse grained sand, up to 4-inch gravel clasts [ROCK DIKE]											

End of boring at approximately 8 feet below ground surface. Groundwater was encountered at approximately 8 feet below ground surface.

SOIL BORING 2-B3

LATITUDE: 37.862968 LONGITUDE: -122.317538

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 02/08/2025
HOLE DEPTH: 25 ft
HOLE DIAMETER: 6.5 in
SURFACE ELEV.: 11.5 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Sonic
HAMMER TYPE: N/A

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0	10		**ASPHALT** (**AC**), 3-inch Asphalt Concrete, **AGGREGATE BASE** (**AB**), 3-inch Aggregate Base, **SANDY LEAN CLAY WITH GRAVEL** (**CL**), light yellowish brown, stiff, wet, ≈30% fine- to medium-grained sand, ≈15% subangular to subrounded fine to coarse gravel, [FILL]											
5	5		**SANDY LEAN CLAY WITH GRAVEL** (**CL**), dark gray, moist, fine- to coarse-grained sand, ≈20% fine- to medium-grained sand, ≈15% subangular to subrounded fine gravel [FILL]											
10	0		**SILTY SAND WITH GRAVEL** (**SM**), dark gray, fine- to coarse-grained sand, ≈35% subangular to angular fine to coarse gravel, ≈15% fines [ROCK DIKE]											
15	-5													

ENGEO
Expect Excellence

SOIL BORING 2-B3

LATITUDE: 37.862968 LONGITUDE: -122.317538

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 02/08/2025
HOLE DEPTH: 25 ft
HOLE DIAMETER: 6.5 in
SURFACE ELEV.: 11.5 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Sonic
HAMMER TYPE: N/A

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			FAT CLAY WITH SAND (**CH**), gray, soft, moist, ≈20% fine-grained sand [YOUNG BAY MUD]											
	-10													
25			End of boring at approximately 25 feet below ground surface. Groundwater was encountered at approximately 8 feet below ground surface.											

ENGEO
Expect Excellence

SOIL BORING 2-B4

LATITUDE: 37.86298 LONGITUDE: -122.317497

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 02/08/2025
HOLE DEPTH: 30 ft
HOLE DIAMETER: 6.5 in
SURFACE ELEV.: 11.5 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Sonic
HAMMER TYPE: N/A

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0	10		**GRAVELLY LEAN CLAY WITH SAND** (**CL**), light yellowish brown, stiff, moist to wet, ≈25% subangular to subrounded fine to coarse gravel, ≈15% fine- to medium-grained sand, [FILL]											
5	5		**POORLY GRADED GRAVEL WITH SAND** (**GP**), gray, dry, subangular to subrounded fine to coarse gravel, up to 4½-inch clasts, [ROCK DIKE]											
10	0		**CLAYEY GRAVEL WITH SAND** (**GC**), dark yellowish brown to dark gray, wet, subangular to subrounded fine to coarse gravel, ≈45% fines, ≈15% fine- to coarse-grained sand, up to 4½ inch clasts, light gray staining [ROCK DIKE] No recovery from 10 to 13½ feet below existing ground surface											
15	-5		becomes dark yellowish brown, ≈35% fine- to coarse-grained sand, becomes dark gray becomes gray, presence of fibrous rootlets											
			CLAYEY SAND WITH GRAVEL (**SC**), dark gray, moist, fine- to coarse-grained sand, ≈25% fines, ≈25% subrounded fine to coarse gravel [ROCK DIKE]											

SOIL BORING 2-B4

LATITUDE: 37.86298 LONGITUDE: -122.317497

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 02/08/2025
HOLE DEPTH: 30 ft
HOLE DIAMETER: 6.5 in
SURFACE ELEV.: 11.5 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Sonic
HAMMER TYPE: N/A

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
			≈25% subrounded fine to coarse gravel [ROCK DIKE]											
	-10		**FAT CLAY** (**CH**), gray, soft, moist, ≈10% fine- to medium-grained sand, ≈10% shells [YOUNG BAY MUD]											
			CLAYEY SAND (**SC**), gray, moist, fine- to coarse-grained sand, ≈30% fines [YOUNG BAY MUD]											
25			**FAT CLAY** (**CH**), gray, soft, moist, ≈10% fine- to medium-grained sand, ≈10% shells [YOUNG BAY MUD]											
	-15		**SANDY LEAN CLAY** (**CL**), light yellowish brown, stiff, moist, ≈35% fine- to coarse-grained sand [ALLUVIUM]											
			CLAYEY SAND (**SC**), light yellowish brown, wet, fine- to coarse-grained sand, ≈35% fines, ≈10% subrounded fine to coarse gravel [ALLUVIUM]											
30			End of boring at approximately 30 feet below ground surface. Groundwater was encountered at approximately 7 feet below ground surface.											

ENGEO
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SOIL BORING 2-B5

LATITUDE: 37.863006 LONGITUDE: -122.31733

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 02/08/2025
HOLE DEPTH: 20 ft
HOLE DIAMETER: 6.5 in
SURFACE ELEV.: 14.5 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Sonic
HAMMER TYPE: -

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0	10		**ASPHALT** (**AC**), 4½-inch Asphalt Concrete **AGGREGATE BASE** (**AB**), 5-inch Aggregate Base **LEAN CLAY WITH SAND** (**CL**), yellowish brown, moist, ≈25% fine- to coarse-grained sand, brick fragments [FILL]											
5														
10	5		**SILTY GRAVEL WITH SAND** (**GM**), dark gray, wet, subangular to subrounded fine gravel, ≈35% fine- to coarse-grained sand, ≈20% fines [FILL] 3½-inch subangular cobble											
			SANDY LEAN CLAY (**CL**), pale olive, moist, ≈40% fine-grained sand [FILL]											
15	0		**SILTY GRAVEL** (**GM**), dark gray, wet, subangular to subrounded fine to coarse gravel, ≈20% fines [FILL]											
	-5													

ENGEO
Expect Excellence

SOIL BORING 2-B6

LATITUDE: 37.863009 LONGITUDE: -122.317313

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 02/08/2025
HOLE DEPTH: 15 ft
HOLE DIAMETER: 6.5 in
SURFACE ELEV.: 14.5 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Sonic
HAMMER TYPE: N/A

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0	10		**ASPHALT** (**AC**), 4½-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 6-inch Aggregate Base											
5			**LEAN CLAY WITH SAND** (**CL**), dark yellowish brown, moist, ≈20% fine- to medium-grained sand, dark gray stains, concrete and brick fragments [FILL]											
	5													
10			**SILTY GRAVEL WITH SAND** (**GM**), dark gray, wet, angular to subrounded fine to coarse gravel, ≈35% fine- to coarse-grained sand, ≈20% fines, up to 1½ inch clasts [FILL]											
	0													
15			End of boring at approximately 15 feet below ground surface. Groundwater was encountered at approximately 12 feet below ground surface.											

ENGEO
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SOIL BORING 2-B7

LATITUDE: 37.863027 LONGITUDE: -122.317221

Berkeley Water Transportation Pier Ferry
Berkeley, CA
25022.000.001

DATE DRILLED: 02/08/2025
HOLE DEPTH: 20 ft
HOLE DIAMETER: 6.5 in
SURFACE ELEV.: 15 ft (MLLW)
LOGGED BY / REVIEWED BY: VZ / JF
DRILLING CONTRACTOR: Gregg Drilling LLC
DRILLING METHOD: Sonic
HAMMER TYPE: N/A

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
0	15		**ASPHALT** (**AC**), 4½-inch Asphalt Concrete											
			AGGREGATE BASE (**AB**), 6-inch Aggregate Base											
			LEAN CLAY WITH SAND (**CL**), dark yellowish brown, stiff, wet, ≈20% fine- to medium-grained sand, textile fragments, dark gray stains [FILL]											
5	10													
			GRAVELLY LEAN CLAY (**CL**), dark yellowish brown, stiff, wet, ≈30% angular to subrounded fine gravel, fiborous roots [FILL]											
10	5		**SANDY FAT CLAY WITH GRAVEL** (**CH**), dark yellowish brown, soft, wet, ≈30% fine angular to subrounded fine gravel [FILL]											
			CLAYEY SAND WITH GRAVEL (**SC**), dark yellowish brown, moist, ≈40% fines, ≈20% subangular fine gravel, fiborous roots [FILL]											
15	0		**FAT CLAY** (**CH**), black, very soft, wet, organic odor [YOUNG BAY MUD]											

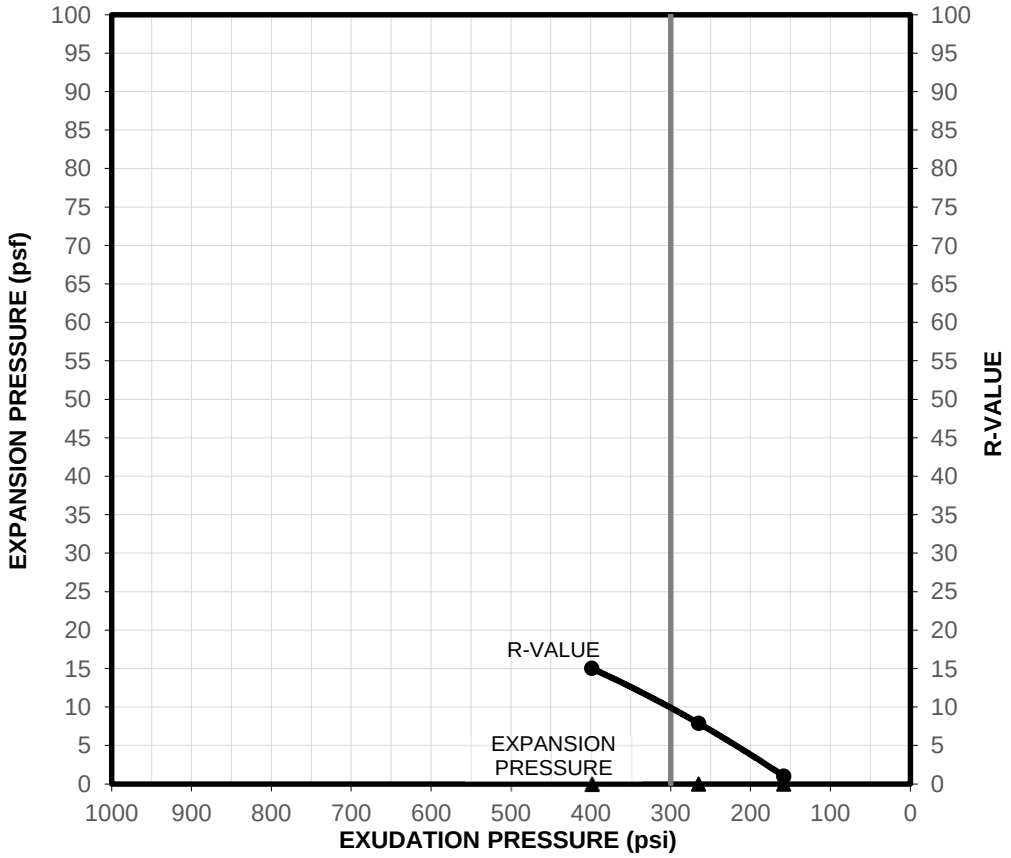


APPENDIX D

LABORATORY TEST RESULTS

R-VALUE TEST REPORT

CTM 301



SAMPLE ID	MATERIAL DESCRIPTION	SAMPLE LOCATION
1-C1@0-5	See exploration logs	1-C1 at 0 to 5 feet

SPECIMENS	1	2	3
EXUDATION PRESSURE (psi)	399	265	158
EXPANSION PRESSURE (psf)	0	0	0
R-VALUE	15	8	1
MOISTURE CONTENT (%)	15.1	17.3	19.1
DRY DENSITY (pcf)	116.1	112.7	108.3
EXPANSION PRESSURE (psf) AT EXUDATION PRESSURE OF 300 psi		0	

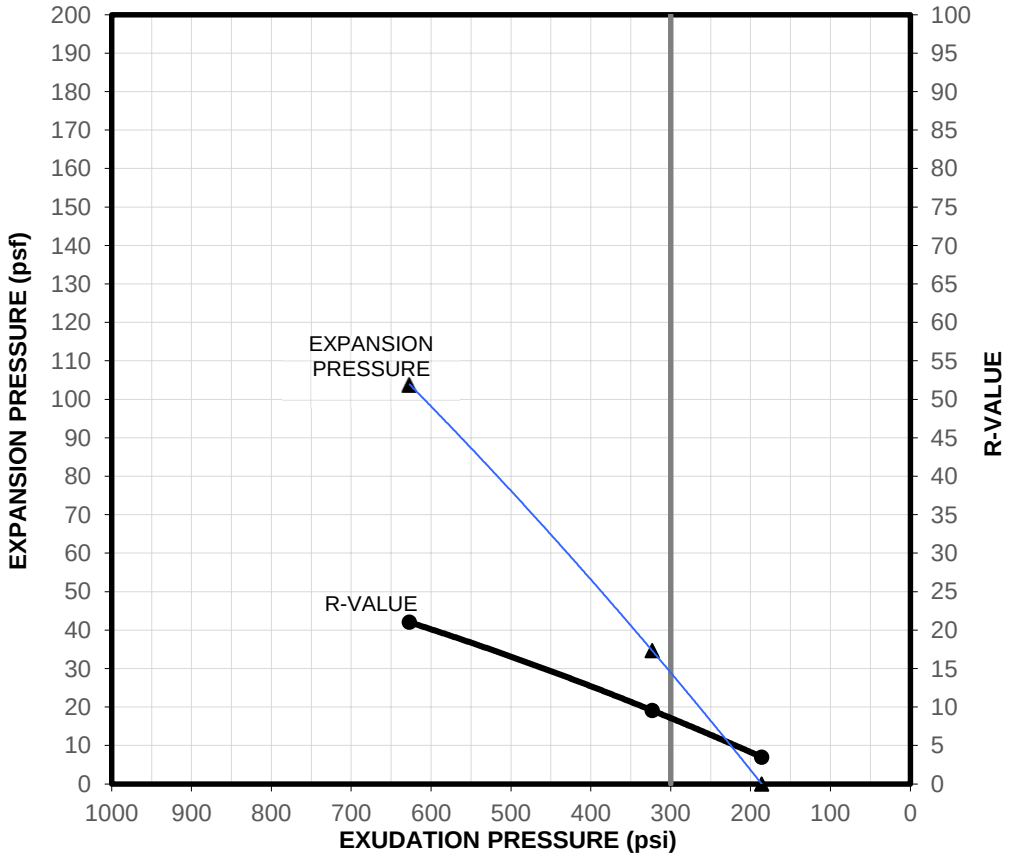
R-VALUE AT EXUDATION PRESSURE OF 300 psi	TEST RESULT
	10

ENGEO
Expect Excellence

CLIENT: COWI North American, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/22/2024
TESTED BY: M. Ryan
REVIEWED BY: M. Gilbert

R-VALUE TEST REPORT

CTM 301



SAMPLE ID	MATERIAL DESCRIPTION	SAMPLE LOCATION
1-C3@0-5	See exploration logs	1-C3 at 0 to 5 feet

SPECIMENS	1	2	3
EXUDATION PRESSURE (psi)	627	323	186
EXPANSION PRESSURE (psf)	104	35	0
R-VALUE	21	10	3
MOISTURE CONTENT (%)	14.7	16.0	17.8
DRY DENSITY (pcf)	118.5	114.0	109.9
EXPANSION PRESSURE (psf) AT EXUDATION PRESSURE OF 300 psi		29	

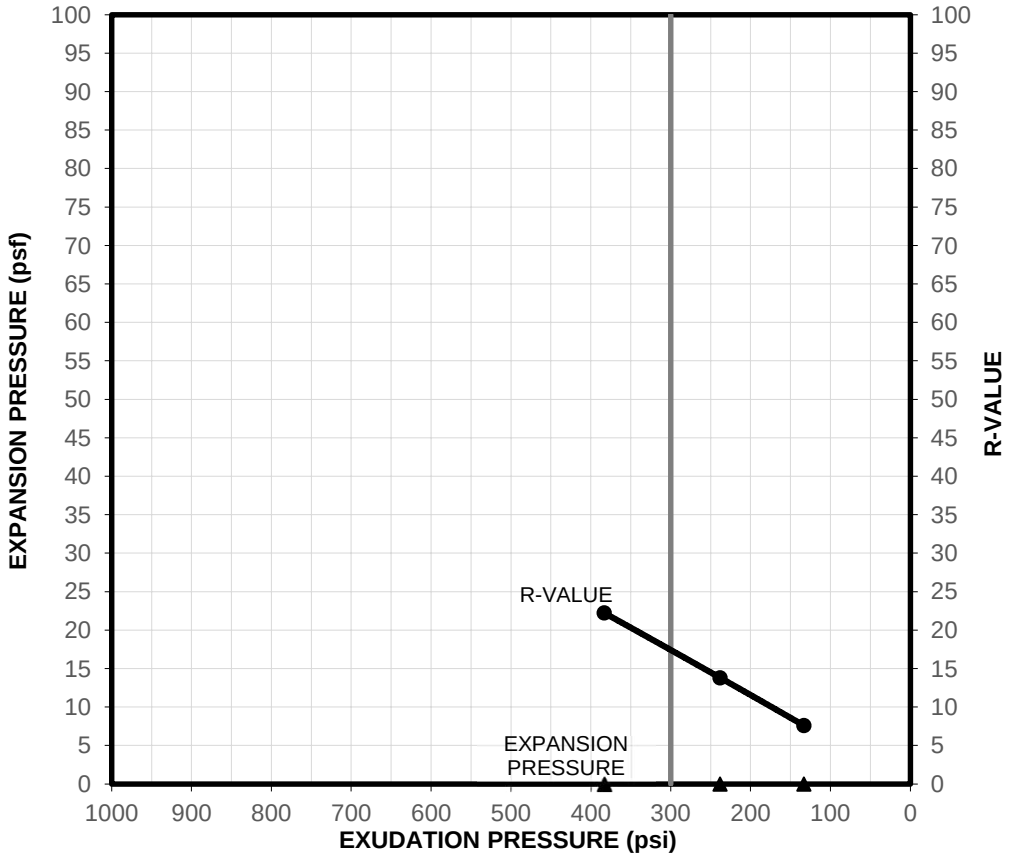
R-VALUE AT EXUDATION PRESSURE OF 300 psi	TEST RESULT
	9

ENGEO
Expect Excellence

CLIENT: COWI North American, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/17/2024
TESTED BY: M. Ryan
REVIEWED BY: M. Gilbert

R-VALUE TEST REPORT

CTM 301



SAMPLE ID	MATERIAL DESCRIPTION	SAMPLE LOCATION
1-C6 @ 0-5	See exploration logs	1-C6 at 0 to 5 feet

SPECIMENS	1	2	3
EXUDATION PRESSURE (psi)	383	238	133
EXPANSION PRESSURE (psf)	0	0	0
R-VALUE	22	14	8
MOISTURE CONTENT (%)	15.8	17.5	19.6
DRY DENSITY (pcf)	117.0	111.8	107.8
EXPANSION PRESSURE (psf) AT EXUDATION PRESSURE OF 300 psi		0	

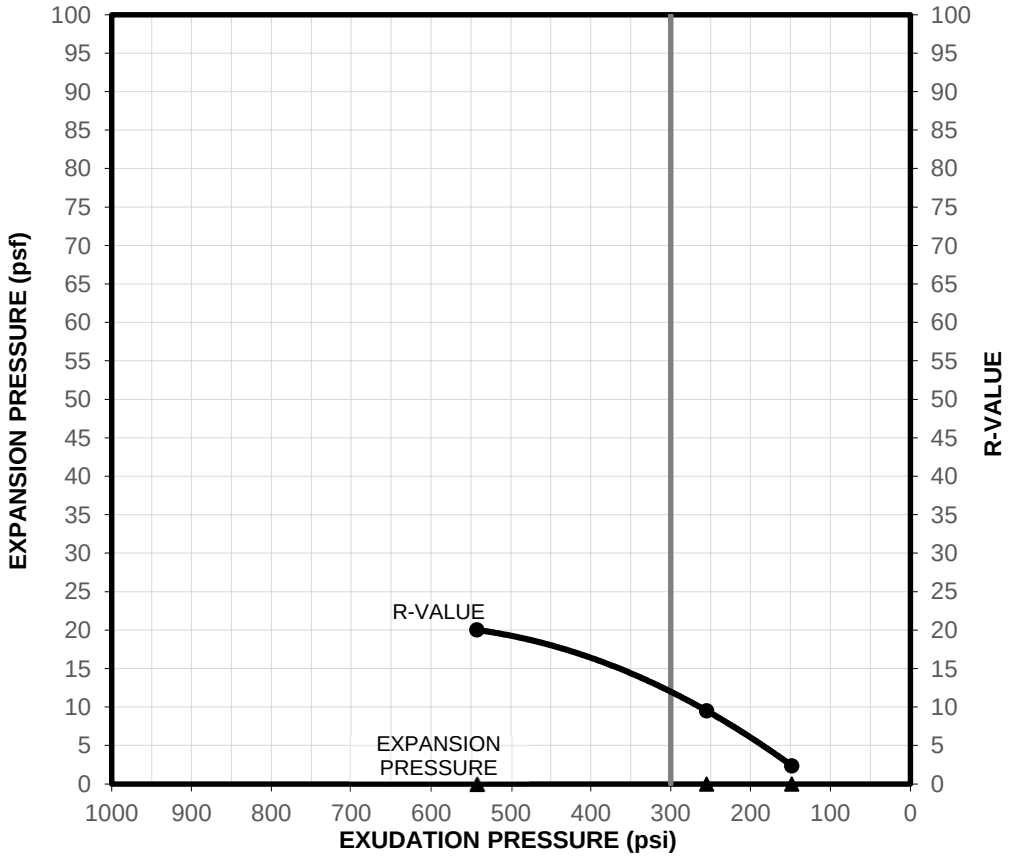
R-VALUE AT EXUDATION PRESSURE OF 300 psi	TEST RESULT
	17

ENGEO
Expect Excellence

CLIENT: COWI North American, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/22/2024
TESTED BY: M. Ryan
REVIEWED BY: M. Gilbert

R-VALUE TEST REPORT

CTM 301



SAMPLE ID	MATERIAL DESCRIPTION	SAMPLE LOCATION
1-C8@0-5	See exploration logs	1-C8 at 0 to 5 feet

SPECIMENS	1	2	3
EXUDATION PRESSURE (psi)	543	255	148
EXPANSION PRESSURE (psf)	0	0	0
R-VALUE	20	10	2
MOISTURE CONTENT (%)	13.8	15.0	16.9
DRY DENSITY (pcf)	118.3	114.3	109.3
EXPANSION PRESSURE (psf) AT EXUDATION PRESSURE OF 300 psi		0	

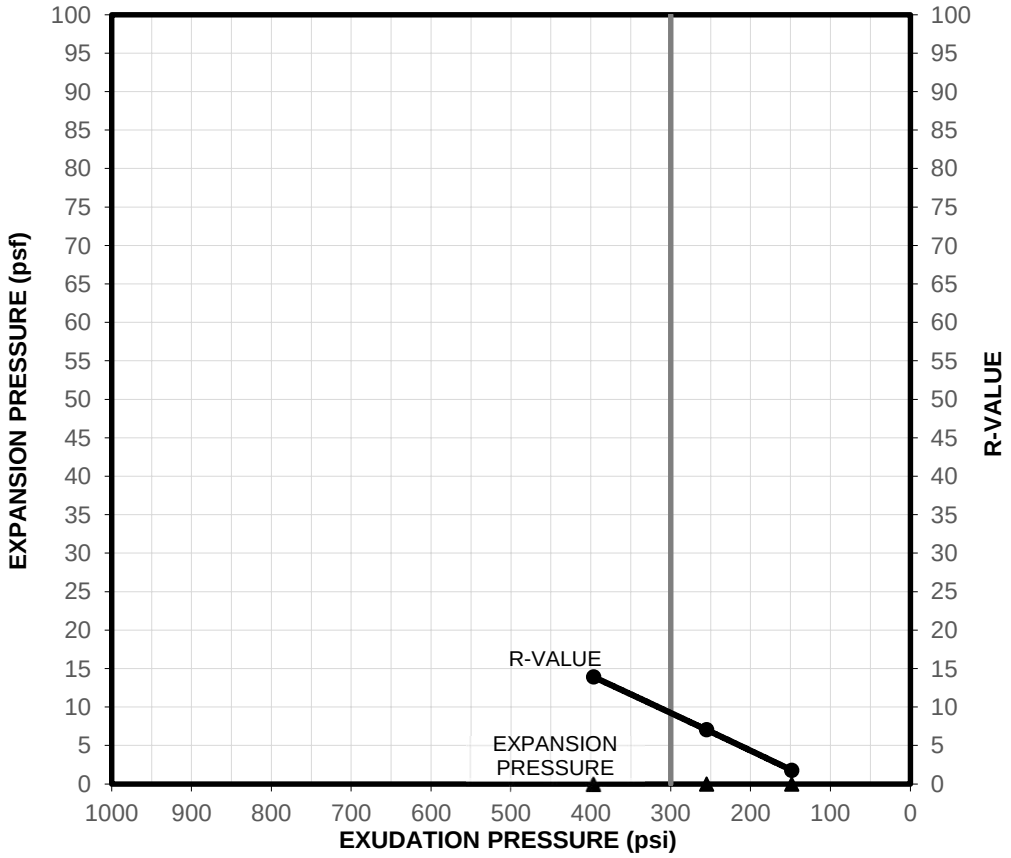
R-VALUE AT EXUDATION PRESSURE OF 300 psi	TEST RESULT
	12

ENGEO
Expect Excellence

CLIENT: COWI North American, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/17/2024
TESTED BY: M. Ryan
REVIEWED BY: M. Gilbert

R-VALUE TEST REPORT

CTM 301



SAMPLE ID	MATERIAL DESCRIPTION	SAMPLE LOCATION
1-B3 @ 0-5	See exploration logs	1-B3 at 0 to 5 feet

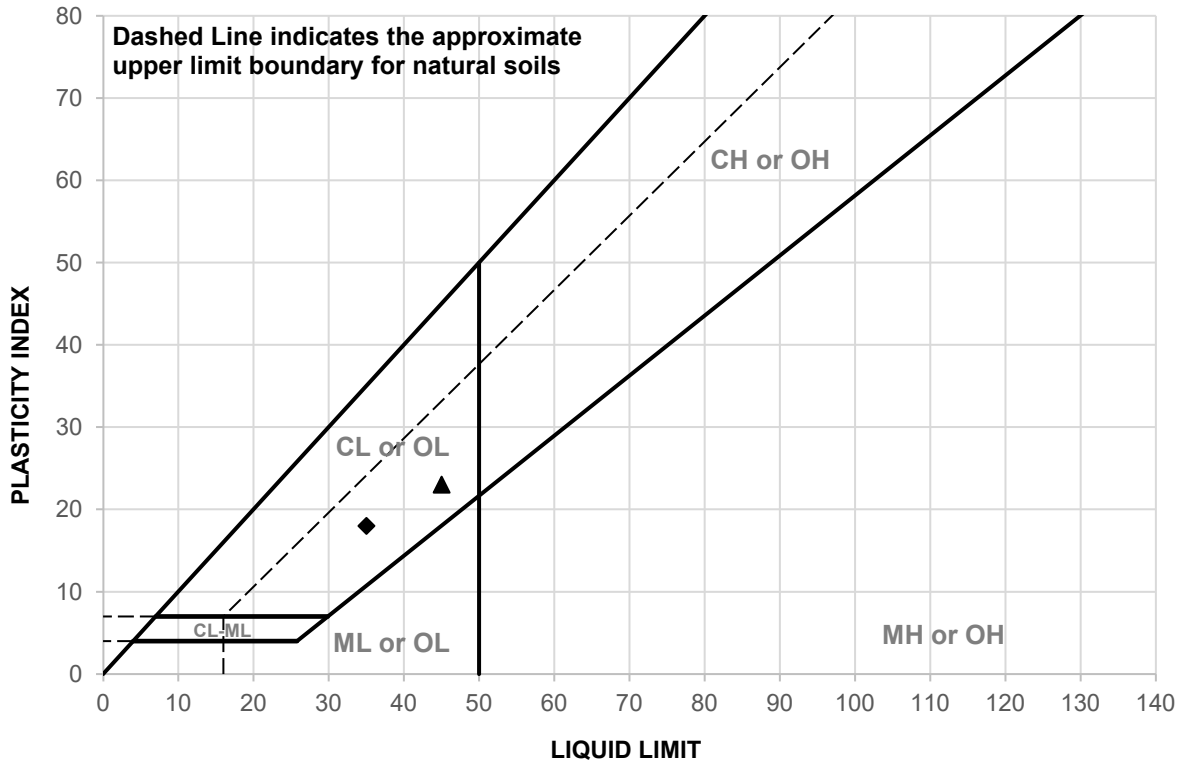
SPECIMENS	1	2	3
EXUDATION PRESSURE (psi)	397	255	148
EXPANSION PRESSURE (psf)	0	0	0
R-VALUE	14	7	2
MOISTURE CONTENT (%)	18.8	20.5	22.3
DRY DENSITY (pcf)	111.7	107.3	104.1
EXPANSION PRESSURE (psf) AT EXUDATION PRESSURE OF 300 psi		0	

R-VALUE AT EXUDATION PRESSURE OF 300 psi	TEST RESULT
	9

ENGEO
Expect Excellence

CLIENT: COWI North American, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/22/2024
TESTED BY: M. Ryan
REVIEWED BY: M. Gilbert

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



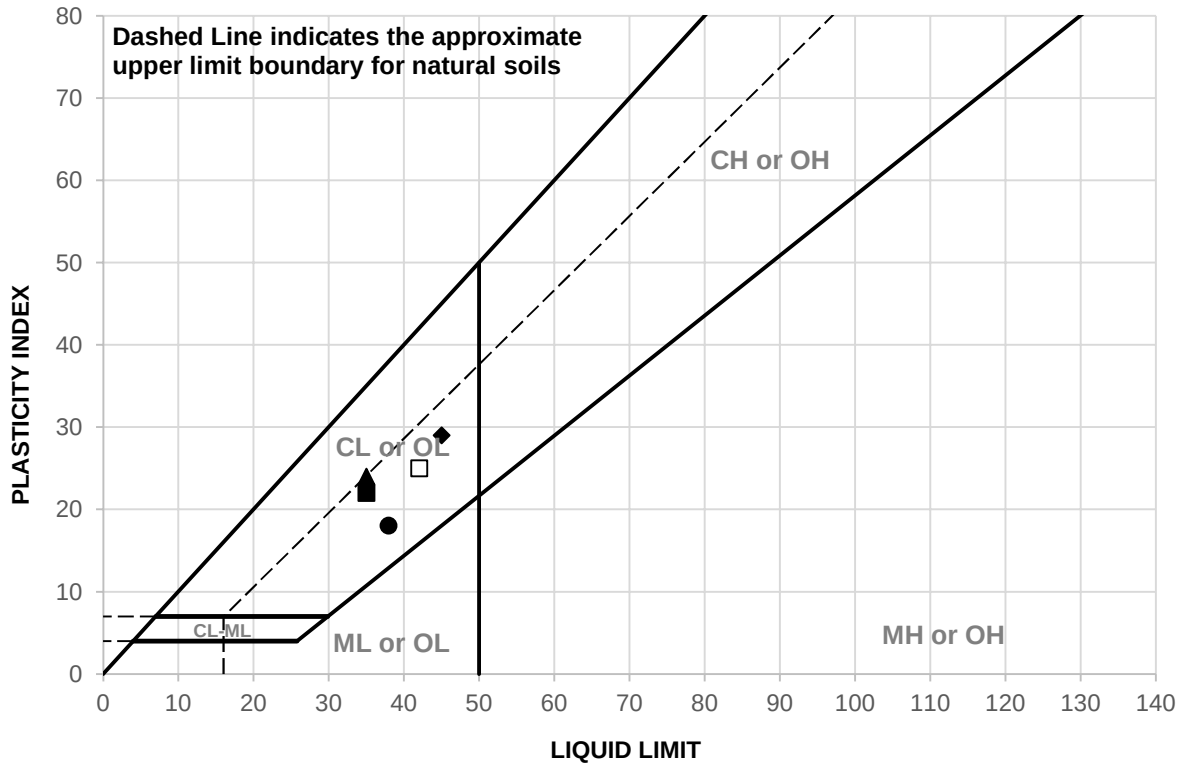
	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B4@10-12	10-12	See exploration logs	45	22	23
◆	1OS-B1@12-14	12-14	See exploration logs	35	17	18

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B4@10-12	PI: ASTM D4318, Wet Method	
◆	1OS-B1@12-14	PI: ASTM D4318, Wet Method	

ENGEO
Expect Excellence

CLIENT: COWI North America
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/11/2024
TESTED BY: S. Fikre-Selassie
REVIEWED BY: G. Criste

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



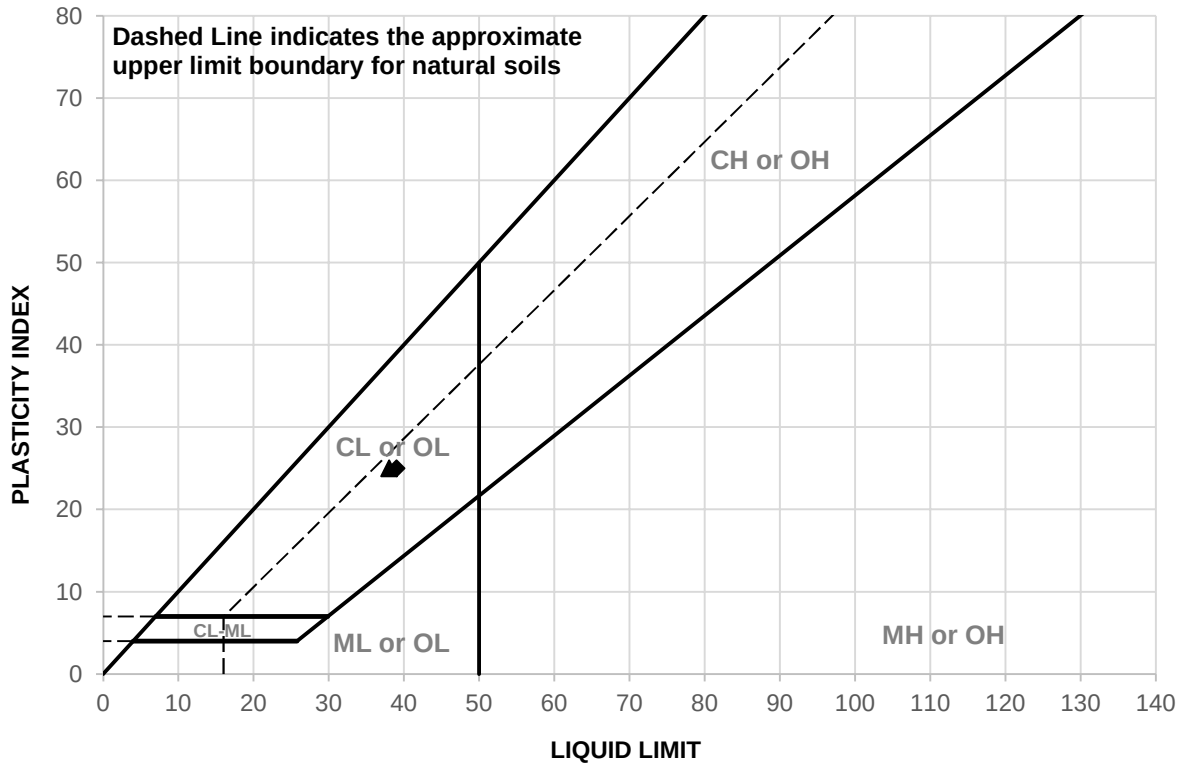
	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B1@6-6.5	6-6.5	See exploration logs	35	11	24
◆	1-B1@11-11.5	11-11.5	See exploration logs	45	16	29
□	1-B2@5.5-6	5.5-6	See exploration logs	42	17	25
●	1-B2@8.5-9	8.5-9	See exploration logs	38	20	18
■	1-B2@30-31.5	30-31.5	See exploration logs	35	13	22

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B1@6-6.5	PI: ASTM D4318, Wet Method	
◆	1-B1@11-11.5	PI: ASTM D4318, Wet Method	
□	1-B2@5.5-6	PI: ASTM D4318, Wet Method	
●	1-B2@8.5-9	PI: ASTM D4318, Wet Method	
■	1-B2@30-31.5	PI: ASTM D4318, Wet Method	

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/17/2024
TESTED BY: R. Montalvo
REVIEWED BY: M. Gilbert

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



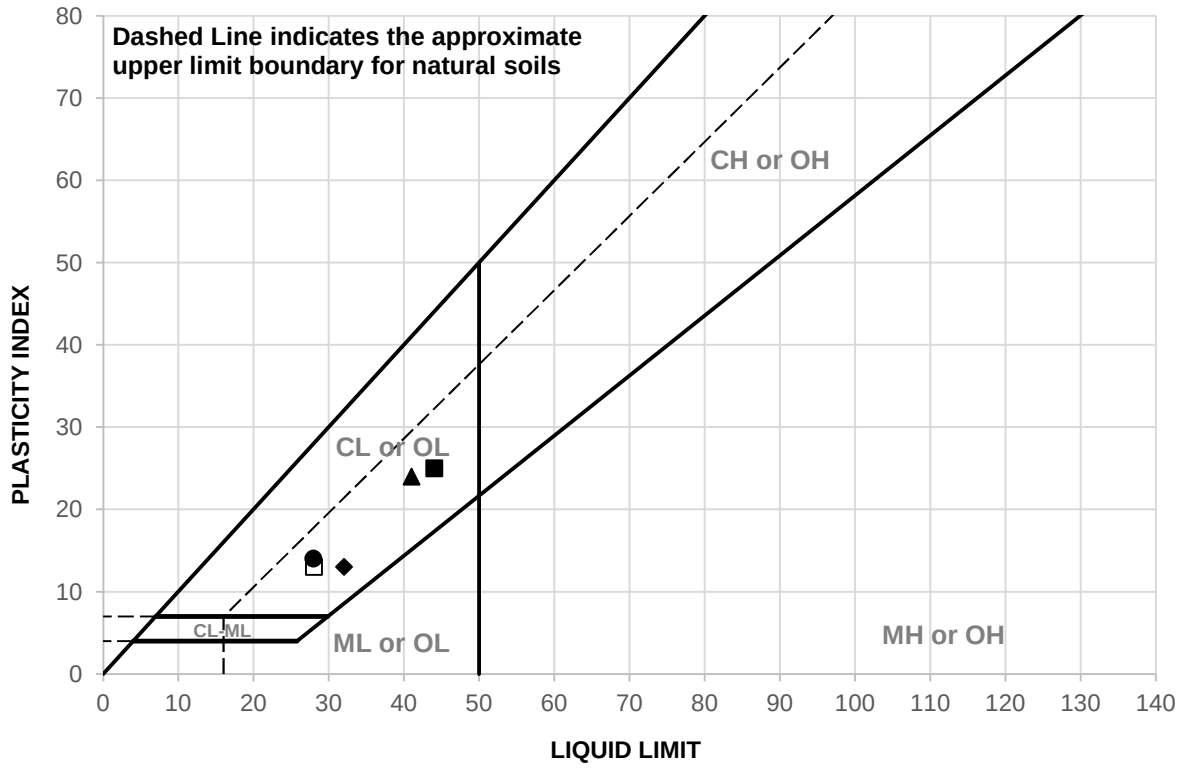
	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B3@6-6.5	6-6.5	See exploration logs	38	13	25
◆	1-B3@15-16.5	15-16.5	See exploration logs	39	14	25

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B3@6-6.5	PI: ASTM D4318, Wet Method	
◆	1-B3@15-16.5	PI: ASTM D4318, Wet Method	

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CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/17/2024
TESTED BY: R. Montalvo
REVIEWED BY: M. Gilbert

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



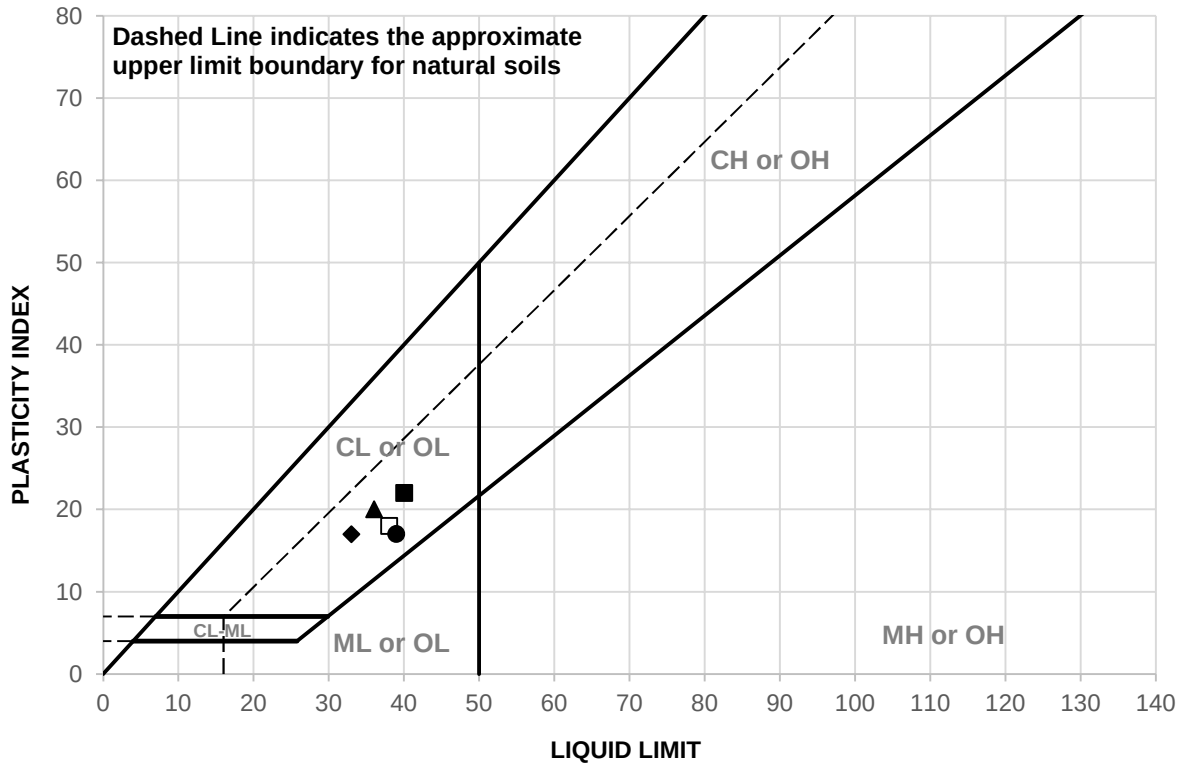
	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B3@0-5	0-5	See exploration logs	41	17	24
◆	1-B3@26-26.5	26-26.5	See exploration logs	32	19	13
□	1OS-CPT1@14-15.5	14-15.5	See exploration logs	28	15	13
●	1OS-B6@25-26	25-26	See exploration logs	28	14	14
■	1-C1@SG	SG	See exploration logs	44	19	25

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B3@0-5	PI: ASTM D4318, Wet Method	
◆	1-B3@26-26.5	PI: ASTM D4318, Wet Method	
□	1OS-CPT1@14-15.5	PI: ASTM D4318, Wet Method	
●	1OS-B6@25-26	PI: ASTM D4318, Wet Method	
■	1-C1@SG	PI: ASTM D4318, Wet Method	

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Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/16/2024
TESTED BY: S.Fikre-Selassie/K. Nguyen/L. Schmitz
REVIEWED BY: O. Espinoza

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



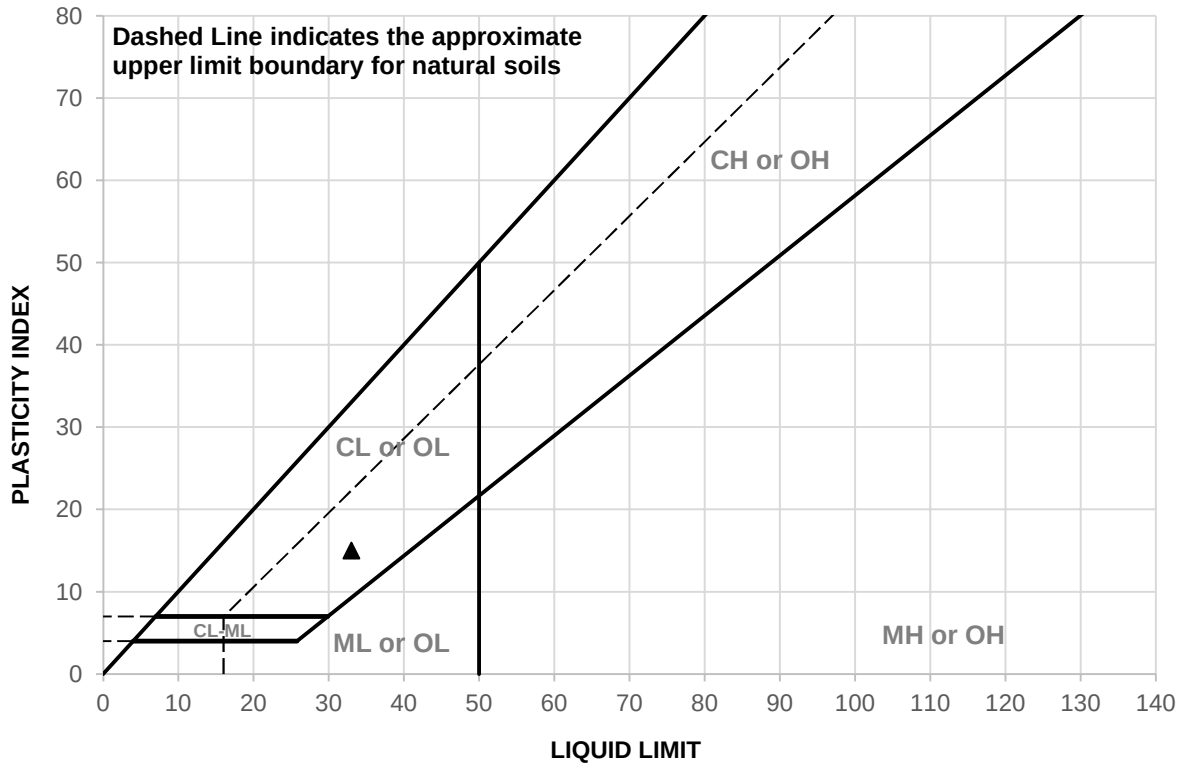
	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-C3@SG	SG	See exploration logs	36	16	20
◆	1-C4@SG	SG	See exploration logs	33	16	17
□	1-C6@SG	SG	See exploration logs	38	20	18
●	1-C8@SG	SG	See exploration logs	39	22	17
■	1-CPT2@SG	SG	See exploration logs	40	18	22

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-C3@SG	PI: ASTM D4318, Wet Method	
◆	1-C4@SG	PI: ASTM D4318, Wet Method	
□	1-C6@SG	PI: ASTM D4318, Wet Method	
●	1-C8@SG	PI: ASTM D4318, Wet Method	
■	1-CPT2@SG	PI: ASTM D4318, Wet Method	

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CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/16/2024
TESTED BY: S.Fikre-Selassie/K. Nguyen
REVIEWED BY: O. Espinoza

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	1OS-B1@7.5-9	7.5-9	See exploration logs	33	18	15

	SAMPLE ID	TEST METHOD	REMARKS
▲	1OS-B1@7.5-9	PI: ASTM D4318, Wet Method	

ENGEO
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CLIENT: COWI North America
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T004
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 11/21/2024
TESTED BY: K. Nguyen
REVIEWED BY: G. Criste

MOISTURE CONTENT REPORT
ASTM D2216

SAMPLE ID	1OSB1@7.5-9							
DEPTH (ft.)	7.5-9							
METHOD A OR B	B							
MOISTURE CONTENT (%)	18.3							

SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								

SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								

SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								

SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								

SAMPLE ID								
DEPTH (ft.)								
METHOD A OR B								
MOISTURE CONTENT (%)								

ENGEO
Expect Excellence

CLIENT: COWI North America
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T004
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 11/21/2024
TESTED BY: K. Nguyen
REVIEWED BY: G. Criste

MOISTURE CONTENT REPORT
ASTM D2216

SAMPLE ID	1-B2	1-B2	1-B2	1-B2	1-B3	1-B3		
DEPTH (ft.)	8-8.5	8.5-9	10-11.5	16-16.5	11-11.5	15-16.5		
METHOD A OR B	B	B	B	B	B	B		
MOISTURE CONTENT (%)	15.6	13.6	8.2	14.9	22.1	19.7		

ENGEO
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CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/11/2024
TESTED BY: L. Schmitz
REVIEWED BY: M. Gilbert

MOISTURE CONTENT DETERMINATION

Revision 01-20-2021

ASTM D2216

PROJECT NAME: Berkeley Water Trasportation Pier Ferry P | **REPORT DATE:** 10/11/24

PROJECT, PHASE, TASK NO.: 25022.000.001 PH001 T003

CLIENT: COWI North America, Inc. | **TESTED BY:** Y. Cabrale

PROJECT LOCATION: Berkeley, CA | **REVIEWED BY:** G. Criste

Sample #	1	2	3	4	5	6	7	8
Sample ID	1-B5@ 16-16.5	1-B5@ 17-17.5	1OS-CPT1@ 14-15.5	1OS-CPT3@ 12.5-14	1OS-CPT3@ 85-86.5	1OS-CPT3@ 101-101.5	1OS-B4@ 10-11.5	1OS-B4@ 26-26.5
Depth (ft)	16-16.5	17-17.5	14-15.5	12.5-14	85-86.5	101-101.5	10-11.5	26-26.5
Method A or B	B	B	B	B	B	B	B	B
Tare name:								
Wet soil + tare:	1306.78	1014.23	617.77	670.62	588.37	781.33	736.87	275.41
Dry soil + tare:	1175.22	960.21	585.34	638.77	509.65	680.59	649.17	237.98
Tare weight:	565.77	409.47	412.23	440.34	84.62	82.53	83.80	86.23
%Moisture:	21.6	9.8	18.7	16.1	18.5	16.8	15.5	24.7

Sample #	9	10	11	12	13	14	15	16
Sample ID	1OS-B5@ 19-20.5	1-C1@SG	1-C3@SG	1-C4@SG	1-C5@SG	1-C8@SG	CPT2@ SG	
Depth (ft)	19-20.5	SG	SG	SG	SG	SG	SG	
Method A or B	B	B	B	B	B	B	B	
Tare name:								
Wet soil + tare:	1047.22	749.07	541.6	1225.60	734.45	744.8	885.43	
Dry soil + tare:	938.81	698.56	505.96	1150.59	665.79	686.49	816.11	
Tare weight:	467.44	283.04	256.6	464.30	311.65	279.9	412.38	
%Moisture:	23.0	12.2	14.3	10.9	19.4	14.3	17.2	

Sample #	17	18	19	20	21	22	23	24
Sample ID								
Depth (ft)								
Method A or B								
Tare name:								
Wet soil + tare:								
Dry soil + tare:								
Tare weight:								
%Moisture:								

MOISTURE-DENSITY DETERMINATION REPORT
ASTM D7263

SAMPLE ID	1-B2	1-B2	1-B2					
DEPTH (ft.)	36-36.5	41-41.5	71-71.5					
METHOD A OR B	B	B	B					
MOISTURE CONTENT (%)	25.8	21.4	35.6					
DRY DENSITY (pcf)	100.1	107.8	84.3					

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/11/2024
TESTED BY: L. Schmitz
REVIEWED BY: M. Gilbert

MOISTURE-DENSITY DETERMINATION REPORT
ASTM D7263

SAMPLE ID	1-B3@ 26-26.5	1-B5@ 5-5.25	1OS-B1@ 16-16.5	1OS-B1@ 21-21.5	1OS-B1@ 46-46.5	1OS-B1@ 96-96.5	1OS-B1@ 101-101.5	1OS-B4@ 16-16.5
DEPTH (ft.)	26-26.5	5-5.25	16-16.5	21-21.5	46-46.5	96-96.5	101-101.5	16-16.5
METHOD A OR B	B	B	B	B	B	B	B	B
MOISTURE CONTENT (%)	29.6	16.2	19.4	32.9	30.4	47.8	20.2	26.0
DRY DENSITY (pcf)	96.2	104.8	113.1	92.5	94.4	86.7	108.6	99.8

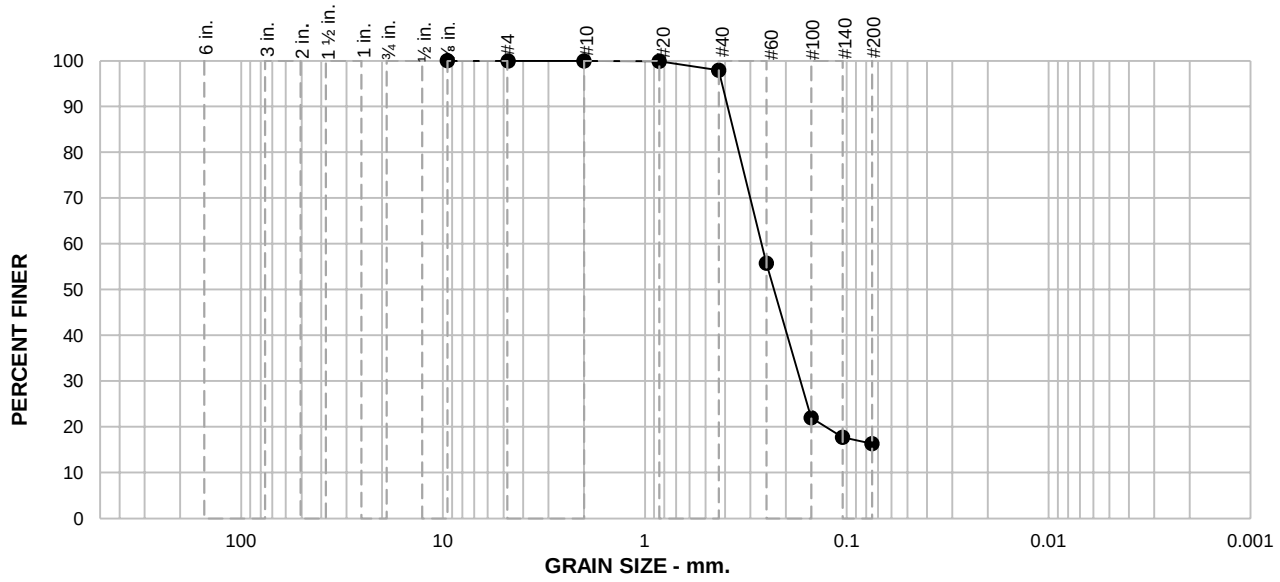
SAMPLE ID	1OS-B4@ 31-31.5	1OS-B4@ 111-111.5	1OS-B5@ 73.5-74	1OS-B6@ 16-16.5	1OS-B6@ 36-36.5	1OS-B6@ 56-56.5	1-B5@5.25-5.75	1OS-B6@ 16-16.5
DEPTH (ft.)	31-31.5	111-111.5	73.5-74	16-16.5	36-36.5	56-56.5	5.25-5.75	16-16.5
METHOD A OR B	B	B	B	B	B	B	B	B
MOISTURE CONTENT (%)	25.3	27.9	40.2	17.6	28.2	20.9	15.7	17.6
DRY DENSITY (pcf)	100.7	96.4	80.7	117.3	96.0	107.8	105.3	117.3

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/11/2024
TESTED BY: K. Nguyen
REVIEWED BY: G. Criste

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method A



SAMPLE ID: 1-B1@41-41.5

DEPTH (ft): 41-41.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
				2	82	16	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
⅜ in.	100		
#4	100		
#10	100		
#20	100		
#40	98		
#60	56		
#100	22		
#140	18		
#200	16		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 0.3878 mm D_{85} = 0.3636 mm D_{60} = 0.2633 mm

D_{50} = 0.2284 mm D_{30} = 0.1692 mm D_{15} =

D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.

PROJECT NAME: Berkeley Water Transportation Pier Ferry Project

PROJECT NO: 25022.000.001 PH001 T003

PROJECT LOCATION: Berkeley, CA

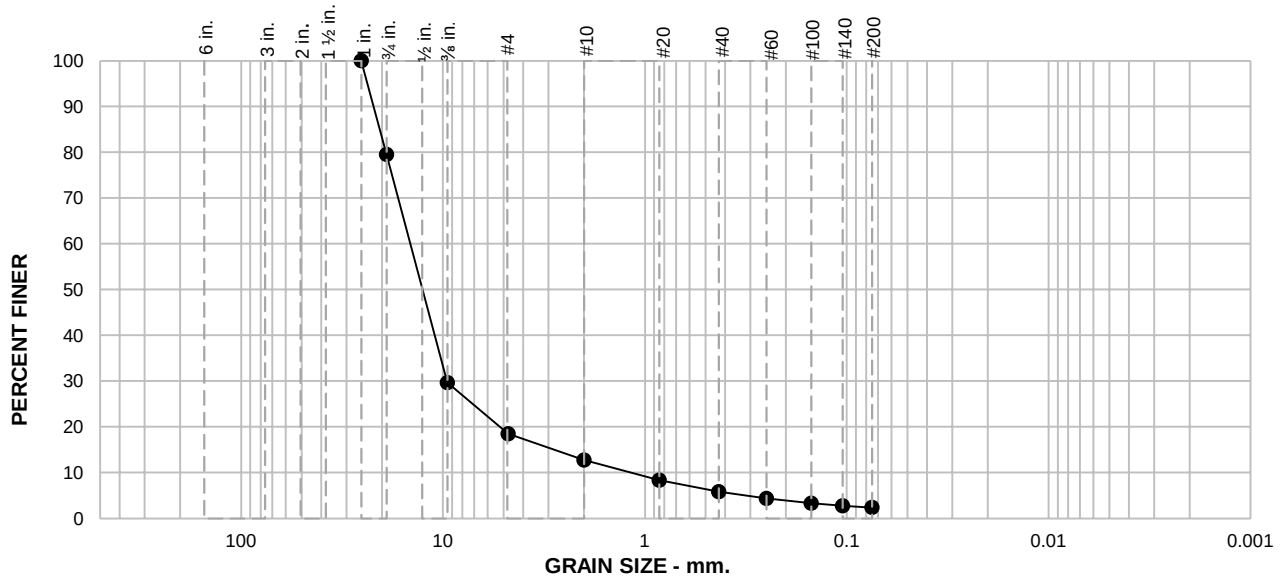
REPORT DATE: 10/16/2024

TESTED BY: L. Schmitz

REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method A



SAMPLE ID: 1-B2@16-16.5

DEPTH (ft): 16-16.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
	20	61	6	7	4	2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100		
¾ in.	80		
⅜ in.	30		
#4	19		
#10	13		
#20	8		
#40	6		
#60	4		
#100	3		
#140	3		
#200	2		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 21.9970 mm D_{85} = 20.4706 mm D_{60} = 14.4372 mm

D_{50} = 12.5683 mm D_{30} = 9.5250 mm D_{15} = 2.6684 mm

D_{10} = 1.1969 mm C_u = 12.06 C_c = 5.25

CLASSIFICATION

USCS = GP

REMARKS

* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.

PROJECT NAME: Berkeley Water Transportation Pier Ferry Project

PROJECT NO: 25022.000.001 PH001 T003

PROJECT LOCATION: Berkeley, CA

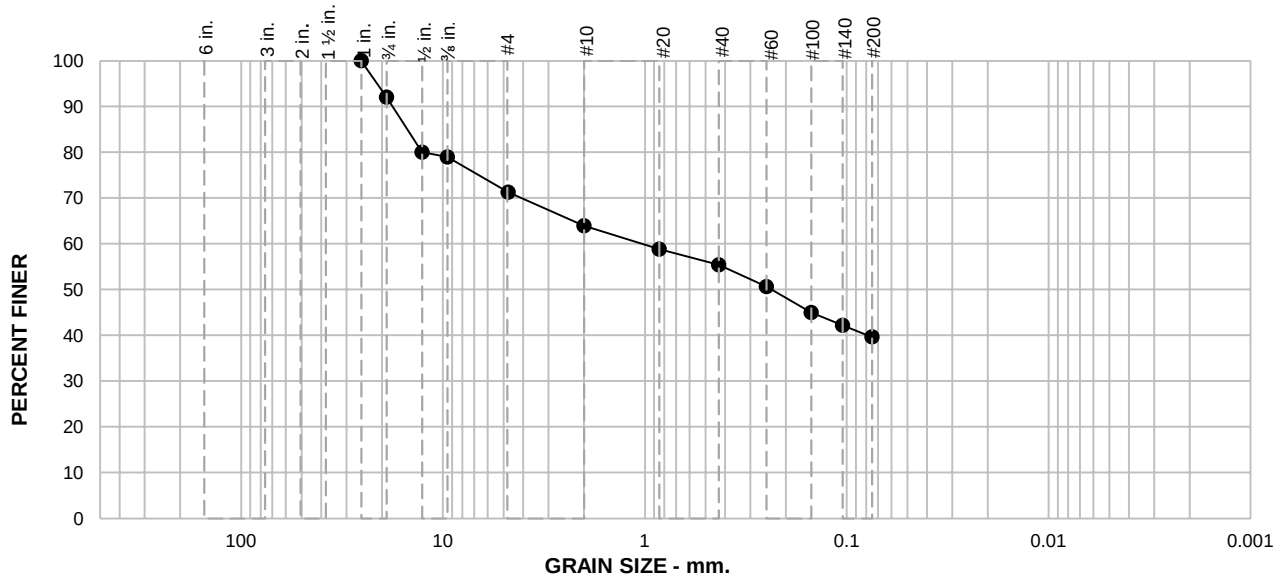
REPORT DATE: 10/16/2024

TESTED BY: L. Schmitz

REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method A



SAMPLE ID: 1-B4@21'-21.5'

DEPTH (ft): 21-21.5

LOCATION: 1-B4@21'-21.5'

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
	8	21	7	9	15	40	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100		
¾ in.	92		
½ in.	80		
⅜ in.	79		
#4	71		
#10	64		
#20	59		
#40	55		
#60	51		
#100	45		
#140	42		
#200	40		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 17.8052 mm D_{85} = 15.0375 mm D_{60} = 1.0087 mm

D_{50} = 0.2296 mm D_{30} = D_{15} =

D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Silt/clay division of 0.002mm used

USCS: ASTM D2488

* (no specification provided)

ENGEO Expect Excellence

CLIENT: COWI North America, Inc.

PROJECT NAME: Berkeley Water Transportation Pier Ferry Project Engineering and Design

PROJECT NO: 25022.000.001 PH001 T003

PROJECT LOCATION: Berkeley, CA

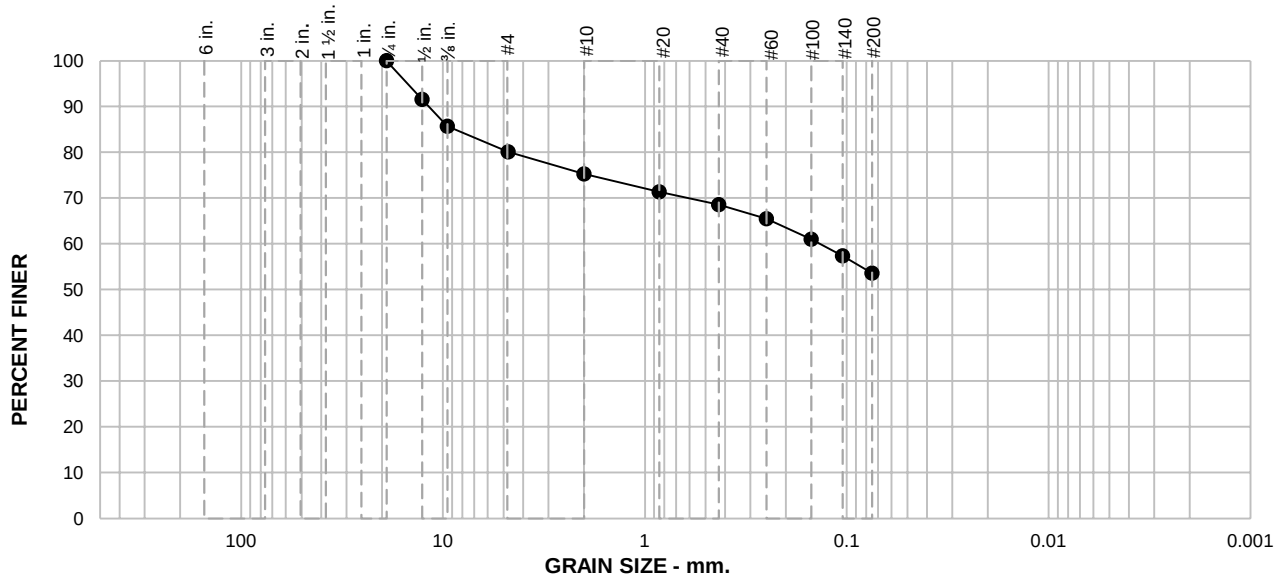
REPORT DATE: 10/15/2024

TESTED BY: K. Nguyen

REVIEWED BY: O. Espinoza

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method A



SAMPLE ID: 1-B4@16'-16.5'

DEPTH (ft): 16-16.5

LOCATION: 1-B4@16'-16.5'

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		20	5	6	15	54	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
¾ in.	100		
½ in.	92		
⅜ in.	86		
#4	80		
#10	75		
#20	71		
#40	69		
#60	65		
#100	61		
#140	57		
#200	54		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 11.5387 mm D_{85} = 8.4821 mm D_{60} = 0.1372 mm

D_{50} = D_{30} = D_{15} =

D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Silt/clay division of 0.002mm used

USCS: ASTM D2488

* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.

PROJECT NAME: Berkeley Water Transportation Pier Ferry Project Engineering and Design

PROJECT NO: 25022.000.001 PH001 T003

PROJECT LOCATION: Berkeley, CA

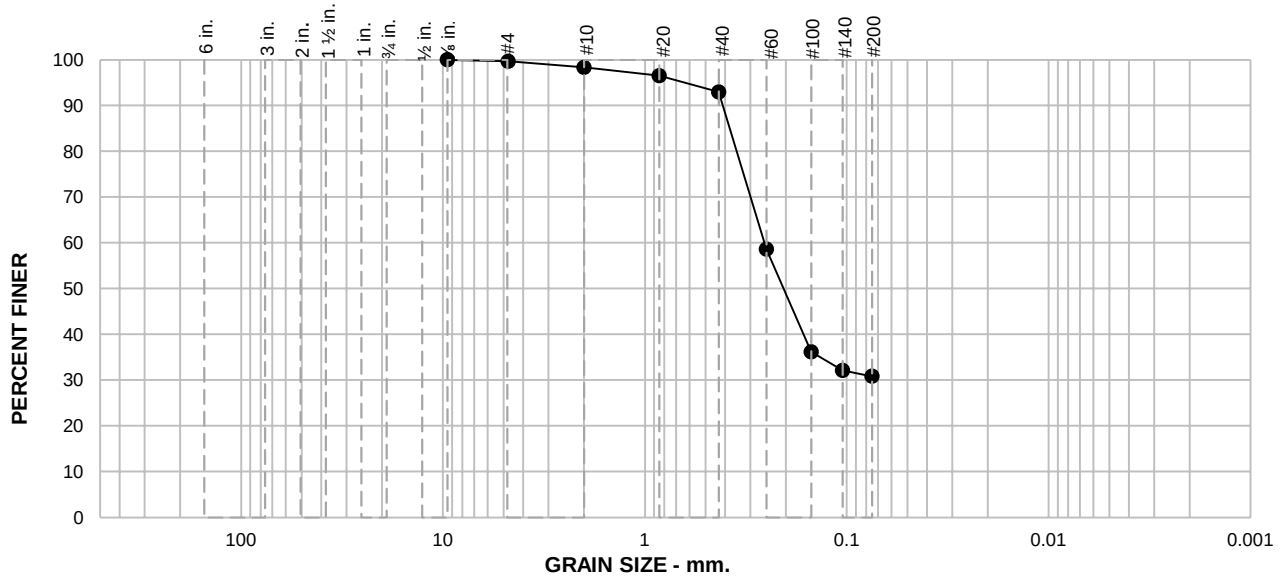
REPORT DATE: 10/15/2024

TESTED BY: K. Nguyen

REVIEWED BY: O. Espinoza

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method A



SAMPLE ID: 1OS-B5@19'-20.5'

DEPTH (ft): 19'-20.5'

LOCATION: 1OS-B5@19'-20.5'

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
			2	5	62	31	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
⅜ in.	100		
#4	100		
#10	98		
#20	97		
#40	93		
#60	59		
#100	36		
#140	32		
#200	31		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 0.4099 mm D_{85} = 0.3785 mm D_{60} = 0.2540 mm

D_{50} = 0.2047 mm D_{30} = D_{15} =

D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Silt/clay division of 0.002mm used
USCS: ASTM D2488

Contains abundance of shell fragments

* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.

PROJECT NAME: Berkeley Water Transportation Pier Ferry Project Engineering and Design

PROJECT NO: 25022.000.001 PH001 T003

PROJECT LOCATION: Berkeley, CA

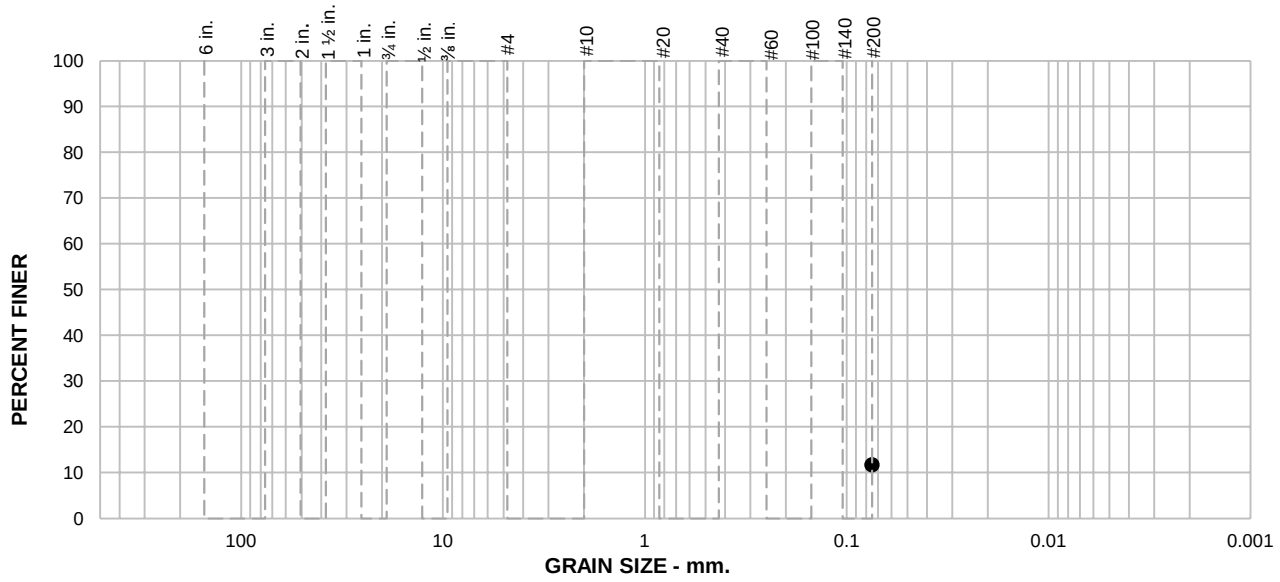
REPORT DATE: 10/15/2024

TESTED BY: K. Nguyen

REVIEWED BY: O. Espinoza

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B1@6-6.5

DEPTH (ft): 6-6.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						12	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	12		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 11 LL = 35 PI = 24

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

PI: ASTM D4318, Wet Method

Soak time = 180 min
Dry sample weight = 526.4 g
Largest particle size ≥ No. 4 Sieve

* (no specification provided)

CLIENT: COWI North America, Inc.

PROJECT NAME: Berkeley Water Transportation Pier Ferry Project

PROJECT NO: 25022.000.001 PH001 T003

PROJECT LOCATION: Berkeley, CA

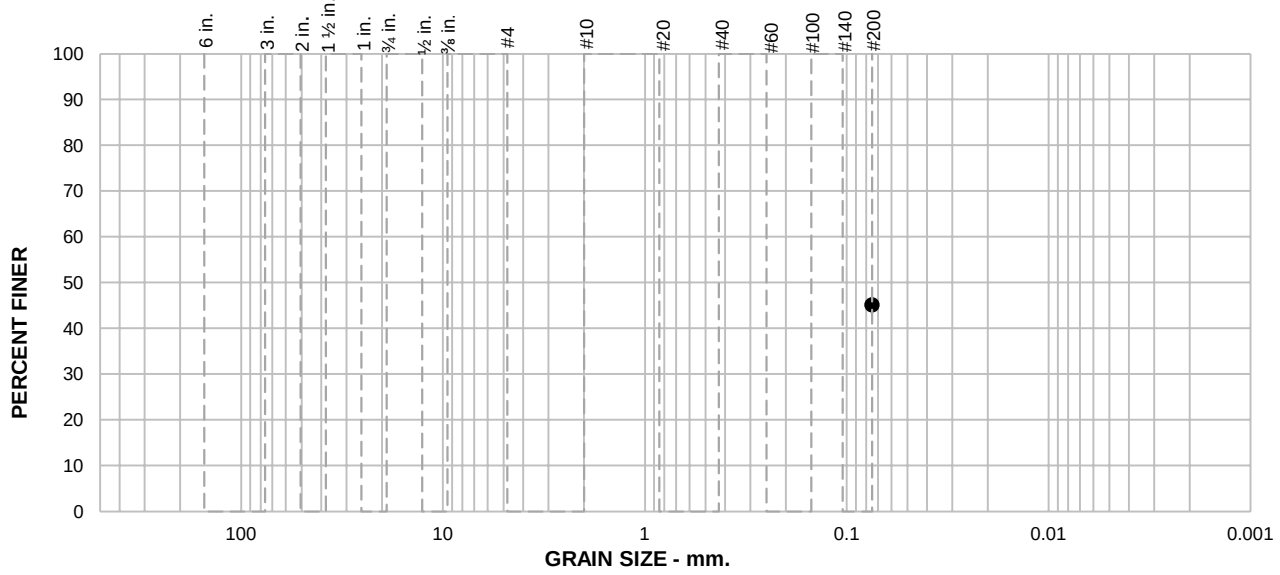
REPORT DATE: 10/17/2024

TESTED BY: L. Schmitz

REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B1@11-11.5

DEPTH (ft): 11-11.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						45	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	45		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 16 LL = 45 PI = 29

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

PI: ASTM D4318, Wet Method

Soak time = 180 min
Dry sample weight = 396.6 g
Largest particle size ≥ No. 4 Sieve

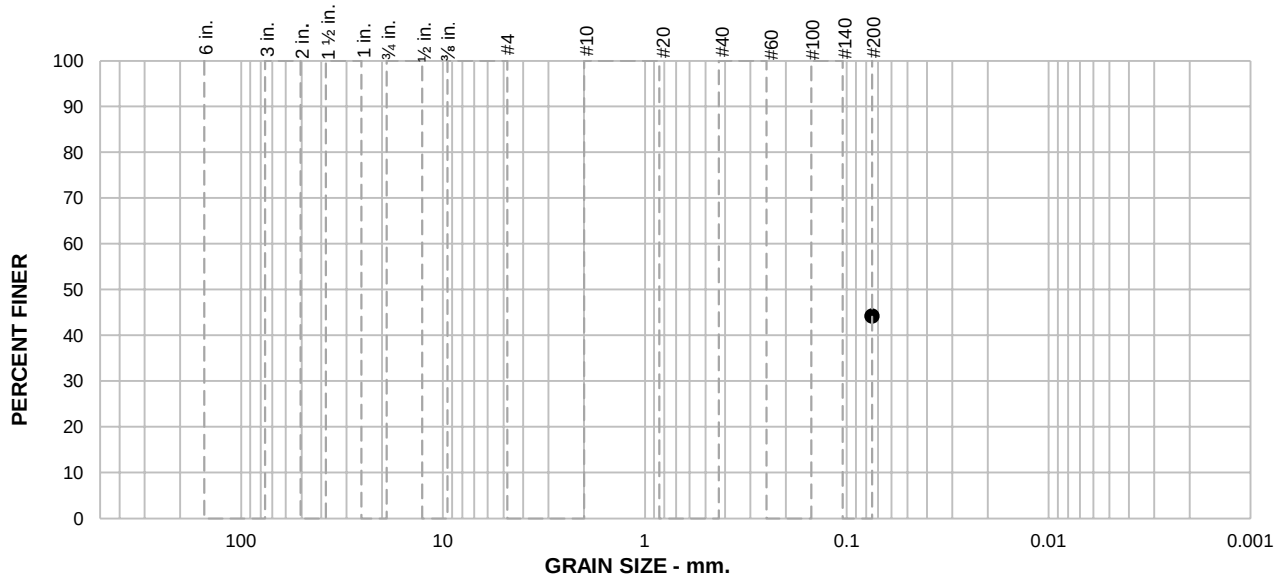
* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/17/2024
TESTED BY: L. Schmitz
REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B1@5.5-6

DEPTH (ft): 5.5-6

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						44	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	44		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 17 LL = 42 PI = 25

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

PI: ASTM D4318, Wet Method

Soak time = 180 min
Dry sample weight = 459.7 g
Largest particle size ≥ No. 4 Sieve

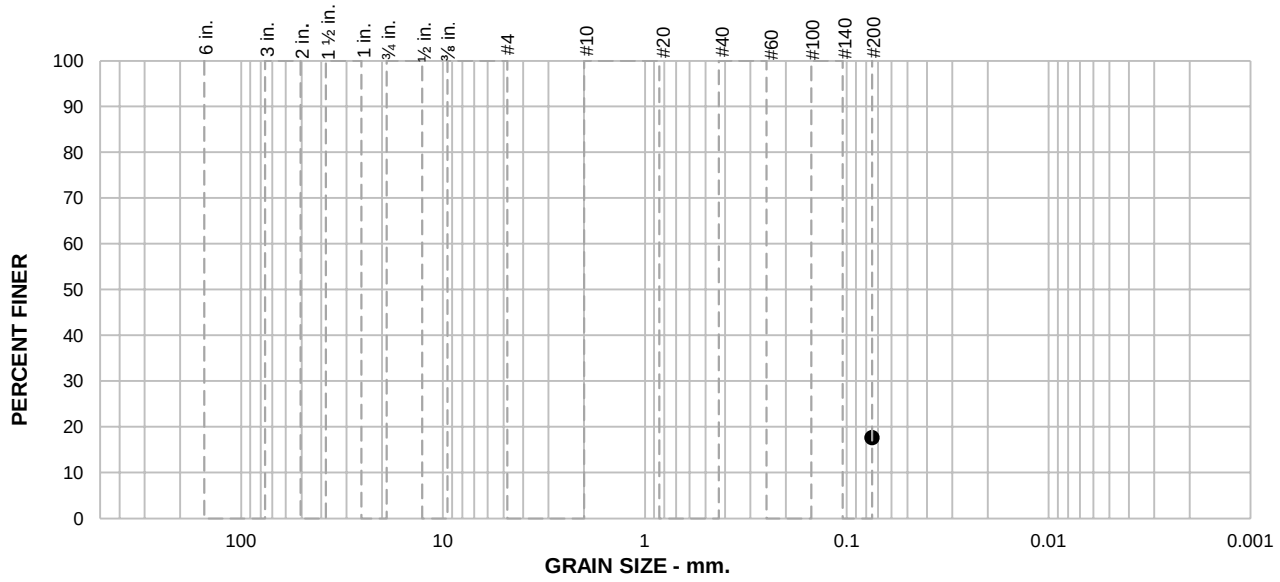
* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/17/2024
TESTED BY: L. Schmitz
REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B2@8-8.5

DEPTH (ft): 8-8.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						18	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	18		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min
Dry sample weight = 613.2 g
Largest particle size ≥ No. 4 Sieve

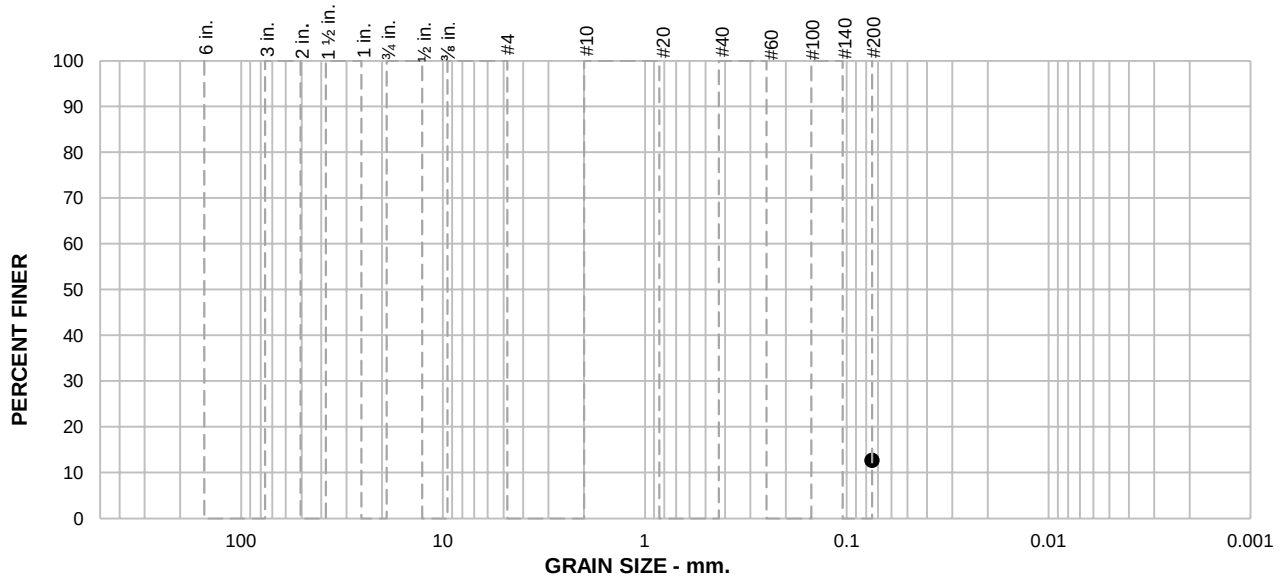
* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/16/2024
TESTED BY: L. Schmitz
REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B2@8.5-9

DEPTH (ft): 8.5-9

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						13	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	13		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 20 LL = 38 PI = 18

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

PI: ASTM D4318, Wet Method

Soak time = 180 min
Dry sample weight = 473.5 g
Largest particle size ≥ No. 4 Sieve

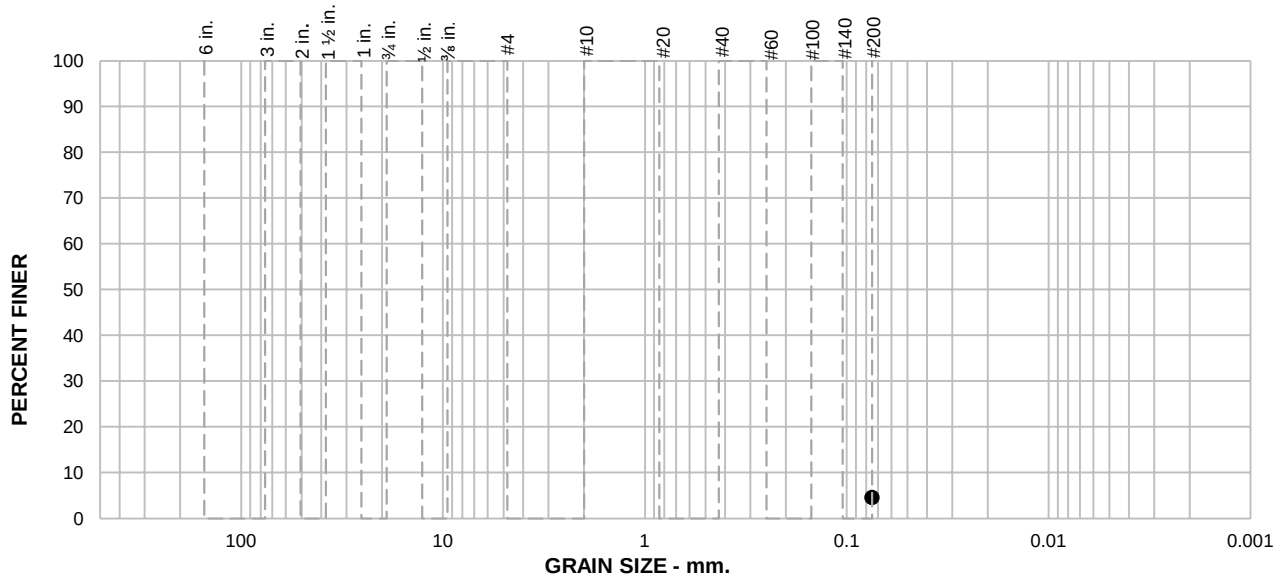
* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/17/2024
TESTED BY: L. Schmitz
REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B2@10-11.5

DEPTH (ft): 10-11.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	5		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min
Dry sample weight = 293.8 g
Largest particle size ≥ No. 4 Sieve

* (no specification provided)

CLIENT: COWI North America, Inc.

PROJECT NAME: Berkeley Water Transportation Pier Ferry Project

PROJECT NO: 25022.000.001 PH001 T003

PROJECT LOCATION: Berkeley, CA

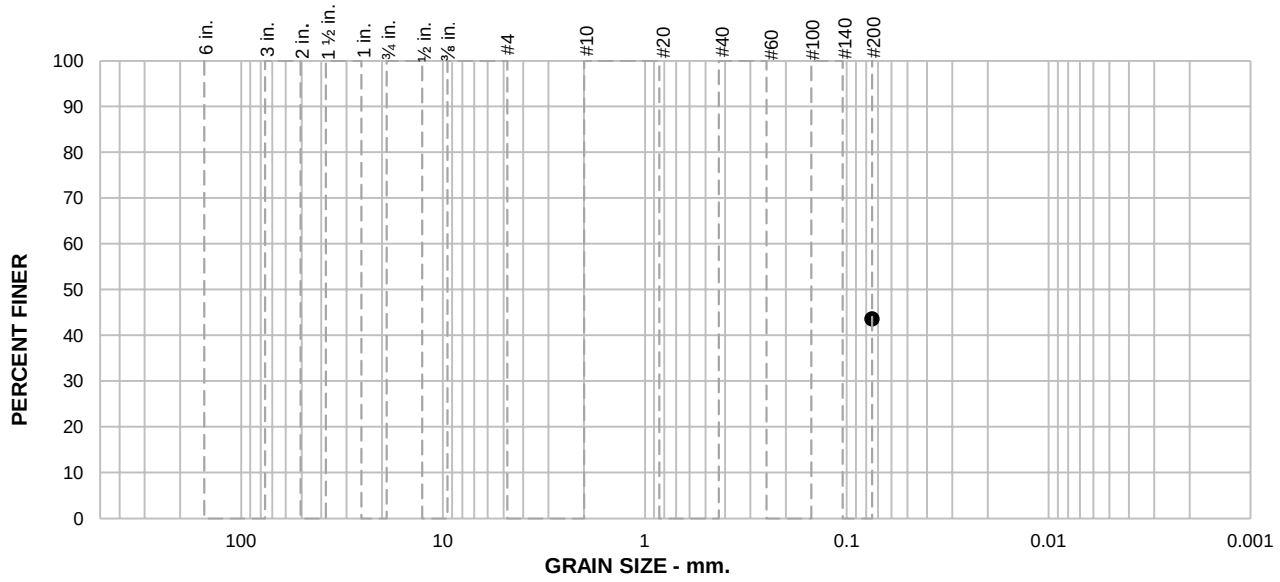
REPORT DATE: 10/16/2024

TESTED BY: L. Schmitz

REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B2@30-31.5

DEPTH (ft): 30-31.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						44	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	44		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 13 LL = 35 PI = 22

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

PI: ASTM D4318, Wet Method

Soak time = 180 min
Dry sample weight = 324.9 g
Largest particle size ≥ No. 4 Sieve

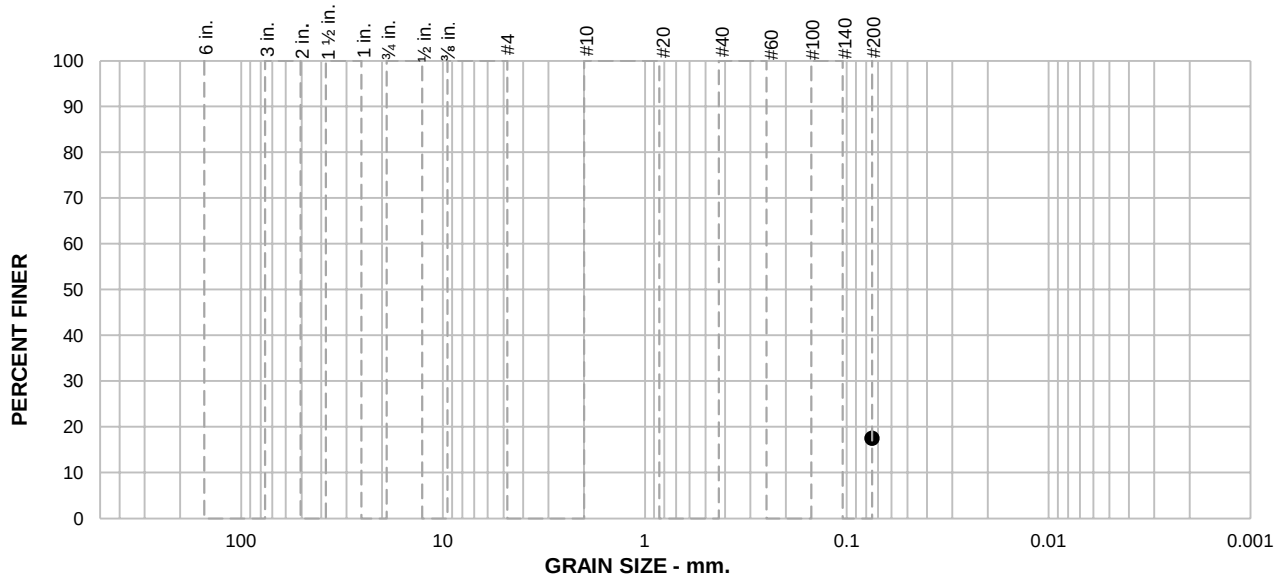
* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/17/2024
TESTED BY: L. Schmitz
REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B3@7.5-9

DEPTH (ft): 7.5-9

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						17	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	17		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min
Dry sample weight = 144.1 g
Largest particle size ≥ No. 4 Sieve

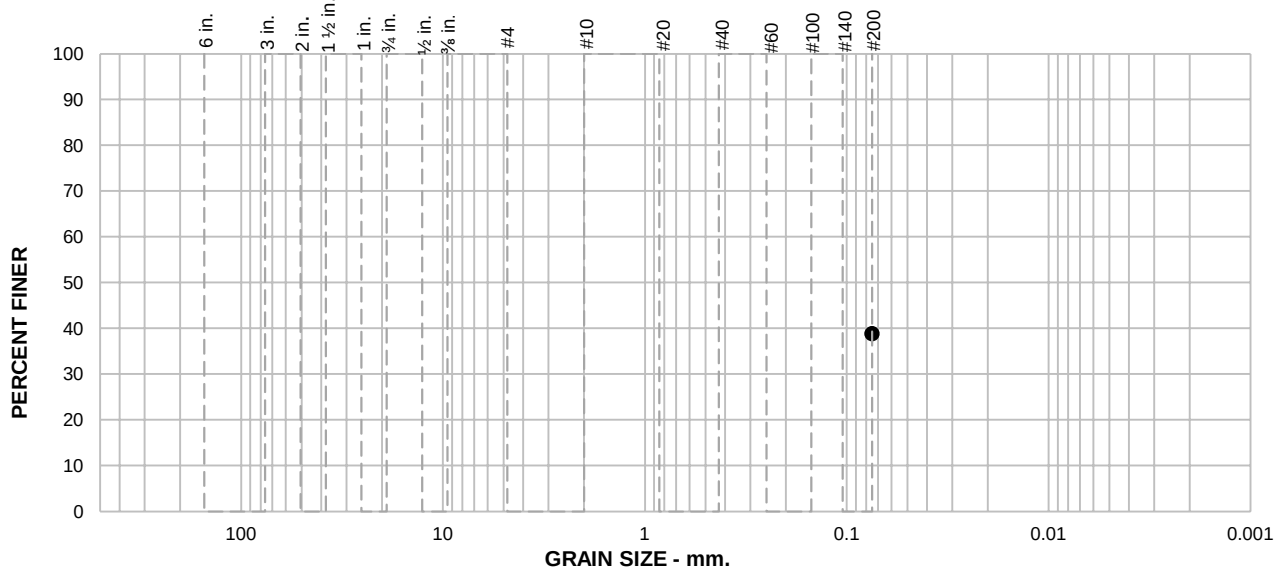
* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/16/2024
TESTED BY: L. Schmitz
REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B5@5.25'-5.75'

DEPTH (ft): 5.25'-5.75'

LOCATION: 1-B5@5.25'-5.75'

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						39	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	39		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Silt/clay division of 0.002mm used

Soak time = 180 min
Dry sample weight = 743.8 g
Largest particle size < No. 4 Sieve

* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.

PROJECT NAME: Berkeley Water Transportation Pier Ferry Project Engineering and Design

PROJECT NO: 25022.000.001 PH001 T003

PROJECT LOCATION: Berkeley, CA

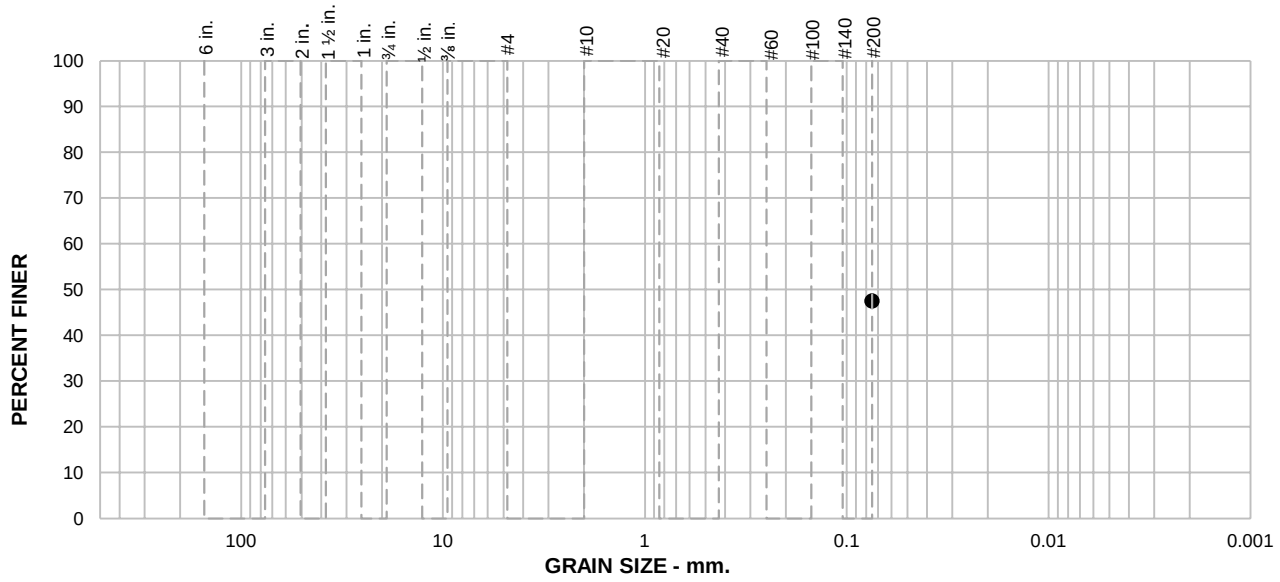
REPORT DATE: 10/15/2024

TESTED BY: K. Nguyen

REVIEWED BY: O. Espinoza

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method A



SAMPLE ID: 1OS-B1@7.5-9

DEPTH (ft): 7.5-9

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						48	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	48		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 18 LL = 33 PI = 15

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Silt/clay division of 0.002mm used
PI: ASTM D4318, Wet Method

Soak time = 1440 min
Dry sample weight = 402.9 g
Largest particle size ≥ No. 4 Sieve

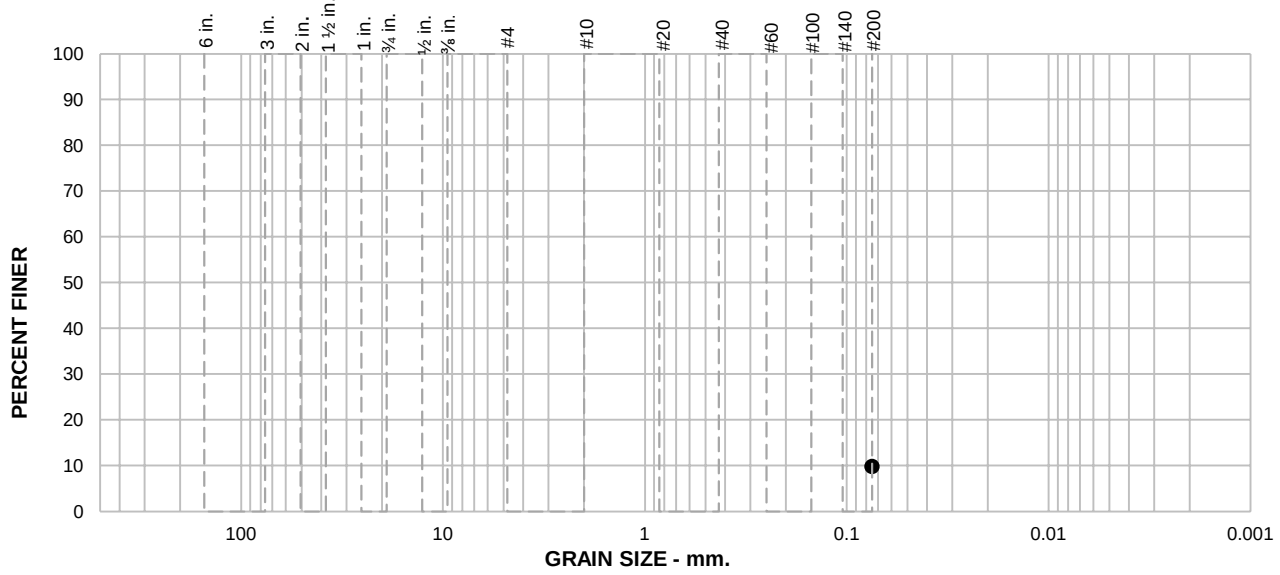
* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T004
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 11/21/2024
TESTED BY: K. Nguyen
REVIEWED BY: G. Criste

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1OS-B1@16'-16.5'

DEPTH (ft): 16'-16.5'

LOCATION: 1OS-B1@16'-16.5'

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						9.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	9.8		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Silt/clay division of 0.002mm used

Soak time = 180 min
Dry sample weight = 179.76 g
Largest particle size < No. 4 Sieve

* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.

PROJECT NAME: Berkeley Water Transportation Pier Ferry Project Engineering and Design

PROJECT NO: 25022.000.001 PH001 T003

PROJECT LOCATION: Berkeley, CA

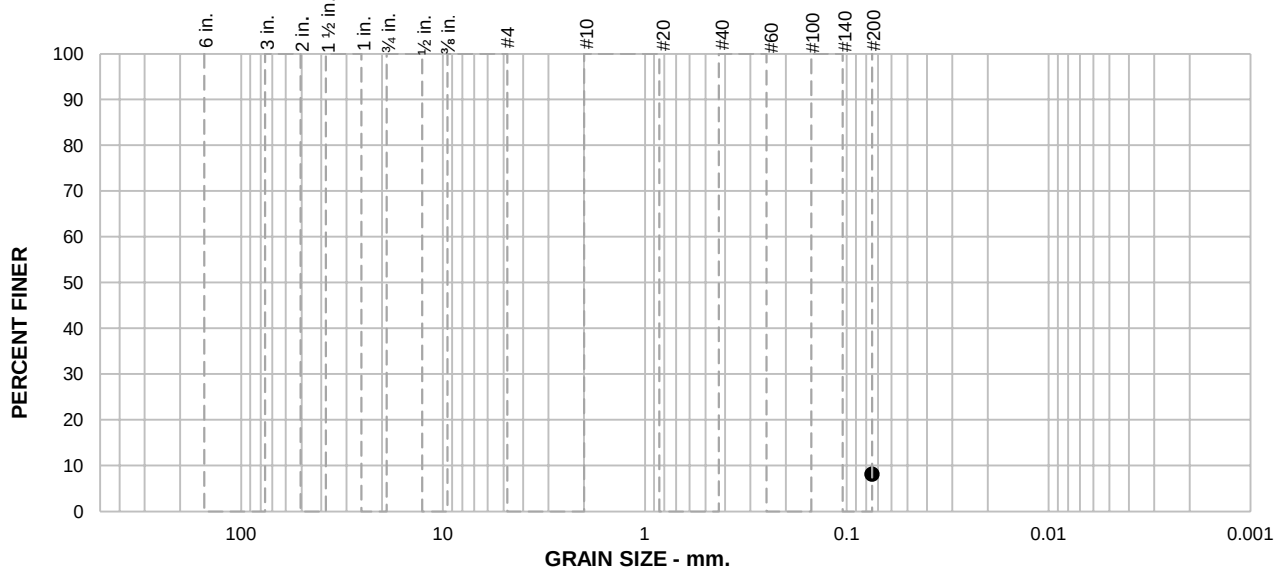
REPORT DATE: 10/15/2024

TESTED BY: K. Nguyen

REVIEWED BY: O. Espinoza

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1OS-B6@16'-16.5'

DEPTH (ft): 16'-16.5'

LOCATION: 1OS-B6@16'-16.5'

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						8.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	8.1		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Silt/clay division of 0.002mm used

Soak time = 180 min
Dry sample weight = 489.96 g
Largest particle size < No. 4 Sieve

* (no specification provided)

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.

PROJECT NAME: Berkeley Water Transportation Pier Ferry Project Engineering and Design

PROJECT NO: 25022.000.001 PH001 T003

PROJECT LOCATION: Berkeley, CA

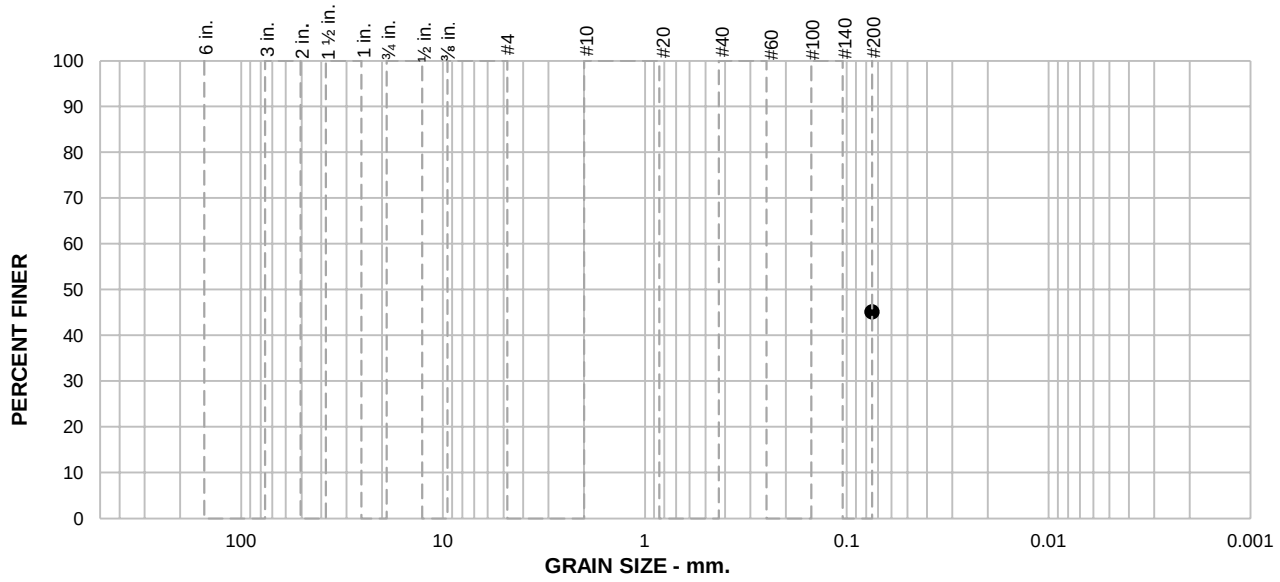
REPORT DATE: 10/15/2024

TESTED BY: K. Nguyen

REVIEWED BY: O. Espinoza

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1OS-CPT1@14'-15.5'

DEPTH (ft): 14'-15.5'

LOCATION: 1OS-CPT1@14'-15.5'

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						45.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	45.1		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Silt/clay division of 0.002mm used

Soak time = 180 min
Dry sample weight = 173.11 g
Largest particle size < No. 4 Sieve

* (no specification provided)

ENGEO
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CLIENT: COWI North America, Inc.

PROJECT NAME: Berkeley Water Transportation Pier Ferry Project Engineering and Design

PROJECT NO: 25022.000.001 PH001 T003

PROJECT LOCATION: Berkeley, CA

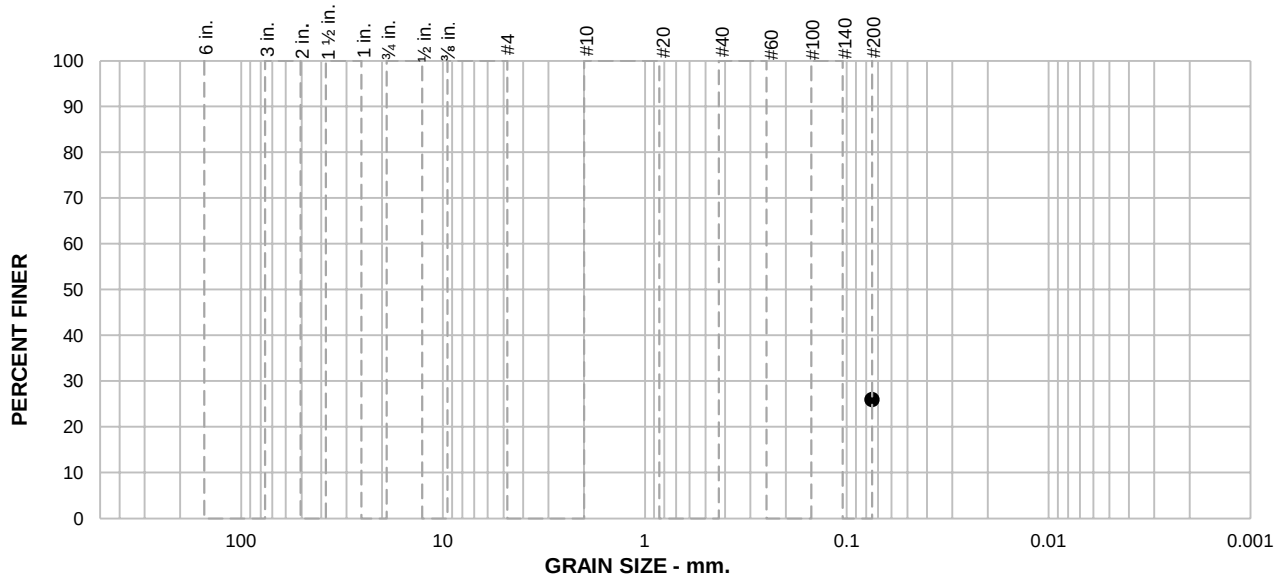
REPORT DATE: 10/15/2024

TESTED BY: K. Nguyen

REVIEWED BY: O. Espinoza

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1OS-CPT3@12.5'-14'

DEPTH (ft): 12.5'-14'

LOCATION: 1OS-CPT3@12.5'-14'

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						26.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	26.0		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Silt/clay division of 0.002mm used

Soak time = 180 min
Dry sample weight = 198.43 g
Largest particle size < No. 4 Sieve

* (no specification provided)

ENGEO
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CLIENT: COWI North America, Inc.

PROJECT NAME: Berkeley Water Transportation Pier Ferry Project Engineering and Design

PROJECT NO: 25022.000.001 PH001 T003

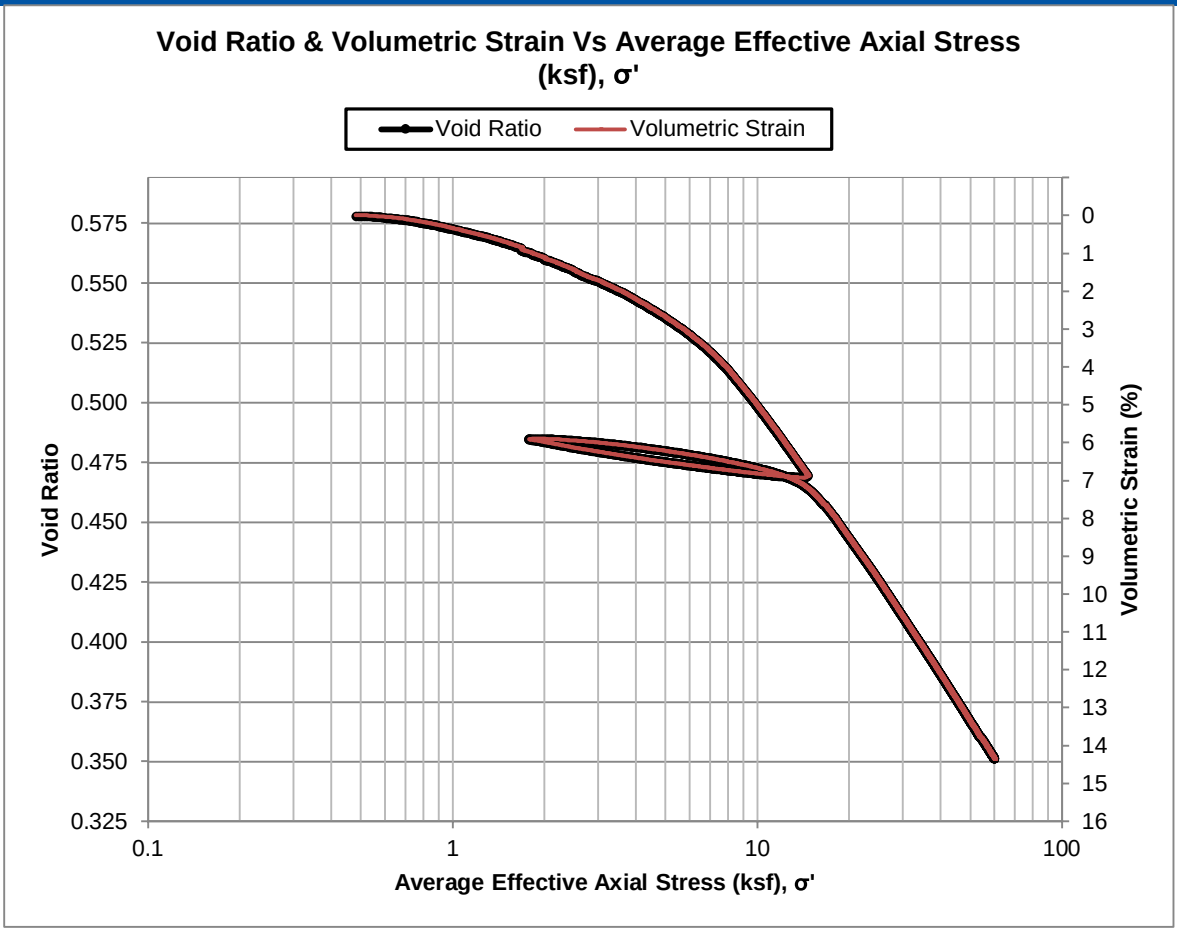
PROJECT LOCATION: Berkeley, CA

REPORT DATE: 10/15/2024

TESTED BY: K. Nguyen

REVIEWED BY: O. Espinoza

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID:	1-B1 at 55-57.5	**DEPTH:**	57-57.5 ft
SOIL DESCRIPTION:	See exploration logs		
REMARKS:			

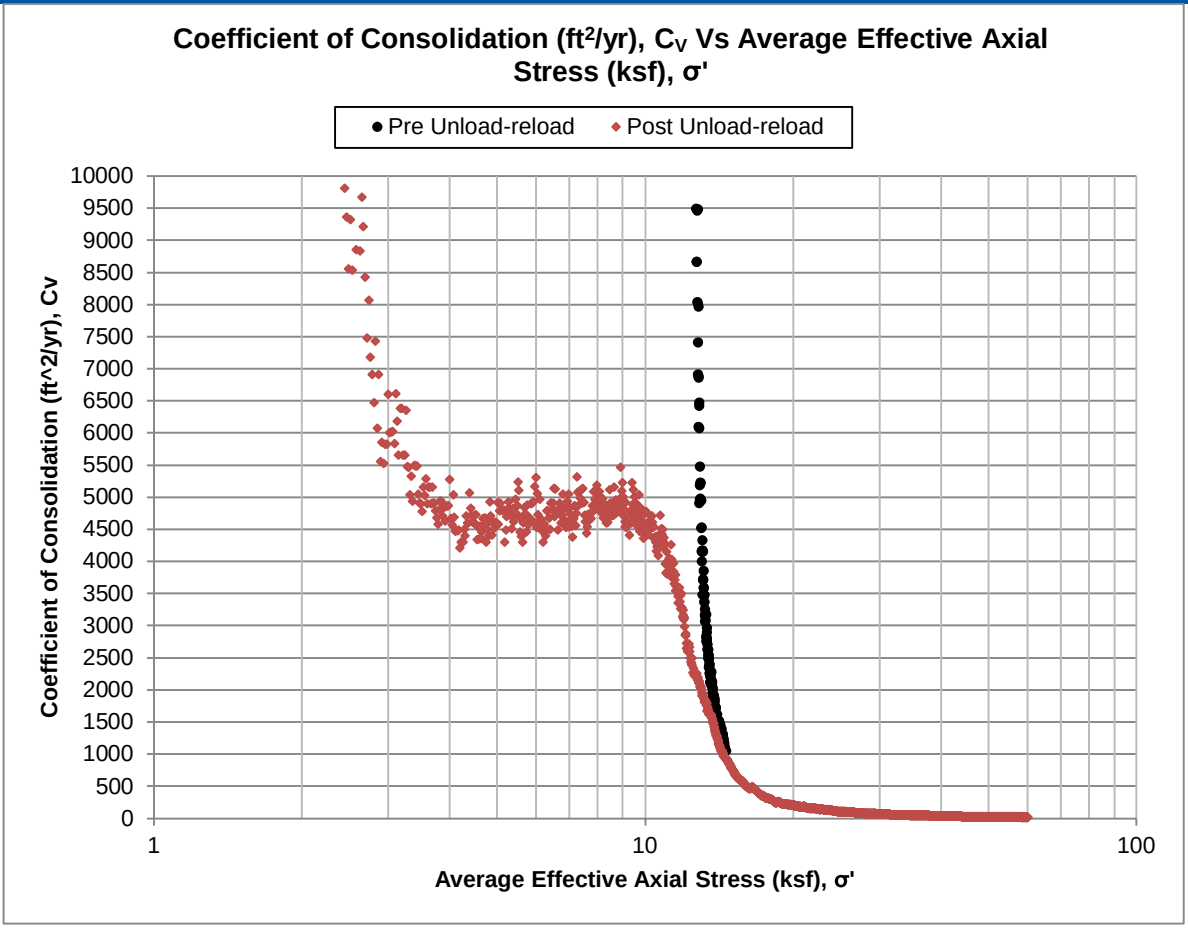
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	20.68	16.90	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	103.76	125.08	**PLASTIC LIMIT:**	
SATURATION (%):	79.31	100.00	**ASTM D854 - Measured**	
VOID RATIO:	0.578	0.309	**SPECIFIC GRAVITY**	2.627
STRAIN RATE (in/min):	0.000062			

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportaion Pier Ferry Project
PROJECT NO: 25022.000.001-P:001-T:003
PROJECT LOCATION: Berkeley, California
REPORT DATE: 10/21/2024
TESTED BY: D. Seibold
REVIEWED BY: O. Espinoza

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID:	1-B1 at 55-57.5	**DEPTH:**	57-57.5 ft
SOIL DESCRIPTION:	See exploration logs		
REMARKS:			

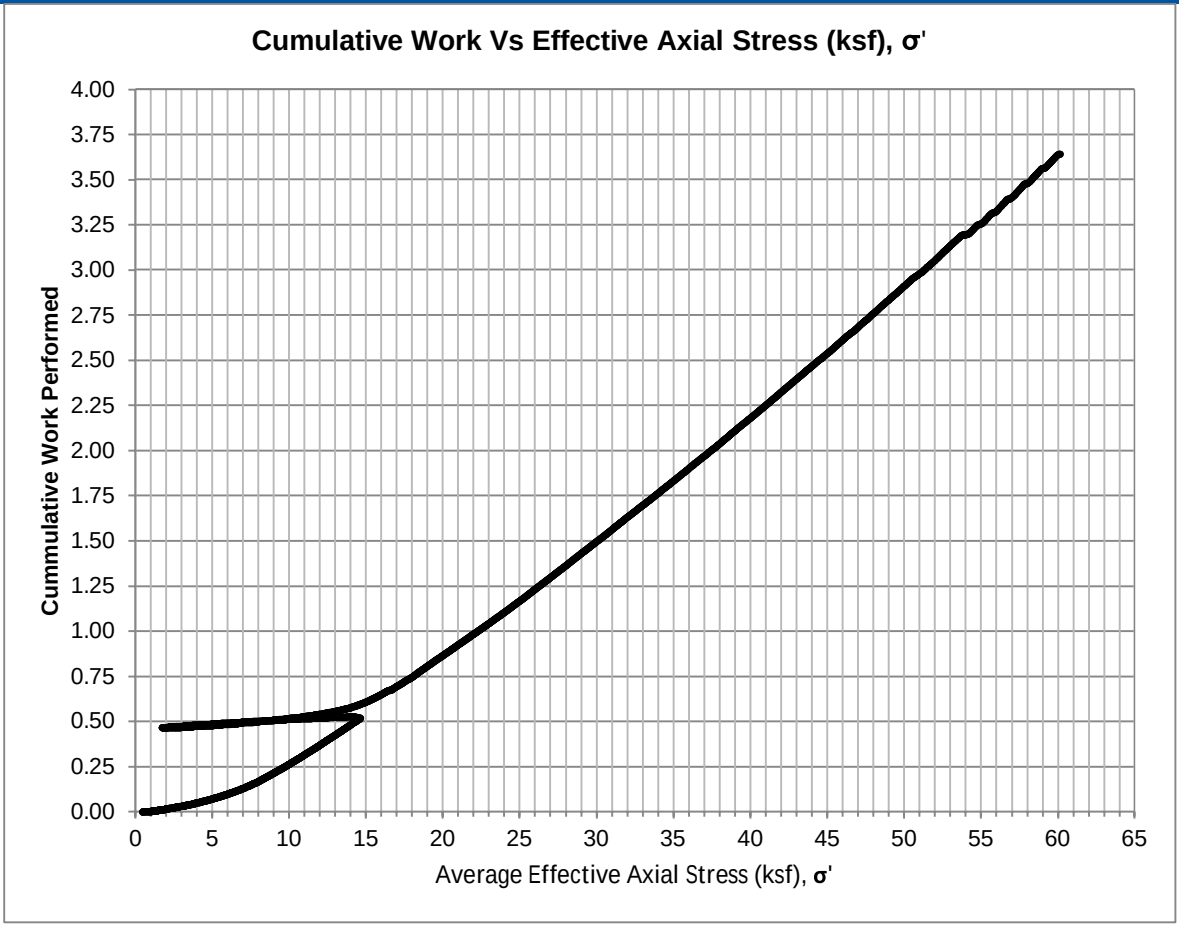
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	20.68	16.90	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	103.76	125.08	**PLASTIC LIMIT:**	
SATURATION (%):	79.31	100.00	**ASTM D854 - Measured**	
VOID RATIO:	0.578	0.309	**SPECIFIC GRAVITY**	2.627
STRAIN RATE (in/min):	0.000062			

ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportaion Pier Ferry Project
PROJECT NO: 25022.000.001-P:001-T:003
PROJECT LOCATION: Berkeley, California
REPORT DATE: 10/21/2024
TESTED BY: D. Seibold
REVIEWED BY: O. Espinoza

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID:	1-B1 at 55-57.5	**DEPTH:**	57-57.5 ft
SOIL DESCRIPTION:	See exploration logs		
REMARKS:			

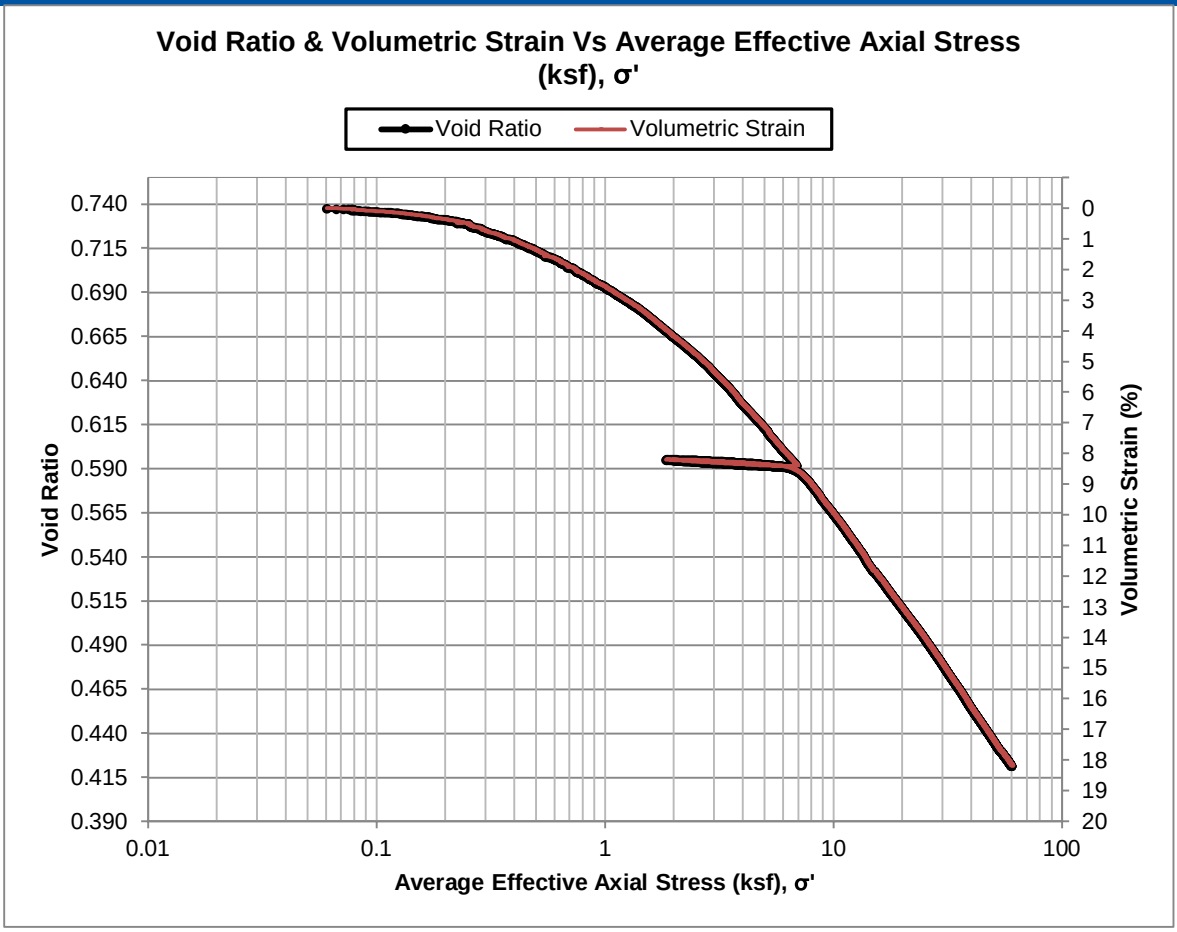
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	20.68	16.90	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	103.76	125.08	**PLASTIC LIMIT:**	
SATURATION (%):	79.31	100.00	**ASTM D854 - Measured**	
VOID RATIO:	0.578	0.309	**SPECIFIC GRAVITY**	2.627
STRAIN RATE (in/min):	0.000062			

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CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportaion Pier Ferry Project
PROJECT NO: 25022.000.001-P:001-T:003
PROJECT LOCATION: Berkeley, California
REPORT DATE: 10/21/2024
TESTED BY: D. Seibold
REVIEWED BY: O. Espinoza

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID: 1-B2 at 20-22.5 **DEPTH:** 22-22.5 ft

SOIL DESCRIPTION: See exploration logs

REMARKS: The specimen contained abundant shell fragments.

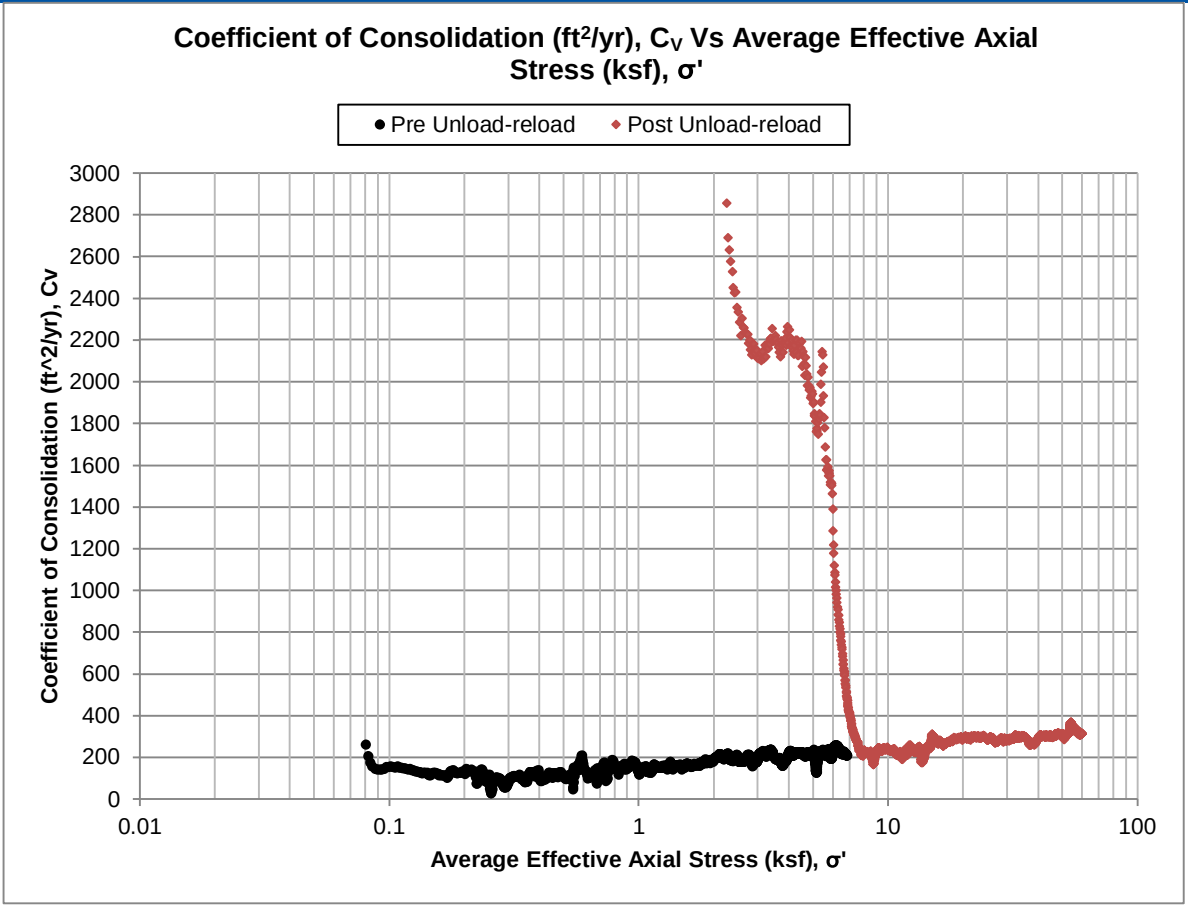
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	27.27	18.99	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	96.96	118.52	**PLASTIC LIMIT:**	
SATURATION (%):	100.00	100.00	**ASTM D854 - Measured**	
VOID RATIO:	0.737	0.421	**SPECIFIC GRAVITY**	2.703
STRAIN RATE (in/min):	0.000062			

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CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportaion Pier Ferry Project
PROJECT NO: 25022.000.001-P:001-T:003
PROJECT LOCATION: Berkeley, California
REPORT DATE: 10/21/2024
TESTED BY: D. Seibold
REVIEWED BY: O. Espinoza

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID:	1-B2 at 20-22.5	**DEPTH:**	22-22.5 ft
SOIL DESCRIPTION:	See exploration logs		
REMARKS:	The specimen contained abundant shell fragments.		

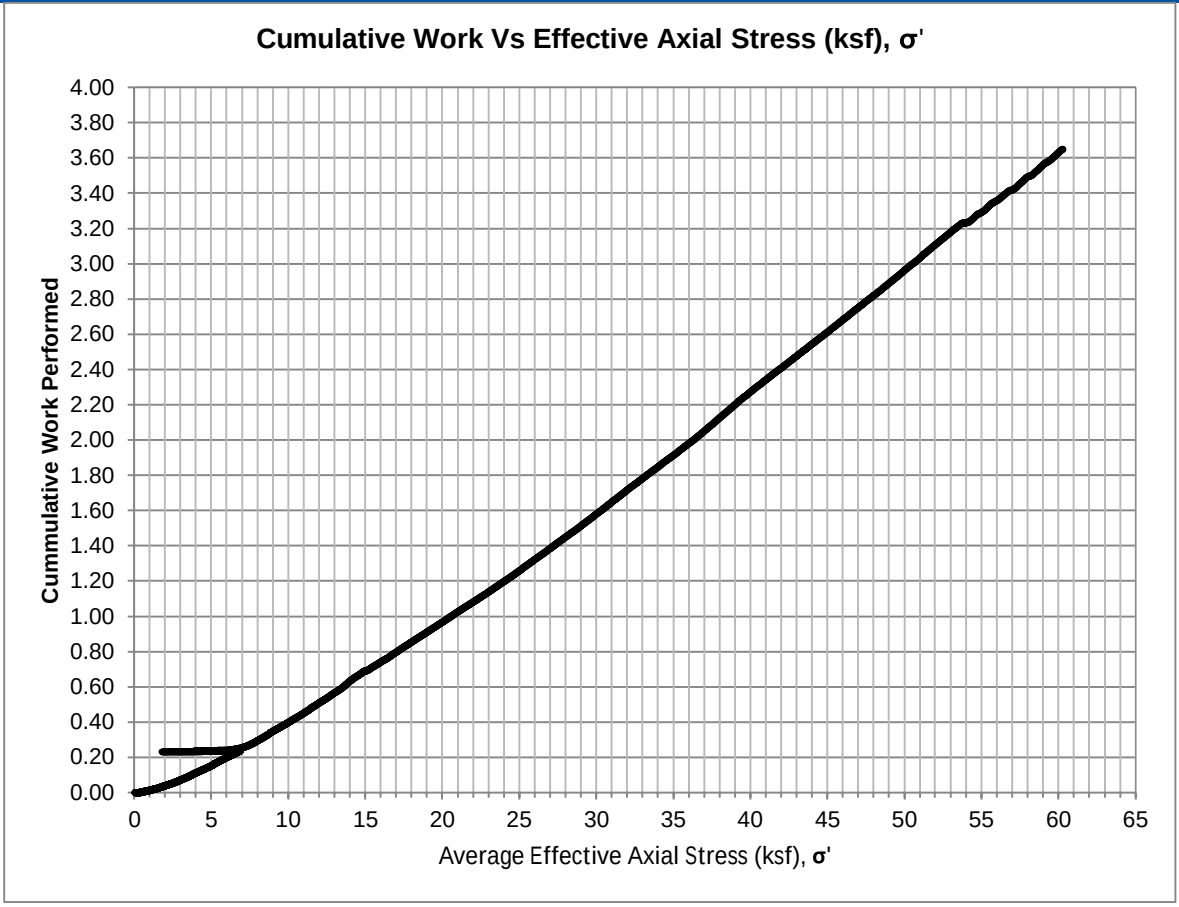
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	27.27	18.99	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	96.96	118.52	**PLASTIC LIMIT:**	
SATURATION (%):	100.00	100.00	**ASTM D854 - Measured**	
VOID RATIO:	0.737	0.421	**SPECIFIC GRAVITY**	2.703
STRAIN RATE (in/min):	0.000062			

ENGEO Expect Excellence

CLIENT:	COWI North America, Inc.
PROJECT NAME:	Berkeley Water Transportaion Pier Ferry Project
PROJECT NO:	25022.000.001-P:001-T:003
PROJECT LOCATION:	Berkeley, California
REPORT DATE:	10/21/2024
TESTED BY:	D. Seibold
REVIEWED BY:	O. Espinoza

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID: 1-B2 at 20-22.5 **DEPTH:** 22-22.5 ft

SOIL DESCRIPTION: See exploration logs

REMARKS: The specimen contained abundant shell fragments.

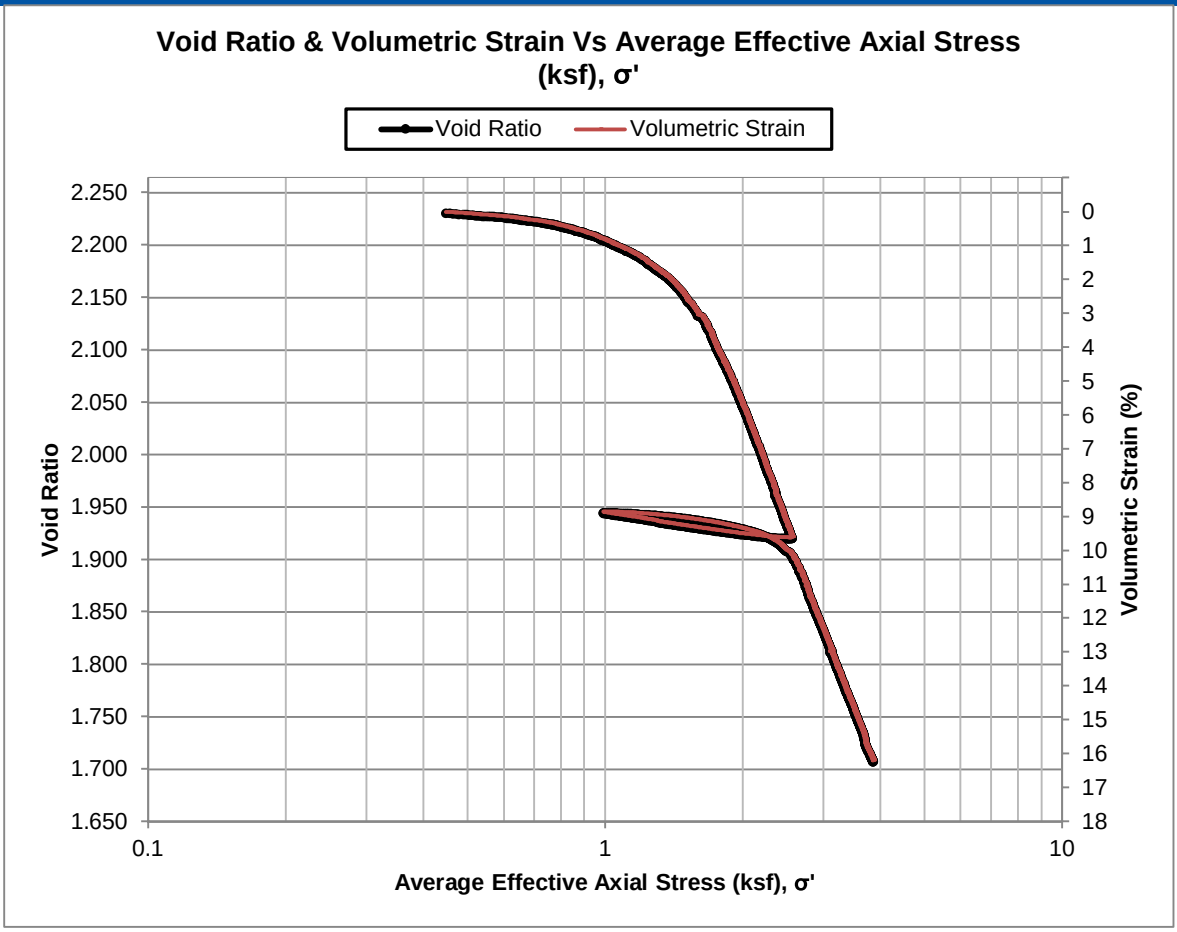
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	27.27	18.99	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	96.96	118.52	**PLASTIC LIMIT:**	
SATURATION (%):	100.00	100.00	**ASTM D854 - Measured**	
VOID RATIO:	0.737	0.421	**SPECIFIC GRAVITY**	2.703
STRAIN RATE (in/min):	0.000062			

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CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportaion Pier Ferry Project
PROJECT NO: 25022.000.001-P:001-T:003
PROJECT LOCATION: Berkeley, California
REPORT DATE: 10/21/2024
TESTED BY: D. Seibold
REVIEWED BY: O. Espinoza

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID:	1-B3 at 20-22.5	**DEPTH:**	21.5-22 ft
SOIL DESCRIPTION:	See exploration logs		
REMARKS:			

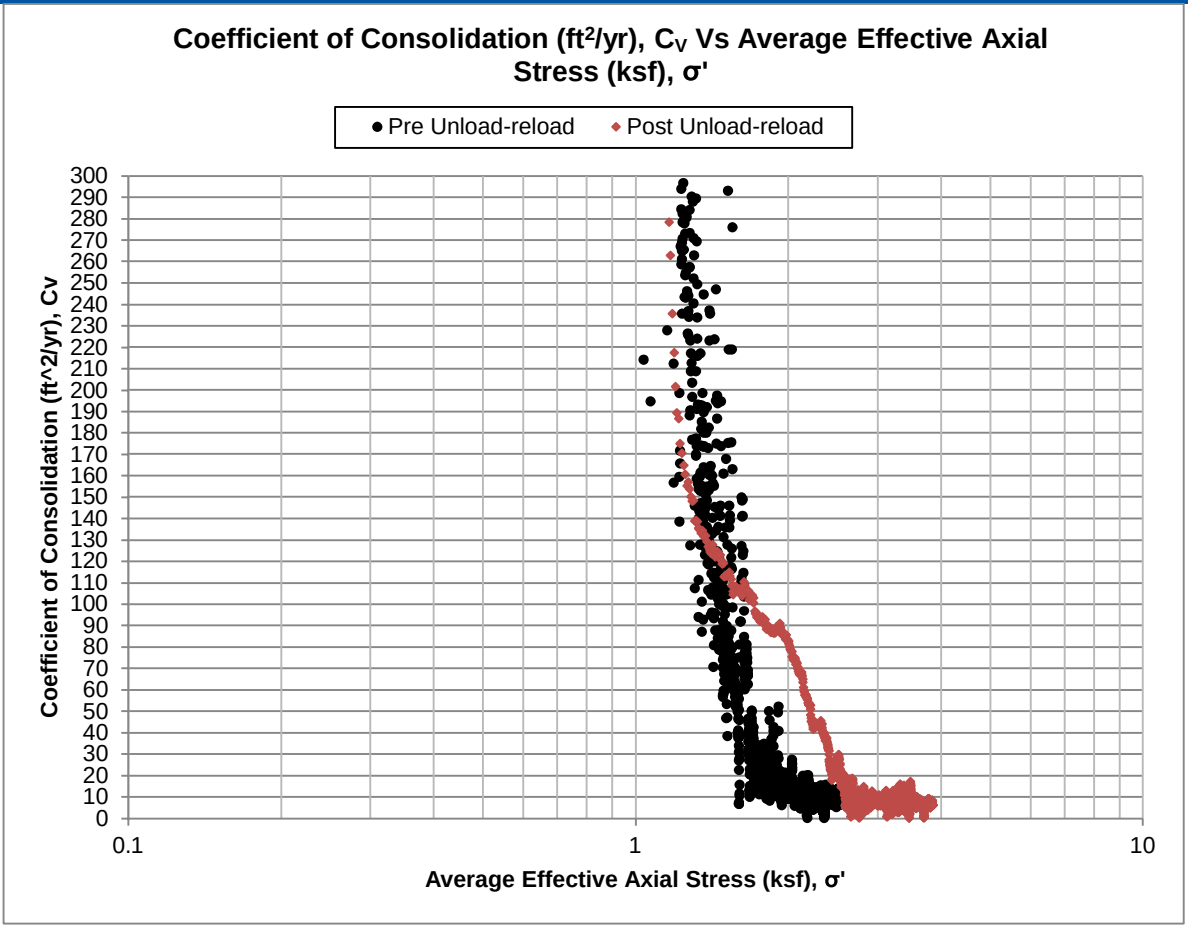
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	86.72	75.48	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	49.16	58.66	**PLASTIC LIMIT:**	
SATURATION (%):	99.08	100.00	**ASTM D854 - Measured**	
VOID RATIO:	2.230	1.707	**SPECIFIC GRAVITY**	2.548
STRAIN RATE (in/min):	0.000042			

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CLIENT:	COWI North America, Inc.
PROJECT NAME:	Berkeley Water Transportaion Pier Ferry Project
PROJECT NO:	25022.000.001-P:001-T:003
PROJECT LOCATION:	Berkeley, California
REPORT DATE:	10/21/2024
TESTED BY:	K. Nguyen
REVIEWED BY:	D. Seibold

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID:	1-B3 at 20-22.5	**DEPTH:**	21.5-22 ft
SOIL DESCRIPTION:	See exploration logs		
REMARKS:			

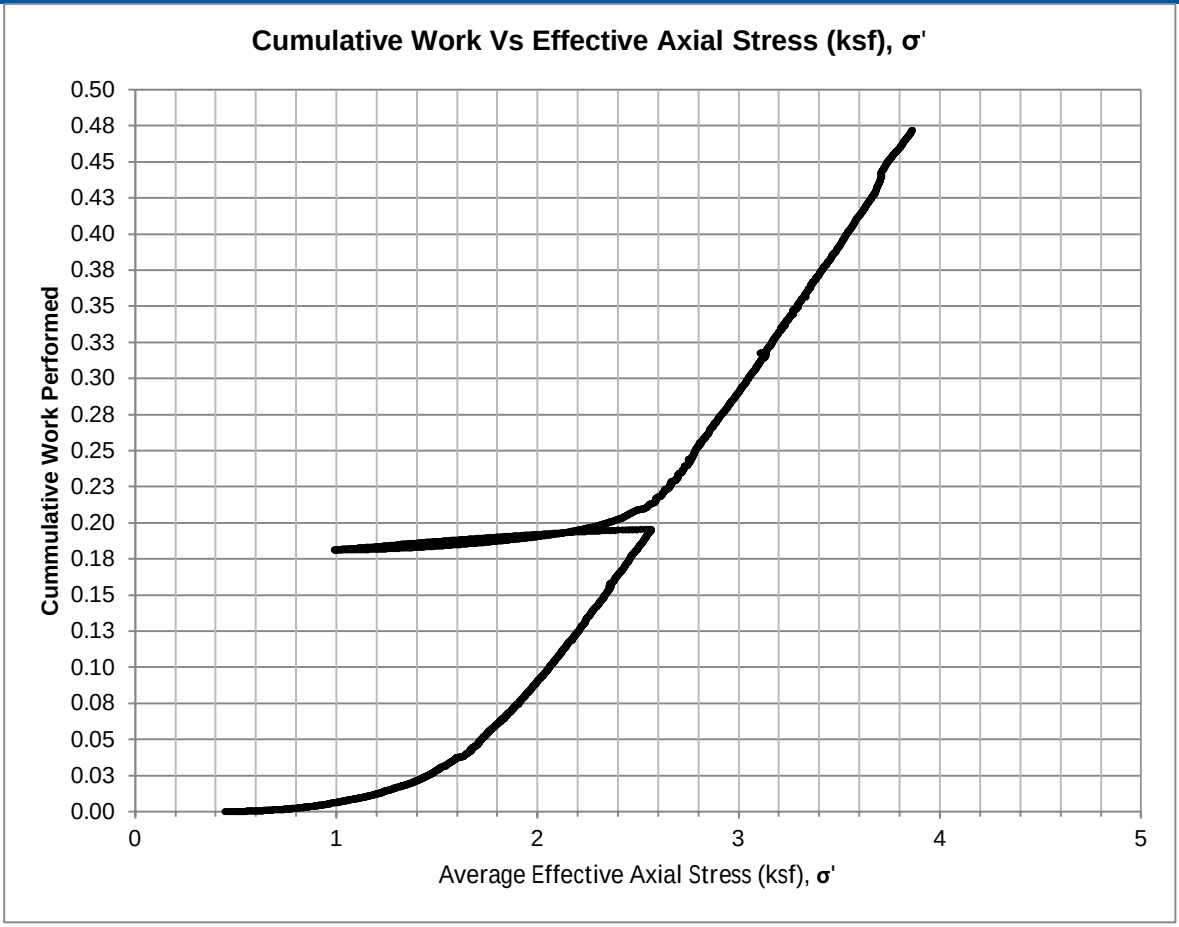
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	86.72	75.48	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	49.16	58.66	**PLASTIC LIMIT:**	
SATURATION (%):	99.08	100.00	**ASTM D854 - Measured**	
VOID RATIO:	2.230	1.707	**SPECIFIC GRAVITY**	2.548
STRAIN RATE (in/min):	0.000042			

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CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportaion Pier Ferry Project
PROJECT NO: 25022.000.001-P:001-T:003
PROJECT LOCATION: Berkeley, California
REPORT DATE: 10/21/2024
TESTED BY: K. Nguyen
REVIEWED BY: D. Seibold

Constant Rate of Strain Consolidation
ASTM D4186



SPECIMEN INFORMATION

SAMPLE ID: 1-B3 at 20-22.5 **DEPTH:** 21.5-22 ft

SOIL DESCRIPTION: See exploration logs

REMARKS:

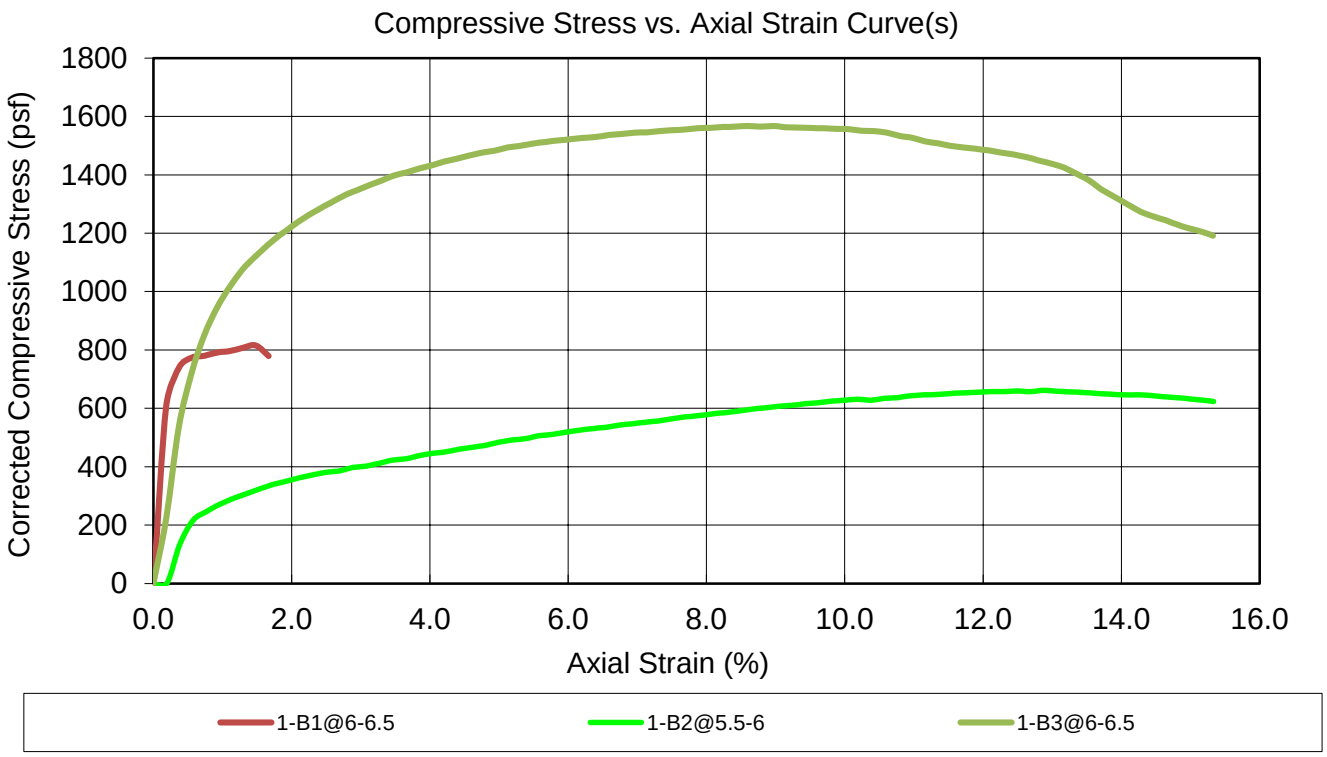
TEST DATA

	INITIAL	FINAL	ASTM D4318 - Wet Method	
MOISTURE CONTENT (%):	86.72	75.48	**LIQUID LIMIT:**	
DRY DENSITY (pcf):	49.16	58.66	**PLASTIC LIMIT:**	
SATURATION (%):	99.08	100.00	**ASTM D854 - Measured**	
VOID RATIO:	2.230	1.707	**SPECIFIC GRAVITY**	2.548
STRAIN RATE (in/min):	0.000042			

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CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportaion Pier Ferry Project
PROJECT NO: 25022.000.001-P:001-T:003
PROJECT LOCATION: Berkeley, California
REPORT DATE: 10/21/2024
TESTED BY: K. Nguyen
REVIEWED BY: D. Seibold

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B1@6-6.5	SPECIMEN 1-B2@5.5-6	SPECIMEN 1-B3@6-6.5
Test Moisture Content (%)	7.26	20.99	23.51
Dry Density (pcf)	110.5	107.8	100.2
Saturation (%)	36.7	99.2	92.1
Void Ratio	0.54	0.58	0.69
Diameter (in)	2.385	2.402	2.399
Height (in)	5.403	5.203	5.467
Height-To-Diameter Ratio	2.27	2.17	2.28
TEST DATA			
Unconfined Compressive Strength (psf)	816	662	1568
Undrained Shear Strength (psf)	408.00	331.11	783.95
Strain Rate (in/min)	0.050	0.050	0.050
Specific Gravity (ASSUMED)	2.720	2.720	2.720
Strain at Failure(%)	1.48	12.88	8.96
Test Remarks			

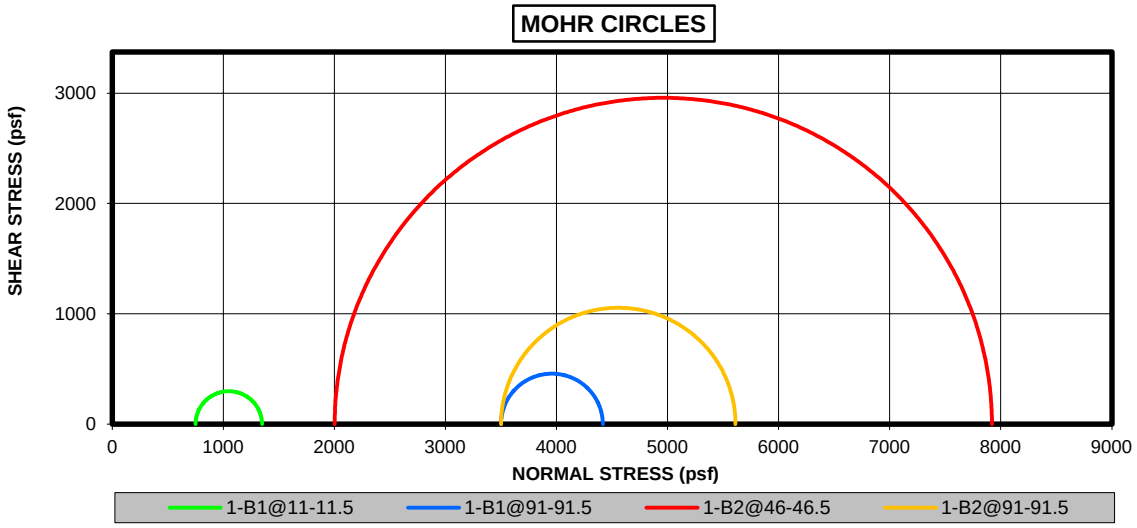
SPECIMEN	DESCRIPTION
1-B1@6-6.5	See exploration logs
1-B2@5.5-6	See exploration logs
1-B3@6-6.5	See exploration logs

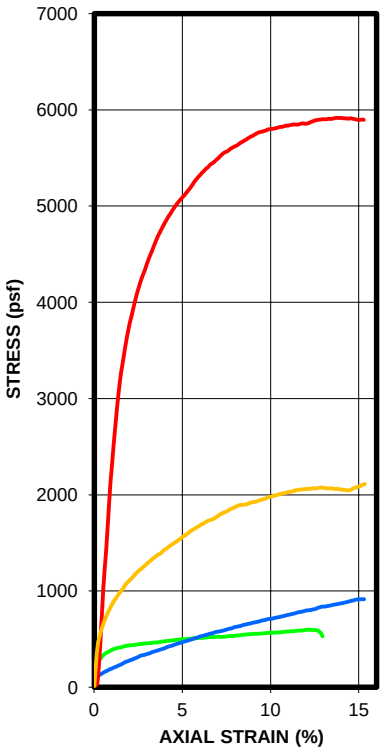
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
CLIENT: COWI North America, Inc.
LOCATION: Berkeley, CA

Test Date: 10/11/24
Tested By: L. Schmitz
Reviewed By: M. Gilbert

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ISOTROPIC UNCONSOLIDATED UNDRAINED TRIAXIAL REPORT
ASTM D2850





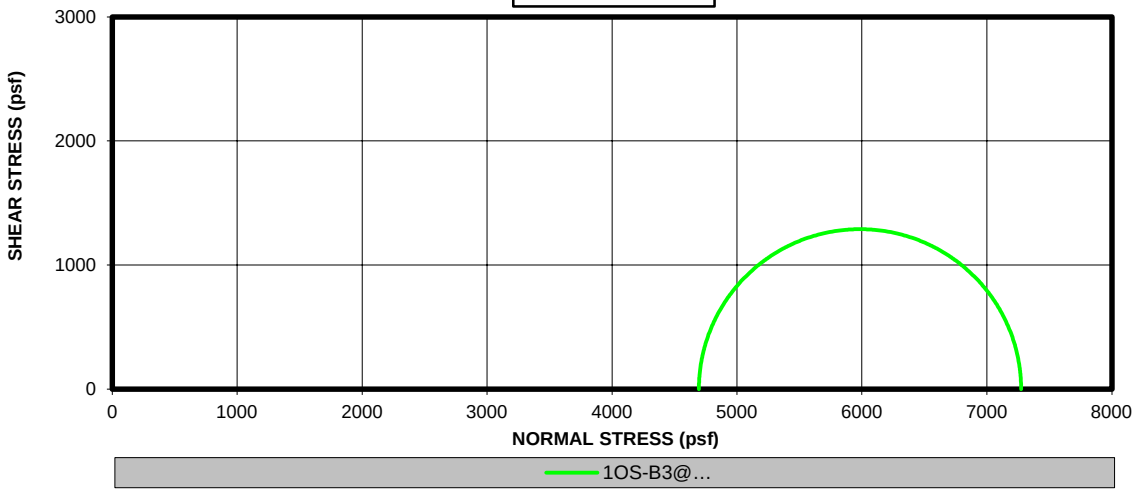
	SPECIMEN			
INITIAL PARAMETERS	**1-B1@11-11.5**	**1-B1@91-91.5**	**1-B2@46-46.5**	**1-B2@91-91.5**
MOISTURE (%)	19.84	23.24	23.61	34.99
DRY DENSITY (PCF)	94.70	103.90	103.20	86.70
SATURATION (%)	68.09	99.56	99.57	99.31
VOID RATIO	0.793	0.635	0.645	0.958
DIAMETER (IN.)	2.403	2.278	2.402	2.392
HEIGHT (IN.)	5.443	5.657	5.920	5.583
DIAMETER-TO-HEIGHT RATIO	2.265	2.483	2.465	2.334
LIQUID LIMIT (ASTM D4318)	n/a	n/a	n/a	n/a
PLASTIC LIMIT (ASTM D4318)	n/a	n/a	n/a	n/a
SPECIFIC GRAVITY (ASTM D854)	2.720	2.720	2.720	2.720
FINAL PARAMETERS	**1-B1@11-11.5**	**1-B1@91-91.5**	**1-B2@46-46.5**	**1-B2@91-91.5**
MOISTURE (%)	19.84	23.24	23.61	34.99
SATURATION (%)	68.09	99.56	99.57	99.31
STRAIN RATE (%/MIN.)	0.920	0.880	0.840	0.900
PEAK DEVIATOR STRESS (PSF)	598.4	916.5	5917.9	2111.5
AXIAL STRAIN AT FAILURE (%)	12.12	15.29	13.68	15.32
CELL PRESSURE				
CELL PRESSURE (PSF)	748.8	3499.2	2001.6	3499.2
BACK PRESSURE (PSF)	n/a	n/a	n/a	n/a
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	1347.2	4415.7	7919.5	5610.7
σ3 (PSF)	748.8	3499.2	2001.6	3499.2
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	299.2	458.3	2959.0	1055.8
REMARKS				

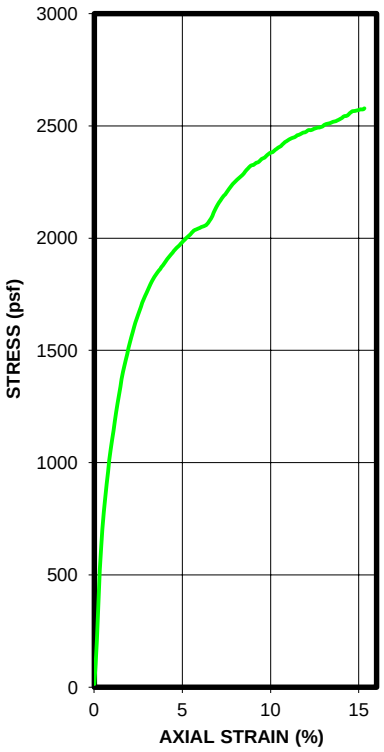
ENGEO
Expect Excellence

CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/18/2024
TESTED BY: L. Schmitz
REVIEWED BY: N. Broussard

ISOTROPIC UNCONSOLIDATED UNDRAINED TRIAXIAL REPORT
ASTM D2850

MOHR CIRCLES





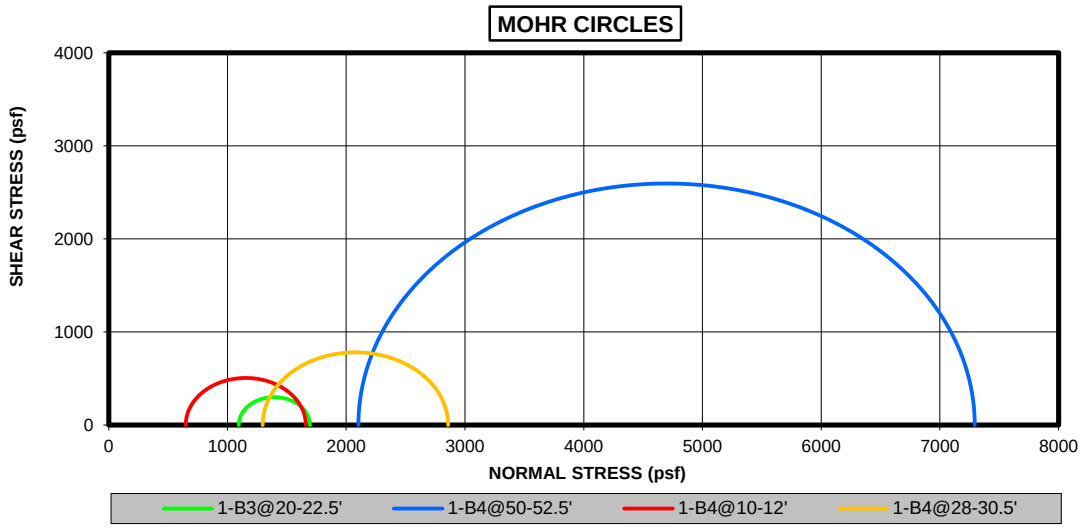
	SPECIMEN			
INITIAL PARAMETERS	1OS-B3@ 122-124			
MOISTURE (%)	39.79			
DRY DENSITY (PCF)	81.40			
SATURATION (%)	99.56			
VOID RATIO	1.087			
DIAMETER (IN.)	2.839			
HEIGHT (IN.)	6.427			
DIAMETER-TO-HEIGHT RATIO	2.264			
LIQUID LIMIT (ASTM D4318)	n/a			
PLASTIC LIMIT (ASTM D4318)	n/a			
SPECIFIC GRAVITY (ASTM D854)	2.720			
FINAL PARAMETERS	1OS-B3@ 122-124			
MOISTURE (%)	39.79			
SATURATION (%)	99.56			
STRAIN RATE (%/MIN.)	0.930			
PEAK DEVIATOR STRESS (PSF)	2578.9			
AXIAL STRAIN AT FAILURE (%)	15.31			
CELL PRESSURE				
CELL PRESSURE (PSF)	4694.4			
BACK PRESSURE (PSF)	n/a			
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	7273.3			
σ3 (PSF)	4694.4			
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	1289.4			
REMARKS				

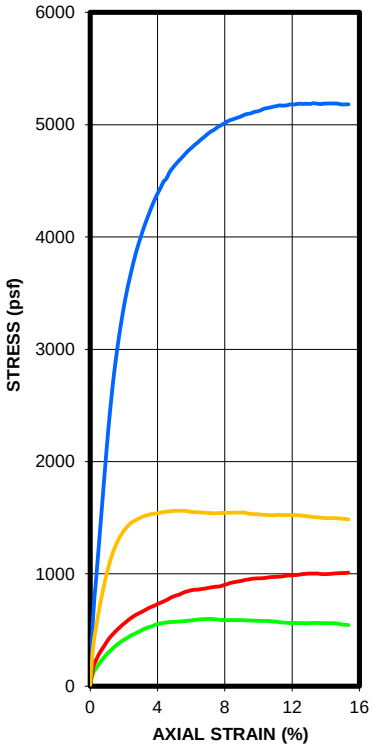
ENGEO
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CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/18/2024
TESTED BY: L. Schmitz
REVIEWED BY: N. Broussard

Please Select Address

ISOTROPIC UNCONSOLIDATED UNDRAINED TRIAXIAL REPORT
ASTM D2850





	SPECIMEN			
INITIAL PARAMETERS	**1-B3@20-22.5'**	**1-B4@50-52.5'**	**1-B4@10-12'**	**1-B4@28-30.5'**
MOISTURE (%)	74.39	18.08	21.79	33.31
DRY DENSITY (PCF)	56.00	111.90	105.10	89.70
SATURATION (%)	99.47	95.02	96.35	100.00
VOID RATIO	2.034	0.517	0.615	0.894
DIAMETER (IN.)	2.852	2.868	2.846	2.838
HEIGHT (IN.)	5.713	5.961	5.634	5.969
DIAMETER-TO-HEIGHT RATIO	2.003	2.078	1.980	2.103
LIQUID LIMIT (ASTM D4318)			45	
PLASTIC LIMIT (ASTM D4318)			22	
SPECIFIC GRAVITY (ASTM D854)	2.720	2.720	2.720	2.720
FINAL PARAMETERS	**1-B3@20-22.5'**	**1-B4@50-52.5'**	**1-B4@10-12'**	**1-B4@28-30.5'**
MOISTURE (%)	74.39	18.08	21.79	33.31
SATURATION (%)	99.47	95.02	96.35	100.00
STRAIN RATE (%/MIN.)	0.057	0.060	0.056	0.060
PEAK DEVIATOR STRESS (PSF)	599.3	5191.0	1011.5	1562.1
AXIAL STRAIN AT FAILURE (%)	7.002	13.253	15.332	5.194
CELL PRESSURE				
CELL PRESSURE (PSF)	1094.4	2102.4	648.0	1296.0
BACK PRESSURE (PSF)	n/a	n/a	n/a	n/a
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	1693.7	7293.4	1659.5	2858.1
σ3 (PSF)	1094.4	2102.4	648.0	1296.0
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	299.6	2595.5	505.7	781.1
REMARKS				

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CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/22/2024
TESTED BY: Y. Cabrales
REVIEWED BY: D. Seibold

ISOTROPIC UNCONSOLIDATED UNDRAINED TRIAXIAL REPORT
ASTM D2850





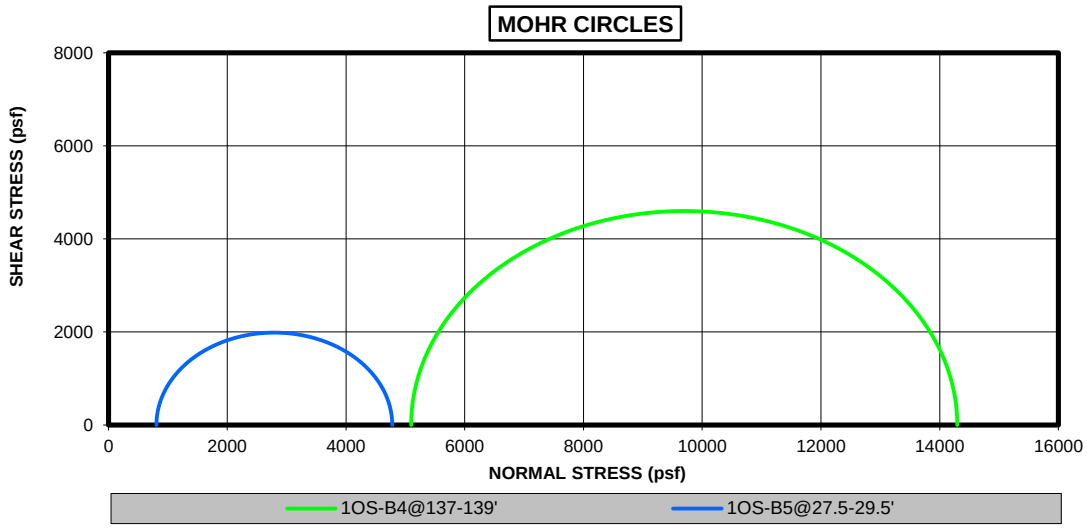


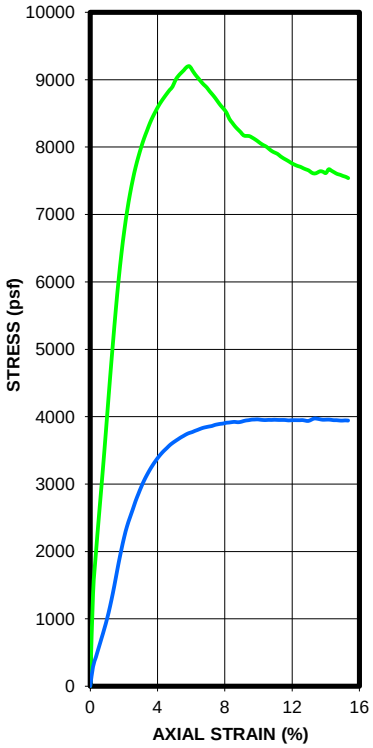


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CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
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REPORT DATE: 10/22/2024
TESTED BY: Y. Cabrales
REVIEWED BY: D. Seibold

ISOTROPIC UNCONSOLIDATED UNDRAINED TRIAXIAL REPORT
ASTM D2850



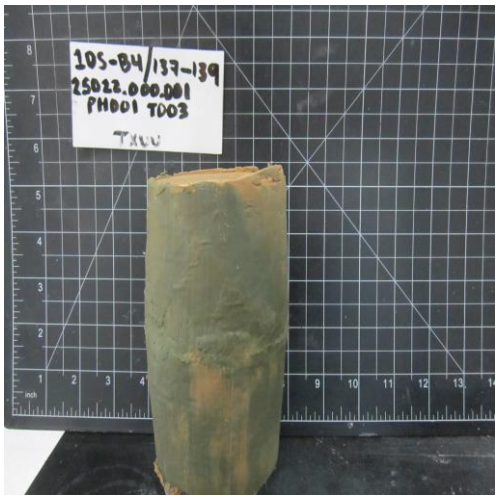


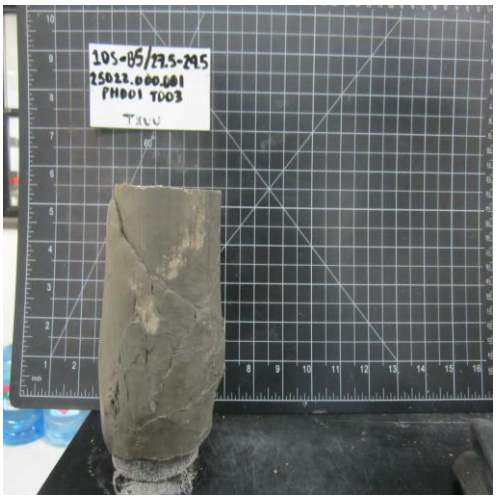
	SPECIMEN			
INITIAL PARAMETERS	**1OS-B4@137-139'**	**1OS-B5@27.5-29.5'**	**C**	**D**
MOISTURE (%)	19.24	25.86		
DRY DENSITY (PCF)	110.80	96.80		
SATURATION (%)	98.20	93.15		
VOID RATIO	0.533	0.755		
DIAMETER (IN.)	2.868	2.897		
HEIGHT (IN.)	5.926	5.942		
DIAMETER-TO-HEIGHT RATIO	2.066	2.051		
LIQUID LIMIT (ASTM D4318)				
PLASTIC LIMIT (ASTM D4318)				
SPECIFIC GRAVITY (ASTM D854)	2.720	2.720		
FINAL PARAMETERS	**1OS-B4@137-139'**	**1OS-B5@27.5-29.5'**	**C**	**D**
MOISTURE (%)	19.24	25.86		
SATURATION (%)	98.20	93.15		
STRAIN RATE (%/MIN.)	0.059	0.059		
PEAK DEVIATOR STRESS (PSF)	9198.9	3971.9		
AXIAL STRAIN AT FAILURE (%)	5.906	13.295		
CELL PRESSURE				
CELL PRESSURE (PSF)	5097.6	806.4		
BACK PRESSURE (PSF)	n/a	n/a		
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	14296.5	4778.3		
σ3 (PSF)	5097.6	806.4		
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	4599.5	1985.9		
REMARKS				

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CLIENT: COWI North America, Inc.
PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/24/2024
TESTED BY: Y. Cabrales
REVIEWED BY: D. Seibold

ISOTROPIC UNCONSOLIDATED UNDRAINED TRIAXIAL REPORT
ASTM D2850

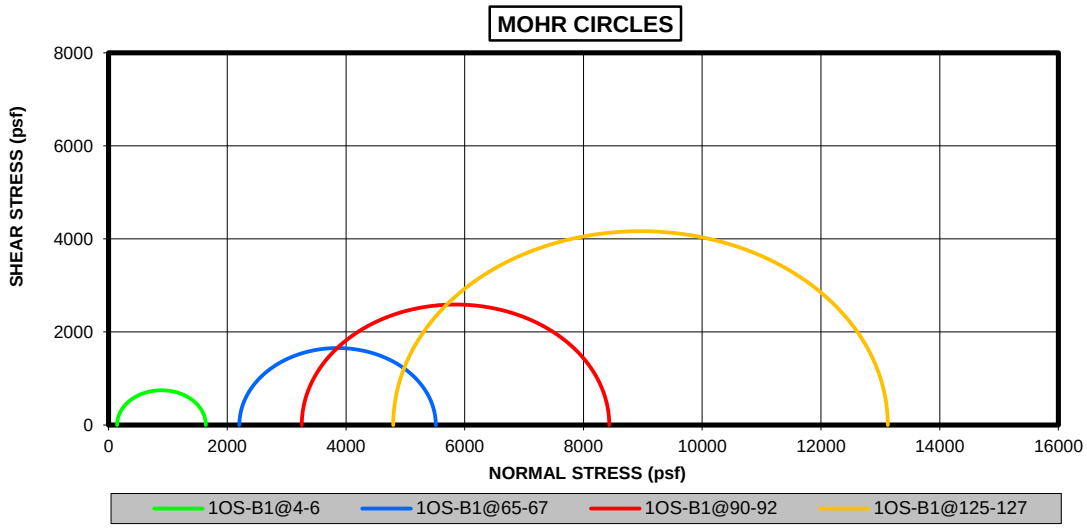


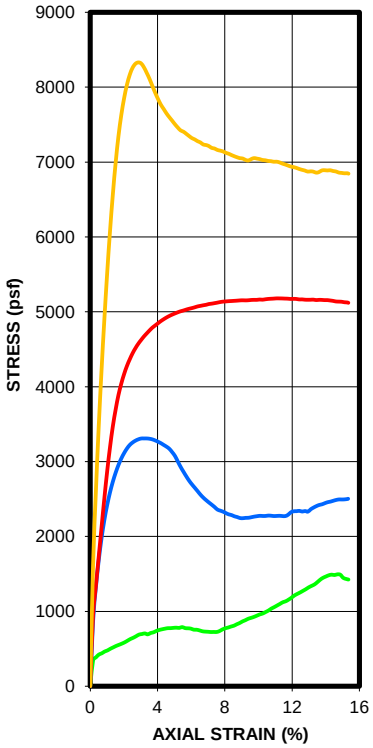


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PROJECT NAME: Berkeley Water Transportation Pier Ferry Project
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/24/2024
TESTED BY: Y. Cabrales
REVIEWED BY: D. Seibold

ISOTROPIC UNCONSOLIDATED UNDRAINED TRIAXIAL REPORT
ASTM D2850



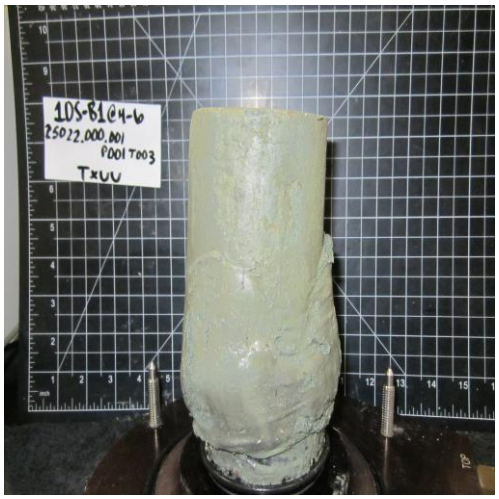


	SPECIMEN			
INITIAL PARAMETERS	**1OS-B1@4-6**	**1OS-B1@65-67**	**1OS-B1@90-92**	**1OS-B1@125-127**
MOISTURE (%)	22.05	43.06	23.61	26.05
DRY DENSITY (PCF)	108.80	78.60	102.60	99.80
SATURATION (%)	100.00	100.00	98.14	100.00
VOID RATIO	0.561	1.160	0.654	0.701
DIAMETER (IN.)	2.858	2.870	2.866	2.868
HEIGHT (IN.)	5.854	5.952	5.958	5.958
DIAMETER-TO-HEIGHT RATIO	2.048	2.074	2.079	2.077
LIQUID LIMIT (ASTM D4318)				
PLASTIC LIMIT (ASTM D4318)				
SPECIFIC GRAVITY (ASTM D854)	2.720	2.720	2.720	2.720
FINAL PARAMETERS	**1OS-B1@4-6**	**1OS-B1@65-67**	**1OS-B1@90-92**	**1OS-B1@125-127**
MOISTURE (%)	22.05	43.06	23.61	26.05
SATURATION (%)	100.00	100.00	98.14	100.00
STRAIN RATE (%/MIN.)	0.059	0.060	0.060	0.060
PEAK DEVIATOR STRESS (PSF)	1494.4	3309.0	5179.8	8330.9
AXIAL STRAIN AT FAILURE (%)	14.691	3.193	11.078	2.853
CELL PRESSURE				
CELL PRESSURE (PSF)	144.0	2203.2	3254.4	4795.2
BACK PRESSURE (PSF)	n/a	n/a	n/a	n/a
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	1638.4	5512.2	8434.2	13126.1
σ3 (PSF)	144.0	2203.2	3254.4	4795.2
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	747.2	1654.5	2589.9	4165.5
REMARKS				

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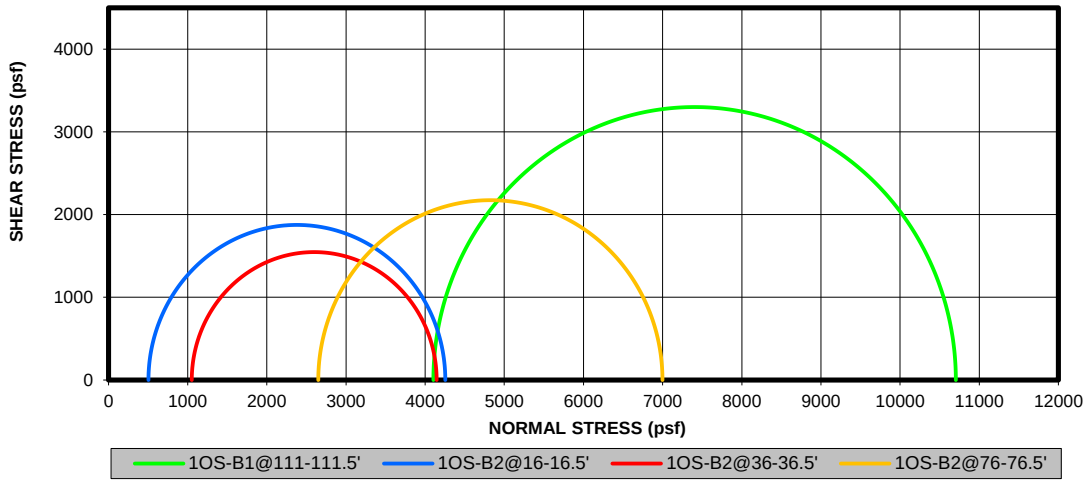


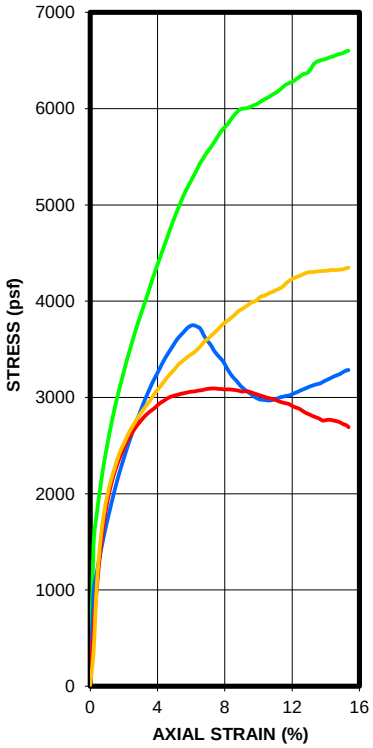
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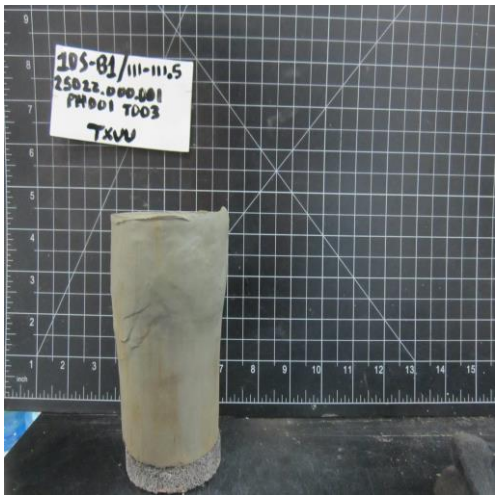


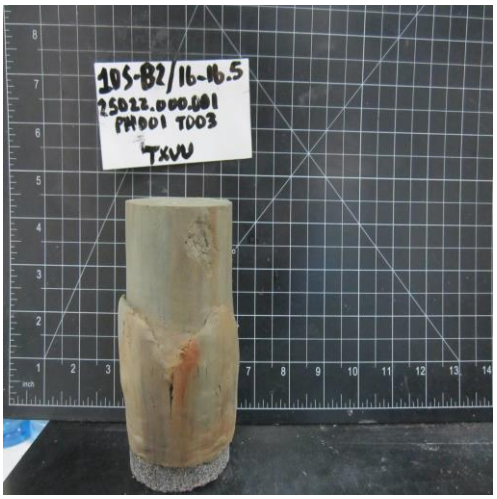
	SPECIMEN			
INITIAL PARAMETERS	1OS-B1@111-111.5'	1OS-B2@16-16.5'	1OS-B2@36-36.5'	1OS-B2@76-76.5'
MOISTURE (%)	27.50	17.98	30.43	32.54
DRY DENSITY (PCF)	96.40	114.00	94.20	90.50
SATURATION (%)	98.26	99.94	100.00	100.00
VOID RATIO	0.761	0.490	0.802	0.876
DIAMETER (IN.)	2.412	2.411	2.407	2.391
HEIGHT (IN.)	5.057	5.049	4.993	5.001
DIAMETER-TO-HEIGHT RATIO	2.097	2.094	2.074	2.092
LIQUID LIMIT (ASTM D4318)				
PLASTIC LIMIT (ASTM D4318)				
SPECIFIC GRAVITY (ASTM D854)	2.720	2.720	2.720	2.720
FINAL PARAMETERS	1OS-B1@111-111.5'	1OS-B2@16-16.5'	1OS-B2@36-36.5'	1OS-B2@76-76.5'
MOISTURE (%)	27.50	17.98	30.43	32.54
SATURATION (%)	98.26	99.93	100.00	100.00
STRAIN RATE (%/MIN.)	0.051	0.050	0.050	0.050
PEAK DEVIATOR STRESS (PSF)	6600.3	3750.2	3093.4	4348.3
AXIAL STRAIN AT FAILURE (%)	15.343	6.140	7.410	15.345
CELL PRESSURE				
CELL PRESSURE (PSF)	4104.0	504.0	1051.2	2649.6
BACK PRESSURE (PSF)	n/a	n/a	n/a	n/a
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	10704.3	4254.2	4144.6	6997.9
σ3 (PSF)	4104.0	504.0	1051.2	2649.6
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	3300.2	1875.1	1546.7	2174.1
REMARKS				

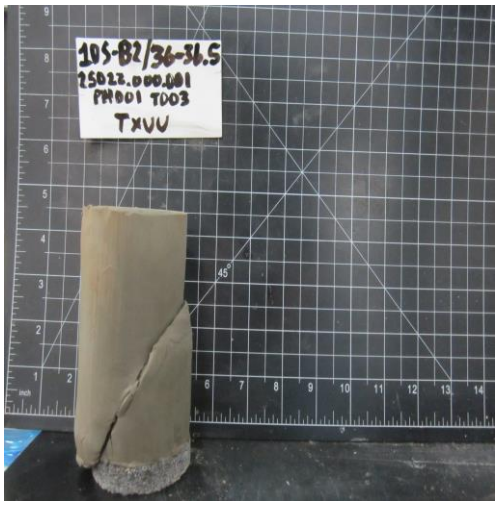
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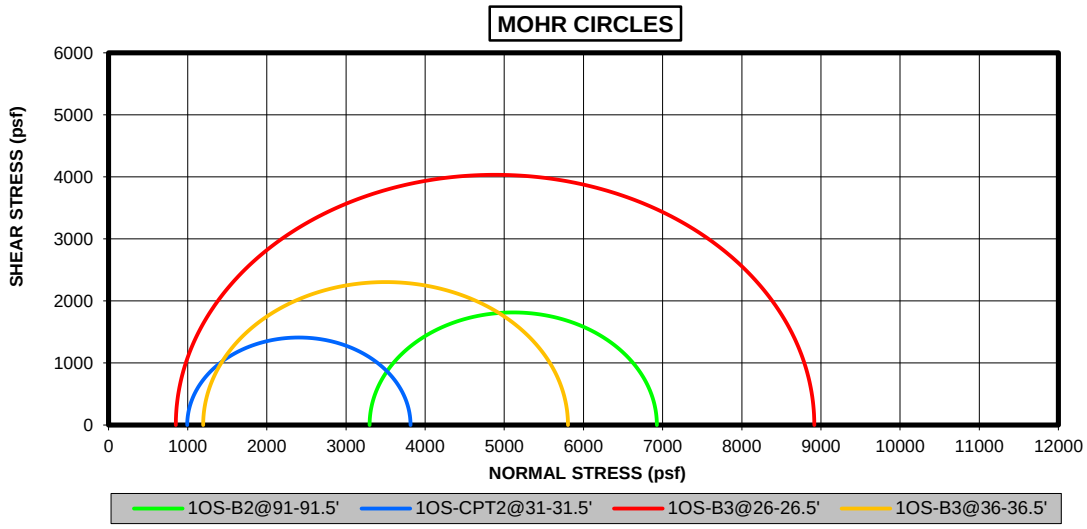


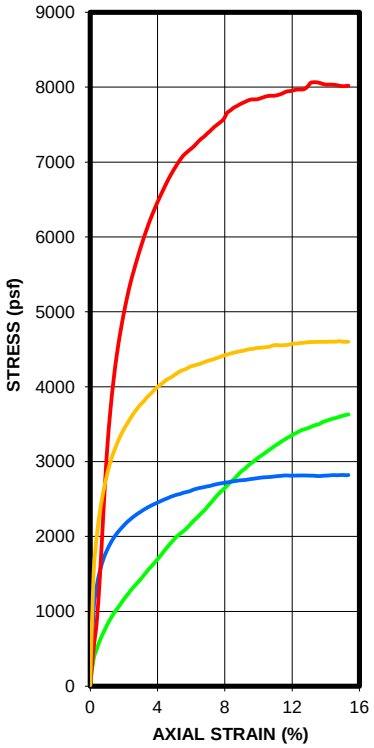


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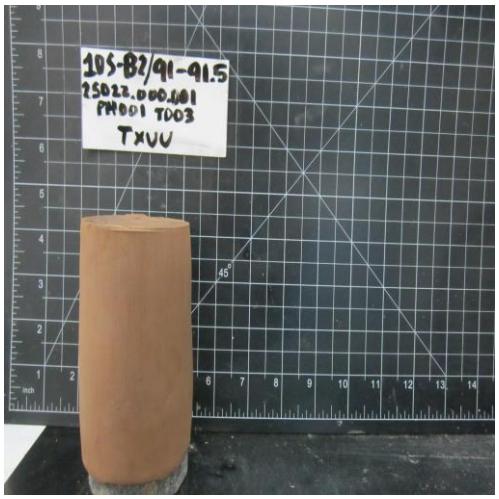


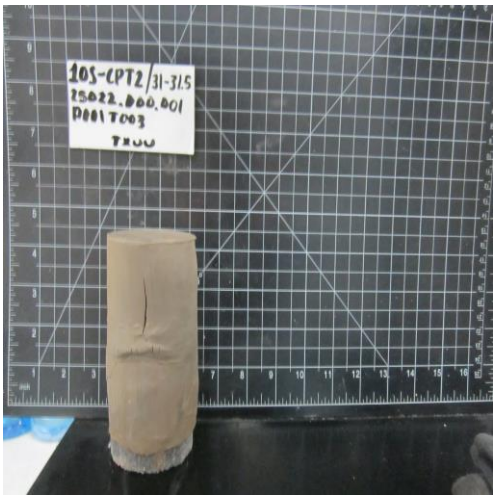
	SPECIMEN			
INITIAL PARAMETERS	1OS-B2@91-91.5'	1OS-CPT2@31-31.5'	1OS-B3@26-26.5'	1OS-B3@36-36.5'
MOISTURE (%)	31.65	28.11	24.76	23.35
DRY DENSITY (PCF)	92.50	96.40	102.30	104.50
SATURATION (%)	100.00	100.00	100.00	100.00
VOID RATIO	0.837	0.761	0.660	0.624
DIAMETER (IN.)	2.405	2.392	2.389	2.398
HEIGHT (IN.)	5.000	5.054	5.040	5.061
DIAMETER-TO-HEIGHT RATIO	2.079	2.113	2.110	2.111
LIQUID LIMIT (ASTM D4318)				
PLASTIC LIMIT (ASTM D4318)				
SPECIFIC GRAVITY (ASTM D854)	2.720	2.720	2.720	2.720
FINAL PARAMETERS	1OS-B2@91-91.5'	1OS-CPT2@31-31.5'	1OS-B3@26-26.5'	1OS-B3@36-36.5'
MOISTURE (%)	31.65	28.11	24.76	23.35
SATURATION (%)	100.00	100.00	100.00	100.00
STRAIN RATE (%/MIN.)	0.050	0.051	0.050	0.051
PEAK DEVIATOR STRESS (PSF)	3629.5	2821.2	8066.6	4608.9
AXIAL STRAIN AT FAILURE (%)	15.353	15.344	13.294	14.819
CELL PRESSURE				
CELL PRESSURE (PSF)	3297.6	993.6	849.6	1195.2
BACK PRESSURE (PSF)	n/a	n/a	n/a	n/a
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	6927.1	3814.8	8916.2	5804.1
σ3 (PSF)	3297.6	993.6	849.6	1195.2
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	1814.8	1410.6	4033.3	2304.4
REMARKS				

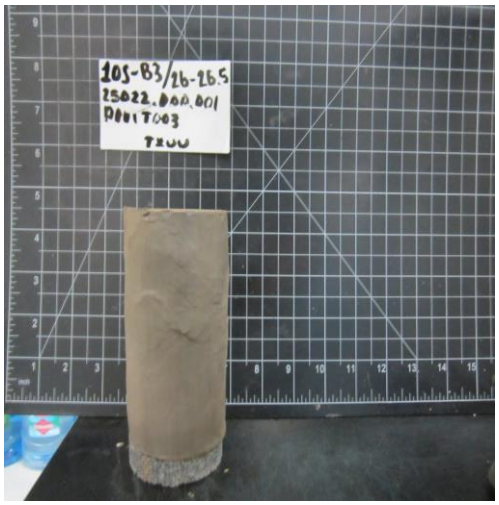
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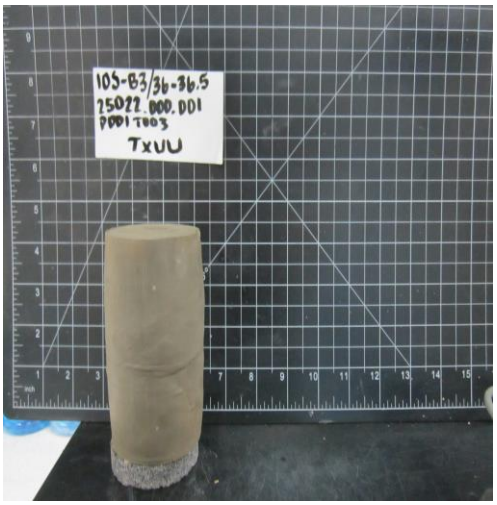
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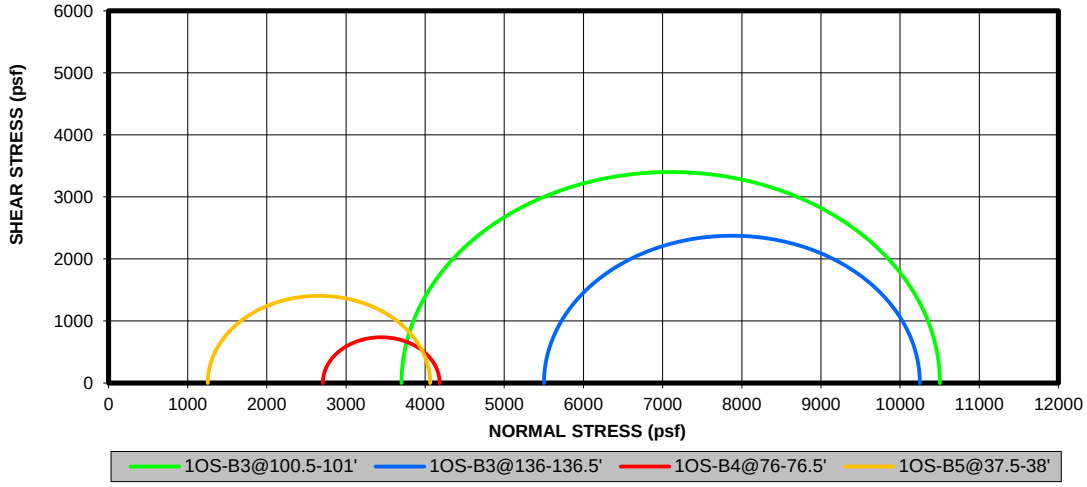


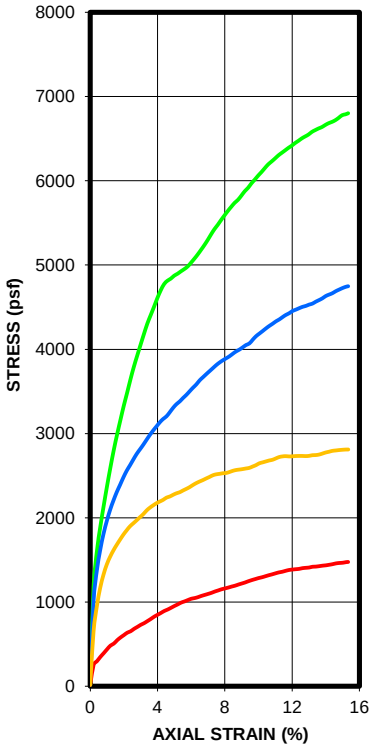
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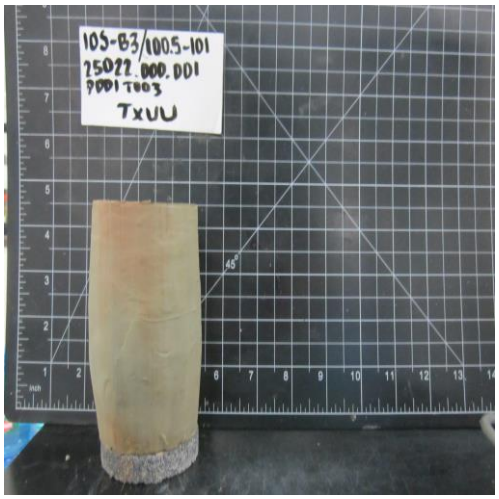


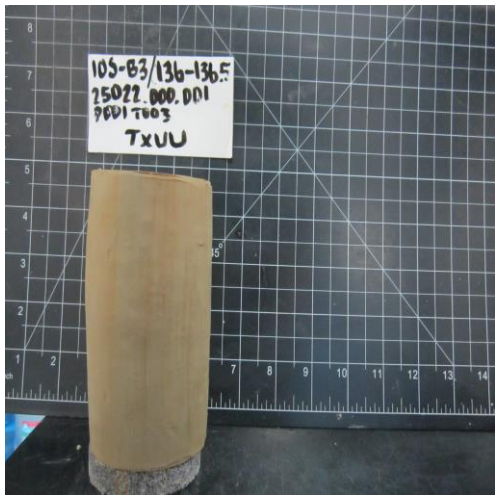
	SPECIMEN			
INITIAL PARAMETERS	1OS-B3@100.5-101'	1OS-B3@136-136.5'	1OS-B4@76-76.5'	1OS-B5@37.5-38'
MOISTURE (%)	22.66	19.13	42.57	26.24
DRY DENSITY (PCF)	106.40	112.00	80.00	99.20
SATURATION (%)	100.00	100.00	100.00	100.00
VOID RATIO	0.595	0.516	1.124	0.711
DIAMETER (IN.)	2.399	2.403	2.383	2.408
HEIGHT (IN.)	5.010	5.073	4.996	5.076
DIAMETER-TO-HEIGHT RATIO	2.088	2.111	2.097	2.108
LIQUID LIMIT (ASTM D4318)				
PLASTIC LIMIT (ASTM D4318)				
SPECIFIC GRAVITY (ASTM D854)	2.720	2.720	2.720	2.720
FINAL PARAMETERS	1OS-B3@100.5-101'	1OS-B3@136-136.5'	1OS-B4@76-76.5'	1OS-B5@37.5-38'
MOISTURE (%)	22.66	19.13	42.57	26.24
SATURATION (%)	100.00	100.00	100.00	100.00
STRAIN RATE (%/MIN.)	0.050	0.051	0.050	0.051
PEAK DEVIATOR STRESS (PSF)	6801.9	4748.4	1475.0	2809.9
AXIAL STRAIN AT FAILURE (%)	15.331	15.331	15.330	15.337
CELL PRESSURE				
CELL PRESSURE (PSF)	3700.8	5500.8	2707.2	1252.8
BACK PRESSURE (PSF)	n/a	n/a	n/a	n/a
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	10502.7	10249.2	4182.2	4062.7
σ3 (PSF)	3700.8	5500.8	2707.2	1252.8
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	3401.0	2374.2	737.5	1405.0
REMARKS				

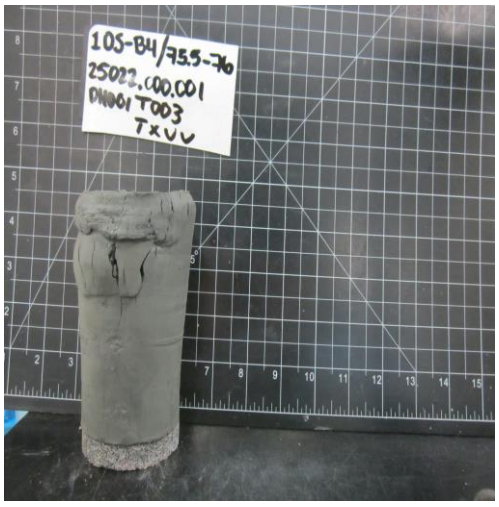
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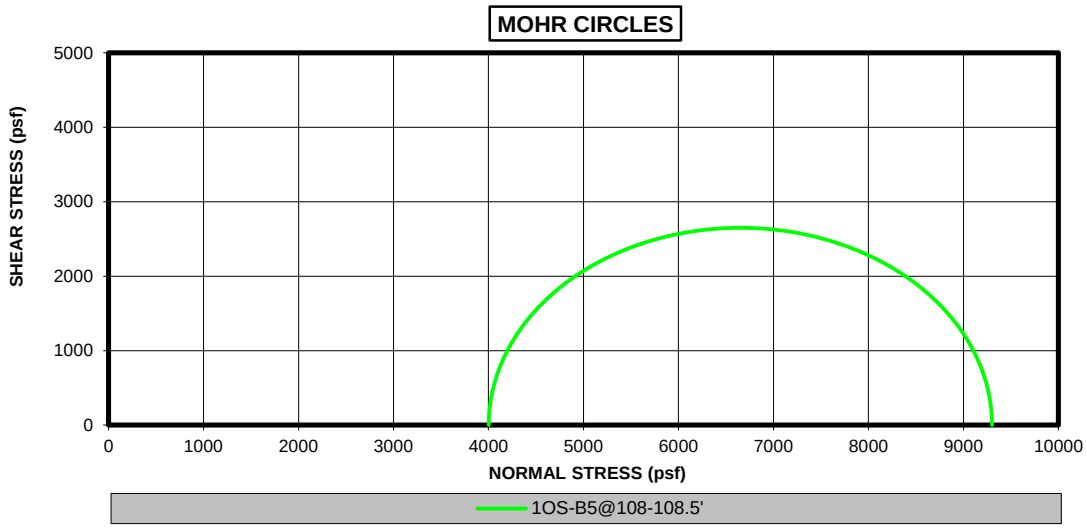


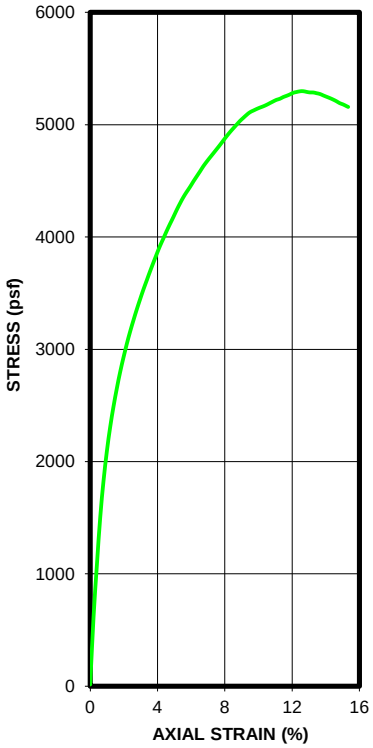


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	SPECIMEN			
INITIAL PARAMETERS	1OS-B5@108-108.5'			
MOISTURE (%)	20.48			
DRY DENSITY (PCF)	108.80			
SATURATION (%)	99.42			
VOID RATIO	0.560			
DIAMETER (IN.)	2.408			
HEIGHT (IN.)	5.056			
DIAMETER-TO-HEIGHT RATIO	2.100			
LIQUID LIMIT (ASTM D4318)				
PLASTIC LIMIT (ASTM D4318)				
SPECIFIC GRAVITY (ASTM D854)	2.720			
FINAL PARAMETERS	1OS-B5@108-108.5'			
MOISTURE (%)	20.48			
SATURATION (%)	99.42			
STRAIN RATE (%/MIN.)	0.051			
PEAK DEVIATOR STRESS (PSF)	5297.9			
AXIAL STRAIN AT FAILURE (%)	12.658			
CELL PRESSURE				
CELL PRESSURE (PSF)	4003.2			
BACK PRESSURE (PSF)	n/a			
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	9301.1			
σ3 (PSF)	4003.2			
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	2648.9			
REMARKS				

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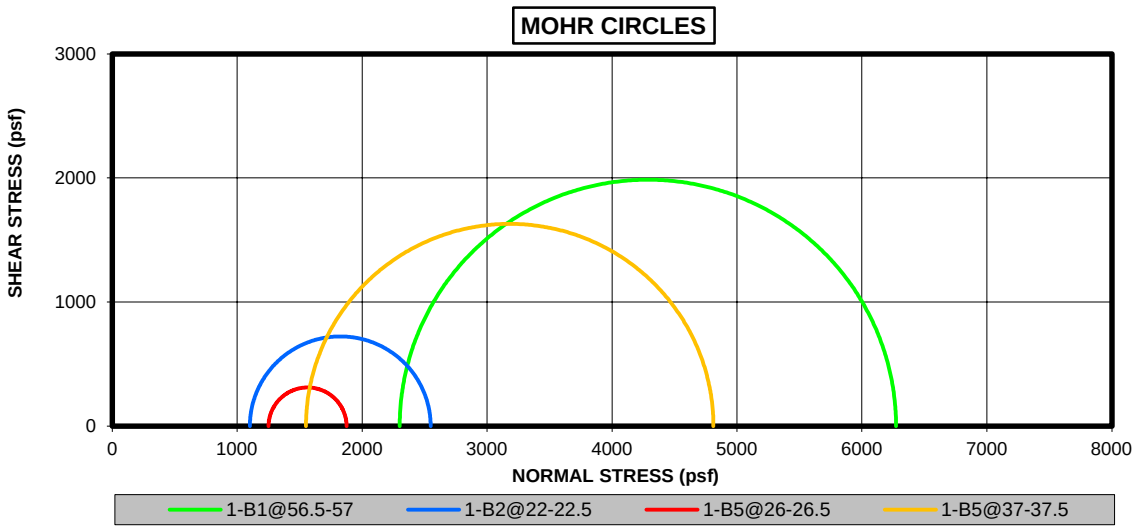
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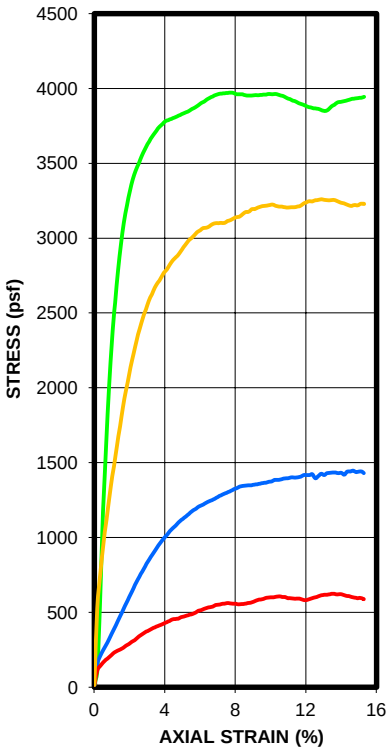


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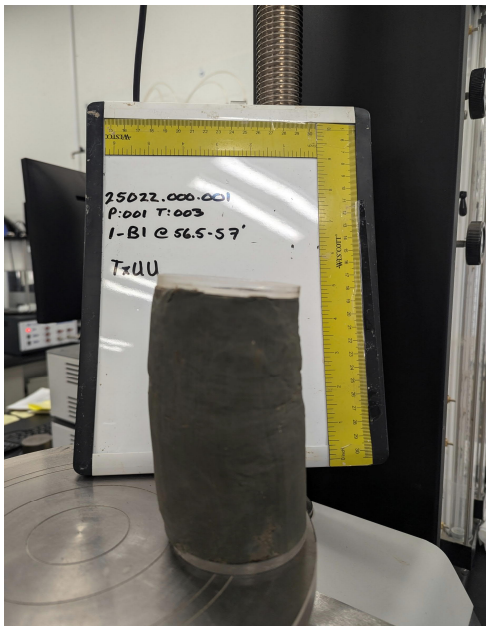


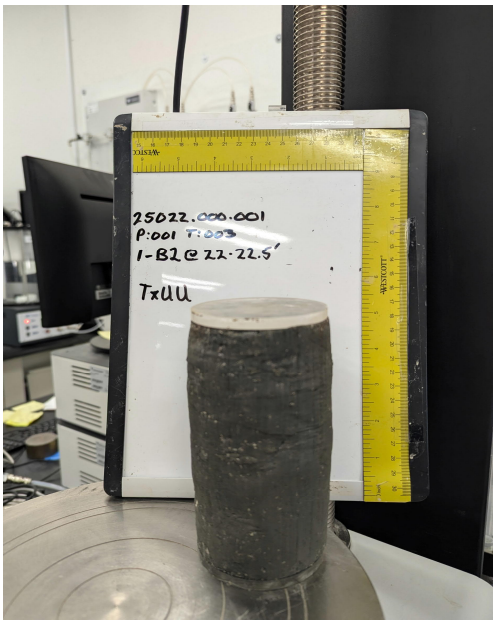
	SPECIMEN			
INITIAL PARAMETERS	**1-B1@56.5-57**	**1-B2@22-22.5**	**1-B5@26-26.5**	**1-B5@37-37.5**
MOISTURE (%)	22.73	27.57	19.40	31.13
DRY DENSITY (PCF)	102.00	96.80	112.20	89.10
SATURATION (%)	92.96	99.49	102.73	93.58
VOID RATIO	0.665	0.754	0.514	0.905
DIAMETER (IN.)	2.869	2.880	2.853	2.855
HEIGHT (IN.)	5.950	6.065	6.101	5.825
DIAMETER-TO-HEIGHT RATIO	2.074	2.106	2.138	2.040
LIQUID LIMIT (ASTM D4318)	n/a	n/a	n/a	n/a
PLASTIC LIMIT (ASTM D4318)	n/a	n/a	n/a	n/a
SPECIFIC GRAVITY (ASTM D854)	2.720	2.720	2.720	2.720
FINAL PARAMETERS	**1-B1@56.5-57**	**1-B2@22-22.5**	**1-B5@26-26.5**	**1-B5@37-37.5**
MOISTURE (%)	22.73	27.57	19.40	31.13
SATURATION (%)	92.96	99.49	100.00	93.58
STRAIN RATE (%/MIN.)	0.840	0.820	0.820	0.840
PEAK DEVIATOR STRESS (PSF)	3972.6	1447.2	624.3	3260.9
AXIAL STRAIN AT FAILURE (%)	7.731	14.674	13.441	12.876
CELL PRESSURE				
CELL PRESSURE (PSF)	2299.7	1100.2	1249.9	1549.4
BACK PRESSURE (PSF)	n/a	n/a	n/a	n/a
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	6272.2	2547.4	1874.2	4810.3
σ3 (PSF)	2299.7	1100.2	1249.9	1549.4
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	1986.3	723.6	312.1	1630.4
REMARKS				

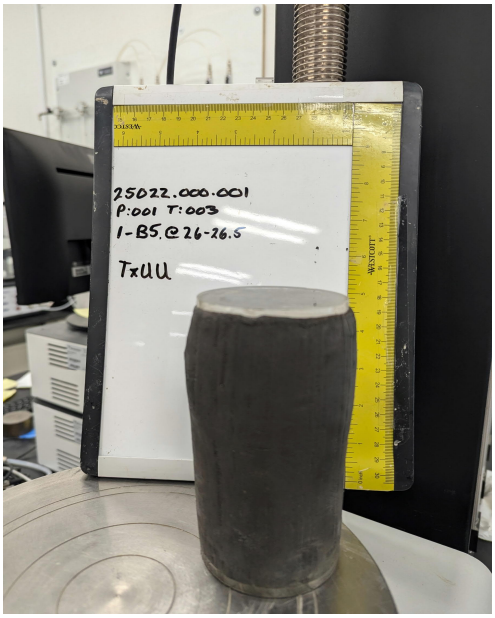
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PROJECT NAME: Berkeley Water Transportation Pier Ferry
PROJECT NO: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/22/2024
TESTED BY: K. Lecce
REVIEWED BY: D. Bryant

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ASTM D2850





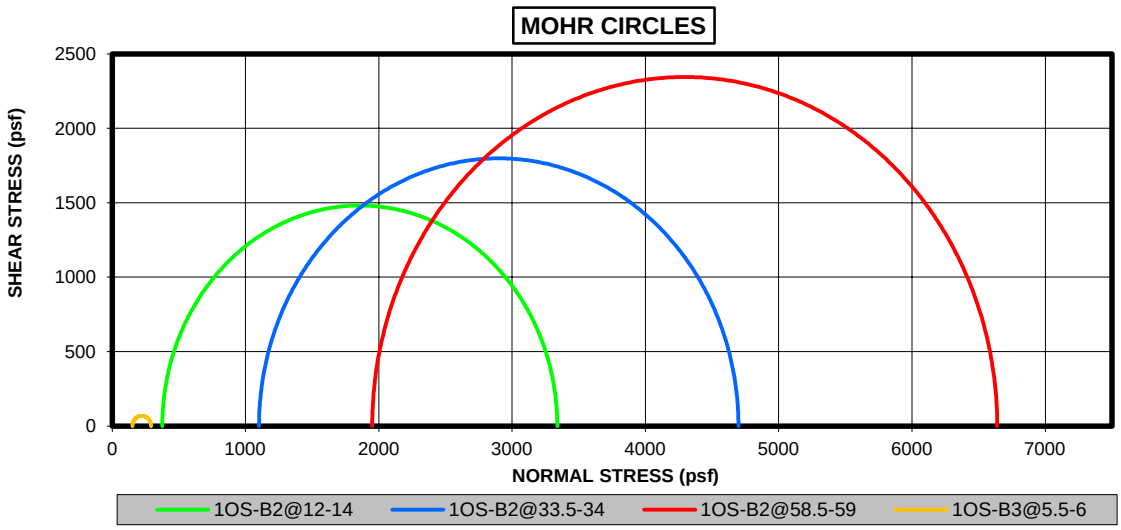


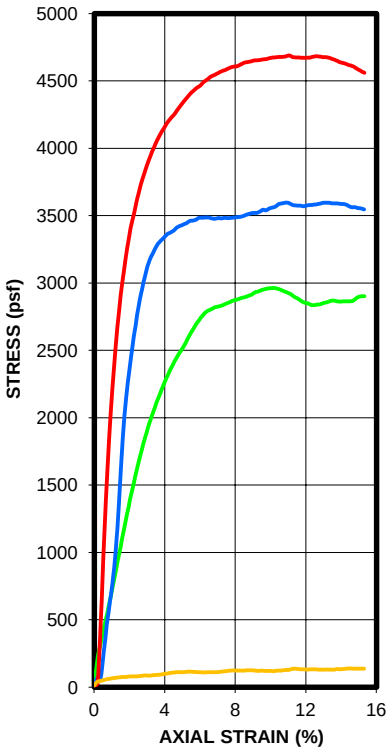


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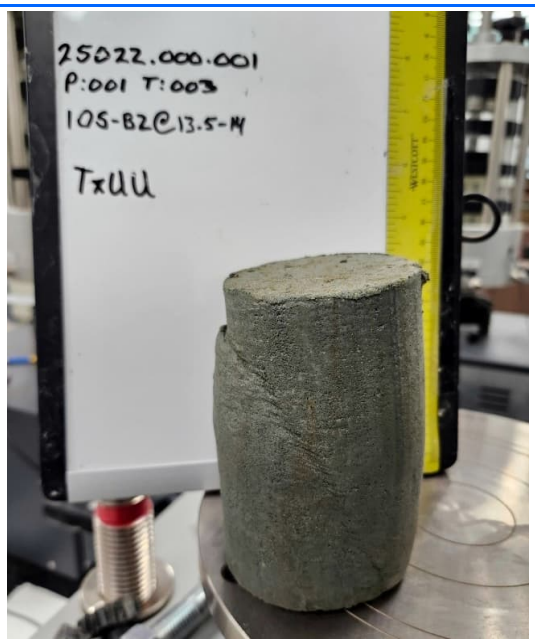


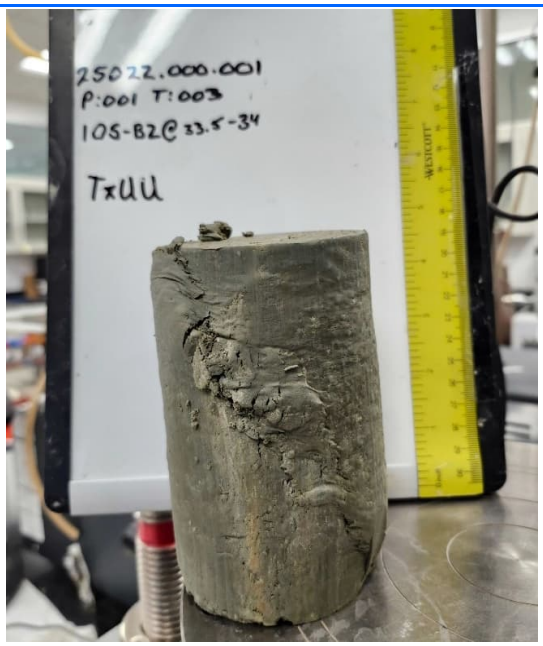
	SPECIMEN			
INITIAL PARAMETERS	**1OS-B2@12-14**	**1OS-B2@33.5-34**	**1OS-B2@58.5-59**	**1OS-B3@5.5-6**
MOISTURE (%)	18.72	23.04	29.04	34.93
DRY DENSITY (PCF)	111.50	102.50	94.40	84.70
SATURATION (%)	97.38	95.39	98.86	94.60
VOID RATIO	0.523	0.657	0.799	1.004
DIAMETER (IN.)	2.850	2.850	2.860	2.850
HEIGHT (IN.)	5.380	5.550	5.330	5.800
DIAMETER-TO-HEIGHT RATIO	1.888	1.947	1.864	2.035
LIQUID LIMIT (ASTM D4318)	n/a	n/a	n/a	n/a
PLASTIC LIMIT (ASTM D4318)	n/a	n/a	n/a	n/a
SPECIFIC GRAVITY (ASTM D854)	2.720	2.720	2.720	2.720
FINAL PARAMETERS	**1OS-B2@12-14**	**1OS-B2@33.5-34**	**1OS-B2@58.5-59**	**1OS-B3@5.5-6**
MOISTURE (%)	18.72	23.04	29.04	34.93
SATURATION (%)	97.37	95.39	98.86	94.60
STRAIN RATE (%/MIN.)	0.840	0.820	0.820	0.840
PEAK DEVIATOR STRESS (PSF)	2964.0	3597.4	4689.6	140.5
AXIAL STRAIN AT FAILURE (%)	10.223	13.153	11.070	14.483
CELL PRESSURE				
CELL PRESSURE (PSF)	374.4	1100.2	1949.8	149.8
BACK PRESSURE (PSF)	n/a	n/a	n/a	n/a
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	3338.4	4697.6	6639.4	290.3
σ3 (PSF)	374.4	1100.2	1949.8	149.8
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	1482.0	1798.7	2344.8	70.2
REMARKS				
Due to its saturation/moisture content, sample 1OS-B3@5.5-6 was collapsing under its own weight when extruded from the liner and prepared for the test. It should be considered disturbed.				

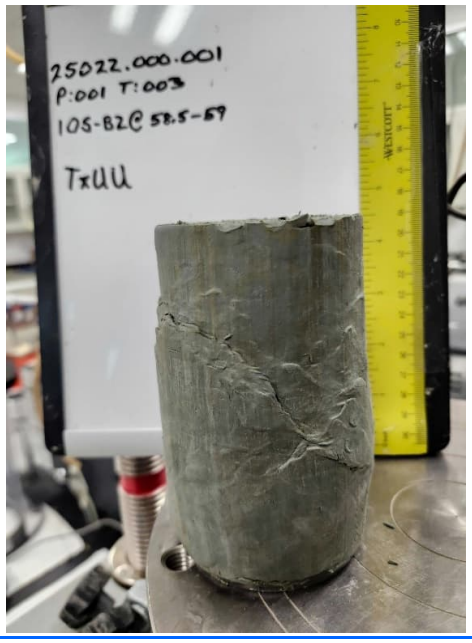
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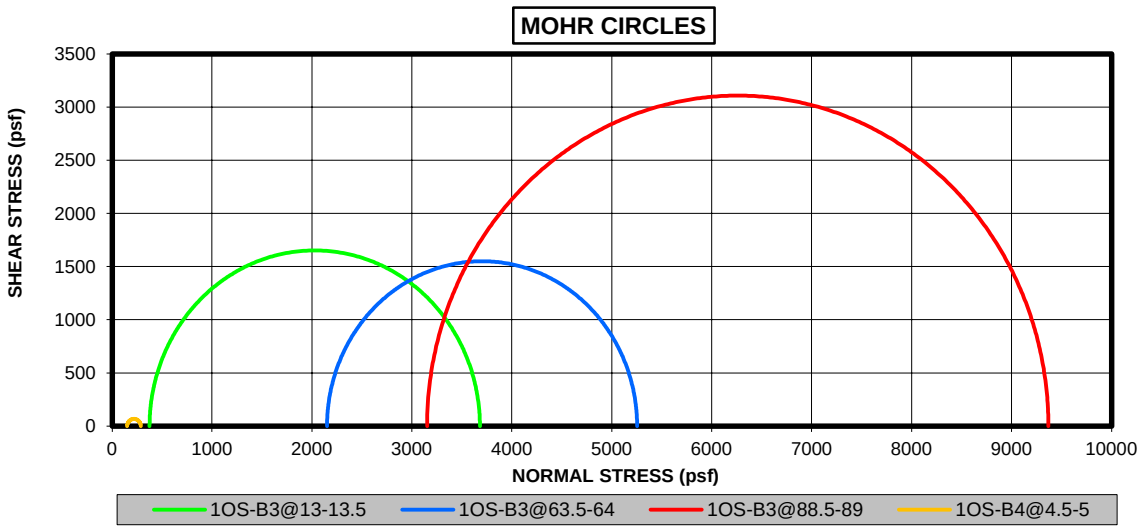


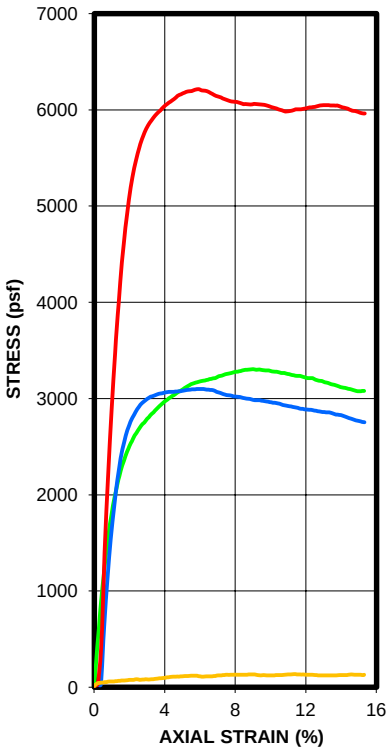
1OS-B3@5.5-6

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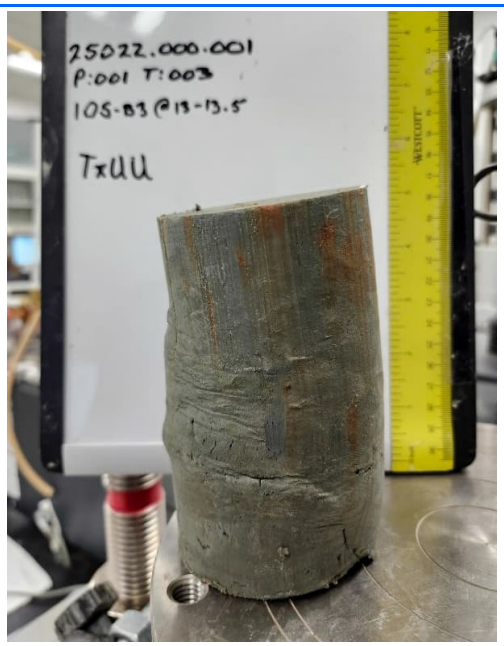


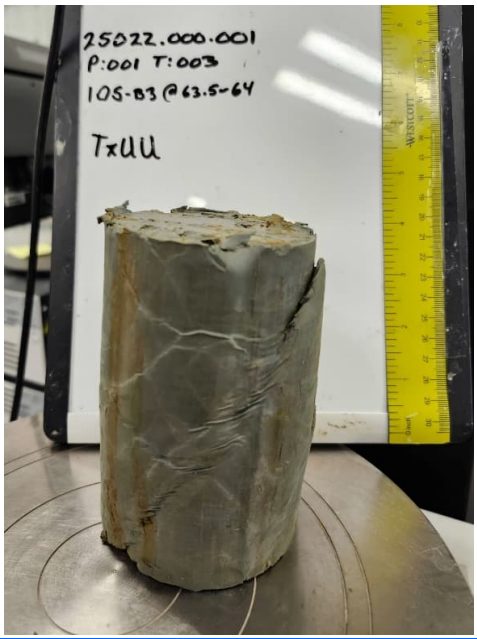
	SPECIMEN			
INITIAL PARAMETERS	**1OS-B3@13-13.5**	**1OS-B3@63.5-64**	**1OS-B3@88.5-89**	**1OS-B4@4.5-5**
MOISTURE (%)	20.43	44.62	25.33	38.18
DRY DENSITY (PCF)	109.70	76.50	99.00	83.30
SATURATION (%)	96.36	99.50	96.38	98.99
VOID RATIO	0.594	1.220	0.715	1.061
DIAMETER (IN.)	2.860	2.860	2.860	2.860
HEIGHT (IN.)	5.660	5.330	5.080	5.830
DIAMETER-TO-HEIGHT RATIO	1.979	1.864	1.776	2.038
LIQUID LIMIT (ASTM D4318)	n/a	n/a	n/a	n/a
PLASTIC LIMIT (ASTM D4318)	n/a	n/a	n/a	n/a
SPECIFIC GRAVITY (ASTM D854)	2.800	2.720	2.720	2.750
FINAL PARAMETERS	**1OS-B3@13-13.5**	**1OS-B3@63.5-64**	**1OS-B3@88.5-89**	**1OS-B4@4.5-5**
MOISTURE (%)	20.43	44.62	25.33	38.18
SATURATION (%)	96.36	99.50	96.38	98.99
STRAIN RATE (%/MIN.)	0.840	0.820	0.820	0.840
PEAK DEVIATOR STRESS (PSF)	3305.0	3102.1	6216.2	137.1
AXIAL STRAIN AT FAILURE (%)	9.011	6.004	5.906	11.321
CELL PRESSURE				
CELL PRESSURE (PSF)	374.4	2149.9	3150.7	149.8
BACK PRESSURE (PSF)	n/a	n/a	n/a	n/a
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	3679.4	5252.0	9367.0	286.8
σ3 (PSF)	374.4	2149.9	3150.7	149.8
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	1652.5	1551.1	3108.1	68.5
REMARKS				
Due to its saturation/moisture content, sample 1OS-B4@4.5-5 was collapsing under its own weight when extruded from the liner and prepared for the test. It should be considered disturbed.				

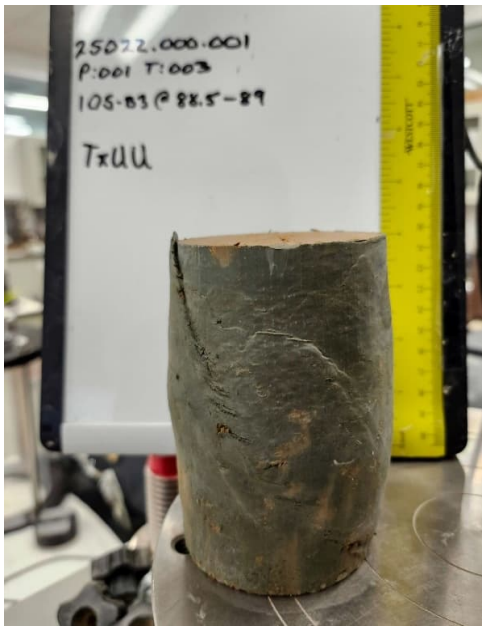
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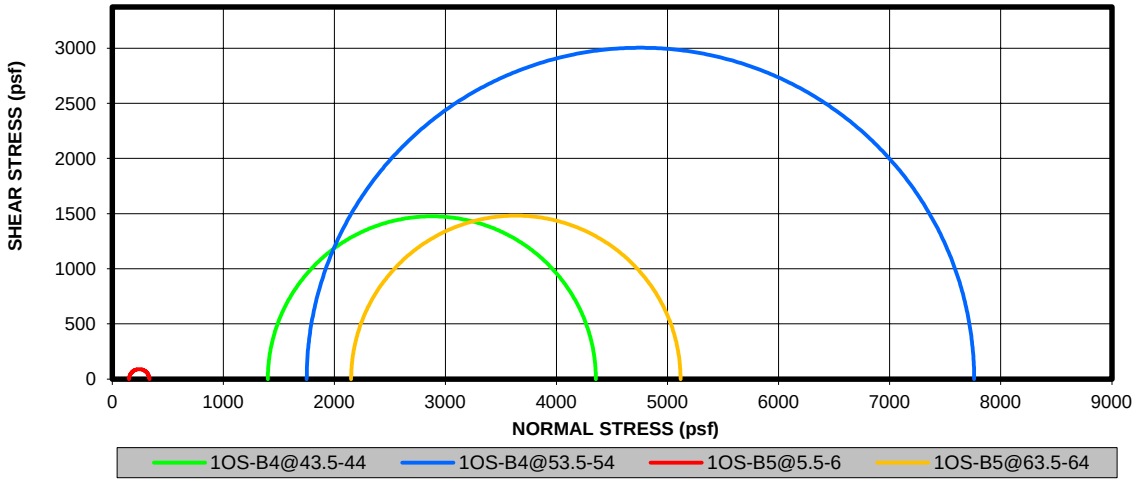
1OS-B4@4.5-5

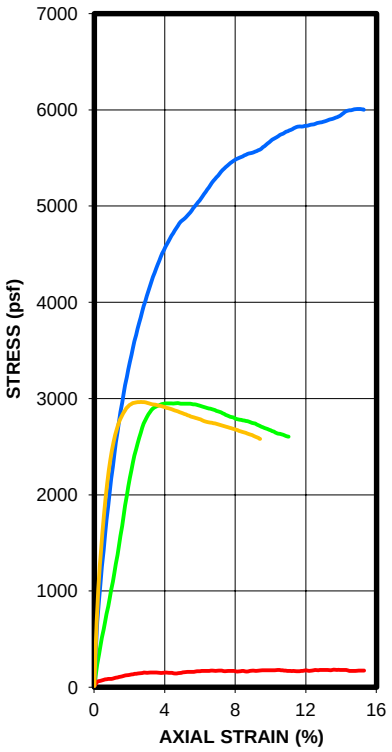
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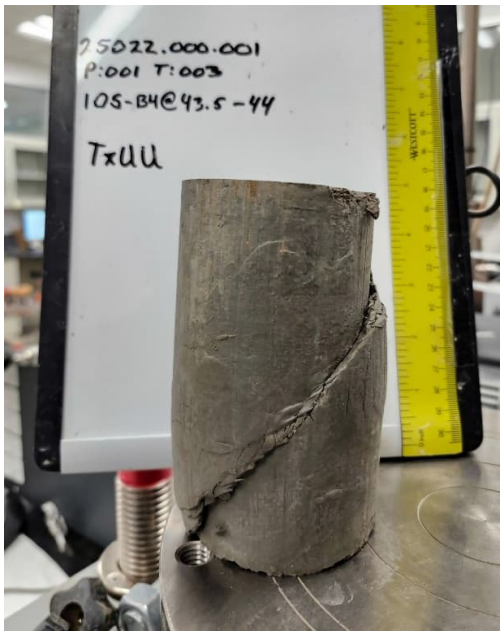


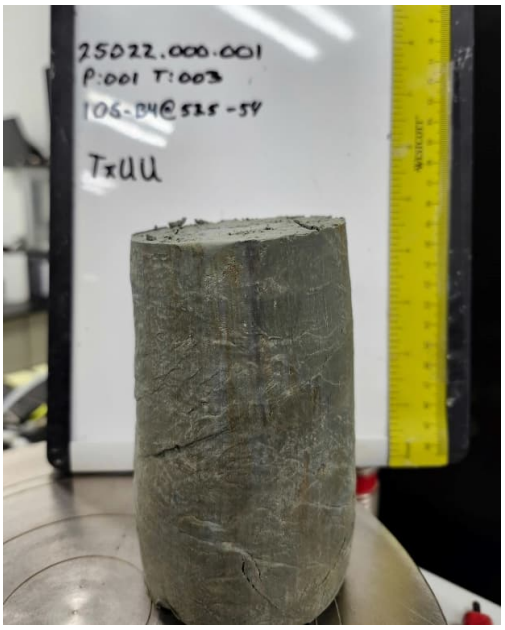
	SPECIMEN			
INITIAL PARAMETERS	**1OS-B4@43.5-44**	**1OS-B4@53.5-54**	**1OS-B5@5.5-6**	**1OS-B5@63.5-64**
MOISTURE (%)	29.55	28.58	35.14	47.12
DRY DENSITY (PCF)	92.00	94.70	78.30	74.30
SATURATION (%)	94.92	98.12	81.85	99.16
VOID RATIO	0.847	0.792	1.168	1.302
DIAMETER (IN.)	2.860	2.860	2.870	2.860
HEIGHT (IN.)	5.700	5.700	5.670	5.480
DIAMETER-TO-HEIGHT RATIO	1.993	1.993	1.976	1.916
LIQUID LIMIT (ASTM D4318)	n/a	n/a	n/a	n/a
PLASTIC LIMIT (ASTM D4318)	n/a	n/a	n/a	n/a
SPECIFIC GRAVITY (ASTM D854)	2.720	2.720	2.720	2.740
FINAL PARAMETERS	**1OS-B4@43.5-44**	**1OS-B4@53.5-54**	**1OS-B5@5.5-6**	**1OS-B5@63.5-64**
MOISTURE (%)	29.55	28.58	35.14	47.12
SATURATION (%)	94.92	98.12	81.85	99.16
STRAIN RATE (%/MIN.)	0.840	0.820	0.820	0.840
PEAK DEVIATOR STRESS (PSF)	2953.6	6009.7	183.1	2965.8
AXIAL STRAIN AT FAILURE (%)	4.737	15.088	13.580	2.555
CELL PRESSURE				
CELL PRESSURE (PSF)	1399.7	1749.6	149.8	2149.9
BACK PRESSURE (PSF)	n/a	n/a	n/a	n/a
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	4353.2	7759.3	332.8	5115.7
σ3 (PSF)	1399.7	1749.6	149.8	2149.9
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	1476.8	3004.9	91.5	1482.9
REMARKS				
Due to its saturation/moisture content, sample 1OS-B5@5.5-6 was collapsing under its own weight when extruded from the liner and prepared for the test. It should be considered disturbed.				

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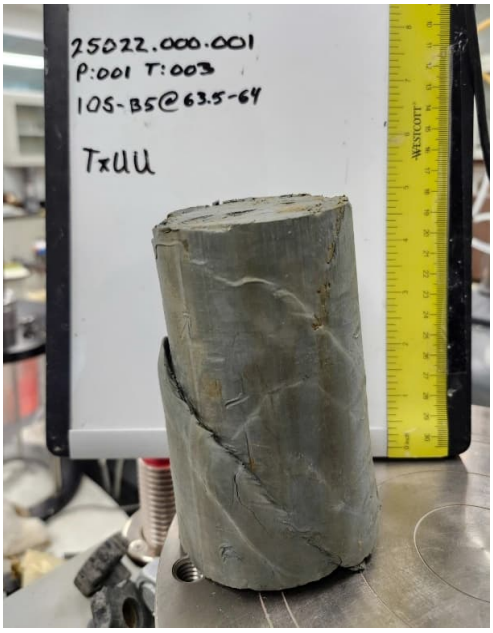
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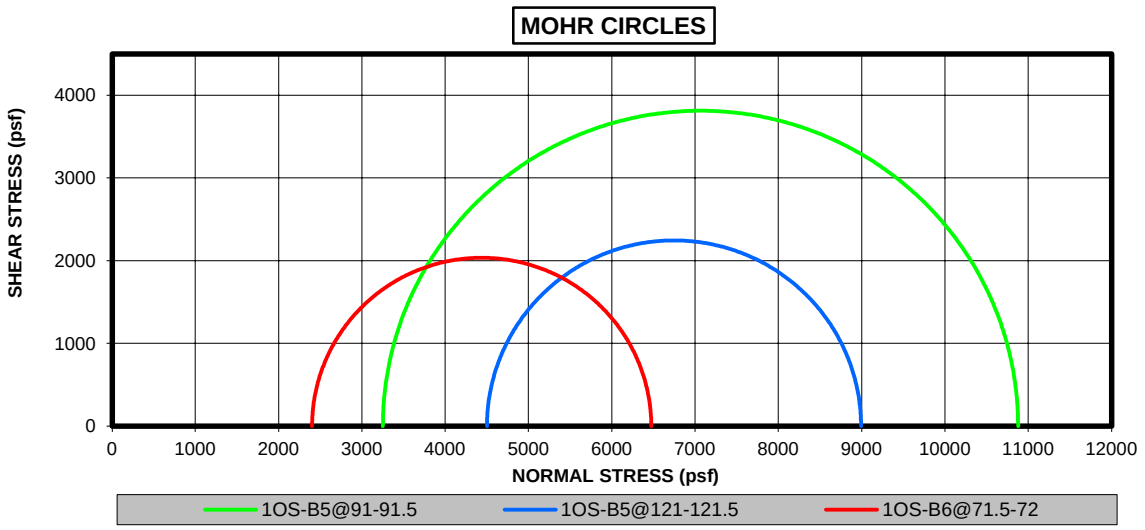


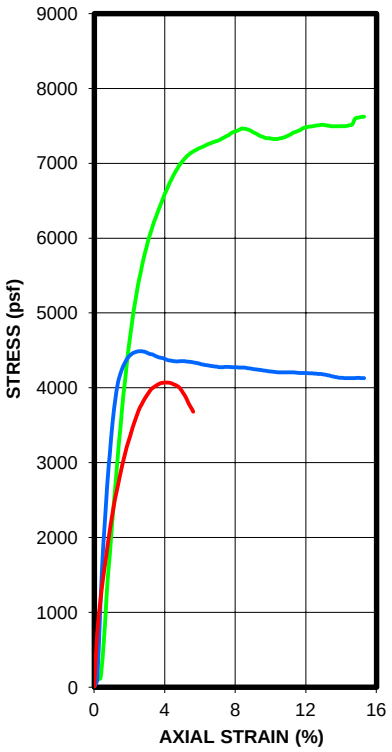


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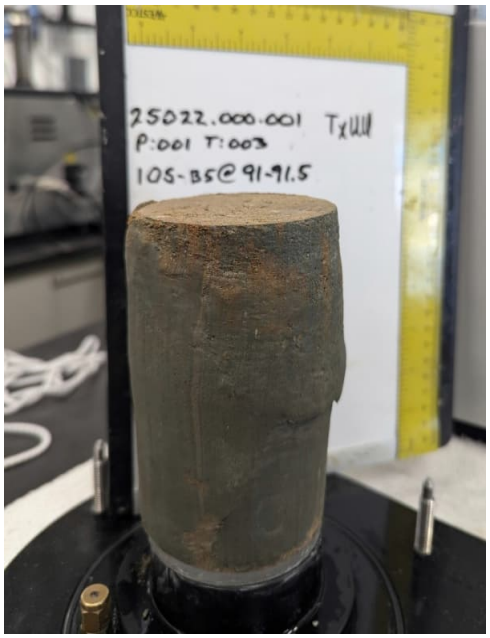


	SPECIMEN			
INITIAL PARAMETERS	1OS-B5@91-91.5	1OS-B5@121-121.5	1OS-B6@71.5-72	
MOISTURE (%)	19.78	46.03	44.46	
DRY DENSITY (PCF)	105.90	74.90	77.20	
SATURATION (%)	89.23	98.84	99.72	
VOID RATIO	0.603	1.267	1.231	
DIAMETER (IN.)	2.870	2.880	2.875	
HEIGHT (IN.)	5.745	5.752	5.965	
DIAMETER-TO-HEIGHT RATIO	2.002	1.997	2.075	
LIQUID LIMIT (ASTM D4318)	n/a	n/a	n/a	
PLASTIC LIMIT (ASTM D4318)	n/a	n/a	n/a	
SPECIFIC GRAVITY (ASTM D854)	2.720	2.720	2.720	
FINAL PARAMETERS	1OS-B5@91-91.5	1OS-B5@121-121.	1OS-B6@71.5-72	
MOISTURE (%)	19.78	46.03	44.46	
SATURATION (%)	89.23	98.84	99.72	
STRAIN RATE (%/MIN.)	0.840	0.820	0.820	
PEAK DEVIATOR STRESS (PSF)	7626.6	4491.2	4072.0	
AXIAL STRAIN AT FAILURE (%)	15.293	2.608	4.024	
CELL PRESSURE				
CELL PRESSURE (PSF)	3250.1	4500.0	2400.5	
BACK PRESSURE (PSF)	n/a	n/a	n/a	
PRINCIPLE STRESSES AT FAILURE				
σ1 (PSF)	10876.7	8991.2	6472.5	
σ3 (PSF)	3250.1	4500.0	2400.5	
COHESION AT FAILURE WITH A ZERO FRICTION ANGLE (Ø=0)				
COHESION, C (PSF)	3813.3	2245.6	2036.0	
REMARKS				

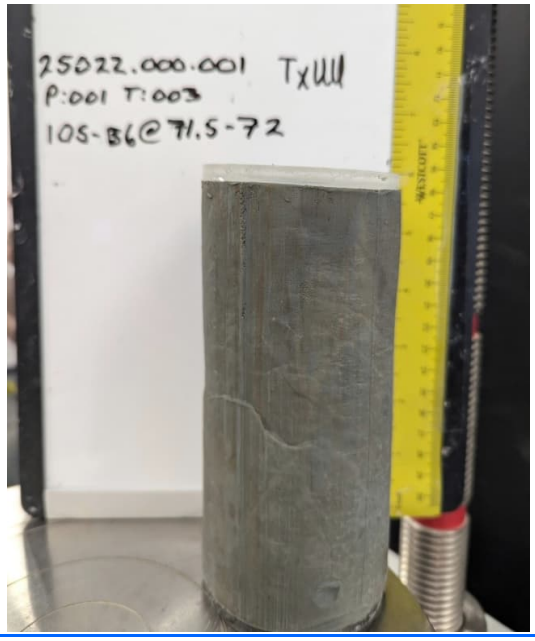
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LABORATORY MINIATURE VANE SHEAR

ASTM D4648

APPARATUS USED: Wykeham Farrance, Model 27-WF1730/4

SAMPLE NUMBER	SAMPLE ID	Remold? (Y/N)	TEST DEPTH, ft	SPRING NUMBER	Shear strength, psf
1	1OS-B1@12-14	N	13.5-13.75	4	2245
2	1-B3@26-26.5	N	26.25	3	498

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PROJECT NUMBER: 25022.000.001 PH001 T003
PROJECT LOCATION: Berkeley, CA
REPORT DATE: 10/09/24
TESTED BY: G. Criste
REVIEWED BY: D. Seibold



APPENDIX E

CORROSION LABORATORY TESTING (CERCO Analytical)

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

5 November, 2024

Job No. 2410076
Cust. No. 10169

Mr. Vlad Zasmolin
ENGEO Inc.
1630 San Pablo Ave Suite 200
Oakland, CA 94612

Subject: Project No.: 25022.000.001
Project Name: Berkeley Marina
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Zasmolin:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on October 31, 2024. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Sample No. 001 is classified as "severely corrosive". Sample No. 002 is classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations are none detected and 570 mg/kg. Chloride ion concentrations greater than 300 mg/kg are considered corrosive to embedded reinforcing steel; and, as such, the concrete mix design shall be adjusted accordingly by a qualified corrosion engineer.

The sulfate ion concentrations are 17 mg/kg and 97 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils are 6.86 and 8.00, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are 200-mV and 270-mV. Both samples are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Julie A for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: ENGEO, Incorporated
Client's Project No.: 25022.000.001
Client's Project Name: Berkeley Marina
Date Sampled: 30-Oct-24
Date Received: 31-Oct-24
Matrix: Soil
Authorization: Chain of Custody

Date of Report: 5-Nov-2024

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2410076-001	1-B2 (4.5'-5')	270	6.86	-	360	-	570	97
2410076-002	1-B3 (5.5-6')	200	8.00	-	760	-	N.D.	17

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	-	15	15
Date Analyzed:	31-Oct-2024	1-Nov-2024	-	31-Oct-2024	-	1-Nov-2024	1-Nov-2024

Julia Clauson
Chemist

* Results Reported on "As Received" Basis
N.D. - None Detected

Quality Control Summary - **All laboratory quality control parameters were found to be within established limits**

Chain of Custody

Page of

1100 Willow Pass Court
Concord, CA 94520-1005
925 462-2771
Fax: 925 462-2775
www.cercoanalytical.com

CERCO analytical

Job No.	CU#		Client Project I.D.	Schedule	Date Sampled	Date Due
2410076	13067		25022.000.001	Analyte	9/31/24	

Full Name Vlad Zasmolin

Phone 510 9772385

Fax

Company and/or Mailing Address ENGEO Incorporated, 1630 San Pablo Ave, VZasmolin@engeo.com

Cell

Sample Source Berkeley Marina

ANALYSIS — **ASTM w/Brief Evaluation**

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qtv.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated		Brief Evaluation
1	1-B2 (4.5-5')	10/30/24		S	Baggie			1	x	x	x	x	x		x
2	1-B3 (5.5-6')	10/30/24		S	Liner			1	"	"	"	"	"		"

MATRIX	ABBREVIATIONS	SAMPLE RECEIPT
DW - Drinking Water	HB - Hosebib	Total No. of Containers
GW - Ground Water	PV - Petcock Valve	Rec'd Good Cond/Cold
SW - Surface Water	PT - Pressure Tank	Conforms to Record
WW - Waste Water	PH - Pump House	Temp. a t Lab - °C
Water	RR - Restroom	Sampler
SL - Sludge	GL - Glass	
S - Soil	PL - Plastic	
Product	ST - Sterile	

Comments:
THERE IS AN ADDITIONAL CHARGE FOR EXTRUDING SOIL FROM TUBES

Relinquished By:	GIS	Date	Time
Received By:	Julia	Date 10/31/24	Time 1000
Relinquished By:		Date	Time
Received By:		Date	Time
Relinquished By:		Date	Time
Received By:		Date	Time

Email Address: [illegible]23



APPENDIX F

DISCUSSION OF SITE-SPECIFIC SEISMIC-HAZARD ANALYSIS

Site-Specific Seismic-Hazard Analysis

We performed a site-specific seismic-hazard analysis in accordance with ASCE 61-14 and ASCE 7-05 to develop response spectra for the seismic hazard and performance levels discussed in Section 3.4.2.

We performed the following tasks to develop the Maximum Considered Earthquake (MCE) and Design Earthquake (DE) response spectra for this site.

- Performed probabilistic seismic-hazard analysis (PSHA) to develop a median-component (RotD50) uniform hazard response spectrum (UHS) corresponding to a 2-percent probability of exceedance in 50 years (2,475 year return period)
- Performed deterministic seismic-hazard analysis (DSHA) to develop a response spectrum equal to 150 percent of the largest 50th-percentile RotD50 response spectrum
- Compared the DSHA response spectrum with the deterministic lower limit in accordance with Section 21.2.2 of ASCE 7-05.
- Compared the probabilistic and the deterministic response spectra to obtain the site-specific MCE response spectrum for the site
- Compared the MCE response spectrum developed in the previous step with 80 percent of the general MCE response spectrum to develop the recommended site-specific MCE response spectrum
- Multiplied the site-specific MCE response spectrum by two-thirds to obtain the site-specific Design Earthquake (DE) response spectrum for the site

As required by ASCE 61-14, we developed additional RotD50 response spectra based on the Operating level earthquake (OLE) and Contingency Level Earthquake (CLE) hazard levels corresponding to a 50-percent probability of exceedance in 50 years (72-year return period) and a 20-percent probability of exceedance in 50 years (225-year return period) or a 10-percent probability of exceedance in 50 years (475-year return period), respectively.

Seismic Source Model

We utilized the 2018 National Seismic Hazard Model (NSHM), as implemented in the United States Geological Survey *nshmp-haz* software. The 2018 NSHM is the most current and publicly available rupture forecast model for the conterminous United States and, as such, is required by ASCE 61-14 and ASCE 7-05. The 2018 NSHM incorporates the Third California Earthquake Rupture Forecast model or UCERF3 (Field et al. 2014 and 2015) for California sources and seismicity rates. Background seismicity, modeled as gridded point sources, is also included in the model. The implementation of the 2018 NSHM in seismic-hazard codes considers many sources of epistemic uncertainty regarding alternate rupture scenarios, maximum magnitudes for individual faults, and alternate magnitude-recurrence relations. This uncertainty affects the mean hazard that is provided by hazard codes implementing the 2018 NSHM and is used in typical applications, including this analysis.

Ground-Motion Models and Site Parameters

We used four semi-empirical ground-motion models (GMMs) from the Next Generation Attenuation West 2 (NGA West 2) project (Ancheta et al., 2014) to estimate ground motion for shallow crustal sources included within the 2018 NSHM. These models include Abrahamson et

al. (2014) [ASK], Boore et al. (2014) [BSSA], Campbell and Bozorgnia (2014) [CB], and Chiou and Youngs (2014) [CY]. We performed our analysis using all four GMMs for a spectral damping of 5 percent of critical damping. We used a logic-tree approach and assigned equal weight (0.25) to each of the four GMMs in our analysis. Other ground-motion models are considered for different source types within the seismic source model, such as subduction sources; however, the contributions from these sources are negligible at the project site for the return periods considered due their large source-to-site distances.

The ground-motion models incorporate "site parameters" to model how subsurface soil will amplify or attenuate ground motions as they propagate from the bedrock below. For the GMMs we implemented, these site parameters include the following.

- Time-averaged shear-wave velocity (V_S) over the top 100 feet or 30 meters (V_{S30})
- Depth at which V_S reaches 3,280 feet/sec or 1.0 kilometer/sec ($z_{1.0}$)
- Depth at which V_S reaches 8,200 feet/sec or 2.5 kilometers/sec ($z_{2.5}$)

We estimated a V_{S30} value of 690feet/sec (210 meters/sec) based on the measured and correlated V_S profiles from the CPTs. To estimate $z_{1.0}$ and $z_{2.5}$, we used the USGS Bay Area Velocity Model version 8.3.0 Basin Depth models, as implemented in the USGS Site Data Application available through OpenSHA. We applied $z_{1.0}$ and $z_{2.5}$ values of 311 and 2,789 feet (95 and 860 meters) in our analysis, respectively.

Directivity Effects

Directivity effects can increase long-period ground motions at near-fault sites (Somerville et al., 1997; Abrahamson 2000). We used the period-dependent models by Chiu and Spudich (2013), Bayless and Somerville (2013), and Bayless et al. (2020) to consider directivity effects, as implemented in the Natural Hazard and Resiliency Research Center (NHR3) directivity based PSHA interactive tool (Mazzoni et al., 2023), to estimate the period dependent directivity factors for return periods of 2,475, 475, 225, and 72 years. We calculated a weighted mean of the directivity factors obtained from these models with weights of 0.5, 0.25, and 0.25, respectively, as recommended by NHR3.

Probabilistic Seismic-Hazard Analysis

We performed a PSHA for the project site to develop a set of hazard curves and resulting uniform hazard response spectra (UHS) for return periods of 72, 225, 475, and 2,475 years. We calculated the seismic hazard using the standard methodology for hazard analysis (McGuire, 2004). The seismic-hazard calculations can be represented by the following equation, which is an application of the total-probability theorem.

$$H(a) = \sum_{i}^{n} v_i \iint P[A > a|m,r]\, f_{Mi}(\mathrm{m})\, f_{Ri|Mi}(\mathrm{r},\mathrm{m}) dr dm$$

In this equation, the hazard **H*(a)*** is the annual frequency of earthquakes that produce a ground motion amplitude ***A*** higher than ***a***. Amplitude ***A*** may represent peak ground acceleration, velocity, or pseudo-spectral acceleration (PSa) at a given frequency. The summation in the equation shown extends over all sources (i.e., over all faults and areas). In the above equation, $\boldsymbol{v_i}$ is the annual rate of earthquakes (with magnitude higher than some threshold $\boldsymbol{M_i}$) in source ***i***, and $\boldsymbol{f_{Mi}(m)}$

and $f_{Ri|Mi}(r,m)$ are the probability density functions for magnitude and distance, respectively. $P[A > a|m, r]$ is the probability that an earthquake of magnitude m at distance r produces a ground-motion amplitude A at the site that is greater than a. Seismic sources may be either faults or background sources; the specification of source geometries and the calculation of $f_{Ri|Mi}$, are performed differently for these two types of sources.

We present the median component (RotD50) hazard curves for the equally weighted mean of all GMMs and the combined branches of UCERF3 (FM3.1 and FM3.2) in Figure 1. We used the hazard curves to calculate the mean RotD50 uniform hazard response spectra for return periods of 72, 225, 475, and 2,475 years. We additionally applied the directivity adjustment factors from the NHR3 tool. We present the uniform hazard response spectra in Figure 2.

Disaggregation of the Seismic Hazard

We performed a disaggregation of the seismic hazard associated with the 72-year, 225-year, 475-year, and 2,475-year return periods at the peak ground acceleration (PGA), and for spectral periods of 0.5, 1.0 and 2.0 seconds. We present the resulting faults and their respective contributions to the seismic hazard at the site for each return period considered in Tables F-1 through F-4.

TABLE F-1: Summary of Disaggregation Results for a 72-Year Return Period*

SOURCE	R_{RUP}		M_W	CONTRIBUTION (%)			
	Km	miles		PGA	0.5 sec	1.0 sec	2.0 sec
Hayward (North) [2]	6.3	3.9	7.1	10.0	9.9	10.5	11.0
San Andreas (Peninsula) [13]	25.0	15.5	7.9	5.3	5.6	6.4	7.5
Calaveras (North) [0]	24.0	14.9	7.0	1.8	1.9	2.0	2.1
San Gregorio (North) [3]	28.4	17.6	7.7	1.5	1.6	1.8	2.1
Concord [2]	28.1	17.5	6.5	1.3	1.5	1.4	1.1
Hayward (South) [7]	15.6	9.7	6.8	1.3	1.3	1.4	1.4
Hayward (South) [6]	21.2	13.2	6.7	1.1	1.2	1.2	1.2
Franklin [2]	18.4	11.4	7.1	1.0	1.1	1.2	1.2
Mount Diablo [2] (3)	26.7	16.6	7.0	<1.0	1.1	1.1	1.1
Hayward (South) [5]	28.7	17.8	6.8	<1.0	<1.0	1.0	<1.0

*Based on the USGS Earthquake Hazard Toolbox: NSHM Conterminous U.S. 2018

TABLE F-2: Summary of Disaggregation Results for a 225-Year Return Period*

SOURCE	R_{RUP}		M_W	CONTRIBUTION (%)			
	km	miles		PGA	0.5 sec	1.0 sec	2.0 sec
Hayward (North) [2]	6.3	3.9	7.2	17.1	17.1	19.1	20.4
San Andreas (Peninsula) [13]	25.0	15.5	7.9	5.8	1.7	7.7	9.4
Calaveras (North) [0]	24.0	14.9	7.1	1.5	1.7	1.7	1.6
San Gregorio (North) [3]	28.4	17.6	7.7	1.4	1.4	1.9	2.2
Hayward (South) [7]	15.6	9.7	6.8	1.3	1.2	1.4	1.3
Franklin [2]	18.4	11.4	7.2	1.1	1.2	1.2	1.2
Hayward (north) [3]	6.5	4.0	6.9	1.1	1.0	1.1	1.1

*Based on the USGS Earthquake Hazard Toolbox: NSHM Conterminous U.S. 2018

TABLE F-3: Summary of Disaggregation Results for a 475-Year Return Period*

SOURCE	R_{RUP}		M_W	CONTRIBUTION (%)			
	km	miles		PGA	0.5 sec	1.0 sec	2.0 sec
Hayward (North) [2]	6.3	3.9	7.2	21.1	21.1	24.2	25.8
San Andreas (Peninsula) [13]	25.0	15.5	7.9	5.6	6.7	7.6	9.2
Calaveras (North) [0]	24.0	14.9	7.1	1.3	1.3	1.4	1.1
San Gregorio (North) [3]	28.4	17.6	7.7	1.2	1.5	1.7	1.9
Hayward (North) [3]	6.5	4.0	6.8	1.2	1.2	1.3	1.2
Hayward (South) [7]	15.6	9.8	6.8	1.2	1.3	1.3	<1.0
Franklin [2]	18.4	11.4	7.2	<1.0	1.1	1.1	1.1

*Based on the USGS Earthquake Hazard Toolbox: NSHM Conterminous U.S. 2018

TABLE F-4: Summary of Disaggregation Results for a 2,475-Year Return Period*

SOURCE	R_{RUP}		M_W	CONTRIBUTION (%)			
	km	miles		PGA	0.5 sec	1.0 sec	2.0 sec
Hayward (North) [2]	6.3	3.9	7.3	56.7	57.7	66.2	69.4
San Andreas (Peninsula) [13]	25.0	15.5	8.0	9.3	12.1	12.2	14.2
Hayward (North) [3]	6.5	4.1	7.0	3.0	2.9	3.1	2.8
Hayward (North) [1]	7.2	4.5	6.9	2.1	2.1	2.1	< 1.0
San Gregorio (North) [3]	28.4	17.7	7.8	<1.0	2.4	2.2	1.0

*Based on the USGS Earthquake Hazard Toolbox: NSHM Conterminous U.S. 2018

These results represent known fault sources contributing at least 1 percent to the seismic hazard at the site for at least one of the spectral periods considered and for the given return period. Background seismicity zones, such as gridded or areal sources, are not presented. The rupture distances (R_{RUP}) and mean moment magnitude (M_W) values listed are based on values assigned according to the 2018 NSHM and UCERF3, and the bracketed numbers after each fault name correspond to fault subsections assigned by the 2018 NSHM and UCERF3. Magnitudes vary slightly between the two fault models (FM 3.1 and 3.2) utilized by the 2018 NSHM and UCERF3 and from one spectral period to another. Therefore, for each source we present the maximum mean magnitude of the spectral periods considered where that source contributes significantly to the hazard. Note that the above fault tables are not exhaustive lists and other faults in the region may generate seismic shaking at the project site.

Deterministic Seismic-Hazard Analysis

The DSHA involves developing the 50th-percentile median component (RotD50) response spectrum for a spectral damping of 5 percent of critical damping considering the characteristic magnitudes of significant faults, without background seismicity, and utilizing the ground-motion models previously discussed. However, the definition of the characteristic magnitude is ambiguous when using the UCERF3 model due to its complexity. Based on the 2020 NEHRP Provisions and ASCE/SEI 7-22, in deterministic analyses, “scenario” earthquakes with significant contribution to hazard should be used in lieu of “characteristic” earthquakes when using the 2018 NSHM and UCERF3. We identified the scenario earthquakes by considering the disaggregation of the PSHA results in Table F-4.

We considered the magnitudes in Table F-4 and associated distances (R_{RUP}, R_{JB}, R_X) to calculate the median deterministic response spectra. We estimated additional ground-motion model parameters (e.g., rupture width, depth to top of rupture, etc.) for each fault/scenario based on the UCERF3 model and fault-specific information published by the United States Geologic Survey (USGS). The DSHA is predominantly controlled by the Hayward (North) [2] fault with a M_W of 7.3 within 3.9 miles (6.3 kilometers) of the site. Similar to the probabilistic response spectrum, we applied the directivity adjustment factors from the NHR3 tool.

We scaled the spectral ordinates of the deterministic response spectra by 150 percent and compared the maximum of the median deterministic response spectra with the deterministic lower limit in accordance with Section 21.2.2 of ASCE 7-05. Per Section 21.2.2, the maximum PSa of the deterministic response spectrum shall not be less than the lower limit determined in accordance with Figure 21.2-1 of ASCE 7-05. In computing the lower-limit, the short-period site coefficient (F_a) is determined using Table 11.4-1 of ASCE 7-05 for an S_S value of 1.5 and the long-period site coefficient (F_v) is determined using Table 11.4-2 of ASCE 7-05 for an S_1 value of 0.6. For this site, the maximum of the deterministic response spectra is less than the deterministic lower limit across most spectral periods. We present the deterministic response spectra in Figure 3.

Resulting Ground Surface Response Spectra

According to Section 21.2.3 of ASCE 7-05, the MCE is controlled by the lesser of the probabilistic and the deterministic response spectra. At this site, the spectral accelerations associated with the deterministic response spectrum are generally lower than the probabilistic response spectrum, except for the period range of 0.01 to 0.12 second where the spectral accelerations associated with the probabilistic response spectrum are lower. Additionally, the MCE is not permitted to be lower than 80 percent of the associated general response spectrum for Site Class D (i.e., the code minimum). Finally, we calculated the DE response spectrum by taking two-thirds of the MCE response spectrum.

We present the site-specific OLE, CLE, DE, and MCE response spectra in Table F-5 and Figure 4.

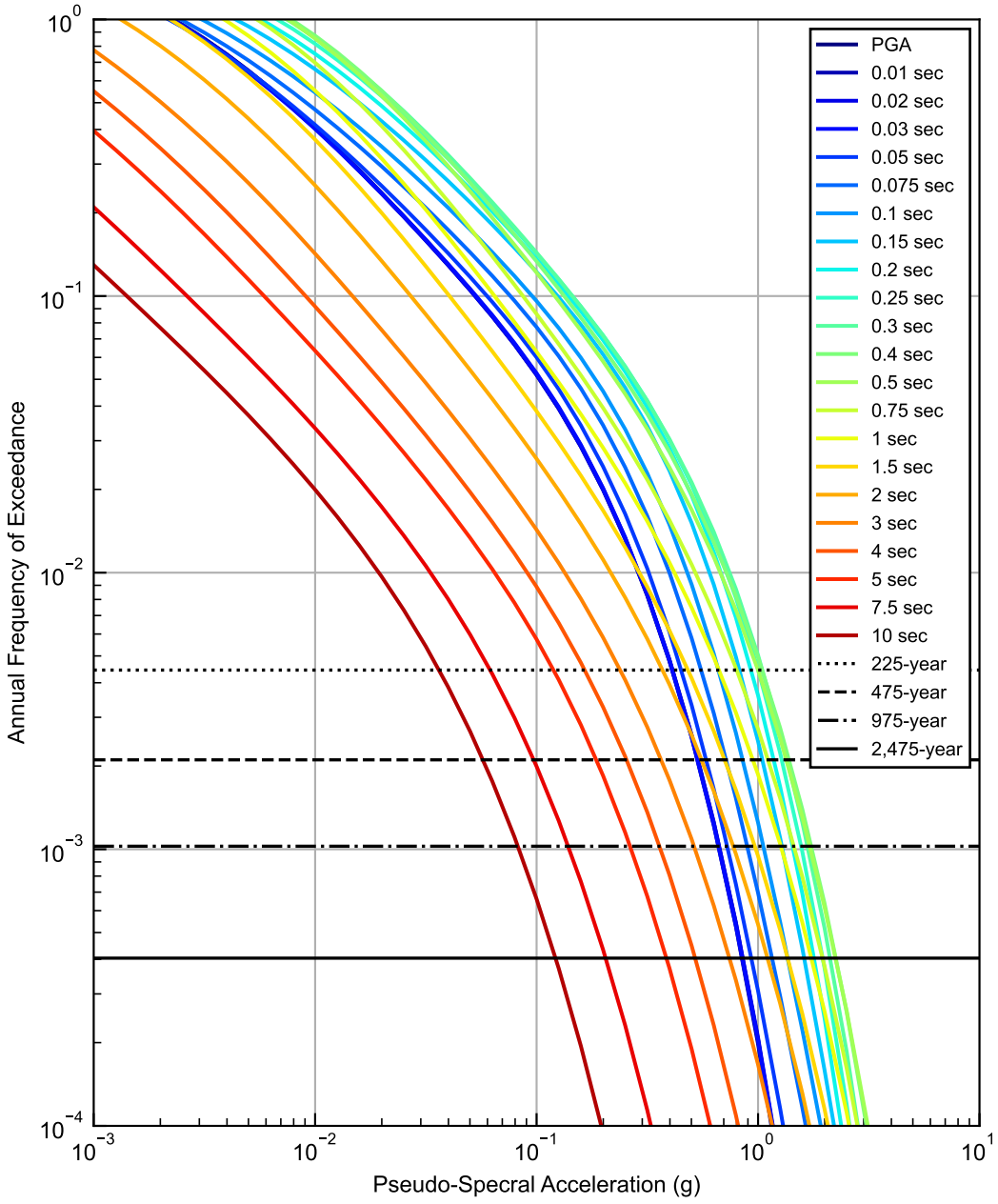
TABLE F-5: Site-Specific OLE, CLE, DE, and MCE Response Spectra

PERIOD (seconds)	PSEUDO-SPECTRAL ACCELERATION (g)				
	OLE (72-year)	CLE (225-year)	CLE (475-year)	DE (ASCE7-05)	MCE (ASCE 7-05)
0.01	0.25	0.41	0.54	0.57	0.85
0.02	0.24	0.41	0.53	0.57	0.85
0.03	0.25	0.41	0.53	0.56	0.85
0.05	0.27	0.45	0.58	0.62	0.93
0.08	0.34	0.55	0.72	0.77	1.16
0.10	0.41	0.66	0.85	0.91	1.36
0.12	0.46	0.73	0.93	0.98	1.47
0.15	0.52	0.82	1.05	1.00	1.50
0.20	0.58	0.93	1.18	1.00	1.50
0.25	0.62	1.00	1.28	1.00	1.50
0.30	0.63	1.05	1.35	1.00	1.50
0.40	0.60	1.04	1.38	1.00	1.50

PERIOD (seconds)	PSEUDO-SPECTRAL ACCELERATION (g)				
	OLE (72-year)	CLE (225-year)	CLE (475-year)	DE (ASCE7-05)	MCE (ASCE 7-05)
0.50	0.57	1.01	1.35	1.00	1.50
0.60	0.51	0.93	1.25	1.00	1.50
0.75	0.43	0.81	1.13	0.80	1.20
1.00	0.35	0.70	1.00	0.60	0.90
1.50	0.24	0.51	0.75	0.42	0.63
2.00	0.18	0.40	0.60	0.32	0.47
3.00	0.11	0.26	0.40	0.20	0.30
4.00	0.07	0.18	0.29	0.15	0.23
5.00	0.05	0.13	0.21	0.12	0.18
7.50	0.03	0.07	0.11	0.08	0.12
8.00	0.02	0.06	0.10	0.08	0.11
10.00	0.02	0.04	0.06	0.06	0.09

We estimated the site-specific MCE peak ground acceleration (PGA) to be 0.85 g. The PGA is a median component (RotD50) value and probabilistically controlled.

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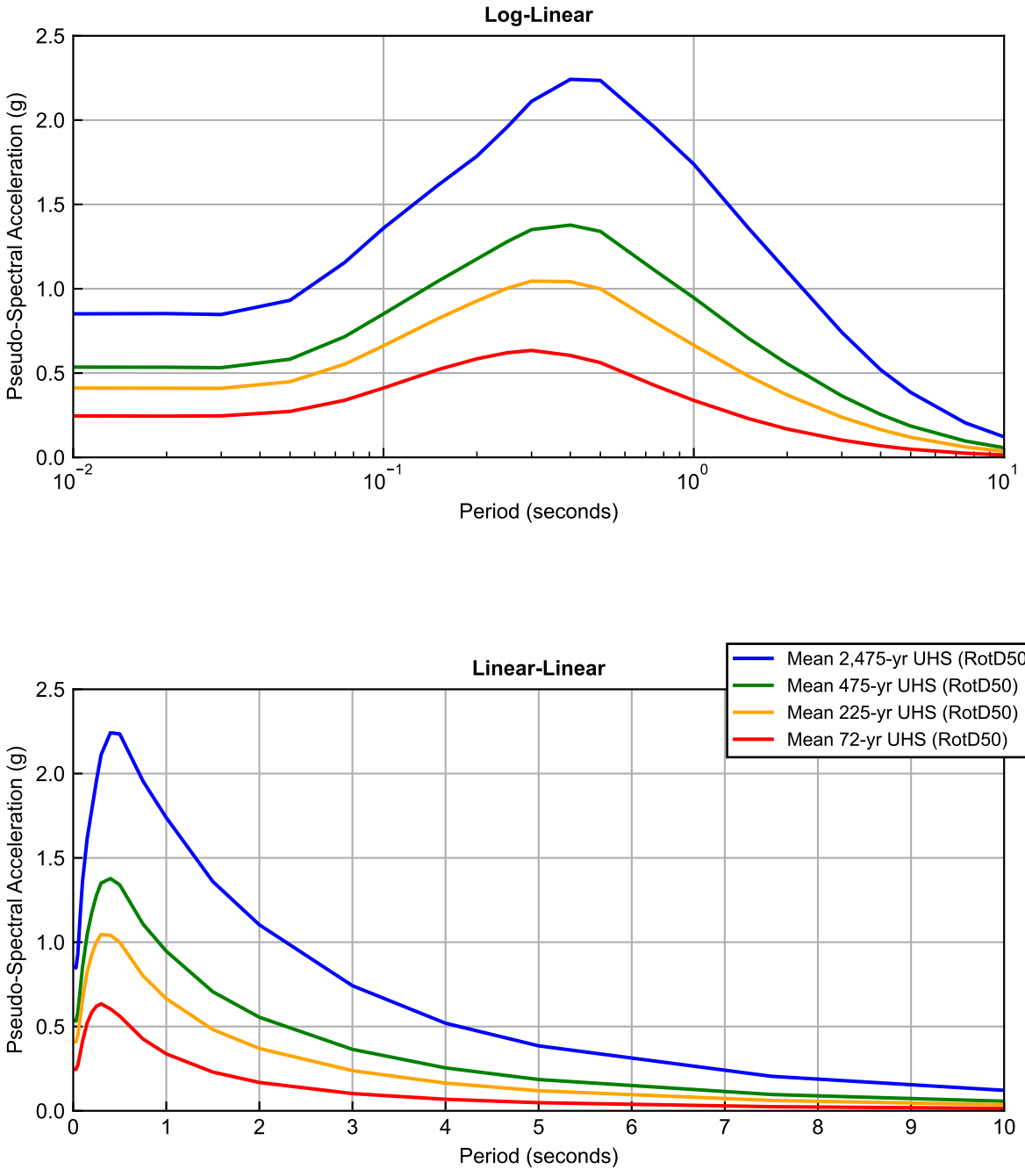


ENGEO Expect Excellence

MEAN ROTD50 HAZARD CURVES
BERKELEY WATER TRANSPORTATION PIER FERRY
BERKELEY, CALIFORNIA

PROJECT NO: 25022.000.001		FIGURE NO. 1
SCALE: AS SHOWN		
DRAWN BY: VZ	CHECKED BY: JF	

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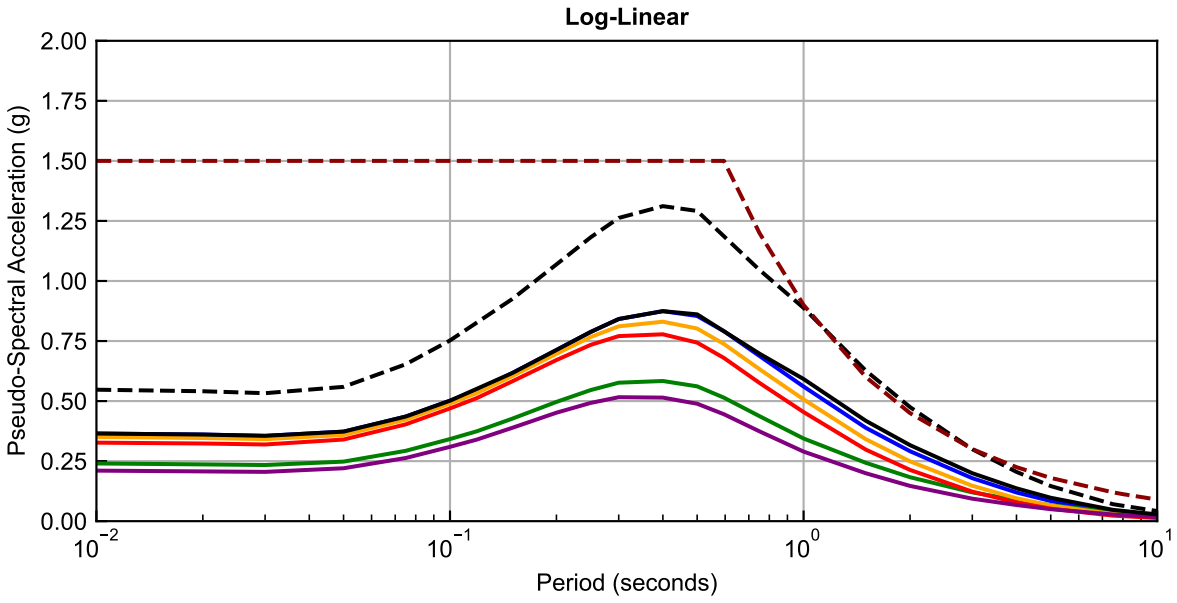


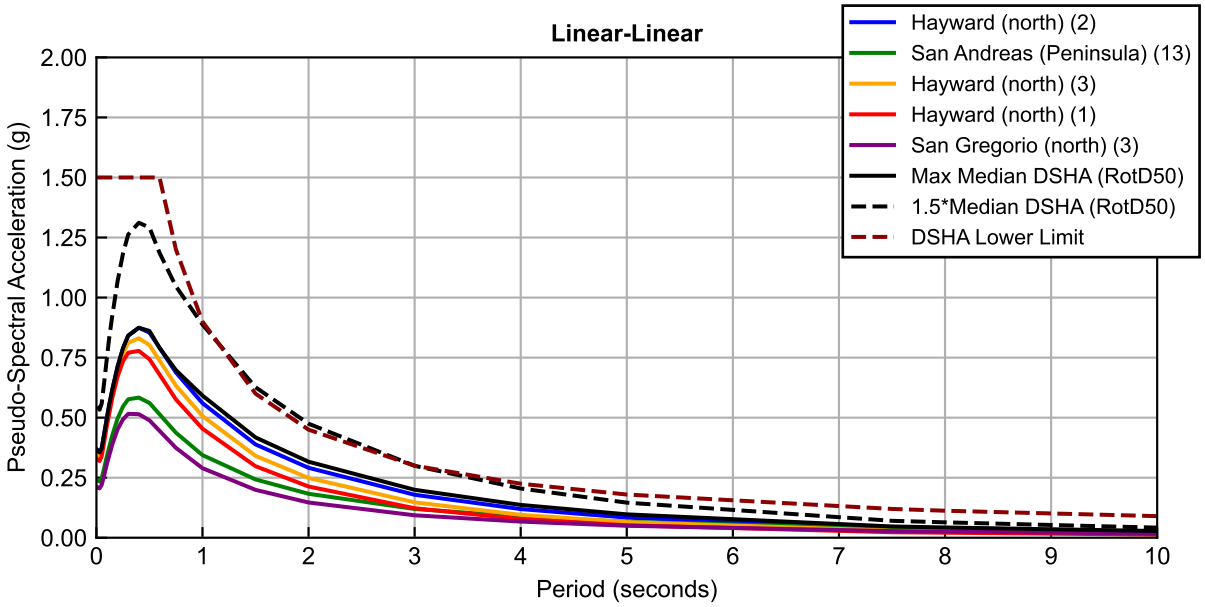
ENGEO
Expect Excellence

MEAN UNIFORM HAZARD RESPONSE SPECTRA
BERKELEY WATER TRANSPORTATION PIER FERRY
BERKELEY, CALIFORNIA

PROJECT NO: 25022.000.001		FIGURE NO. 2
SCALE: AS SHOWN		
DRAWN BY: VZ	CHECKED BY: JF	

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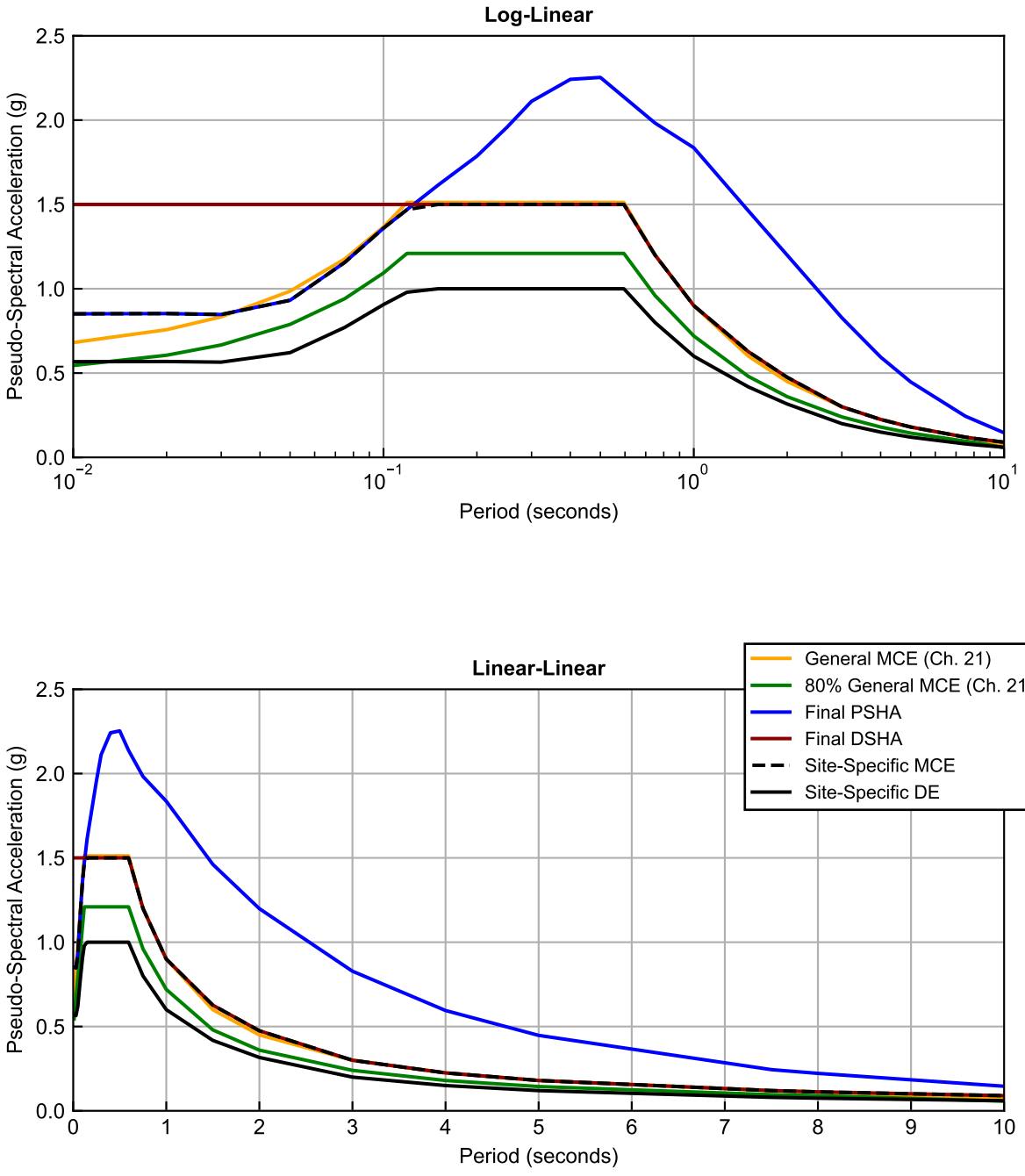


ENGEO Expect Excellence

DETERMINISTIC RESPONSE SPECTRA
BERKELEY WATER TRANSPORTATION PIER FERRY
BERKELEY, CALIFORNIA

PROJECT NO: 25022.000.001		FIGURE NO. 3
SCALE: AS SHOWN		
DRAWN BY: VZ	CHECKED BY: JF	

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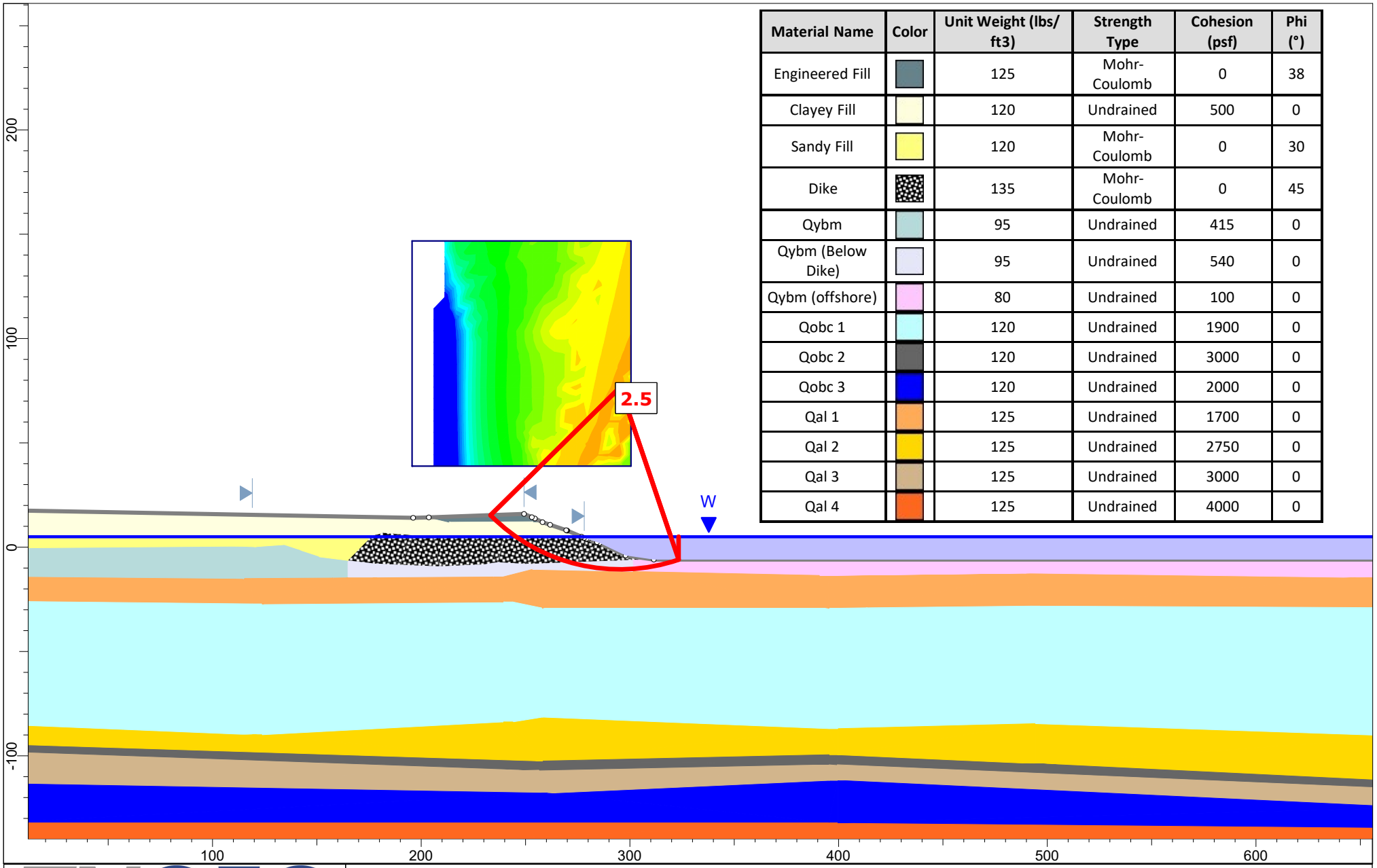
SITE-SPECIFIC RESPONSE SPECTRA
BERKELEY WATER TRANSPORTATION PIER FERRY
BERKELEY, CALIFORNIA

PROJECT NO: 25022.000.001		FIGURE NO. 4
SCALE: AS SHOWN		
DRAWN BY: VZ	CHECKED BY: JF	



APPENDIX G.1

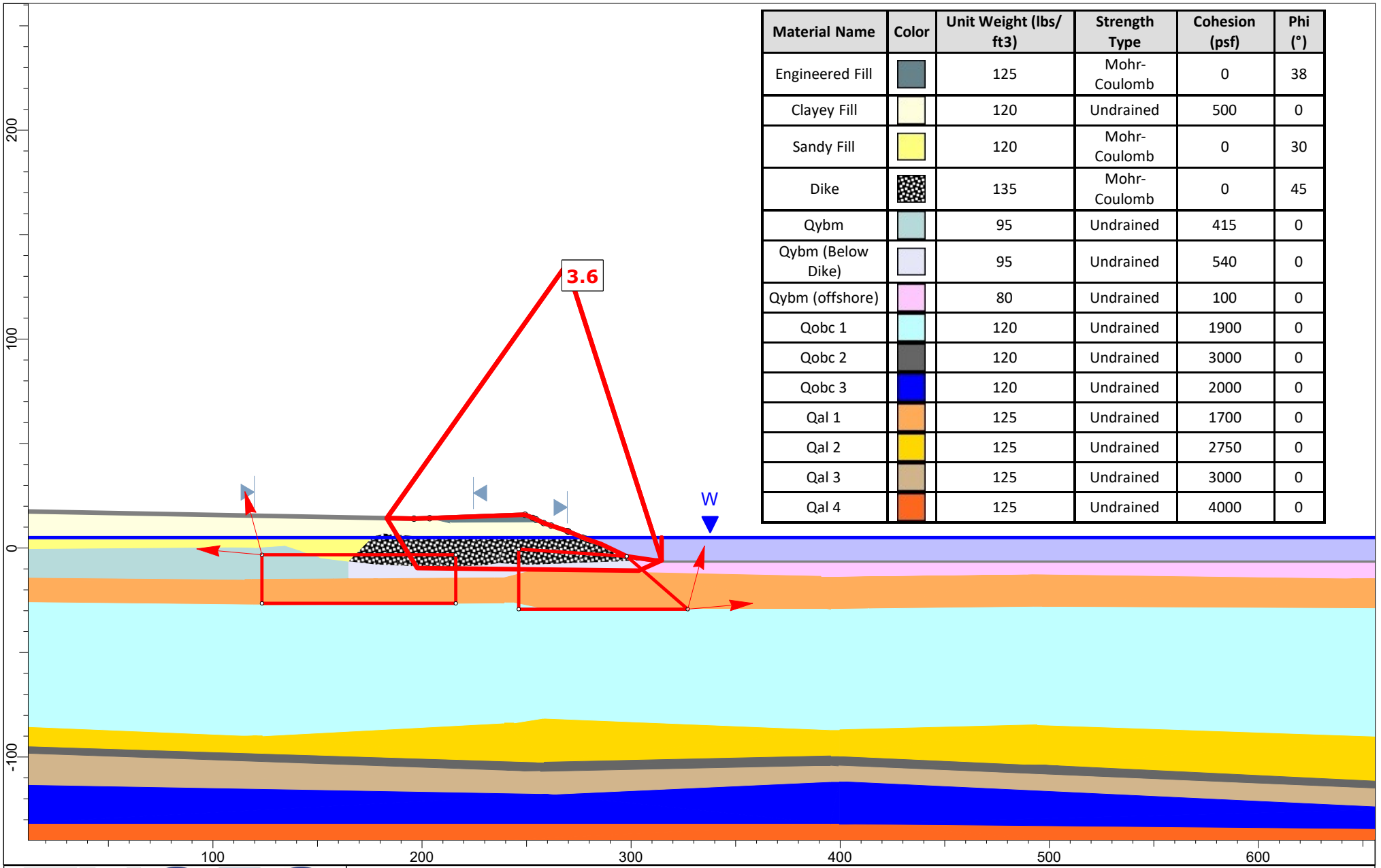
SLOPE STABILITY ANALYSIS – EXISTING SLOPE



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
Engineered Fill		125	Mohr-Coulomb	0	38
Clayey Fill		120	Undrained	500	0
Sandy Fill		120	Mohr-Coulomb	0	30
Dike		135	Mohr-Coulomb	0	45
Qybm		95	Undrained	415	0
Qybm (Below Dike)		95	Undrained	540	0
Qybm (offshore)		80	Undrained	100	0
Qobc 1		120	Undrained	1900	0
Qobc 2		120	Undrained	3000	0
Qobc 3		120	Undrained	2000	0
Qal 1		125	Undrained	1700	0
Qal 2		125	Undrained	2750	0
Qal 3		125	Undrained	3000	0
Qal 4		125	Undrained	4000	0

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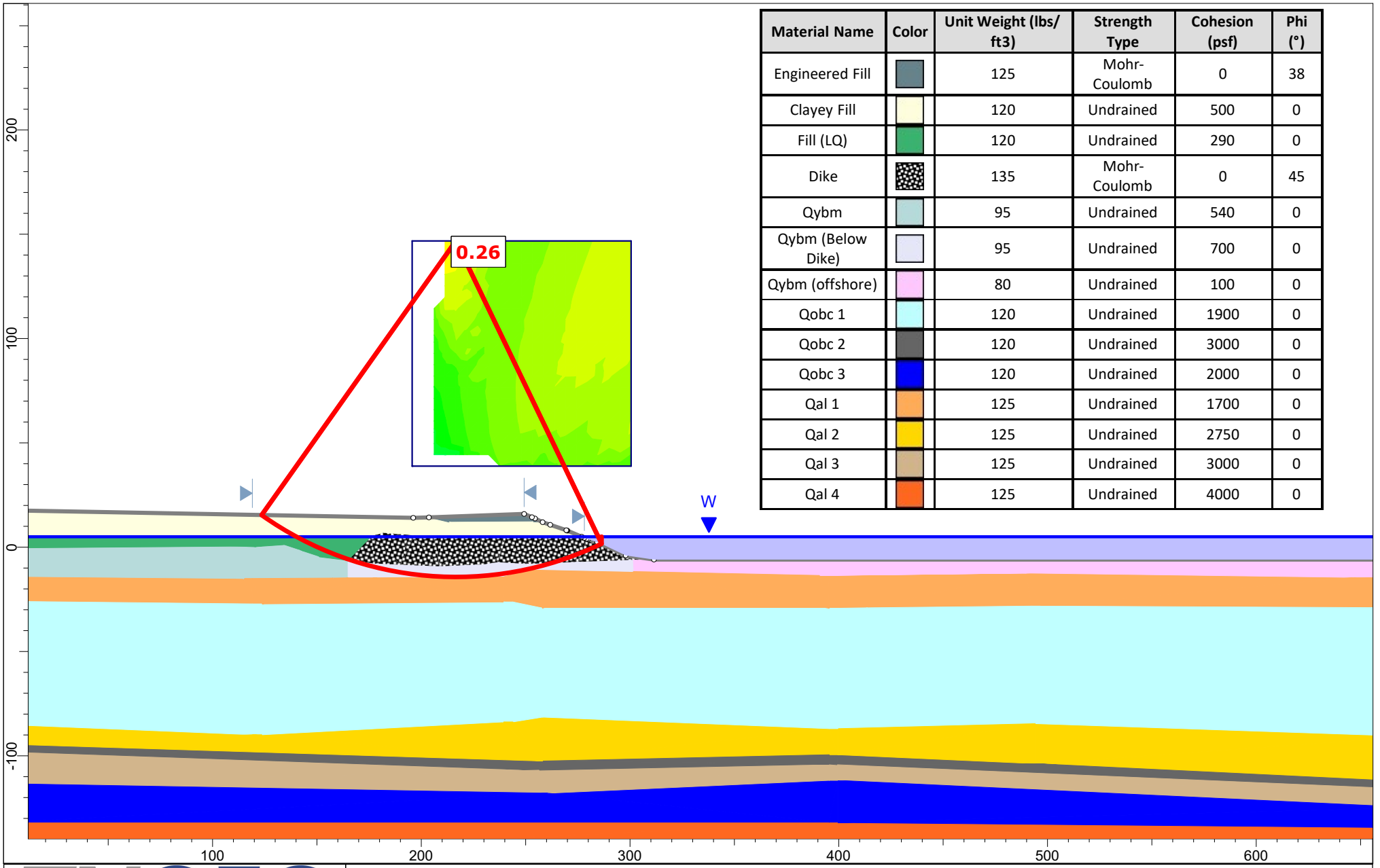
Project	Berkeley Water Transportation Pier Ferry		
Scale 1:750		*Author* VZ/JF	*Project No.* 25022.000.001
Date 5/28/2025		*Condition* Factor of Safety for Circular Failure Surface Near Waterside Slope Face	



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
Engineered Fill		125	Mohr-Coulomb	0	38
Clayey Fill		120	Undrained	500	0
Sandy Fill		120	Mohr-Coulomb	0	30
Dike		135	Mohr-Coulomb	0	45
Qybm		95	Undrained	415	0
Qybm (Below Dike)		95	Undrained	540	0
Qybm (offshore)		80	Undrained	100	0
Qobc 1		120	Undrained	1900	0
Qobc 2		120	Undrained	3000	0
Qobc 3		120	Undrained	2000	0
Qal 1		125	Undrained	1700	0
Qal 2		125	Undrained	2750	0
Qal 3		125	Undrained	3000	0
Qal 4		125	Undrained	4000	0

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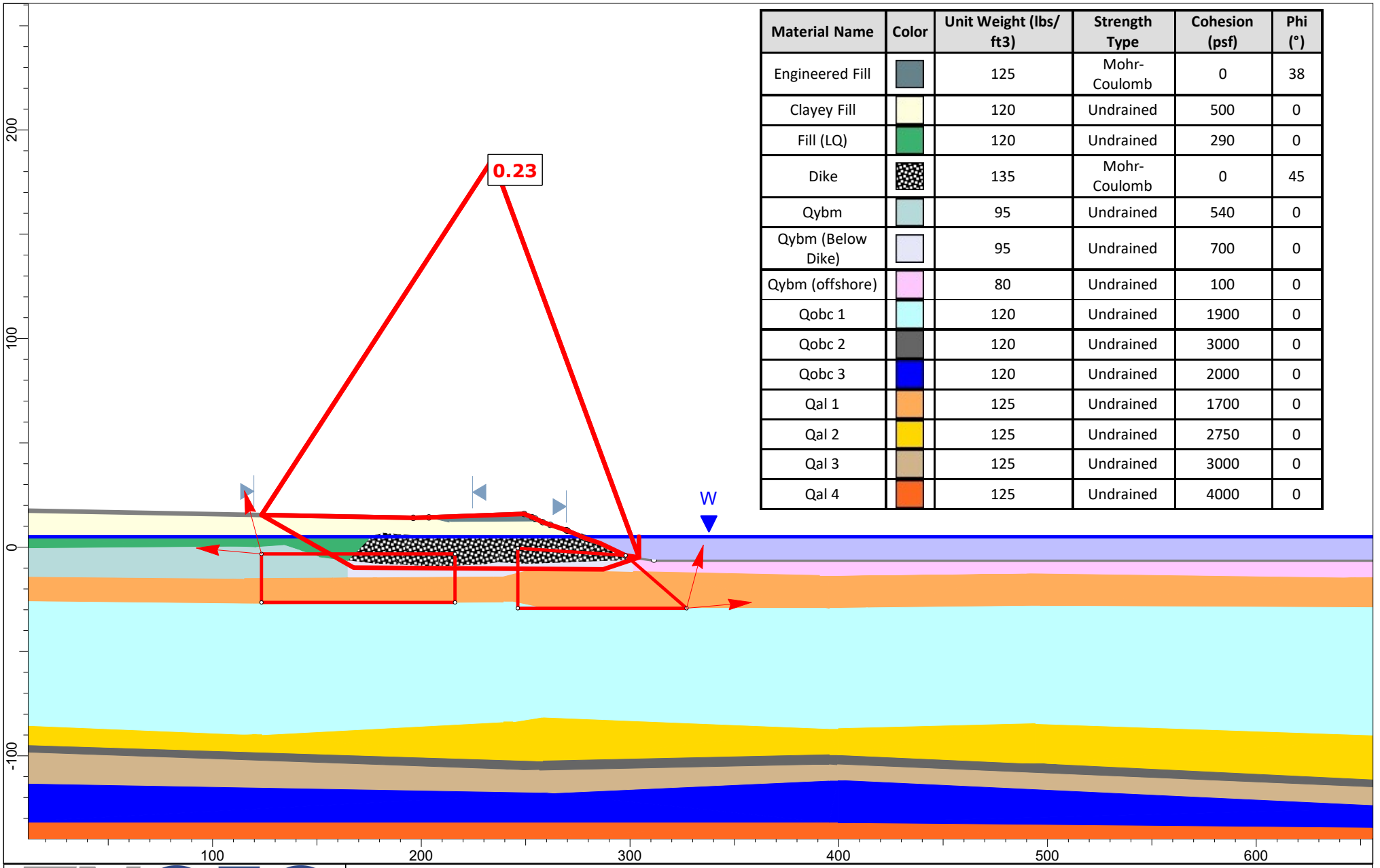
Project	Berkeley Water Transportation Pier Ferry		
Scale 1:750		*Author* VZ/JF	*Project No.* 25022.000.001
Date 5/28/2025		*Condition* Factor of Safety for Block Failure Surface Near Waterside Slope Face	



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
Engineered Fill		125	Mohr-Coulomb	0	38
Clayey Fill		120	Undrained	500	0
Fill (LQ)		120	Undrained	290	0
Dike		135	Mohr-Coulomb	0	45
Qybm		95	Undrained	540	0
Qybm (Below Dike)		95	Undrained	700	0
Qybm (offshore)		80	Undrained	100	0
Qobc 1		120	Undrained	1900	0
Qobc 2		120	Undrained	3000	0
Qobc 3		120	Undrained	2000	0
Qal 1		125	Undrained	1700	0
Qal 2		125	Undrained	2750	0
Qal 3		125	Undrained	3000	0
Qal 4		125	Undrained	4000	0

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Project: Berkeley Water Transportation Pier Ferry			
Scale: 1:750	Author: VZ/JF	Project No.: 25022.000.001	
Date: 5/28/2025	Condition: Yield Acceleration for Circular Failure Surface Near Waterside Slope Face		



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
Engineered Fill		125	Mohr-Coulomb	0	38
Clayey Fill		120	Undrained	500	0
Fill (LQ)		120	Undrained	290	0
Dike		135	Mohr-Coulomb	0	45
Qybm		95	Undrained	540	0
Qybm (Below Dike)		95	Undrained	700	0
Qybm (offshore)		80	Undrained	100	0
Qobc 1		120	Undrained	1900	0
Qobc 2		120	Undrained	3000	0
Qobc 3		120	Undrained	2000	0
Qal 1		125	Undrained	1700	0
Qal 2		125	Undrained	2750	0
Qal 3		125	Undrained	3000	0
Qal 4		125	Undrained	4000	0

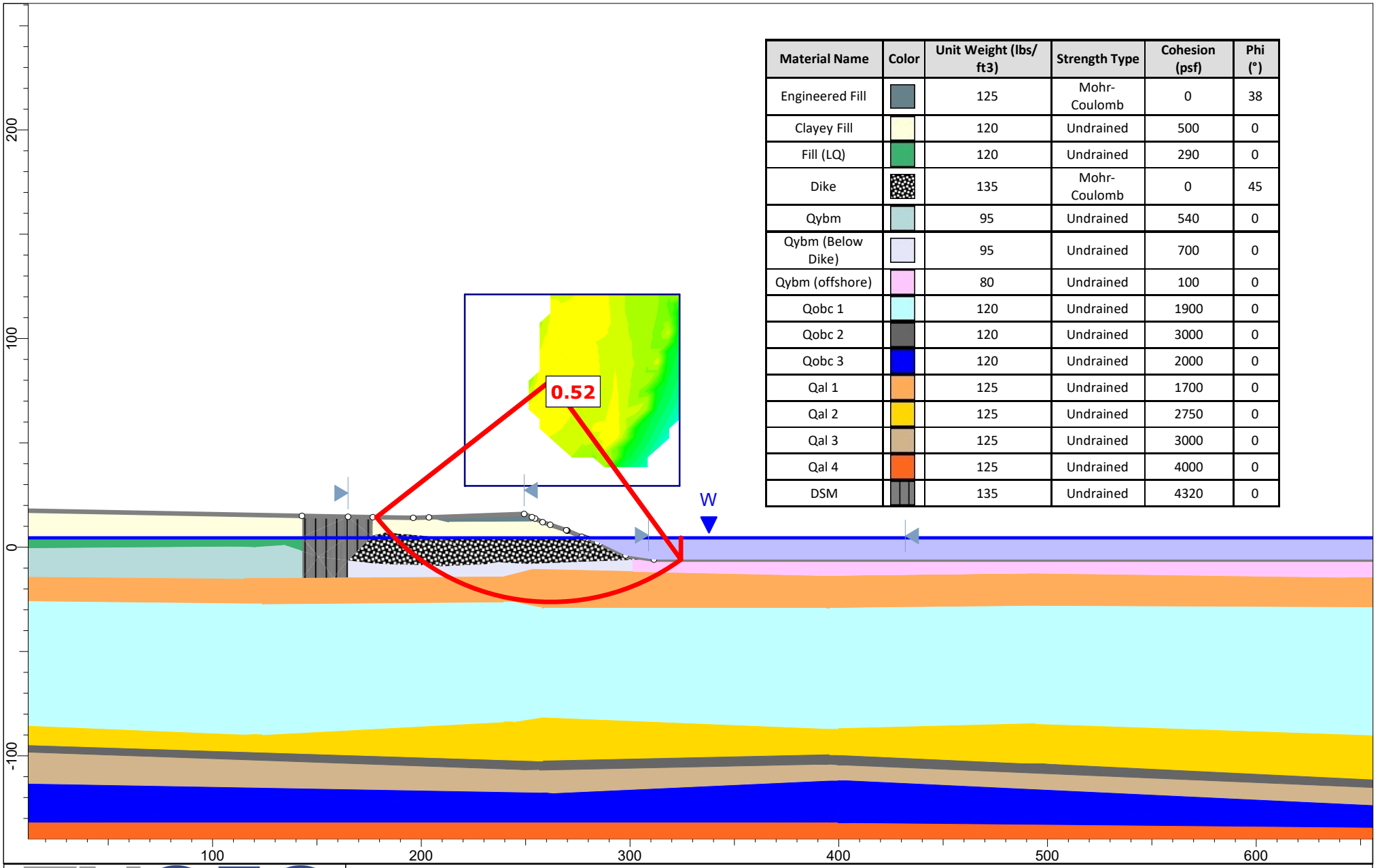
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Project: Berkeley Water Transportation Pier Ferry			
Scale: 1:750	Author: VZ/JF	Project No.: 25022.000.001	
Date: 5/28/2025	Condition: Yield Acceleration for Block Failure Surface Near Waterside Slope Face		



APPENDIX G.2

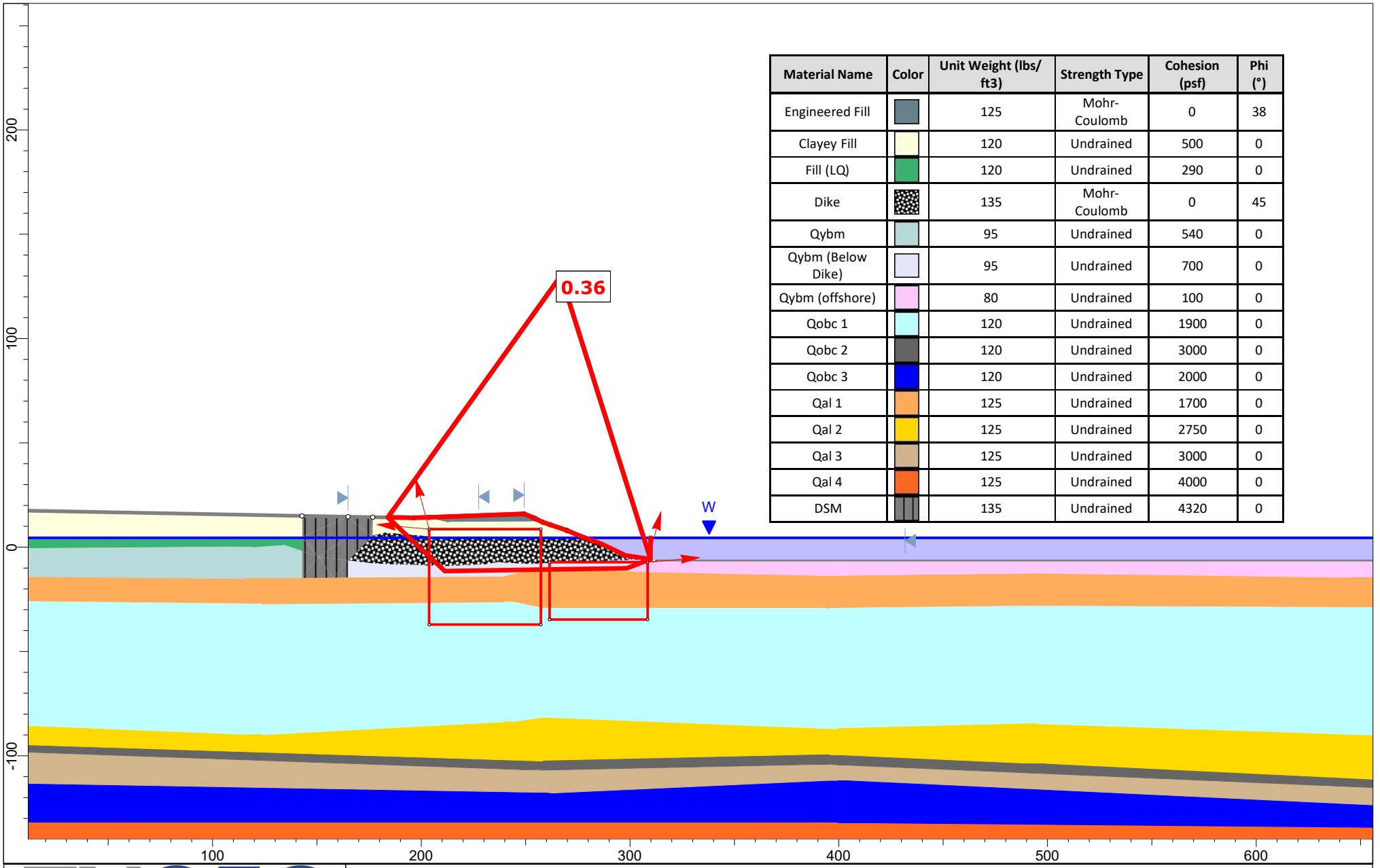
SLOPE STABILITY ANALYSIS – DEEP SOIL MIXING



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
Engineered Fill		125	Mohr-Coulomb	0	38
Clayey Fill		120	Undrained	500	0
Fill (LQ)		120	Undrained	290	0
Dike		135	Mohr-Coulomb	0	45
Qybm		95	Undrained	540	0
Qybm (Below Dike)		95	Undrained	700	0
Qybm (offshore)		80	Undrained	100	0
Qobc 1		120	Undrained	1900	0
Qobc 2		120	Undrained	3000	0
Qobc 3		120	Undrained	2000	0
Qal 1		125	Undrained	1700	0
Qal 2		125	Undrained	2750	0
Qal 3		125	Undrained	3000	0
Qal 4		125	Undrained	4000	0
DSM		135	Undrained	4320	0

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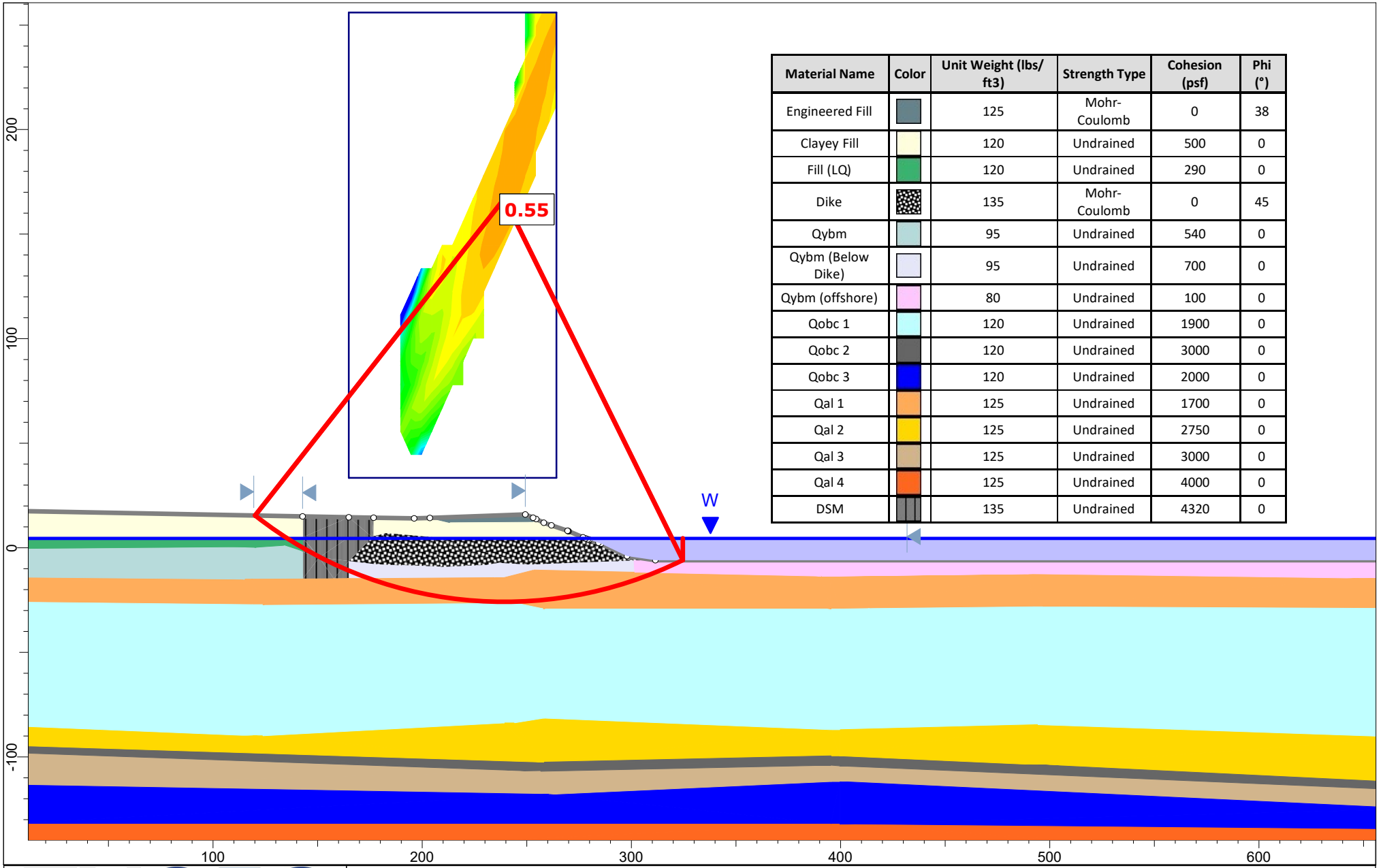
Project	Berkeley Water Transportation Pier Ferry		
Scale 1:750		*Author* VZ/JF	*Project No.* 25022.000.001
Date 5/28/2025		*Condition* Yield Acceleration for Circular Failure Surface in Front of the DSM Ground Improvement	



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
Engineered Fill		125	Mohr-Coulomb	0	38
Clayey Fill		120	Undrained	500	0
Fill (LQ)		120	Undrained	290	0
Dike		135	Mohr-Coulomb	0	45
Qybm		95	Undrained	540	0
Qybm (Below Dike)		95	Undrained	700	0
Qybm (offshore)		80	Undrained	100	0
Qobc 1		120	Undrained	1900	0
Qobc 2		120	Undrained	3000	0
Qobc 3		120	Undrained	2000	0
Qal 1		125	Undrained	1700	0
Qal 2		125	Undrained	2750	0
Qal 3		125	Undrained	3000	0
Qal 4		125	Undrained	4000	0
DSM		135	Undrained	4320	0

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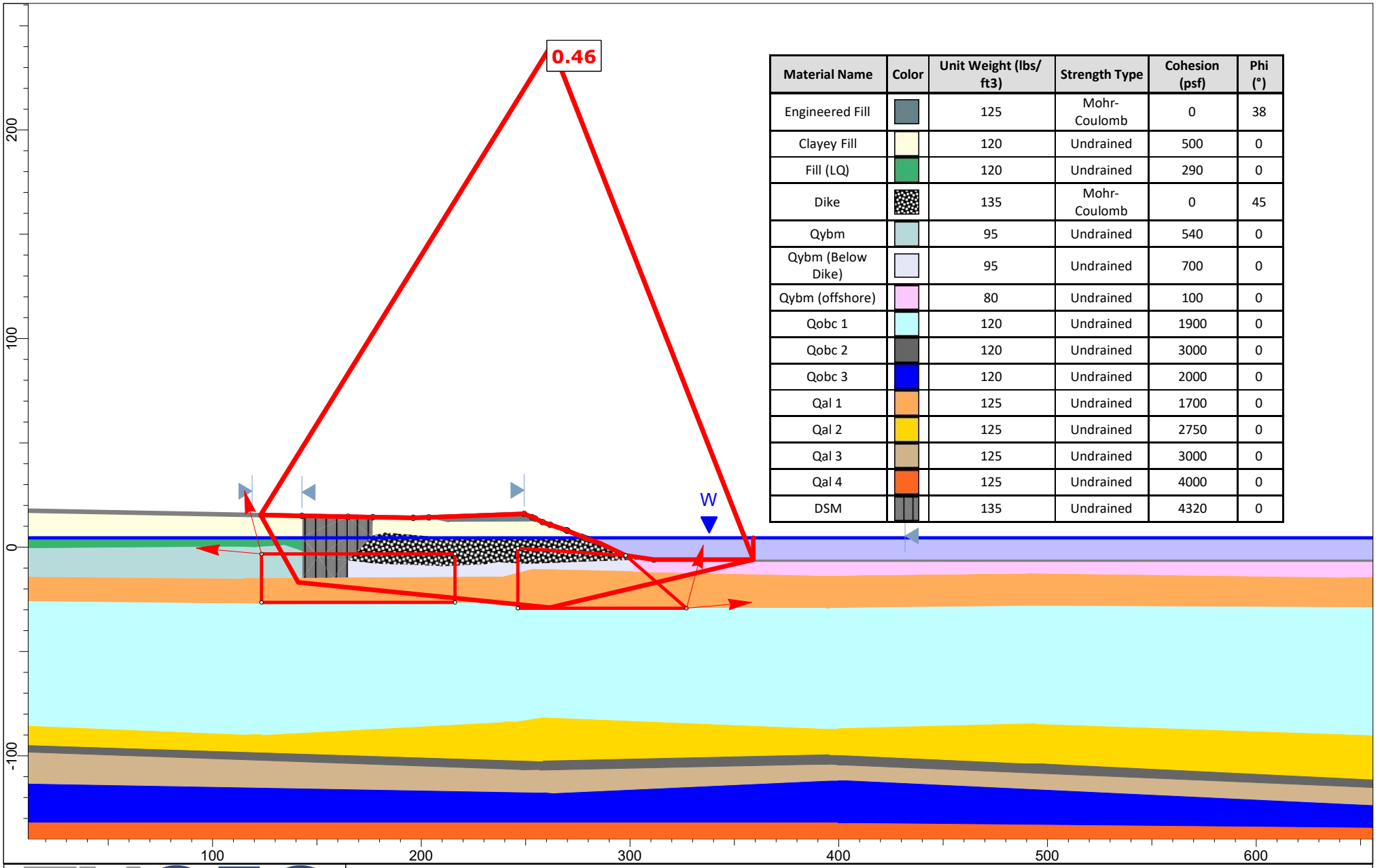
Project	Berkeley Water Transportation Pier Ferry		
Scale 1:750		*Author* VZ/JF	*Project No.* 25022.000.001
Date 5/28/2025		*Condition* Yield Acceleration for Block Failure Surface in Front of the DSM Ground Improvement	



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
Engineered Fill		125	Mohr-Coulomb	0	38
Clayey Fill		120	Undrained	500	0
Fill (LQ)		120	Undrained	290	0
Dike		135	Mohr-Coulomb	0	45
Qybm		95	Undrained	540	0
Qybm (Below Dike)		95	Undrained	700	0
Qybm (offshore)		80	Undrained	100	0
Qobc 1		120	Undrained	1900	0
Qobc 2		120	Undrained	3000	0
Qobc 3		120	Undrained	2000	0
Qal 1		125	Undrained	1700	0
Qal 2		125	Undrained	2750	0
Qal 3		125	Undrained	3000	0
Qal 4		125	Undrained	4000	0
DSM		135	Undrained	4320	0

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Project	Berkeley Water Transportation Pier Ferry		
Scale 1:750	*Author* VZ/JF	*Project No.* 25022.000.001	
Date 5/28/2025	*Condition* Yield Acceleration for Circular Failure Surface Behind the DSM Ground Improvement		



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
Engineered Fill		125	Mohr-Coulomb	0	38
Clayey Fill		120	Undrained	500	0
Fill (LQ)		120	Undrained	290	0
Dike		135	Mohr-Coulomb	0	45
Qybm		95	Undrained	540	0
Qybm (Below Dike)		95	Undrained	700	0
Qybm (offshore)		80	Undrained	100	0
Qobc 1		120	Undrained	1900	0
Qobc 2		120	Undrained	3000	0
Qobc 3		120	Undrained	2000	0
Qal 1		125	Undrained	1700	0
Qal 2		125	Undrained	2750	0
Qal 3		125	Undrained	3000	0
Qal 4		125	Undrained	4000	0
DSM		135	Undrained	4320	0

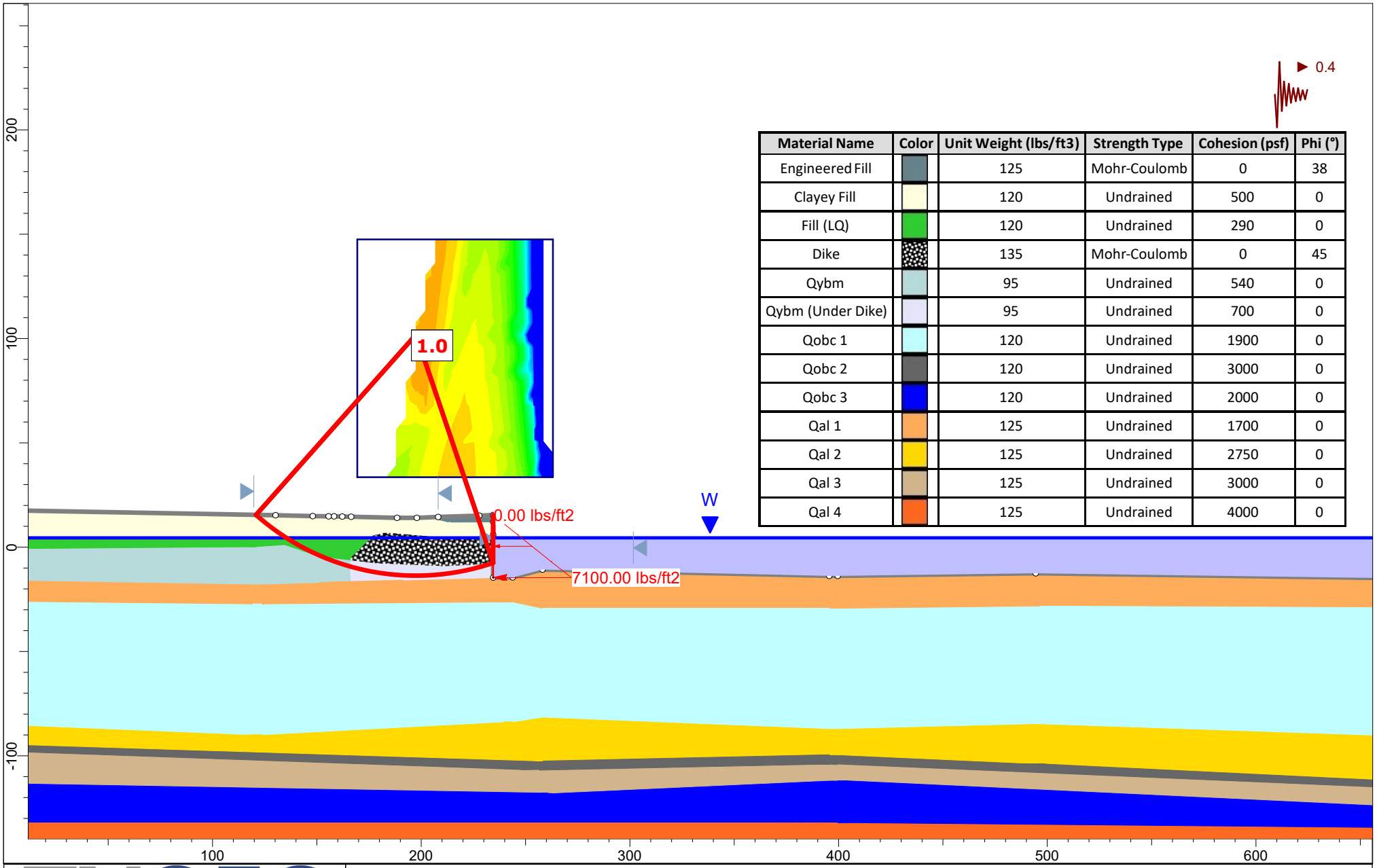
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Project	Berkeley Water Transportation Pier Ferry		
Scale 1:750	*Author* VZ/JF	*Project No.* 25022.000.001	
Date 5/28/2025	*Condition* Yield Acceleration for Block Failure Surface Behind the DSM Ground Improvement		



APPENDIX G.3

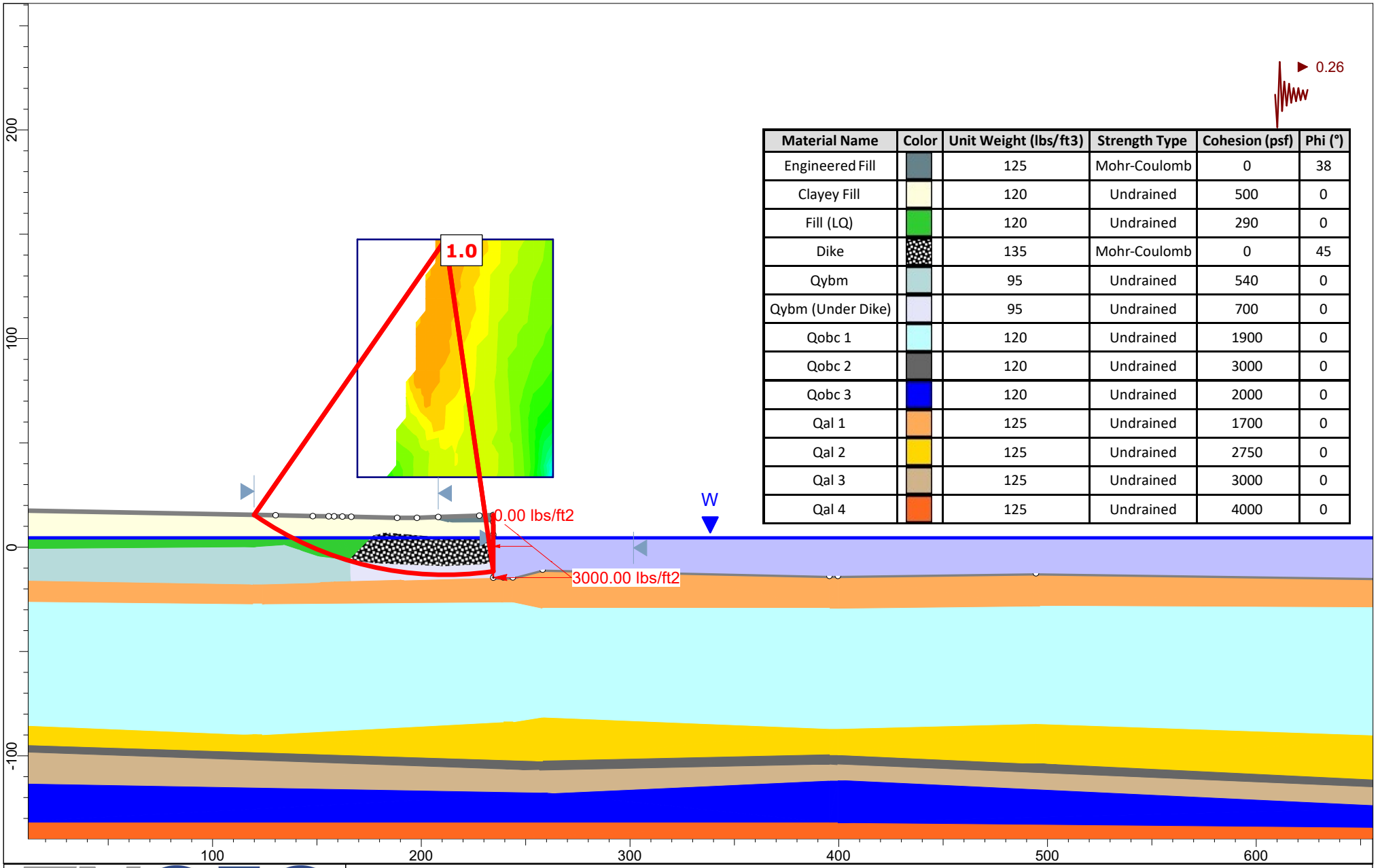
SLOPE STABILITY ANALYSIS – RETAINING WALL



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Engineered Fill		125	Mohr-Coulomb	0	38
Clayey Fill		120	Undrained	500	0
Fill (LQ)		120	Undrained	290	0
Dike		135	Mohr-Coulomb	0	45
Qybm		95	Undrained	540	0
Qybm (Under Dike)		95	Undrained	700	0
Qobc 1		120	Undrained	1900	0
Qobc 2		120	Undrained	3000	0
Qobc 3		120	Undrained	2000	0
Qal 1		125	Undrained	1700	0
Qal 2		125	Undrained	2750	0
Qal 3		125	Undrained	3000	0
Qal 4		125	Undrained	4000	0

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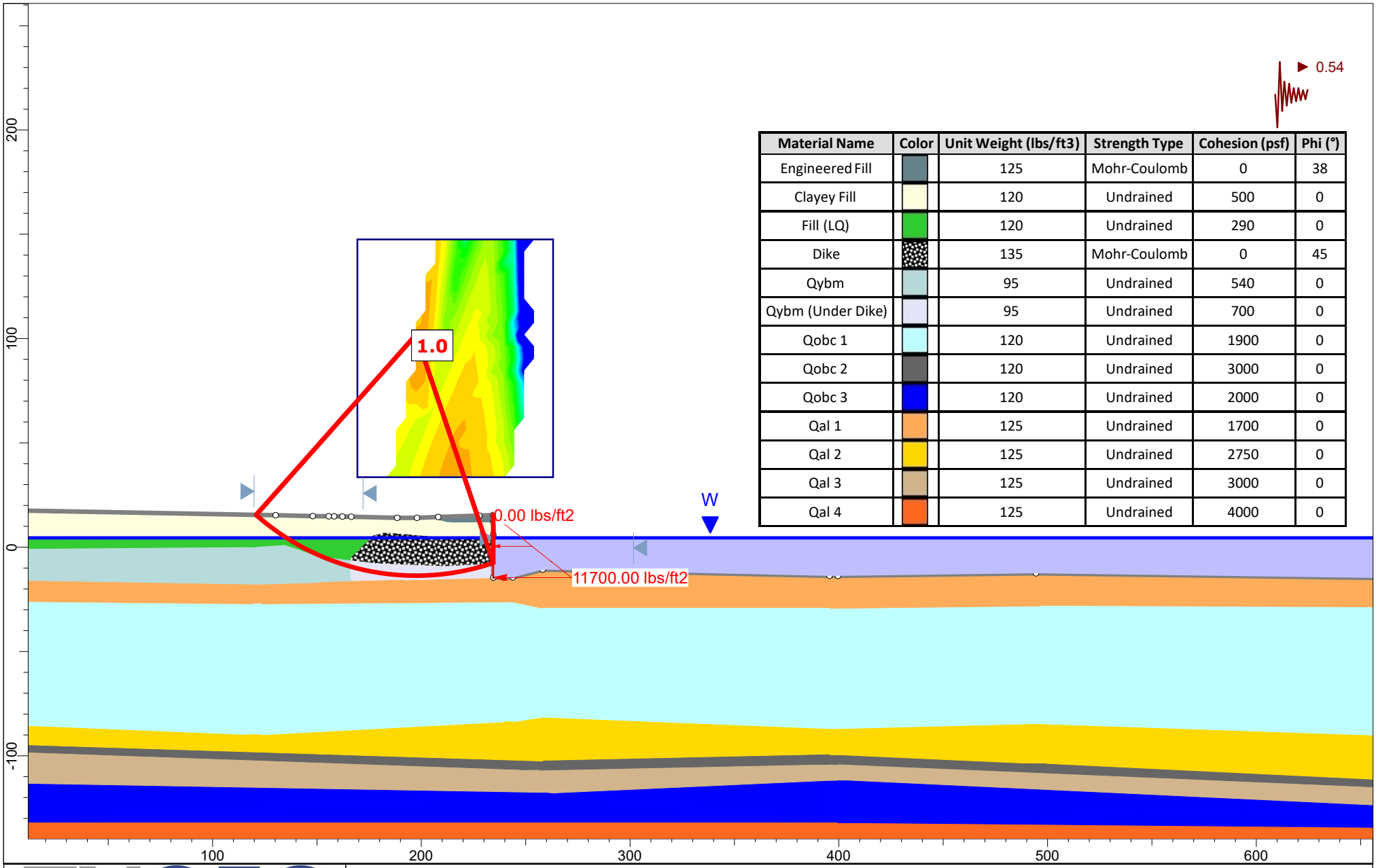
Project	Berkeley Water Transportation Pier Ferry		
Scale 1:750		Author VZ/JF	Project No. 25022.000.001
Date 5/28/2025		Condition Estimate of Earthquake and Soil Pressure on Retaining Wall Based on 2 inches of Lateral Movement During CLE-Level Earthquake (Bray and Macedo, 2019)	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Engineered Fill		125	Mohr-Coulomb	0	38
Clayey Fill		120	Undrained	500	0
Fill (LQ)		120	Undrained	290	0
Dike		135	Mohr-Coulomb	0	45
Qybm		95	Undrained	540	0
Qybm (Under Dike)		95	Undrained	700	0
Qobc 1		120	Undrained	1900	0
Qobc 2		120	Undrained	3000	0
Qobc 3		120	Undrained	2000	0
Qal 1		125	Undrained	1700	0
Qal 2		125	Undrained	2750	0
Qal 3		125	Undrained	3000	0
Qal 4		125	Undrained	4000	0

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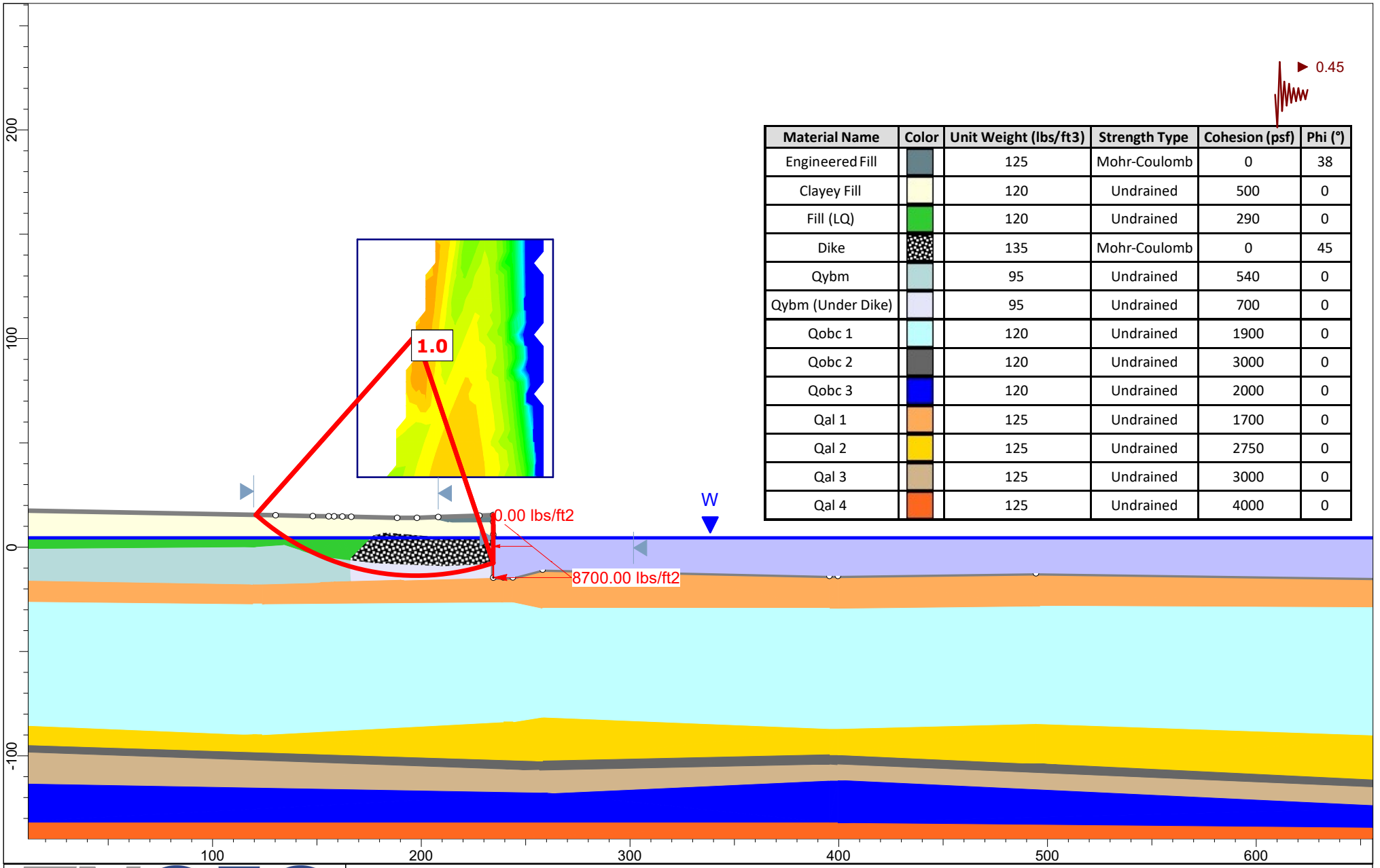
Project	Berkeley Water Transportation Pier Ferry		
Scale 1:750		Author VZ/JF	Project No. 25022.000.001
Date 5/28/2025		Condition Estimate of Earthquake and Soil Pressure on Retaining Wall Based on 2 inches of Lateral Movement During CLE-Level Earthquake (NCHRP, 2008)	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Engineered Fill		125	Mohr-Coulomb	0	38
Clayey Fill		120	Undrained	500	0
Fill (LQ)		120	Undrained	290	0
Dike		135	Mohr-Coulomb	0	45
Qybm		95	Undrained	540	0
Qybm (Under Dike)		95	Undrained	700	0
Qobc 1		120	Undrained	1900	0
Qobc 2		120	Undrained	3000	0
Qobc 3		120	Undrained	2000	0
Qal 1		125	Undrained	1700	0
Qal 2		125	Undrained	2750	0
Qal 3		125	Undrained	3000	0
Qal 4		125	Undrained	4000	0

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Project	Berkeley Water Transportation Pier Ferry		
Scale 1:750		Author VZ/JF	Project No. 25022.000.001
Date 5/28/2025		Condition Estimate of Earthquake and Soil Pressure on Retaining Wall Based on 2 inches of Lateral Movement During MCE-Level Earthquake (Bray and Macedo, 2019)	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Engineered Fill		125	Mohr-Coulomb	0	38
Clayey Fill		120	Undrained	500	0
Fill (LQ)		120	Undrained	290	0
Dike		135	Mohr-Coulomb	0	45
Qybm		95	Undrained	540	0
Qybm (Under Dike)		95	Undrained	700	0
Qobc 1		120	Undrained	1900	0
Qobc 2		120	Undrained	3000	0
Qobc 3		120	Undrained	2000	0
Qal 1		125	Undrained	1700	0
Qal 2		125	Undrained	2750	0
Qal 3		125	Undrained	3000	0
Qal 4		125	Undrained	4000	0

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Project	Berkeley Water Transportation Pier Ferry		
Scale 1:750	*Author* VZ/JF	*Project No.* 25022.000.001	
Date 5/28/2025	*Condition* Estimate of Earthquake and Soil Pressure on Retaining Wall Based on 2 inches of Lateral Movement During MCE-Level Earthquake (NCHRP, 2008)		



APPENDIX H

P-Y SPRINGS

4ft Concrete Breakwater p-y Springs

Section	Pier Alignment						Best Estimate						
Material	Depth Below Mudline(feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Qybm	0.5	0	0	9	2.84	16	16.59	21	39.32	26	76.80	26	81.60
Qybm	6.5	0	0	14	2.84	25	16.59	33	39.32	42	76.80	42	81.60
Qal 1 (Clay)	7.5	0	0	149	0.66	267	6.97	357	22.02	446	53.76	446	67.20
Qal 1 (Clay)	12.5	0	0	184	0.66	331	6.97	441	22.02	552	53.76	552	67.20
Qal 1 (Sand)	13.5	0	0	116	1.23	153	2.36	175	3.20	271	8.64	271	10.08
Qal 1 (Sand)	16.5	0	0	129	1.19	224	3.20	303	5.20	381	7.20	381	10.08
Qal 1 (Sand)	17.5	0	0	221	0.66	398	6.97	531	22.02	664	53.76	664	67.20
Qal 1 (Sand)	22.5	0	0	259	0.66	466	6.97	621	22.02	777	53.76	777	67.20
Qobc 1	23.5	0	0	284	0.66	512	6.97	682	22.02	853	53.76	853	67.20
Qobc 1	28	0	0	321	0.17	577	1.74	770	5.51	962	13.44	962	16.80
Qobc 1	33	0	0	362	0.66	651	6.97	868	22.02	1085	53.76	1085	67.20
Qobc 1	38	0	0	402	0.66	724	6.97	966	22.02	1207	53.76	1207	67.20
Qobc 1	43	0	0	443	0.66	798	6.97	1064	22.02	1330	53.76	1330	67.20
Qobc 1	48	0	0	475	0.66	855	6.97	1140	22.02	1425	53.76	1425	67.20
Qobc 1	79.5	0	0	475	0.66	855	6.97	1140	22.02	1425	53.76	1425	67.20
Qal 2 (Clay)	80.5	0	0	688	0.47	1238	4.98	1650	15.73	2063	38.40	2063	48.00
Qal 2 (Clay)	92.5	0	0	688	0.47	1238	4.98	1650	15.73	2063	38.40	2063	48.00
Qobc 2	93.5	0	0	750	0.47	1350	4.98	1800	15.73	2250	38.40	2250	48.00
Qobc 2	97.5	0	0	3000	0.47	5400	4.98	7200	15.73	9000	38.40	9000	48.00
Qal 3 (Clay)	98.5	0	0	750	0.47	1350	4.98	1800	15.73	2250	38.40	2250	48.00
Qal 3 (Clay)	105.5	0	0	750	0.47	1350	4.98	1800	15.73	2250	38.40	2250	48.00
Qobc 3	106.5	0	0	500	0.47	900	4.98	1200	15.73	1500	38.40	1500	48.00
Qobc 3	126.5	0	0	500	0.47	900	4.98	1200	15.73	1500	38.40	1500	48.00
Qal 4 (Clay)	127.5	0	0	750	0.47	1350	4.98	1800	15.73	2250	38.40	2250	48.00
Qal 4 (Clay)	139.5	0	0	750	0.47	1350	4.98	1800	15.73	2250	38.40	2250	48.00

Section	Pier Alignment						Upper Bound						
Material	Depth Below Mudline(feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Qybm	0.5	0	0	18	2.84	32	16.59	42	39.32	53	76.80	53	81.60
Qybm	6.5	0	0	28	2.84	50	16.59	66	39.32	83	76.80	83	81.60
Qal 1 (Clay)	7.5	0	0	297	0.66	535	6.97	713	22.02	891	53.76	891	67.20
Qal 1 (Clay)	12.5	0	0	368	0.66	662	6.97	882	22.02	1103	53.76	1103	67.20
Qal 1 (Sand)	13.5	0	0	231	1.23	307	2.36	351	3.20	541	8.64	541	10.08
Qal 1 (Sand)	16.5	0	0	257	1.19	448	3.20	605	5.20	763	7.20	763	10.08
Qal 1 (Sand)	17.5	0	0	443	0.66	797	6.97	1062	22.02	1328	53.76	1328	67.20
Qal 1 (Sand)	22.5	0	0	518	0.66	932	6.97	1243	22.02	1553	53.76	1553	67.20
Qobc 1	23.5	0	0	568	0.66	1023	6.97	1364	22.02	1705	53.76	1705	67.20
Qobc 1	28	0	0	642	0.17	1155	1.74	1540	5.51	1925	13.44	1925	16.80
Qobc 1	33	0	0	723	0.66	1302	6.97	1736	22.02	2170	53.76	2170	67.20
Qobc 1	38	0	0	805	0.66	1449	6.97	1932	22.02	2415	53.76	2415	67.20
Qobc 1	43	0	0	887	0.66	1596	6.97	2128	22.02	2660	53.76	2660	67.20
Qobc 1	48	0	0	950	0.66	1710	6.97	2280	22.02	2850	53.76	2850	67.20
Qobc 1	79.5	0	0	950	0.66	1710	6.97	2280	22.02	2850	53.76	2850	67.20
Qal 2 (Clay)	80.5	0	0	1375	0.47	2475	4.98	3300	15.73	4125	38.40	4125	48.00
Qal 2 (Clay)	92.5	0	0	1375	0.47	2475	4.98	3300	15.73	4125	38.40	4125	48.00
Qobc 2	93.5	0	0	1500	0.47	2700	4.98	3600	15.73	4500	38.40	4500	48.00
Qobc 2	97.5	0	0	6000	0.47	10800	4.98	14400	15.73	18000	38.40	18000	48.00
Qal 3 (Clay)	98.5	0	0	1500	0.47	2700	4.98	3600	15.73	4500	38.40	4500	48.00
Qal 3 (Clay)	105.5	0	0	1500	0.47	2700	4.98	3600	15.73	4500	38.40	4500	48.00
Qobc 3	106.5	0	0	1000	0.47	1800	4.98	2400	15.73	3000	38.40	3000	48.00
Qobc 3	126.5	0	0	1000	0.47	1800	4.98	2400	15.73	3000	38.40	3000	48.00
Qal 4 (Clay)	127.5	0	0	1500	0.47	2700	4.98	3600	15.73	4500	38.40	4500	48.00
Qal 4 (Clay)	139.5	0	0	1500	0.47	2700	4.98	3600	15.73	4500	38.40	4500	48.00

Section	Pier Alignment						Lower Bound						
Material	Depth Below Mudline(feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Qybm	0.5	0	0	4	2.84	8	16.59	11	39.32	13	76.80	13	81.60
Qybm	6.5	0	0	7	2.84	12	16.59	17	39.32	21	76.80	21	81.60
Qal 1 (Clay)	7.5	0	0	74	0.66	134	6.97	178	22.02	223	53.76	223	67.20
Qal 1 (Clay)	12.5	0	0	92	0.66	165	6.97	221	22.02	276	53.76	276	67.20
Qal 1 (Sand)	13.5	0	0	58	1.23	77	2.36	88	3.20	135	8.64	135	10.08
Qal 1 (Sand)	16.5	0	0	64	1.19	112	3.20	151	5.20	191	7.20	191	10.08
Qal 1 (Sand)	17.5	0	0	111	0.66	199	6.97	266	22.02	332	53.76	332	67.20
Qal 1 (Sand)	22.5	0	0	129	0.66	233	6.97	311	22.02	388	53.76	388	67.20
Qobc 1	23.5	0	0	142	0.66	256	6.97	341	22.02	426	53.76	426	67.20
Qobc 1	28	0	0	160	0.17	289	1.74	385	5.51	481	13.44	481	16.80
Qobc 1	33	0	0	181	0.66	325	6.97	434	22.02	542	53.76	542	67.20
Qobc 1	38	0	0	201	0.66	362	6.97	483	22.02	604	53.76	604	67.20
Qobc 1	43	0	0	222	0.66	399	6.97	532	22.02	665	53.76	665	67.20
Qobc 1	48	0	0	238	0.66	428	6.97	570	22.02	713	53.76	713	67.20
Qobc 1	79.5	0	0	238	0.66	428	6.97	570	22.02	713	53.76	713	67.20
Qal 2 (Clay)	80.5	0	0	344	0.47	619	4.98	825	15.73	1031	38.40	1031	48.00
Qal 2 (Clay)	92.5	0	0	344	0.47	619	4.98	825	15.73	1031	38.40	1031	48.00
Qobc 2	93.5	0	0	375	0.47	675	4.98	900	15.73	1125	38.40	1125	48.00
Qobc 2	97.5	0	0	1500	0.47	2700	4.98	3600	15.73	4500	38.40	4500	48.00
Qal 3 (Clay)	98.5	0	0	375	0.47	675	4.98	900	15.73	1125	38.40	1125	48.00
Qal 3 (Clay)	105.5	0	0	375	0.47	675	4.98	900	15.73	1125	38.40	1125	48.00
Qobc 3	106.5	0	0	250	0.47	450	4.98	600	15.73	750	38.40	750	48.00
Qobc 3	126.5	0	0	250	0.47	450	4.98	600	15.73	750	38.40	750	48.00
Qal 4 (Clay)	127.5	0	0	375	0.47	675	4.98	900	15.73	1125	38.40	1125	48.00
Qal 4 (Clay)	139.5	0	0	375	0.47	675	4.98	900	15.73	1125	38.40	1125	48.00

24-inch Concrete Precast Prestressed Octagonal Pile P-y Springs

Section	Pier Alignment						Best Estimate						
Material	Depth Below Mudline(feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Qybm	0.5	0	0	18	0.36	32	2.07	43	4.92	54	9.60	54	10.20
Qybm	6.5	0	0	32	0.36	58	2.07	77	4.92	97	9.60	97	10.20
Qal 1 (Clay)	7.5	0	0	311	0.08	560	0.87	747	2.75	933	6.72	933	8.40
Qal 1 (Clay)	12.5	0	0	441	0.08	793	0.87	1058	2.75	1322	6.72	1322	8.40
Qal 1 (Sand)	13.5	0	0	338	0.08	700	0.26	1614	0.90	1614	1.08	1614	1.26
Qal 1 (Sand)	16.5	0	0	569	0.08	1142	0.26	2619	0.90	2619	1.08	2619	1.26
Qal 1 (Clay)	17.5	0	0	608	0.08	1094	0.87	1459	2.75	1824	6.72	1824	8.40
Qal 1 (Clay)	22.5	0	0	742	0.08	1336	0.87	1782	2.75	2227	6.72	2227	8.40
Qobc 1	23.5	0	0	816	0.08	1468	0.87	1957	2.75	2447	6.72	2447	8.40
Qobc 1	28	0	0	948	0.08	1707	0.87	2276	2.75	2845	6.72	2845	8.40
Qobc 1	33	0	0	950	0.08	1710	0.87	2280	2.75	2850	6.72	2850	8.40
Qobc 1	79.5	0	0	1710	0.87	2280	2.75	2660	5.10	2850	6.72	2850	8.40
Qal 2 (Clay)	80.5	0	0	2475	0.62	3300	1.97	3850	3.64	4125	4.80	4125	6.00
Qal 2 (Clay)	92.5	0	0	2475	0.62	3300	1.97	3850	3.64	4125	4.80	4125	6.00
Qobc 2	93.5	0	0	2700	0.62	3600	1.97	4200	3.64	4500	4.80	4500	6.00
Qobc 2	97.5	0	0	2700	0.62	3600	1.97	4200	3.64	4500	4.80	4500	6.00
Qal 3 (Clay)	98.5	0	0	2700	0.62	3600	1.97	4200	3.64	4500	4.80	4500	6.00
Qal 3 (Clay)	105.5	0	0	2700	0.62	3600	1.97	4200	3.64	4500	4.80	4500	6.00
Qobc 3	106.5	0	0	1800	0.62	2400	1.97	2800	3.64	3000	4.80	3000	6.00
Qobc 3	126.5	0	0	1800	0.62	2400	1.97	2800	3.64	3000	4.80	3000	6.00
Qal 4 (Clay)	127.5	0	0	4000	0.76	5200	2.17	5600	2.91	6000	3.84	6000	4.80
Qal 4 (Clay)	139.5	0	0	4000	0.76	5200	2.17	5600	2.91	6000	3.84	6000	4.80

Section	Pier Alignment						Upper Bound						
Material	Depth Below Mudline(feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Qybm	0.5	0	0	36	0.36	64	2.07	86	4.92	107	9.60	107	10.20
Qybm	6.5	0	0	64	0.36	116	2.07	155	4.92	193	9.60	193	10.20
Qal 1 (Clay)	7.5	0	0	622	0.08	1120	0.87	1493	2.75	1866	6.72	1866	8.40
Qal 1 (Clay)	12.5	0	0	881	0.08	1586	0.87	2115	2.75	2644	6.72	2644	8.40
Qal 1 (Sand)	13.5	0	0	676	0.08	1401	0.26	3229	0.90	3229	1.08	3229	1.26
Qal 1 (Sand)	16.5	0	0	1138	0.08	2284	0.26	5238	0.90	5238	1.08	5238	1.26
Qal 1 (Clay)	17.5	0	0	1216	0.08	2189	0.87	2918	2.75	3648	6.72	3648	8.40
Qal 1 (Clay)	22.5	0	0	1485	0.08	2673	0.87	3563	2.75	4454	6.72	4454	8.40
Qobc 1	23.5	0	0	1631	0.08	2936	0.87	3915	2.75	4894	6.72	4894	8.40
Qobc 1	28	0	0	1897	0.08	3414	0.87	4552	2.75	5690	6.72	5690	8.40
Qobc 1	33	0	0	1900	0.08	3420	0.87	4560	2.75	5700	6.72	5700	8.40
Qobc 1	79.5	0	0	3420	0.87	4560	2.75	5320	5.10	5700	6.72	5700	8.40
Qal 2 (Clay)	80.5	0	0	4950	0.62	6600	1.97	7700	3.64	8250	4.80	8250	6.00
Qal 2 (Clay)	92.5	0	0	4950	0.62	6600	1.97	7700	3.64	8250	4.80	8250	6.00
Qobc 2	93.5	0	0	5400	0.62	7200	1.97	8400	3.64	9000	4.80	9000	6.00
Qobc 2	97.5	0	0	5400	0.62	7200	1.97	8400	3.64	9000	4.80	9000	6.00
Qal 3 (Clay)	98.5	0	0	5400	0.62	7200	1.97	8400	3.64	9000	4.80	9000	6.00
Qal 3 (Clay)	105.5	0	0	5400	0.62	7200	1.97	8400	3.64	9000	4.80	9000	6.00
Qobc 3	106.5	0	0	3600	0.62	4800	1.97	5600	3.64	6000	4.80	6000	6.00
Qobc 3	126.5	0	0	3600	0.62	4800	1.97	5600	3.64	6000	4.80	6000	6.00
Qal 4 (Clay)	127.5	0	0	8000	0.76	10400	2.17	11200	2.91	12000	3.84	12000	4.80
Qal 4 (Clay)	139.5	0	0	8000	0.76	10400	2.17	11200	2.91	12000	3.84	12000	4.80

Section	Pier Alignment						Lower Bound						
Material	Depth Below Mudline(feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Qybm	0.5	0	0	9	0.36	16	2.07	21	4.92	27	9.60	27	10.20
Qybm	6.5	0	0	16	0.36	29	2.07	39	4.92	48	9.60	48	10.20
Qal 1 (Clay)	7.5	0	0	156	0.08	280	0.87	373	2.75	467	6.72	467	8.40
Qal 1 (Clay)	12.5	0	0	220	0.08	397	0.87	529	2.75	661	6.72	661	8.40
Qal 1 (Sand)	13.5	0	0	169	0.08	350	0.26	807	0.90	807	1.08	807	1.26
Qal 1 (Sand)	16.5	0	0	285	0.08	571	0.26	1310	0.90	1310	1.08	1310	1.26
Qal 1 (Clay)	17.5	0	0	304	0.08	547	0.87	730	2.75	912	6.72	912	8.40
Qal 1 (Clay)	22.5	0	0	371	0.08	668	0.87	891	2.75	1114	6.72	1114	8.40
Qobc 1	23.5	0	0	408	0.08	734	0.87	979	2.75	1223	6.72	1223	8.40
Qobc 1	28	0	0	474	0.08	854	0.87	1138	2.75	1423	6.72	1423	8.40
Qobc 1	33	0	0	475	0.08	855	0.87	1140	2.75	1425	6.72	1425	8.40
Qobc 1	79.5	0	0	855	0.87	1140	2.75	1330	5.10	1425	6.72	1425	8.40
Qal 2 (Clay)	80.5	0	0	1238	0.62	1650	1.97	1925	3.64	2063	4.80	2063	6.00
Qal 2 (Clay)	92.5	0	0	1238	0.62	1650	1.97	1925	3.64	2063	4.80	2063	6.00
Qobc 2	93.5	0	0	1350	0.62	1800	1.97	2100	3.64	2250	4.80	2250	6.00
Qobc 2	97.5	0	0	1350	0.62	1800	1.97	2100	3.64	2250	4.80	2250	6.00
Qal 3 (Clay)	98.5	0	0	1350	0.62	1800	1.97	2100	3.64	2250	4.80	2250	6.00
Qal 3 (Clay)	105.5	0	0	1350	0.62	1800	1.97	2100	3.64	2250	4.80	2250	6.00
Qobc 3	106.5	0	0	900	0.62	1200	1.97	1400	3.64	1500	4.80	1500	6.00
Qobc 3	126.5	0	0	900	0.62	1200	1.97	1400	3.64	1500	4.80	1500	6.00
Qal 4 (Clay)	127.5	0	0	2000	0.76	2600	2.17	2800	2.91	3000	3.84	3000	4.80
Qal 4 (Clay)	139.5	0	0	2000	0.76	2600	2.17	2800	2.91	3000	3.84	3000	4.80

36-inch Steel Pipe Pile p-y Springs

Section	Pier Alignment						Best Estimate						
Material	Depth Below Mudline(feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Qybm	0.5	0	0	26	0.53	48	3.11	63	7.37	79	14.40	79	15.30
Qybm	6.5	0	0	44	0.53	79	3.11	105	7.37	131	14.40	131	15.30
Qal 1 (Clay)	7.5	0	0	452	0.12	814	1.31	1086	4.13	1357	10.08	1357	12.60
Qal 1 (Clay)	12.5	0	0	588	0.12	1058	1.31	1411	4.13	1763	10.08	1763	12.60
Qal 1 (Sand)	13.5	0	0	407	0.11	825	0.38	1850	1.35	1850	1.62	1850	1.89
Qal 1 (Sand)	16.5	0	0	608	0.12	1261	0.38	2908	1.35	2908	1.62	2908	1.89
Qal 1 (Clay)	17.5	0	0	746	0.12	1344	1.31	1791	4.13	2239	10.08	2239	12.60
Qal 1 (Clay)	22.5	0	0	889	0.12	1600	1.31	2133	4.13	2666	10.08	2666	12.60
Qobc 1	23.5	0	0	976	0.12	1756	1.31	2341	4.13	2927	10.08	2927	12.60
Qobc 1	28	0	0	1115	0.12	2008	1.31	2677	4.13	3346	10.08	3346	12.60
Qobc 1	33	0	0	1271	0.12	2287	1.31	3050	4.13	3812	10.08	3812	12.60
Qobc 1	38	0	0	1425	0.12	2565	1.31	3420	4.13	4275	10.08	4275	12.60
Qobc 1	79.5	0	0	1425	0.12	2565	1.31	3420	4.13	4275	10.08	4275	12.60
Qal 1 (Clay)	80.5	0	0	2063	0.09	3713	0.93	4950	2.95	6188	7.20	6188	9.00
Qal 1 (Clay)	92.5	0	0	2063	0.09	3713	0.93	4950	2.95	6188	7.20	6188	9.00
Qobc 2	93.5	0	0	2250	0.09	4050	0.93	5400	2.95	6750	7.20	6750	9.00
Qobc 2	97.5	0	0	2250	0.09	4050	0.93	5400	2.95	6750	7.20	6750	9.00
Qal 3 (Clay)	98.5	0	0	2250	0.09	4050	0.93	5400	2.95	6750	7.20	6750	9.00
Qal 3 (Clay)	105.5	0	0	2250	0.09	4050	0.93	5400	2.95	6750	7.20	6750	9.00
Qobc 3	106.5	0	0	1500	0.09	2700	0.93	3600	2.95	4500	7.20	4500	9.00
Qobc 3	126.5	0	0	1500	0.09	2700	0.93	3600	2.95	4500	7.20	4500	9.00
Qal 4 (Clay)	127.5	0	0	2250	0.09	4050	0.93	5400	2.95	6750	7.20	6750	9.00
Qal 4 (Clay)	139.5	0	0	2250	0.09	4050	0.93	5400	2.95	6750	7.20	6750	9.00

Section	Pier Alignment						Upper Bound						
Material	Depth Below Mudline(feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Qybm	0.5	0	0	53	0.53	95	3.11	127	7.37	159	14.40	159	15.30
Qybm	6.5	0	0	88	0.53	158	3.11	210	7.37	263	14.40	263	15.30
Qal 1 (Clay)	7.5	0	0	905	0.12	1629	1.31	2172	4.13	2715	10.08	2715	12.60
Qal 1 (Clay)	12.5	0	0	1176	0.12	2116	1.31	2821	4.13	3527	10.08	3527	12.60
Qal 1 (Sand)	13.5	0	0	813	0.11	1650	0.38	3699	1.35	3699	1.62	3699	1.89
Qal 1 (Sand)	16.5	0	0	1215	0.12	2523	0.38	5816	1.35	5816	1.62	5816	1.89
Qal 1 (Clay)	17.5	0	0	1493	0.12	2687	1.31	3583	4.13	4478	10.08	4478	12.60
Qal 1 (Clay)	22.5	0	0	1778	0.12	3200	1.31	4266	4.13	5333	10.08	5333	12.60
Qobc 1	23.5	0	0	1951	0.12	3512	1.31	4683	4.13	5854	10.08	5854	12.60
Qobc 1	28	0	0	2231	0.12	4015	1.31	5353	4.13	6692	10.08	6692	12.60
Qobc 1	33	0	0	2542	0.12	4575	1.31	6100	4.13	7625	10.08	7625	12.60
Qobc 1	38	0	0	2850	0.12	5130	1.31	6840	4.13	8550	10.08	8550	12.60
Qobc 1	79.5	0	0	2850	0.12	5130	1.31	6840	4.13	8550	10.08	8550	12.60
Qal 1 (Clay)	80.5	0	0	4125	0.09	7425	0.93	9900	2.95	12375	7.20	12375	9.00
Qal 1 (Clay)	92.5	0	0	4125	0.09	7425	0.93	9900	2.95	12375	7.20	12375	9.00
Qobc 2	93.5	0	0	4500	0.09	8100	0.93	10800	2.95	13500	7.20	13500	9.00
Qobc 2	97.5	0	0	4500	0.09	8100	0.93	10800	2.95	13500	7.20	13500	9.00
Qal 3 (Clay)	98.5	0	0	4500	0.09	8100	0.93	10800	2.95	13500	7.20	13500	9.00
Qal 3 (Clay)	105.5	0	0	4500	0.09	8100	0.93	10800	2.95	13500	7.20	13500	9.00
Qobc 3	106.5	0	0	3000	0.09	5400	0.93	7200	2.95	9000	7.20	9000	9.00
Qobc 3	126.5	0	0	3000	0.09	5400	0.93	7200	2.95	9000	7.20	9000	9.00
Qal 4 (Clay)	127.5	0	0	4500	0.09	8100	0.93	10800	2.95	13500	7.20	13500	9.00
Qal 4 (Clay)	139.5	0	0	4500	0.09	8100	0.93	10800	2.95	13500	7.20	13500	9.00

Section	Pier Alignment						Lower Bound						
Material	Depth Below Mudline(feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Qybm	0.5	0	0	13	0.53	24	3.11	32	7.37	40	14.40	40	15.30
Qybm	6.5	0	0	22	0.53	39	3.11	53	7.37	66	14.40	66	15.30
Qal 1 (Clay)	7.5	0	0	226	0.12	407	1.31	543	4.13	679	10.08	679	12.60
Qal 1 (Clay)	12.5	0	0	294	0.12	529	1.31	705	4.13	882	10.08	882	12.60
Qal 1 (Sand)	13.5	0	0	203	0.11	413	0.38	925	1.35	925	1.62	925	1.89
Qal 1 (Sand)	16.5	0	0	304	0.12	631	0.38	1454	1.35	1454	1.62	1454	1.89
Qal 1 (Clay)	17.5	0	0	373	0.12	672	1.31	896	4.13	1120	10.08	1120	12.60
Qal 1 (Clay)	22.5	0	0	444	0.12	800	1.31	1067	4.13	1333	10.08	1333	12.60
Qobc 1	23.5	0	0	488	0.12	878	1.31	1171	4.13	1463	10.08	1463	12.60
Qobc 1	28	0	0	558	0.12	1004	1.31	1338	4.13	1673	10.08	1673	12.60
Qobc 1	33	0	0	635	0.12	1144	1.31	1525	4.13	1906	10.08	1906	12.60
Qobc 1	38	0	0	713	0.12	1283	1.31	1710	4.13	2138	10.08	2138	12.60
Qobc 1	79.5	0	0	713	0.12	1283	1.31	1710	4.13	2138	10.08	2138	12.60
Qal 1 (Clay)	80.5	0	0	1031	0.09	1856	0.93	2475	2.95	3094	7.20	3094	9.00
Qal 1 (Clay)	92.5	0	0	1031	0.09	1856	0.93	2475	2.95	3094	7.20	3094	9.00
Qobc 2	93.5	0	0	1125	0.09	2025	0.93	2700	2.95	3375	7.20	3375	9.00
Qobc 2	97.5	0	0	1125	0.09	2025	0.93	2700	2.95	3375	7.20	3375	9.00
Qal 3 (Clay)	98.5	0	0	1125	0.09	2025	0.93	2700	2.95	3375	7.20	3375	9.00
Qal 3 (Clay)	105.5	0	0	1125	0.09	2025	0.93	2700	2.95	3375	7.20	3375	9.00
Qobc 3	106.5	0	0	750	0.09	1350	0.93	1800	2.95	2250	7.20	2250	9.00
Qobc 3	126.5	0	0	750	0.09	1350	0.93	1800	2.95	2250	7.20	2250	9.00
Qal 4 (Clay)	127.5	0	0	1125	0.09	2025	0.93	2700	2.95	3375	7.20	3375	9.00
Qal 4 (Clay)	139.5	0	0	1125	0.09	2025	0.93	2700	2.95	3375	7.20	3375	9.00

24-inch Pier Steel Pipe Pile p-y Springs

Section	Pier Pipe Pile Within Rock Dike						Best Estimate						
Material	Depth Below Ground Surface (feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Rock Dike Above GWT	0.2	0	0	25	0.04	32	0.11	43	0.33	60	0.9	60	1.26
Rock Dike Above GWT	3.7	0	0	638	0.06	813	0.12	1138	0.34	1684	0.9	1684	1.26
Rock Dike Below GWT	4.7	0	0	908	0.11	1037	0.16	1340	0.35	2000	0.9	2000	1.26
Rock Dike Below GWT	17.7	0	0	1788	0.06	2792	0.12	5242	0.34	10226	0.9	10226	1.26
Qybm	18.7	0	0	270	0.18	486	1.04	648	2.46	810	4.8	810	5.1
Qybm	22.7	0	0	270	0.18	486	1.04	648	2.46	810	4.8	810	5.1
Qal 1 (Clay)	23.7	0	0	850	0.08	1530	0.87	2040	2.75	2550	6.72	2550	8.4
Qal 1 (Clay)	27.2	0	0	850	0.08	1530	0.87	2040	2.75	2550	6.72	2550	8.4
Qal 1 (Sand)	28.2	0	0	2744	0.11	3498	0.16	5576	0.35	10702	0.9	10702	1.26
Qal 1 (Sand)	31.2	0	0	3874	0.14	4641	0.19	6853	0.35	13029	0.9	13029	1.26
Qal 1 (Clay)	32.2	0	0	850	0.08	1530	0.87	2040	2.75	2550	6.72	2550	8.4
Qal 1 (Clay)	37.2	0	0	850	0.08	1530	0.87	2040	2.75	2550	6.72	2550	8.4
Qobc 1	38.2	0	0	950	0.08	1710	0.87	2280	2.75	2850	6.72	2850	8.4
Qobc 1	82.7	0	0	950	0.08	1710	0.87	2280	2.75	2850	6.72	2850	8.4
Qal 2 (Clay)	95.2	0	0	1375	0.06	2475	0.62	3300	1.97	4125	4.8	4125	6
Qal 2 (Clay)	107.2	0	0	1375	0.06	2475	0.62	3300	1.97	4125	4.8	4125	6
Qobc 2	108.2	0	0	1500	0.06	2700	0.62	3600	1.97	4500	4.8	4500	6
Qobc 2	112.2	0	0	1500	0.06	2700	0.62	3600	1.97	4500	4.8	4500	6
Qal 3 (Clay)	113.2	0	0	1500	0.06	2700	0.62	3600	1.97	4500	4.8	4500	6
Qal 3 (Clay)	120.2	0	0	1500	0.06	2700	0.62	3600	1.97	4500	4.8	4500	6
Qobc 3	121.2	0	0	1000	0.06	1800	0.62	2400	1.97	3000	4.8	3000	6
Qobc 3	141.2	0	0	1000	0.06	1800	0.62	2400	1.97	3000	4.8	3000	6
Qal 4 (Clay)	142.2	0	0	1500	0.06	2700	0.62	3600	1.97	4500	4.8	4500	6
Qal 4 (Clay)	154.2	0	0	1500	0.06	2700	0.62	3600	1.97	4500	4.8	4500	6

Section	Pier Pipe Pile Within Rock Dike						Upper Bound						
Material	Depth Below Rock Dike(feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Rock Dike Above GWT	0.2	0	0	49	0.04	64	0.11	86	0.33	120	0.9	120	1.26
Rock Dike Above GWT	3.7	0	0	1276	0.06	1625	0.12	2276	0.34	3369	0.9	3369	1.26
Rock Dike Below GWT	4.7	0	0	1816	0.11	2073	0.16	2679	0.35	4000	0.9	4000	1.26
Rock Dike Below GWT	17.7	0	0	3576	0.06	5584	0.12	10484	0.34	20452	0.9	20452	1.26
Qybm	18.7	0	0	540	0.18	972	1.04	1296	2.46	1620	4.8	1620	5.1
Qybm	22.7	0	0	540	0.18	972	1.04	1296	2.46	1620	4.8	1620	5.1
Qal 1 (Clay)	23.7	0	0	1700	0.08	3060	0.87	4080	2.75	5100	6.72	5100	8.4
Qal 1 (Clay)	27.2	0	0	1700	0.08	3060	0.87	4080	2.75	5100	6.72	5100	8.4
Qal 1 (Sand)	28.2	0	0	5487	0.11	6996	0.16	11152	0.35	21403	0.9	21403	1.26
Qal 1 (Sand)	31.2	0	0	7749	0.14	9281	0.19	13707	0.35	26058	0.9	26058	1.26
Qal 1 (Clay)	32.2	0	0	1700	0.08	3060	0.87	4080	2.75	5100	6.72	5100	8.4
Qal 1 (Clay)	37.2	0	0	1700	0.08	3060	0.87	4080	2.75	5100	6.72	5100	8.4
Qobc 1	38.2	0	0	1900	0.08	3420	0.87	4560	2.75	5700	6.72	5700	8.4
Qobc 1	82.7	0	0	1900	0.08	3420	0.87	4560	2.75	5700	6.72	5700	8.4
Qal 2 (Clay)	95.2	0	0	2750	0.06	4950	0.62	6600	1.97	8250	4.8	8250	6
Qal 2 (Clay)	107.2	0	0	2750	0.06	4950	0.62	6600	1.97	8250	4.8	8250	6
Qobc 2	108.2	0	0	3000	0.06	5400	0.62	7200	1.97	9000	4.8	9000	6
Qobc 2	112.2	0	0	3000	0.06	5400	0.62	7200	1.97	9000	4.8	9000	6
Qal 3 (Clay)	113.2	0	0	3000	0.06	5400	0.62	7200	1.97	9000	4.8	9000	6
Qal 3 (Clay)	120.2	0	0	3000	0.06	5400	0.62	7200	1.97	9000	4.8	9000	6
Qobc 3	121.2	0	0	2000	0.06	3600	0.62	4800	1.97	6000	4.8	6000	6
Qobc 3	141.2	0	0	2000	0.06	3600	0.62	4800	1.97	6000	4.8	6000	6
Qal 4 (Clay)	142.2	0	0	3000	0.06	5400	0.62	7200	1.97	9000	4.8	9000	6
Qal 4 (Clay)	154.2	0	0	3000	0.06	5400	0.62	7200	1.97	9000	4.8	9000	6

Section	Pier Pipe Pile Within Rock Dike						Lower Bound						
Material	Depth Below Rock Dike(feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Rock Dike Above GWT	0.2	0	0	12	0.04	16	0.11	22	0.33	30	0.9	30	1.26
Rock Dike Above GWT	3.7	0	0	319	0.06	406	0.12	569	0.34	842	0.9	842	1.26
Rock Dike Below GWT	4.7	0	0	454	0.11	518	0.16	670	0.35	1000	0.9	1000	1.26
Rock Dike Below GWT	17.7	0	0	894	0.06	1396	0.12	2621	0.34	5113	0.9	5113	1.26
Qybm	18.7	0	0	135	0.18	243	1.04	324	2.46	405	4.8	405	5.1
Qybm	22.7	0	0	135	0.18	243	1.04	324	2.46	405	4.8	405	5.1
Qal 1 (Clay)	23.7	0	0	425	0.08	765	0.87	1020	2.75	1275	6.72	1275	8.4
Qal 1 (Clay)	27.2	0	0	425	0.08	765	0.87	1020	2.75	1275	6.72	1275	8.4
Qal 1 (Sand)	28.2	0	0	1372	0.11	1749	0.16	2788	0.35	5351	0.9	5351	1.26
Qal 1 (Sand)	31.2	0	0	1937	0.14	2320	0.19	3427	0.35	6514	0.9	6514	1.26
Qal 1 (Clay)	32.2	0	0	425	0.08	765	0.87	1020	2.75	1275	6.72	1275	8.4
Qal 1 (Clay)	37.2	0	0	425	0.08	765	0.87	1020	2.75	1275	6.72	1275	8.4
Qobc 1	38.2	0	0	475	0.08	855	0.87	1140	2.75	1425	6.72	1425	8.4
Qobc 1	82.7	0	0	475	0.08	855	0.87	1140	2.75	1425	6.72	1425	8.4
Qal 2 (Clay)	95.2	0	0	688	0.06	1238	0.62	1650	1.97	2063	4.8	2063	6
Qal 2 (Clay)	107.2	0	0	688	0.06	1238	0.62	1650	1.97	2063	4.8	2063	6
Qobc 2	108.2	0	0	750	0.06	1350	0.62	1800	1.97	2250	4.8	2250	6
Qobc 2	112.2	0	0	750	0.06	1350	0.62	1800	1.97	2250	4.8	2250	6
Qal 3 (Clay)	113.2	0	0	750	0.06	1350	0.62	1800	1.97	2250	4.8	2250	6
Qal 3 (Clay)	120.2	0	0	750	0.06	1350	0.62	1800	1.97	2250	4.8	2250	6
Qobc 3	121.2	0	0	500	0.06	900	0.62	1200	1.97	1500	4.8	1500	6
Qobc 3	141.2	0	0	500	0.06	900	0.62	1200	1.97	1500	4.8	1500	6
Qal 4 (Clay)	142.2	0	0	750	0.06	1350	0.62	1800	1.97	2250	4.8	2250	6
Qal 4 (Clay)	154.2	0	0	750	0.06	1350	0.62	1800	1.97	2250	4.8	2250	6

24-inch Abutment Steel Pipe Pile p-y Springs

Section	Abutment Landside Pipe Pile						Best Estimate						
Material	Depth Below Bottom of Abutment (feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Engineered Fill	0.5	0	0	18	0.03	24	0.10	33	0.33	46	0.9	46	1.26
Rock Dike Above GWT	1.5	0	0	101	0.02	152	0.09	221	0.33	317	0.9	317	1.26
Rock Dike Above GWT	6	0	0	576	0.03	899	0.10	1442	0.33	2297	0.9	2297	1.26
Rock Dike Below GWT	7	0	0	736	0.06	1001	0.12	1560	0.34	2589	0.9	2589	1.26
Rock Dike Below GWT	20	0	0	2523	0.07	3643	0.13	6493	0.34	12607	0.9	12607	1.26
Qybm	21	0	0	270	0.18	486	1.04	648	2.46	810	4.8	810	5.1
Qybm	25	0	0	270	0.18	486	1.04	648	2.46	810	4.8	810	5.1
Qal 1 (Clay)	26	0	0	850	0.08	1530	0.87	2040	2.75	2550	6.72	2550	8.4
Qal 1 (Clay)	29.5	0	0	850	0.08	1530	0.87	2040	2.75	2550	6.72	2550	8.4
Qal 1 (Sand)	30.5	0	0	3741	0.14	4494	0.18	6664	0.35	12675	0.9	12675	1.26
Qal 1 (Sand)	33.5	0	0	5143	0.17	5876	0.21	8078	0.36	15203	0.9	15203	1.26
Qal 1 (Clay)	34.5	0	0	850	0.08	1530	0.87	2040	2.75	2550	6.72	2550	8.4
Qal 1 (Clay)	39.5	0	0	850	0.08	1530	0.87	2040	2.75	2550	6.72	2550	8.4
Qobc 1	40.5	0	0	950	0.08	1710	0.87	2280	2.75	2850	6.72	2850	8.4
Qobc 1	96.5	0	0	950	0.08	1710	0.87	2280	2.75	2850	6.72	2850	8.4
Qal 2 (Clay)	97.5	0	0	1375	0.06	2475	0.62	3300	1.97	4125	4.8	4125	6
Qal 2 (Clay)	109.5	0	0	1375	0.06	2475	0.62	3300	1.97	4125	4.8	4125	6
Qobc 2	110.5	0	0	1500	0.06	2700	0.62	3600	1.97	4500	4.8	4500	6
Qobc 2	114.5	0	0	1500	0.06	2700	0.62	3600	1.97	4500	4.8	4500	6
Qal 3 (Clay)	115.5	0	0	1500	0.06	2700	0.62	3600	1.97	4500	4.8	4500	6
Qal 3 (Clay)	122.5	0	0	1500	0.06	2700	0.62	3600	1.97	4500	4.8	4500	6
Qobc 3	123.5	0	0	1000	0.06	1800	0.62	2400	1.97	3000	4.8	3000	6
Qobc 3	143.5	0	0	1000	0.06	1800	0.62	2400	1.97	3000	4.8	3000	6
Qal 4 (Clay)	144.5	0	0	1500	0.06	2700	0.62	3600	1.97	4500	4.8	4500	6
Qal 4 (Clay)	156.5	0	0	1500	0.06	2700	0.62	3600	1.97	4500	4.8	4500	6

Section	Abutment Landside Pipe Pile						Upper Bound						
Material	Elevation, City of Berkeley Datum (feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Engineered Fill	0.5	0	0	36	0.03	47	0.10	66	0.33	92	0.9	92	1.26
Rock Dike Above GWT	1.5	0	0	201	0.02	304	0.09	441	0.33	634	0.9	634	1.26
Rock Dike Above GWT	6	0	0	1151	0.03	1798	0.10	2885	0.33	4594	0.9	4594	1.26
Rock Dike Below GWT	7	0	0	1471	0.06	2002	0.12	3120	0.34	5178	0.9	5178	1.26
Rock Dike Below GWT	20	0	0	5047	0.07	7286	0.13	12986	0.34	25214	0.9	25214	1.26
Qybm	21	0	0	540	0.18	972	1.04	1296	2.46	1620	4.8	1620	5.1
Qybm	25	0	0	540	0.18	972	1.04	1296	2.46	1620	4.8	1620	5.1
Qal 1 (Clay)	26	0	0	1700	0.08	3060	0.87	4080	2.75	5100	6.72	5100	8.4
Qal 1 (Clay)	29.5	0	0	1700	0.08	3060	0.87	4080	2.75	5100	6.72	5100	8.4
Qal 1 (Sand)	30.5	0	0	7482	0.14	8989	0.18	13328	0.35	25351	0.9	25351	1.26
Qal 1 (Sand)	33.5	0	0	10287	0.17	11751	0.21	16157	0.36	30405	0.9	30405	1.26
Qal 1 (Clay)	34.5	0	0	1700	0.08	3060	0.87	4080	2.75	5100	6.72	5100	8.4
Qal 1 (Clay)	39.5	0	0	1700	0.08	3060	0.87	4080	2.75	5100	6.72	5100	8.4
Qobc 1	40.5	0	0	1900	0.08	3420	0.87	4560	2.75	5700	6.72	5700	8.4
Qobc 1	96.5	0	0	1900	0.08	3420	0.87	4560	2.75	5700	6.72	5700	8.4
Qal 2 (Clay)	97.5	0	0	2750	0.06	4950	0.62	6600	1.97	8250	4.8	8250	6
Qal 2 (Clay)	109.5	0	0	2750	0.06	4950	0.62	6600	1.97	8250	4.8	8250	6
Qobc 2	110.5	0	0	3000	0.06	5400	0.62	7200	1.97	9000	4.8	9000	6
Qobc 2	114.5	0	0	3000	0.06	5400	0.62	7200	1.97	9000	4.8	9000	6
Qal 3 (Clay)	115.5	0	0	3000	0.06	5400	0.62	7200	1.97	9000	4.8	9000	6
Qal 3 (Clay)	122.5	0	0	3000	0.06	5400	0.62	7200	1.97	9000	4.8	9000	6
Qobc 3	123.5	0	0	2000	0.06	3600	0.62	4800	1.97	6000	4.8	6000	6
Qobc 3	143.5	0	0	2000	0.06	3600	0.62	4800	1.97	6000	4.8	6000	6
Qal 4 (Clay)	144.5	0	0	3000	0.06	5400	0.62	7200	1.97	9000	4.8	9000	6
Qal 4 (Clay)	156.5	0	0	3000	0.06	5400	0.62	7200	1.97	9000	4.8	9000	6

Section	Abutment Landside Pipe Pile						Lower Bound						
Material	Elevation, City of Berkeley Datum (feet)	p0 (psi)	y0 (in)	p1 (psi)	y1 (in)	p2 (psi)	y2 (in)	p3 (psi)	y3 (in)	p4 (psi)	y4 (in)	p5 (psi)	y5 (in)
Engineered Fill	0.5	0	0	9	0.03	12	0.10	16	0.33	23	0.9	23	1.26
Rock Dike Above GWT	1.5	0	0	50	0.02	76	0.09	110	0.33	158	0.9	158	1.26
Rock Dike Above GWT	6	0	0	288	0.03	449	0.10	721	0.33	1149	0.9	1149	1.26
Rock Dike Below GWT	7	0	0	368	0.06	500	0.12	780	0.34	1295	0.9	1295	1.26
Rock Dike Below GWT	20	0	0	1262	0.07	1822	0.13	3247	0.34	6304	0.9	6304	1.26
Qybm	21	0	0	135	0.18	243	1.04	324	2.46	405	4.8	405	5.1
Qybm	25	0	0	135	0.18	243	1.04	324	2.46	405	4.8	405	5.1
Qal 1 (Clay)	26	0	0	425	0.08	765	0.87	1020	2.75	1275	6.72	1275	8.4
Qal 1 (Clay)	29.5	0	0	425	0.08	765	0.87	1020	2.75	1275	6.72	1275	8.4
Qal 1 (Sand)	30.5	0	0	1871	0.14	2247	0.18	3332	0.35	6338	0.9	6338	1.26
Qal 1 (Sand)	33.5	0	0	2572	0.17	2938	0.21	4039	0.36	7601	0.9	7601	1.26
Qal 1 (Clay)	34.5	0	0	425	0.08	765	0.87	1020	2.75	1275	6.72	1275	8.4
Qal 1 (Clay)	39.5	0	0	425	0.08	765	0.87	1020	2.75	1275	6.72	1275	8.4
Qobc 1	40.5	0	0	475	0.08	855	0.87	1140	2.75	1425	6.72	1425	8.4
Qobc 1	96.5	0	0	475	0.08	855	0.87	1140	2.75	1425	6.72	1425	8.4
Qal 2 (Clay)	97.5	0	0	688	0.06	1238	0.62	1650	1.97	2063	4.8	2063	6
Qal 2 (Clay)	109.5	0	0	688	0.06	1238	0.62	1650	1.97	2063	4.8	2063	6
Qobc 2	110.5	0	0	750	0.06	1350	0.62	1800	1.97	2250	4.8	2250	6
Qobc 2	114.5	0	0	750	0.06	1350	0.62	1800	1.97	2250	4.8	2250	6
Qal 3 (Clay)	115.5	0	0	750	0.06	1350	0.62	1800	1.97	2250	4.8	2250	6
Qal 3 (Clay)	122.5	0	0	750	0.06	1350	0.62	1800	1.97	2250	4.8	2250	6
Qobc 3	123.5	0	0	500	0.06	900	0.62	1200	1.97	1500	4.8	1500	6
Qobc 3	143.5	0	0	500	0.06	900	0.62	1200	1.97	1500	4.8	1500	6
Qal 4 (Clay)	144.5	0	0	750	0.06	1350	0.62	1800	1.97	2250	4.8	2250	6
Qal 4 (Clay)	156.5	0	0	750	0.06	1350	0.62	1800	1.97	2250	4.8	2250	6



APPENDIX I

T-Z AND Q-W SPRINGS

4ft Concrete Breakwater t-z Springs

Section	Pier Alignment						Best Estimate						
Material	Depth Below Mudline (feet)	t0 (psi)	z0 (in)	t1 (psi)	z1 (in)	t2 (psi)	z2 (in)	t3 (psi)	z3 (in)	t4 (psi)	z4 (in)	t5 (psi)	z5 (in)
Qybm	0.5	0	0	0	0.17	0	0.24	0	0.31	0	0.61	0	6.11
Qybm	6.5	0	0	0	0.17	0	0.24	0	0.31	0	0.61	0	6.11
Qal 1 (Clay)	7.5	0	0	1	0.17	2	0.24	2	0.31	2	0.61	2	6.11
Qal 1 (Clay)	12.5	0	0	1	0.17	1	0.24	1	0.31	1	0.61	1	6.11
Qal 1 (Sand)	13.5	0	0	5	0.17	7	0.24	7	0.31	7	0.61	7	6.11
Qal 1 (Sand)	16.5	0	0	10	0.17	12	0.24	13	0.31	13	0.61	13	6.11
Qal 1 (Clay)	17.5	0	0	7	0.17	8	0.24	9	0.31	8	0.61	8	6.11
Qal 1 (Clay)	22.5	0	0	4	0.17	5	0.24	5	0.31	5	0.61	5	6.11
Qobc 1	23.5	0	0	4	0.17	5	0.24	6	0.31	5	0.61	5	6.11
Qobc 1	28	0	0	5	0.17	6	0.24	6	0.31	6	0.61	6	6.11
Qobc 1	33	0	0	5	0.17	6	0.24	6	0.31	6	0.61	6	6.11
Qobc 1	38	0	0	5	0.17	6	0.24	7	0.31	6	0.61	6	6.11
Qobc 1	43	0	0	5	0.17	7	0.24	7	0.31	7	0.61	7	6.11
Qobc 1	48	0	0	6	0.17	7	0.24	8	0.31	7	0.61	7	6.11
Qobc 1	53	0	0	6	0.17	7	0.24	8	0.31	7	0.61	7	6.11
Qobc 1	58	0	0	6	0.17	8	0.24	9	0.31	8	0.61	8	6.11
Qobc 1	63	0	0	7	0.17	8	0.24	9	0.31	8	0.61	8	6.11
Qobc 1	68	0	0	7	0.17	8	0.24	9	0.31	8	0.61	8	6.11
Qobc 1	73	0	0	7	0.17	9	0.24	10	0.31	9	0.61	9	6.11
Qobc 1	79.5	0	0	8	0.17	9	0.24	10	0.31	9	0.61	9	6.11
Qal 2 (Clay)	80.5	0	0	8	0.17	10	0.24	11	0.31	10	0.61	10	6.11
Qal 2 (Clay)	92.5	0	0	10	0.17	12	0.24	13	0.31	12	0.61	12	6.11
Qobc 2	93.5	0	0	10	0.17	12	0.24	14	0.31	12	0.61	12	6.11
Qobc 2	97.5	0	0	11	0.17	13	0.24	14	0.31	13	0.61	13	6.11
Qal 3 (Clay)	98.5	0	0	11	0.17	13	0.24	14	0.31	13	0.61	13	6.11
Qal 3 (Clay)	105.5	0	0	11	0.17	13	0.24	15	0.31	13	0.61	13	6.11
Qobc 3	106.5	0	0	10	0.17	12	0.24	13	0.31	12	0.61	12	6.11
Qobc 3	126.5	0	0	10	0.17	12	0.24	13	0.31	12	0.61	12	6.11
Qal 4 (Clay)	127.5	0	0	12	0.17	14	0.24	16	0.31	14	0.61	14	6.11
Qal 4 (Clay)	139.5	0	0	15	0.17	18	0.24	20	0.31	18	0.61	18	6.11

Section	Pier Alignment						Upper Bound						
Material	Depth Below Mudline (feet)	t0 (psi)	z0 (in)	t1 (psi)	z1 (in)	t2 (psi)	z2 (in)	t3 (psi)	z3 (in)	t4 (psi)	z4 (in)	t5 (psi)	z5 (in)
Qybm	0.5	0	0	0	0.17	0	0.24	0	0.31	0	0.61	0	6.11
Qybm	6.5	0	0	0	0.17	0	0.24	0	0.31	0	0.61	0	6.11
Qal 1 (Clay)	7.5	0	0	3	0.17	3	0.24	4	0.31	3	0.61	3	6.11
Qal 1 (Clay)	12.5	0	0	2	0.17	3	0.24	3	0.31	3	0.61	3	6.11
Qal 1 (Sand)	13.5	0	0	11	0.17	13	0.24	15	0.31	15	0.61	15	6.11
Qal 1 (Sand)	16.5	0	0	20	0.17	24	0.24	27	0.31	27	0.61	27	6.11
Qal 1 (Clay)	17.5	0	0	14	0.17	17	0.24	18	0.31	17	0.61	17	6.11
Qal 1 (Clay)	22.5	0	0	8	0.17	10	0.24	11	0.31	10	0.61	10	6.11
Qobc 1	23.5	0	0	8	0.17	10	0.24	11	0.31	10	0.61	10	6.11
Qobc 1	28	0	0	9	0.17	11	0.24	12	0.31	11	0.61	11	6.11
Qobc 1	33	0	0	10	0.17	12	0.24	13	0.31	12	0.61	12	6.11
Qobc 1	38	0	0	10	0.17	12	0.24	14	0.31	12	0.61	12	6.11
Qobc 1	43	0	0	11	0.17	13	0.24	14	0.31	13	0.61	13	6.11
Qobc 1	48	0	0	12	0.17	14	0.24	15	0.31	14	0.61	14	6.11
Qobc 1	53	0	0	12	0.17	15	0.24	16	0.31	15	0.61	15	6.11
Qobc 1	58	0	0	13	0.17	15	0.24	17	0.31	15	0.61	15	6.11
Qobc 1	63	0	0	13	0.17	16	0.24	18	0.31	16	0.61	16	6.11
Qobc 1	68	0	0	14	0.17	17	0.24	19	0.31	17	0.61	17	6.11
Qobc 1	73	0	0	14	0.17	17	0.24	19	0.31	17	0.61	17	6.11
Qobc 1	79.5	0	0	15	0.17	18	0.24	20	0.31	18	0.61	18	6.11
Qal 2 (Clay)	80.5	0	0	17	0.17	20	0.24	22	0.31	20	0.61	20	6.11
Qal 2 (Clay)	92.5	0	0	20	0.17	24	0.24	26	0.31	24	0.61	24	6.11
Qobc 2	93.5	0	0	20	0.17	24	0.24	27	0.31	24	0.61	24	6.11
Qobc 2	97.5	0	0	21	0.17	25	0.24	28	0.31	25	0.61	25	6.11
Qal 2 (Clay)	98.5	0	0	21	0.17	26	0.24	28	0.31	26	0.61	26	6.11
Qal 2 (Clay)	105.5	0	0	22	0.17	27	0.24	30	0.31	27	0.61	27	6.11
Qobc 3	106.5	0	0	20	0.17	24	0.24	27	0.31	24	0.61	24	6.11
Qobc 3	126.5	0	0	20	0.17	24	0.24	26	0.31	24	0.61	24	6.11
Qal 4 (Clay)	127.5	0	0	24	0.17	29	0.24	32	0.31	29	0.61	29	6.11
Qal 4 (Clay)	139.5	0	0	30	0.17	35	0.24	39	0.31	35	0.61	35	6.11

Section	Pier Alignment						Lower Bound						
Material	Depth Below Mudline (feet)	t0 (psi)	z0 (in)	t1 (psi)	z1 (in)	t2 (psi)	z2 (in)	t3 (psi)	z3 (in)	t4 (psi)	z4 (in)	t5 (psi)	z5 (in)
Qybm	0.5	0	0	0	0.17	0	0.24	0	0.31	0	0.61	0	6.11
Qybm	6.5	0	0	0	0.17	0	0.24	0	0.31	0	0.61	0	6.11
Qal 1 (Clay)	7.5	0	0	1	0.17	1	0.24	1	0.31	1	0.61	1	6.11
Qal 1 (Clay)	12.5	0	0	1	0.17	1	0.24	1	0.31	1	0.61	1	6.11
Qal 1 (Sand)	13.5	0	0	3	0.17	3	0.24	4	0.31	4	0.61	4	6.11
Qal 1 (Sand)	16.5	0	0	5	0.17	6	0.24	7	0.31	7	0.61	7	6.11
Qal 1 (Clay)	17.5	0	0	3	0.17	4	0.24	5	0.31	4	0.61	4	6.11
Qal 1 (Clay)	22.5	0	0	2	0.17	2	0.24	3	0.31	2	0.61	2	6.11
Qobc 1	23.5	0	0	2	0.17	3	0.24	3	0.31	3	0.61	3	6.11
Qobc 1	28	0	0	2	0.17	3	0.24	3	0.31	3	0.61	3	6.11
Qobc 1	33	0	0	2	0.17	3	0.24	3	0.31	3	0.61	3	6.11
Qobc 1	38	0	0	3	0.17	3	0.24	3	0.31	3	0.61	3	6.11
Qobc 1	43	0	0	3	0.17	3	0.24	4	0.31	3	0.61	3	6.11
Qobc 1	48	0	0	3	0.17	3	0.24	4	0.31	3	0.61	3	6.11
Qobc 1	53	0	0	3	0.17	4	0.24	4	0.31	4	0.61	4	6.11
Qobc 1	58	0	0	3	0.17	4	0.24	4	0.31	4	0.61	4	6.11
Qobc 1	63	0	0	3	0.17	4	0.24	4	0.31	4	0.61	4	6.11
Qobc 1	68	0	0	3	0.17	4	0.24	5	0.31	4	0.61	4	6.11
Qobc 1	73	0	0	4	0.17	4	0.24	5	0.31	4	0.61	4	6.11
Qobc 1	79.5	0	0	4	0.17	5	0.24	5	0.31	5	0.61	5	6.11
Qal 2 (Clay)	80.5	0	0	4	0.17	5	0.24	6	0.31	5	0.61	5	6.11
Qal 2 (Clay)	92.5	0	0	5	0.17	6	0.24	7	0.31	6	0.61	6	6.11
Qobc 2	93.5	0	0	5	0.17	6	0.24	7	0.31	6	0.61	6	6.11
Qobc 2	97.5	0	0	5	0.17	6	0.24	7	0.31	6	0.61	6	6.11
Qal 2 (Clay)	98.5	0	0	5	0.17	6	0.24	7	0.31	6	0.61	6	6.11
Qal 2 (Clay)	105.5	0	0	6	0.17	7	0.24	7	0.31	7	0.61	7	6.11
Qobc 3	106.5	0	0	5	0.17	6	0.24	7	0.31	6	0.61	6	6.11
Qobc 3	126.5	0	0	5	0.17	6	0.24	7	0.31	6	0.61	6	6.11
Qal 4 (Clay)	127.5	0	0	6	0.17	7	0.24	8	0.31	7	0.61	7	6.11
Qal 4 (Clay)	139.5	0	0	7	0.17	9	0.24	10	0.31	9	0.61	9	6.11

24-inch Concrete Precast Prestressed Octagonal Pile t-z Springs

Section	Pier Alignment						Best Estimate						
Material	Depth Below Mudline (feet)	t0 (psi)	z0 (in)	t1 (psi)	z1 (in)	t2 (psi)	z2 (in)	t3 (psi)	z3 (in)	t4 (psi)	z4 (in)	t5 (psi)	z5 (in)
Qybm	0.5	0	0.00	0	0.14	0	0.20	0	0.25	0	0.51	0	5.08
Qybm	6.5	0	0.00	0	0.14	0	0.20	0	0.25	0	0.51	0	5.08
Qal 1 (Clay)	7.5	0	0.00	0	0.14	0	0.20	0	0.25	0	0.51	0	5.08
Qal 1 (Clay)	12.5	0	0.00	1	0.14	2	0.20	2	0.25	2	0.51	2	5.08
Qal 1 (Sand)	13.5	0	0.00	1	0.14	1	0.20	1	0.25	1	0.51	1	5.08
Qal 1 (Sand)	16.5	0	0.00	10	0.14	12	0.20	13	0.25	13	0.51	13	5.08
Qal 1 (Clay)	17.5	0	0.00	10	0.14	12	0.20	13	0.25	12	0.51	12	5.08
Qal 1 (Clay)	22.5	0	0.00	4	0.14	5	0.20	5	0.25	5	0.51	5	5.08
Qobc 1	23.5	0	0.00	4	0.14	5	0.20	5	0.25	5	0.51	5	5.08
Qobc 1	28	0	0.00	5	0.14	5	0.20	6	0.25	5	0.51	5	5.08
Qobc 1	33	0	0.00	5	0.14	6	0.20	6	0.25	6	0.51	6	5.08
Qobc 1	38	0	0.00	5	0.14	6	0.20	7	0.25	6	0.51	6	5.08
Qobc 1	43	0	0.00	5	0.14	6	0.20	7	0.25	6	0.51	6	5.08
Qobc 1	48	0	0.00	6	0.14	7	0.20	7	0.25	7	0.51	7	5.08
Qobc 1	53	0	0.00	6	0.14	7	0.20	8	0.25	7	0.51	7	5.08
Qobc 1	58	0	0.00	6	0.14	7	0.20	8	0.25	7	0.51	7	5.08
Qobc 1	63	0	0.00	6	0.14	8	0.20	9	0.25	8	0.51	8	5.08
Qobc 1	68	0	0.00	7	0.14	8	0.20	9	0.25	8	0.51	8	5.08
Qobc 1	73	0	0.00	7	0.14	8	0.20	9	0.25	8	0.51	8	5.08
Qobc 1	79.5	0	0.00	7	0.14	9	0.20	10	0.25	9	0.51	9	5.08
Qal 2 (Clay)	80.5	0	0.00	7	0.14	9	0.20	10	0.25	9	0.51	9	5.08
Qal 2 (Clay)	92.5	0	0.00	10	0.14	11	0.20	13	0.25	11	0.51	11	5.08
Qobc 2	93.5	0	0.00	10	0.14	12	0.20	13	0.25	12	0.51	12	5.08
Qobc 2	97.5	0	0.00	10	0.14	12	0.20	13	0.25	12	0.51	12	5.08
Qal 3 (Clay)	98.5	0	0.00	10	0.14	12	0.20	13	0.25	12	0.51	12	5.08
Qal 3 (Clay)	105.5	0	0.00	11	0.14	13	0.20	14	0.25	13	0.51	13	5.08
Qobc 3	106.5	0	0.00	11	0.14	13	0.20	14	0.25	13	0.51	13	5.08
Qobc 3	126.5	0	0.00	10	0.14	12	0.20	13	0.25	12	0.51	12	5.08
Qal 4 (Clay)	127.5	0	0.00	10	0.14	12	0.20	13	0.25	12	0.51	12	5.08
Qal 4 (Clay)	137.5	0	0.00	14	0.14	17	0.20	19	0.25	17	0.51	17	5.08
Qal 4 (Clay)	145.5	0	0.00	14	0.14	17	0.20	19	0.25	17	0.51	17	5.08

Section	Pier Alignment						Upper Bound						
Material	Depth Below Mudline (feet)	t0 (psi)	z0 (in)	t1 (psi)	z1 (in)	t2 (psi)	z2 (in)	t3 (psi)	z3 (in)	t4 (psi)	z4 (in)	t5 (psi)	z5 (in)
Qybm	0.5	0	0.00	0	0.14	0	0.20	0	0.25	0	0.51	0	5.08
Qybm	6.5	0	0.00	0	0.14	0	0.20	0	0.25	0	0.51	0	5.08
Qal 1 (Clay)	7.5	0	0.00	0	0.14	0	0.20	0	0.25	0	0.51	0	5.08
Qal 1 (Clay)	12.5	0	0.00	3	0.14	3	0.20	3	0.25	3	0.51	3	5.08
Qal 1 (Sand)	13.5	0	0.00	2	0.14	2	0.20	2	0.25	2	0.51	2	5.08
Qal 1 (Sand)	16.5	0	0.00	20	0.14	24	0.20	27	0.25	27	0.51	27	5.08
Qal 1 (Clay)	17.5	0	0.00	20	0.14	24	0.20	27	0.25	24	0.51	24	5.08
Qal 1 (Clay)	22.5	0	0.00	8	0.14	9	0.20	10	0.25	9	0.51	9	5.08
Qobc 1	23.5	0	0.00	8	0.14	9	0.20	10	0.25	9	0.51	9	5.08
Qobc 1	28	0	0.00	9	0.14	11	0.20	12	0.25	11	0.51	11	5.08
Qobc 1	33	0	0.00	9	0.14	11	0.20	13	0.25	11	0.51	11	5.08
Qobc 1	38	0	0.00	10	0.14	12	0.20	13	0.25	12	0.51	12	5.08
Qobc 1	43	0	0.00	11	0.14	13	0.20	14	0.25	13	0.51	13	5.08
Qobc 1	48	0	0.00	11	0.14	13	0.20	15	0.25	13	0.51	13	5.08
Qobc 1	53	0	0.00	12	0.14	14	0.20	16	0.25	14	0.51	14	5.08
Qobc 1	58	0	0.00	12	0.14	15	0.20	16	0.25	15	0.51	15	5.08
Qobc 1	63	0	0.00	13	0.14	16	0.20	17	0.25	16	0.51	16	5.08
Qobc 1	68	0	0.00	13	0.14	16	0.20	18	0.25	16	0.51	16	5.08
Qobc 1	73	0	0.00	14	0.14	17	0.20	19	0.25	17	0.51	17	5.08
Qobc 1	79.5	0	0.00	15	0.14	18	0.20	20	0.25	18	0.51	18	5.08
Qal 2 (Clay)	80.5	0	0.00	15	0.14	18	0.20	20	0.25	18	0.51	18	5.08
Qal 2 (Clay)	92.5	0	0.00	19	0.14	23	0.20	26	0.25	23	0.51	23	5.08
Qobc 2	93.5	0	0.00	19	0.14	23	0.20	26	0.25	23	0.51	23	5.08
Qobc 2	97.5	0	0.00	20	0.14	24	0.20	26	0.25	24	0.51	24	5.08
Qal 3 (Clay)	98.5	0	0.00	20	0.14	24	0.20	26	0.25	24	0.51	24	5.08
Qal 3 (Clay)	105.5	0	0.00	21	0.14	26	0.20	29	0.25	26	0.51	26	5.08
Qobc 3	106.5	0	0.00	22	0.14	26	0.20	29	0.25	26	0.51	26	5.08
Qobc 3	126.5	0	0.00	19	0.14	23	0.20	26	0.25	23	0.51	23	5.08
Qal 4 (Clay)	127.5	0	0.00	19	0.14	23	0.20	26	0.25	23	0.51	23	5.08
Qal 4 (Clay)	137.5	0	0.00	28	0.14	34	0.20	38	0.25	34	0.51	34	5.08
Qal 4 (Clay)	145.5	0	0.00	29	0.14	35	0.20	38	0.25	35	0.51	35	5.08

Section	Pier Alignment						Lower Bound						
Material	Depth Below Mudline (feet)	t0 (psi)	z0 (in)	t1 (psi)	z1 (in)	t2 (psi)	z2 (in)	t3 (psi)	z3 (in)	t4 (psi)	z4 (in)	t5 (psi)	z5 (in)
Qybm	0.5	0	0	0	0.14	0	0.20	0	0.25	0	0.51	0	5.08
Qybm	6.5	0	0	0	0.14	0	0.20	0	0.25	0	0.51	0	5.08
Qal 1 (Clay)	7.5	0	0	0	0.14	0	0.20	0	0.25	0	0.51	0	5.08
Qal 1 (Clay)	12.5	0	0	1	0.14	1	0.20	1	0.25	1	0.51	1	5.08
Qal 1 (Sand)	13.5	0	0	0	0.14	1	0.20	1	0.25	1	0.51	1	5.08
Qal 1 (Sand)	16.5	0	0	5	0.14	6	0.20	7	0.25	7	0.51	7	5.08
Qal 1 (Clay)	17.5	0	0	5	0.14	6	0.20	7	0.25	6	0.51	6	5.08
Qal 1 (Clay)	22.5	0	0	2	0.14	2	0.20	3	0.25	2	0.51	2	5.08
Qobc 1	23.5	0	0	2	0.14	2	0.20	3	0.25	2	0.51	2	5.08
Qobc 1	28	0	0	2	0.14	3	0.20	3	0.25	3	0.51	3	5.08
Qobc 1	33	0	0	2	0.14	3	0.20	3	0.25	3	0.51	3	5.08
Qobc 1	38	0	0	2	0.14	3	0.20	3	0.25	3	0.51	3	5.08
Qobc 1	43	0	0	3	0.14	3	0.20	4	0.25	3	0.51	3	5.08
Qobc 1	48	0	0	3	0.14	3	0.20	4	0.25	3	0.51	3	5.08
Qobc 1	53	0	0	3	0.14	4	0.20	4	0.25	4	0.51	4	5.08
Qobc 1	58	0	0	3	0.14	4	0.20	4	0.25	4	0.51	4	5.08
Qobc 1	63	0	0	3	0.14	4	0.20	4	0.25	4	0.51	4	5.08
Qobc 1	68	0	0	3	0.14	4	0.20	4	0.25	4	0.51	4	5.08
Qobc 1	73	0	0	3	0.14	4	0.20	5	0.25	4	0.51	4	5.08
Qobc 1	79.5	0	0	4	0.14	4	0.20	5	0.25	4	0.51	4	5.08
Qal 2 (Clay)	80.5	0	0	4	0.14	4	0.20	5	0.25	4	0.51	4	5.08
Qal 2 (Clay)	92.5	0	0	5	0.14	6	0.20	6	0.25	6	0.51	6	5.08
Qobc 2	93.5	0	0	5	0.14	6	0.20	6	0.25	6	0.51	6	5.08
Qobc 2	97.5	0	0	5	0.14	6	0.20	7	0.25	6	0.51	6	5.08
Qal 3 (Clay)	98.5	0	0	5	0.14	6	0.20	7	0.25	6	0.51	6	5.08
Qal 3 (Clay)	105.5	0	0	5	0.14	6	0.20	7	0.25	6	0.51	6	5.08
Qobc 3	106.5	0	0	5	0.14	6	0.20	7	0.25	6	0.51	6	5.08
Qobc 3	126.5	0	0	5	0.14	6	0.20	6	0.25	6	0.51	6	5.08
Qal 4 (Clay)	127.5	0	0	5	0.14	6	0.20	6	0.25	6	0.51	6	5.08
Qal 4 (Clay)	137.5	0	0	7	0.14	9	0.20	9	0.25	9	0.51	9	5.08
Qal 4 (Clay)	145.5	0	0	7	0.14	9	0.20	10	0.25	9	0.51	9	5.08

24-inch Concrete Precast Prestressed Octagonal Pile Q-w Springs

Section	Pier Alignment						Best Estimate						
Material	Depth Below Mudline (feet)	t0 (psi)	z0 (in)	Q1 (kips)	w1 (in)	Q2 (kips)	w2 (in)	Q3 (kips)	w3 (in)	Q4 (kips)	w4 (in)	Q5 (kips)	w5 (in)
Qobc 1	50	0	0	14	0.05	28	0.32	42	1.03	56	2.46	56	4.92
Qobc 1	75	0	0	14	0.05	28	0.32	42	1.03	56	2.46	56	4.92
Qal 2 (Clay)	86	0	0	20	0.05	41	0.32	61	1.03	82	2.46	82	4.92
Qal 3 (Clay)	102	0	0	22	0.05	45	0.32	67	1.03	89	2.46	89	4.92
Qobc 3	116	0	0	15	0.05	30	0.32	45	1.03	59	2.46	59	4.92
Qal 4 (Clay)	135	0	0	30	0.05	59	0.32	89	1.03	119	2.46	119	4.92

Section	Pier Alignment						Upper Bound						
Material	Depth Below Mudline (feet)	t0 (psi)	z0 (in)	Q1 (kips)	w1 (in)	Q2 (kips)	w2 (in)	Q3 (kips)	w3 (in)	Q4 (kips)	w4 (in)	Q5 (kips)	w5 (in)
Qobc 1	50	0	0	28	0.05	56	0.32	85	1.03	113	2.46	113	4.92
Qobc 1	75	0	0	28	0.05	56	0.32	85	1.03	113	2.46	113	4.92
Qal 2 (Clay)	86	0	0	41	0.05	82	0.32	123	1.03	163	2.46	163	4.92
Qal 3 (Clay)	102	0	0	45	0.05	89	0.32	134	1.03	178	2.46	178	4.92
Qobc 3	116	0	0	30	0.05	59	0.32	89	1.03	119	2.46	119	4.92
Qal 4 (Clay)	135	0	0	59	0.05	119	0.32	178	1.03	238	2.46	238	4.92

Section	Pier Alignment						Lower Bound						
Material	Depth Below Mudline (feet)	t0 (psi)	z0 (in)	Q1 (kips)	w1 (in)	Q2 (kips)	w2 (in)	Q3 (kips)	w3 (in)	Q4 (kips)	w4 (in)	Q5 (kips)	w5 (in)
Qobc 1	50	0	0	7	0.05	14	0.32	21	1.03	28	2.46	28	4.92
Qobc 1	75	0	0	7	0.05	14	0.32	21	1.03	28	2.46	28	4.92
Qal 2 (Clay)	86	0	0	10	0.05	20	0.32	31	1.03	41	2.46	41	4.92
Qal 3 (Clay)	102	0	0	11	0.05	22	0.32	33	1.03	45	2.46	45	4.92
Qobc 3	116	0	0	7	0.05	15	0.32	22	1.03	30	2.46	30	4.92
Qal 4 (Clay)	135	0	0	15	0.05	30	0.32	45	1.03	59	2.46	59	4.92



APPENDIX J

PIER PLAZA SETTLEMENT MITIGATION

Project No.
25022.000.001

June 2, 2025

Mr. James Connolly
COWI
555 12th Street, Suite 1700
Oakland, CA 94601

Subject: Berkeley Water Transportation Pier Ferry
Berkeley, California

PIER PLAZA SETTLEMENT MITIGATION

Reference: ENGEO. 2024. Geotechnical Report, Berkeley Water Transportation Pier Ferry, Berkeley. December 16, 2024, Revised May 28, 2025. Project No. 25022.000.001.

Dear Mr. Connolly:

Based on our understanding, grades within the Pier Plaza portion of the Berkeley Water Transportation Pier Ferry project in Berkeley, California will be raised by up to 5½ feet to facilitate the Berkeley Water Transportation Pier Ferry project. The Pier Plaza area is shown in Exhibit 1. During our geotechnical exploration for our referenced Geotechnical Report, we encountered 4 to 16 feet of compressible Young Bay Mud in each of our explorations. Table 1 summarizes the thickness of Young Bay Mud that we encountered in the area within or directly adjacent to Pier Plaza. As discussed in the referenced Geotechnical Report, we estimate the resulting settlement from fill placement will be approximately 2½ to 5 inches.

TABLE 1: Summary of Young Bay Mud Thickness in Borehole and CPT Explorations near the Pier Plaza

EXPLORATION ID	THICKNESS OF YOUNG BAY MUD (ft)
1-B1	16
1-B2	7½
1-B3	6
2-B1	4
2-B4	5
1-CPT1	4

EXHIBIT 1: Pier Plaza Area

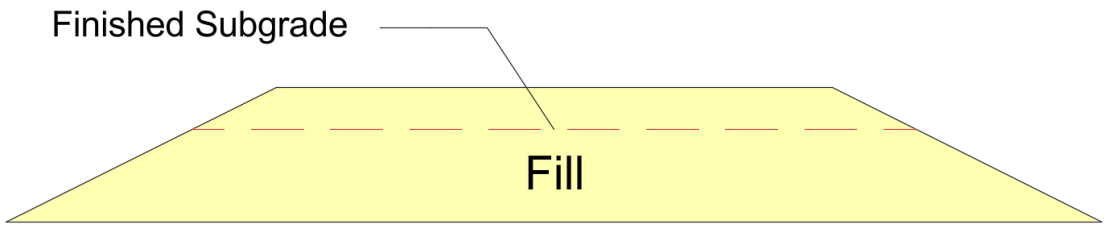


Based on our discussions with you, this range of potential long-term settlement is excessive for the planned improvements. In our referenced Geotechnical Report, we recommend the use of cellular concrete and/or surcharging to limit the impacts of long-term consolidation settlement on the site. Due to the existing shoreline slope, it is not feasible to place surcharge at the location of the wall as that would require placing new fill beyond the existing shoreline.

Therefore, through our conversations with the design team, we selected a segmental wall with precast concrete panels for the facing. This segmental wall will allow for some settlement since the panels can move independently to address differential settlement. Further, the team elected to backfill the wall with cellular concrete to limit that settlement. Once the fill at the wall is far enough from the retaining wall that it will not add additional loading on the wall, a surcharge fill should be placed to cost-effectively mitigate the settlement of the rest of the Pier Plaza. We recommend the following construction sequence below.

1. Construct an 8-foot-high engineered fill embankment with slope gradients not exceeding 2:1 (horizontal:vertical), as shown in Exhibit 2. The embankment should begin at the shoreline and extend 65 feet wide.

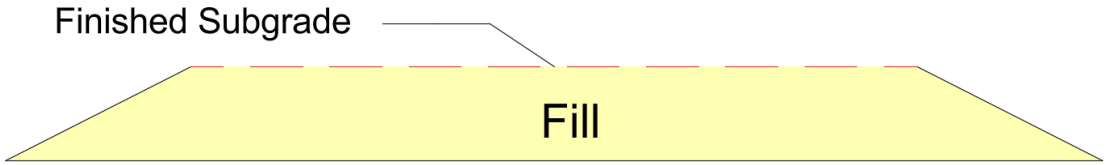
EXHIBIT 2: Construct Embankment Surcharge



2. Allow the Young Bay Mud to settle for approximately 3 months. Surcharge release should be based on accomplishing at least 75 percent consolidation of the fill and surcharge based on settlement monitoring.

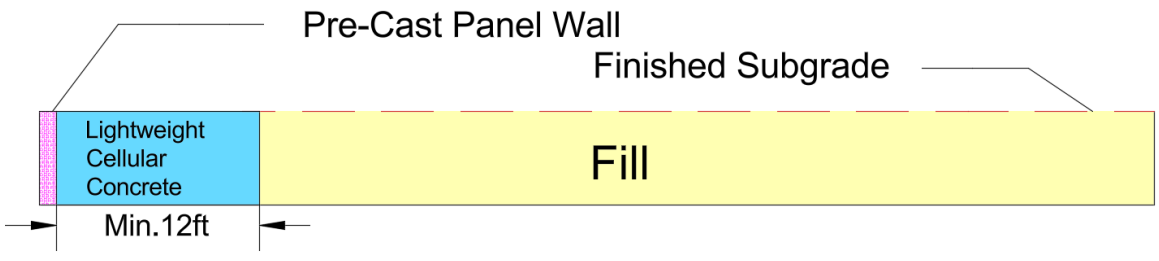
3. Cut the embankment to finished subgrade as shown in Exhibit 3.

EXHIBIT 3: Cut Embankment Surcharge to Final Grade



4. Excavate the embankment at the wall location to allow for a minimum 12-foot-wide lightweight cellular concrete backfill behind the wall as shown in Exhibit 4.

EXHIBIT 4: Excavate Embankment Surcharge far Enough Back to Allow for Construction of Pre-Cast Segmental Panel Wall and Minimum 12-foot-wide area of Lightweight Cellular Concrete Backfill



If the above steps are taken, we expect that the post-construction settlement will be reduced to approximately 1 to 2 inches within 50 years. Cellular concrete lift heights will be restricted by the vertical locations of the reinforcement strips and should be kept to no more than the height of material that is capable of supporting its own weight.

If you have any questions or comments regarding this letter, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated



Vlad Zasmolin, PE



Jeff Fippin, GE

vz/jaf/cb



APPENDIX K

PIER ABUTMENT LATERAL EARTH PRESSURE RECOMMENDATIONS

Project No.
25022.000.001

November 24, 2025

Mr. James Connolly
COWI
555 12th Street, Suite 1700
Oakland, CA 94601

Subject: Berkeley Water Transportation Pier Ferry
Berkeley, California

PIER ABUTMENT LATERAL EARTH PRESSURE RECOMMENDATIONS

References:
1. ENGEO. 2025. Geotechnical Report, Berkeley Water Transportation PierFerry. December 12, 2024; Revised June 2, 2025. Project No. 25022.000.001.
2. California Geologic Survey (CGS). 2008. Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, California Department of Conservation.

Dear Mr. Connolly:

At your request, we have prepared this letter to provide design recommendations for lateral earth pressure on the proposed concrete pier abutment wall at the Berkeley Water Transportation Pier Ferry at the Berkeley Marina in Berkeley, California. We understand the abutment will support a portion of the pier entrance, and a separate retaining structure will support the edge of the entrance and plaza extending north and south of the pier. The abutment wall will be supported on 24-inch-diameter, 1-inch-thick, steel-driven pipe piles and will retain lightweight cellular concrete (LWCC) extending approximately 14 feet laterally from the back of the wall.

The project elevation datum referred to in this letter is the City of Berkeley Mean Lower Low Water tidal datum.

GROUNDWATER CONDITIONS

We anticipate the landside groundwater level to be tidally influenced due to the proximity to the San Francisco Bay and the fill material type. Based on our discussions with the design team, we developed design recommendations assuming that water may accumulate behind the abutment up to Elevation 14.5 feet due to sea level rise, extreme rainfall, and wave run-up conditions. Although the LWCC backfill is designed to be free-draining (permeability of $1x10^{-2}$ centimeters per second or greater) and the wall will be designed with a perforated subdrain, the design team elected to assume a hydraulic lag will create a differential hydrostatic load on the wall. The specific groundwater assumptions for static and seismic design are detailed below.

- Static Loading Conditions: For static load evaluation, the design team elected to use a landside groundwater level at Elevation 14.5 based on sea level rise, extreme rainfall, and wave run-up. On the bayside we assumed a water level below the retained height of the proposed abutment (a conservative estimate).
- Seismic Loading Conditions: Since it would be unlikely that a seismic event occur during an extreme storm event, we assumed the landside and bayside groundwater table to be below the retained soil height, a condition that holds true even when considering the Mean Higher High Water level with a long-term sea level rise scenario.

LATERAL EARTH PRESSURES

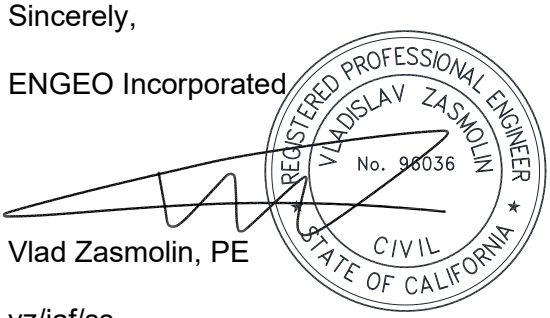
The wall should be designed for static load conditions using a combination of active earth pressures, surcharge loading, and hydrostatic pressures. For static loading, the walls may be designed for the equivalent fluid pressures of 64 pounds per cubic foot. To accommodate for fire truck and vehicular loading at the top of the wall, a uniform horizontal lateral load of 125 pounds per square foot (psf) should be applied to the entire height of the abutment wall.

Because the ferry terminal is an essential facility, we recommend designing the abutment wall for the Maximum Considered Earthquake (MCE) defined in our geotechnical report (Reference 1). To evaluate pressures on the wall during an MCE-level seismic event, we utilized the General Limit Equilibrium (GLE) method. In this analysis, we modeled the wall as a vertical cut subject to a driving horizontal seismic coefficient (k_h) equal to 0.48g. We then applied a counteracting external force to the face of the cut, iteratively adjusting the load magnitude until the system reached equilibrium (factor of safety of 1.0). We developed the seismic coefficient (k_h) in accordance with CGS Special Publication 117A (Reference 2), utilizing the MCE Geometric Mean Peak Ground Acceleration (PGA) equal to 0.85g (per the seismic hazard analysis in Reference 1), a Magnitude 8.1 event on the San Andreas Fault, and a maximum wall deflection allowance of 2 inches (5 centimeters). Based on this analysis, we recommend applying an equivalent lateral earth pressure of 70 pounds per cubic foot (pcf) for seismic loading. This value is equal to the sum of the active lateral earth pressure plus the seismic increment. We show our slope stability results as an attachment to this letter.

Resistance to overturning of the seawall is developed through the passive earth pressure generated by deflection of the wall into the embedded foreground as well as the rotation of the driven piles. We recommend using an ultimate passive resistance of 150 pcf for the Class V armor rock.

Sincerely,

ENGEO Incorporated



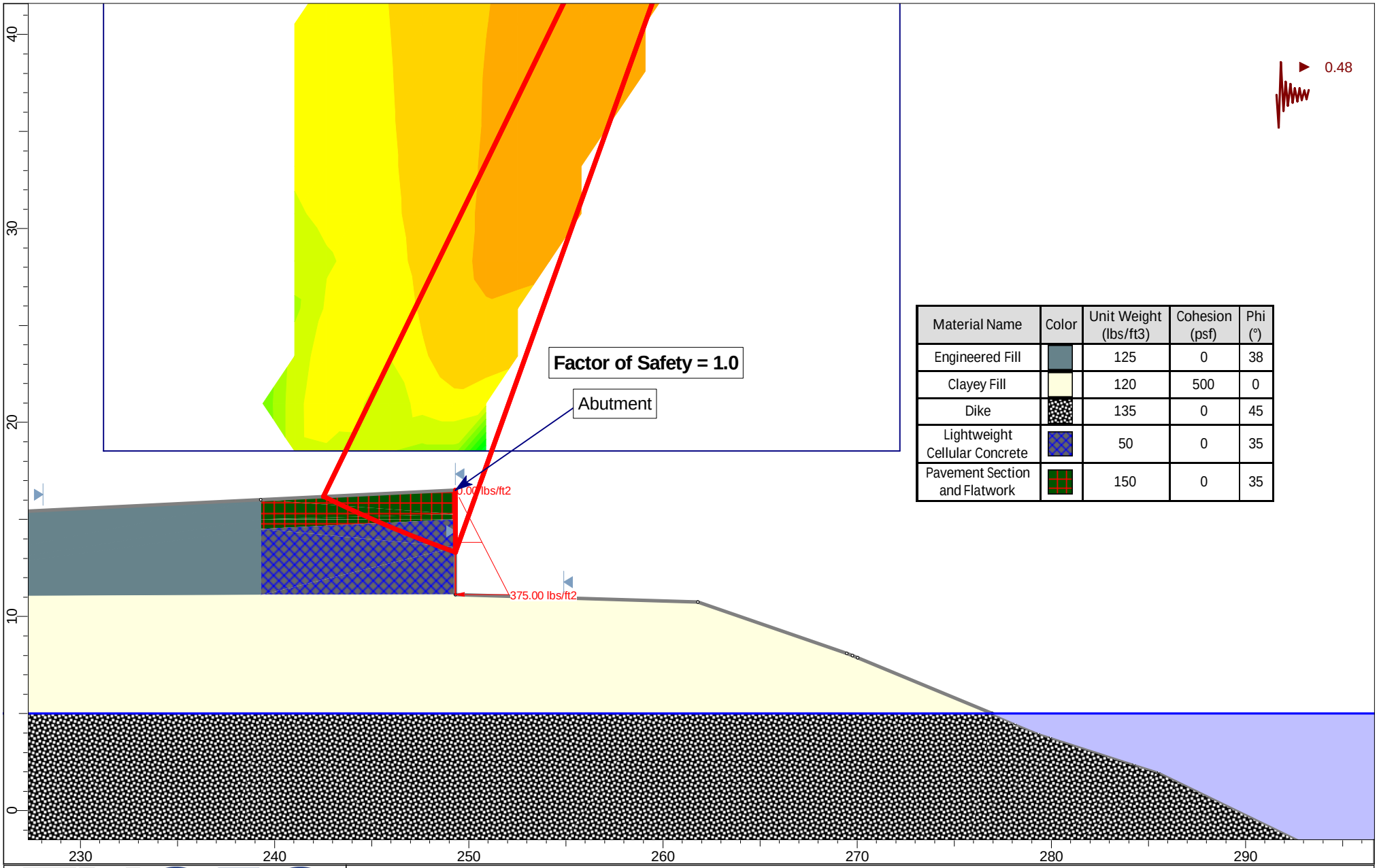
Vlad Zasmolin, PE



Jeff Fippin, GE

vz/jaf/ss

Attachment: Seismic Slope Stability Analysis



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (°)
Engineered Fill		125	0	38
Clayey Fill		120	500	0
Dike		135	0	45
Lightweight Cellular Concrete		50	0	35
Pavement Section and Flatwork		150	0	35

ENGEO
Expect Excellence

Project	Berkeley Water Transportation Pier Ferry		
Scale 1:81	*Author* VZ/JF	*Project No.* 25022.000.001	
Date 11/14/2025	*Condition* Estimate of Earthquake and Soil Pressure on Abutment based on 2 inches of Lateral Movement during MCE-Level Earthquake (SP117A)		



APPENDIX L

PRECAST PANEL MECHANICALLY STABILIZED EARTH SEAWALL SUBMITTAL

Project No.
25022.000.001

December 12, 2025

Mr. James Connolly
COWI
555 12th Street, Suite 1700
Oakland, CA 94601

Subject: Berkeley Water Transportation Pier Ferry
Berkeley Marina
Berkeley, California

PRECAST PANEL MECHANICALLY STABILIZED EARTH RETAINING WALL DESIGN

Dear Mr. Connolly:

This submittal presents our narrative, calculation package, and plan set for the proposed precast panel mechanically stabilized earth (MSE) seawall (retaining wall) for the Berkeley Water Transportation Pier Ferry (BWTPF) project. We used our findings, recommendations and conclusions outlined in our Geotechnical Report (ENGEO, 2025) to support the design of this MSE retaining wall. All project elevations referenced herein are based on the City of Berkeley Mean Lower Low Water (MLLW) datum.

PROJECT OVERVIEW AND SYSTEM DESCRIPTION

The Request for Qualifications (RFQ) document prepared by the City of Berkeley, dated December 15, 2023, describes that the project consists of demolishing 1,080 linear feet of the existing Municipal Berkeley Pier and replacing it with a new 1,480 linear foot sword-shaped pier and breakwater. The new pier will serve as a dual-use facility, functioning as a Water Transportation Authority (WETA) electric-ready ferry terminal and a recreational pier for walking and fishing. At the pier entrance, the grade will be raised from Elevation 11 feet to Elevation 16½ feet to facilitate construction of the entrance plaza. To the north and south of the pier the ground surface will slope down at a gradient of 1.4 and 1.7 percent, respectively.

The design team has selected to construct a precast panel MSE retaining wall to retain the new fill. This system was chosen to mitigate architectural distress in the wall due to consolidation settlement of the underlying Young Bay Mud following fill placement. Unlike conventional gravity or cantilever concrete walls, the joints between the precast panels are can move relative to each other to accommodate minor differential settlement with negligible to minor distress.

The proposed MSE retaining wall will comprise precast concrete facing panels (Maccaferri MacRes or approved equivalent) connected to discrete high-adherence polymeric soil reinforcing strips (Paraweb 2D30 Series or approved equivalent). The reinforcing strips will be mechanically connected to the rear face of the panels by threading them through cast-in-panel connection cavities. The reinforced backfill will consist of permeable Class II Lightweight Cellular Concrete (LWCC) extending 14 feet behind the wall. A Mirafi 180N non-woven geotextile will encapsulate the LWCC to prevent soil fines from clogging the pervious cellular structure. Water will be captured in a perforated pipe placed at the base of the fill that outlets through holes drilled in the wall panels; these outlets will be buried under the surface of the revetment placed at the base of the wall.

We designed the wall to retain up to 5½ feet of level backfill with a 2:1 (horizontal:vertical) sloping foreground. In addition, the design accounts for hydrostatic lateral earth pressure due to temporary inundation and surcharge loading from handrails, pedestrians and periodic emergency vehicles.

SOIL CONDITIONS

During our geotechnical exploration we drilled one mud rotary boring (1-B2), four sonic borings (2-B1, 2-B2, 2-B3, and 2-B4) and advanced one cone penetration test (1-CPT1) at the pier plaza. We present the idealized soil units and their approximate depth and elevation in Table 1.

TABLE 1: Idealized Soil Conditions at the Pier Plaza

APPROXIMATE LAYER ELEVATION (FEET)	LAYER THICKNESS (FEET)	MATERIAL	DESCRIPTION
Ground Surface to 4	6 to 10	Artificial Fill	Medium stiff lean clay and medium dense clayey sand with intermittent debris.
4 to -8	10 to 16	Rock Dike	Dense to very dense, well-graded gravels and sand containing cobble up to 4½ inches in diameter. Intermixed with accreted bay sediments.
-8 to -15	4 to 7½	Young Bay Mud	Soft to medium stiff compressible silt and clay.
-15 and below	100+	Alluvium/Old Bay Clay/Upper Alameda Formation	Stiff to very stiff clay with silt/sand interbeds

A detailed description of each geologic unit and a geologic cross section is provided in our Geotechnical Report (ENGEO, 2025).

GROUNDWATER CONDITIONS

We anticipate the landside groundwater level to be tidally influenced due to the proximity to the San Francisco Bay. Based on our discussions with the design team, we developed design recommendations assuming that water may accumulate behind the retaining wall up to Elevation 14.5 feet due to sea level rise, extreme rainfall, and wave run-up conditions. Although the LWCC backfill is designed to be free-draining (permeability of $1x10^{-2}$ centimeters per second or greater) and the wall will be designed with a perforated subdrain, the design team elected to assume a hydraulic lag will create a differential hydrostatic load on the wall. The specific groundwater assumptions for static and seismic design are detailed below.

- Static Loading Conditions: For static load evaluation, the design team elected to use a landside groundwater level at Elevation 14.5 based on sea level rise, extreme rainfall, and wave run-up conditions. On the bayside we assumed a water level below the retained height of the proposed wall (a conservative estimate).
- Seismic Loading Conditions: Since it would be unlikely that a seismic event occur during an extreme storm event, we assumed the landside and bayside groundwater table to be below the retained soil height to represent Mean Sea Level, a condition that holds true even when considering the long-term sea level rise scenario.

SETTLEMENT

As discussed in our Geotechnical Report (ENGEO, 2025), we estimate the settlement resulting from fill placement in the pier plaza to be approximately 2½ to 5 inches. Post-construction settlement will be mitigated through a combination of a surcharge program and load compensation with LWCC. The surcharge program involves constructing an 8-foot-high engineered fill embankment extending 65 feet behind the shoreline, which will remain in place for approximately 3 months or until 75 percent of primary consolidation in the Young Bay Mud is achieved. Subsequently, the embankment will be cut to finished subgrade and excavated at the wall location to accommodate the construction of the MSE wall and a 14-foot-wide zone of lightweight cellular concrete backfill. The construction sequence for the proposed surcharge program is detailed in Appendix J of our Geotechnical Report (ENGEO, 2025). If the surcharge program is implemented, we anticipate 1 inch of post-construction settlement over 50 years, which can be accommodated by the MSE retaining wall with negligible to minor distress.

DESIGN CRITERIA

General Design Standard

We designed the MSE retaining wall in accordance with the AASHTO LRFD Bridge Design Specifications, 9th Edition (2020). The design follows the load and resistance factor design (LRFD) methodology to evaluate internal stability of the geostrips (tensile strength, connection strength, and pullout capacity of reinforcement) and external stability of the wall (sliding, overturning, and bearing capacity) for strength limit state (static) and extreme limit state (seismic) loading conditions.

Surchage Loading

We applied surcharge loads behind the MSE retaining wall to account for handrails, pedestrian traffic, and periodic fire truck loading. The coping is structurally connected to the panels via dowels; therefore, our model considers it part of the retaining height rather than a surcharge load. Table 2 summarizes the loading magnitudes we used in our analysis.

TABLE 2: Surcharge Loading Summary

ITEM	DEAD LOAD (DIRECTION)	LIVE LOAD #1 (DIRECTION)	LIVE LOAD #2	SOURCE
Handrail	50 pounds per lineal foot (plf) (Vertical)	50 plf (Vertical and Horizontal)	200 pounds (lbs) (Vertical and Horizontal)	Estimated handrail weight and AASHTO LRFD SECTION 13.8.2
Pedestrian Traffic	-	75 pounds per square foot (psf) (Vertical)	-	AASHTO LRFD SECTION 3.6.1.6
Fire Truck Loading (Applied to Static Condition only)	-	250 psf (Vertical)	-	Standard of Practice for Highways

Seismic Design Criteria

We understand the pier is classified as an essential facility and require ongoing access during and following seismic events. In our Geotechnical Report (ENGEO, 2025) we developed seismic-

design response spectra for the pier structure in accordance with the guidelines presented in the 2014 ASCE/COPRI 61 Standard: Seismic Design of Piers and Wharves (ASCE 61-14). We estimated the site-specific MCE peak ground acceleration (PGA_M) to be 0.85g. Based on experience with similar waterfront structures, we assume a displacement of no more than 6 inches is allowable during an MCE level event. The global seismic slope stability analysis in our Geotechnical Report (ENGEO, 2025) shows conformance to this requirement.

In this submittal, we analyzed the seismic stability of the MSE retaining wall using a horizontal seismic coefficient equal to half of the PGA_M. According to AASHTO LRFD Article 11.6.5, achieving a capacity to demand ratio greater than 1.0 using half of the PGA_M will limit permanent wall displacement to approximately 2 inches or less following a seismic event.

MATERIAL PARAMETERS

Physical Parameters of Soil and Lightweight Cellular Concrete

The soil parameters selected for the analysis are described below and summarized in Table 3.

- **Existing Fill**: Physical properties of the existing fill were derived from triaxial laboratory testing. Our laboratory tests are presented in Appendix D of our Geotechnical Report (ENGEO, 2025).
- **Lightweight Cellular Concrete**: We used the "Guide to Lightweight Cellular Concrete" manual written by the Portland Cement Association in 2021 and a conference presentation titled "The Design of Mechanically Stabilized Earth Walls with Lightweight Cellular" presented by Reinforced Earth in 2017 to derive the properties of the LWCC.

  It should be noted that although the specified dry cast density of Class II LWCC is 30 pounds per cubic foot (pcf), we anticipate the material will become saturated upon inundation during its lifetime. Therefore, we adopted a higher design unit weight of 50 pcf to reflect a permanent increase in density.
- **Engineered Fill**: The material type and source of the engineered fill is unknown. Therefore, we assumed a conservative friction angle and typical unit weight for compacted fill.

TABLE 3: Physical Properties of Soil and Lightweight Cellular Concrete

SOIL LAYER	UNIT WEIGHT, γ (pcf)	COHESION, c (psf)	FRICTION ANGLE, ϕ (deg)	COEFFICIENT OF FRICTION, f*
Existing Clayey Fill	120	500	-	-
Lightweight Cellular Concrete (Saturated)	50	-	35	2.0 at top of wall, decreasing linearly to tan(φ) at a depth of 20 feet
Engineered Fill	125	-	32	-

Geostrip Reinforcement

Our design utilizes discrete high-adherence polymeric strips from the Paraweb 2D30 Series. These geostrips are composed of high-tenacity polyester fibers encased in a durable polyethylene sheath. If alternative geostrip products are used they should be polymeric strips, meet the minimum performance criteria outlined in Table 4, and be submitted to the Engineer for approval.

TABLE 4: Minimum Geostrip Tensile and Connection Strength Requirements

MECHANICAL PROPERTY	VALUE	TEST METHOD
Ultimate Tensile Strength, Tult	24,749 pounds per foot (lbs/ft)	ASTM D6637
Connection Strength	35,073 lbs/ft	ASTM D6638

Based on manufacturer recommendations, we used tensile strength reduction factors to account for creep, installation damage, and durability. Additionally, we applied a Factor of Safety of 2.0 to the tensile strength based on Duncan and Wright (2014) criteria for reinforced slopes where the consequences of failure are high. These design factors are summarized in Table 5.

TABLE 5: Ultimate Tensile Strength Reduction Factors

REDUCTION FACTOR	VALUE	SOURCE
Creep Reduction Factor (RF_{cr})	1.38	Manufacturer Recommendation
Installation Damage Factor (RF_{id})	1.1	Manufacturer Recommendation
Deterioration in Service Factor (RF_{d})	1.1	Manufacturer Recommendation
Factor of Safety	2.0	Duncan and Wright (2014)

Our selected reinforcement layout consists of six connections per 4-foot-10¼-inch tall panel (Panel Type A) and three connections per 2-foot-4¾-inch tall panel (Panel Type B). This configuration results in a reinforcement coverage ratio (R_c) of 33.2 percent.

Precast Facing Panels

Our design utilizes the Maccaferri MacRes precast concrete panels. The panels will be manufactured to be suitable for installation in a marine environment in accordance with American Concrete Institute (ACI) 318-19 and 357R standard. The minimum precast concrete production requirements are summarized in Table 6.

TABLE 6: Precast Panel Concrete Production Requirements

MECHANICAL PROPERTY		VALUE	TEST METHOD
Concrete Compressive Strength		5,000psi	ASTM C39
Cement Type		Type II/V (Sulfate Resistant)	ASTM C150
Reinforcement	Reinforcing Steel	Grade 60 (Deformed) or Grade 75 (Plain)	ASTM A615
	Welded Wire Mesh	-	ASTM 1064
Corrosion Protection		Epoxy Coated or Dual Coated	ASTM A934/ ASTM A1055
Maximum Water to Cement Ratio		0.40	-
Minimum Concrete Cover		3 inches	-

METHOD OF ANALYSIS AND RESULTS

We used the Rocscience RSWall software to evaluate the internal and external stability of the MSE retaining wall. For internal reinforcement stability we calculated the tensile rupture and pullout capacity at each geostrip level. We did not include the connection rupture failure mode in our analysis because the geostrip's tensile strength is lower than the connection strength, and therefore, governs the design. For external stability, we assessed the sliding, overturning, and bearing capacity of the wall.

We performed our analyses on three select wall sections that correspond with the project's three panel configurations. Segment 1 is located at Station 0+00 to 0+45 and 2+52 to 3+57, comprising only Panel Type B; Segment 2 is located at Station 0+45 to 1+32 and 2+13 to 2+52, comprising only Panel Type A; Segment 3 is located at Station 1+32 to 1+62 and 1+84 to 2+13, comprising stacked Panel Type A and Panel Type B. We evaluated the maximum retained soil height for each segment. Table 7 summarizes the retaining properties for each case. All sections were conservatively designed assuming a minimum embedment depth of 2.0 feet, satisfying AASHTO (2020) Table 11.10.2.2-1 requirements for walls with a 2:1 (horizontal:vertical) sloping foreground.

TABLE 7: Segment Configuration and Stationing

SEGMENT NUMBER	STATIONING	PANEL TYPE	TOP/BOTTOM OF WALL ELEVATION (FEET)	HEIGHT OF RETAINED SOIL (FEET)
1	0+00 to 0+45 and 2+52 to 3+57	B	14.8/10.5	2.2
2	0+45 to 1+32 and 2+13 to 2+52	A	16.1/9.75	4.3
3	1+32 to 1+62 and 1+84 to 2+13	A and B	16.5/7.25	7.25

We present the results of our analysis in Table 8 and Table 9 for strength limit state and extreme limit state conditions, respectively. A capacity to demand ratio (CDR) greater than 1.0 indicates that the design meets the prescribed criteria.

TABLE 8: Strength Limit State (Static) Results

SEGMENT NUMBER	ANALYSIS TYPE	SOURCE	CDR
1	Internal	Tensile Rupture	2.2*
		Pullout Resistance	4.1*
	External	Base Sliding	2.2
		Overturning	2.4
		Bearing	2.6
2	Internal	Tensile Rupture	2.9*
		Pullout Resistance	2.9*
	External	Base Sliding	2.0
		Overturning	1.9
		Bearing	2.0
3	Internal	Tensile Rupture	2.3*
		Pullout Resistance	2.6*
	External	Base Sliding	1.8
		Overturning	1.2
		Bearing	1.4

*Critical reinforcement level

TABLE 9: Extreme Limit State (Seismic) Results

SEGMENT NUMBER	ANALYSIS TYPE	FAILURE MODE	CDR
1	Internal	Tensile Rupture	9.6*
		Pullout Resistance	6.9*
	External	Base Sliding	1.8
		Overturning	5.4
		Bearing	8.4
2	Internal	Tensile Rupture	9.4*
		Pullout Resistance	5.4*
	External	Base Sliding	1.5
		Overturning	3.6
		Bearing	5.0
3	Internal	Tensile Rupture	7.5*
		Pullout Resistance	4.4*
	External	Base Sliding	1.4
		Overturning	2.4
		Bearing	2.7

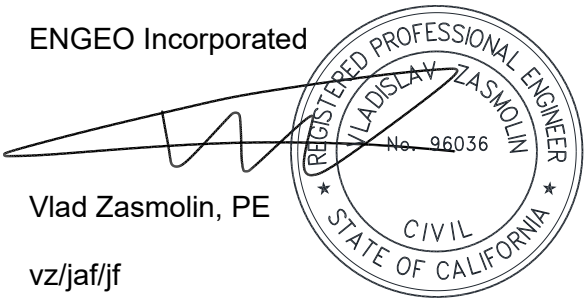
*Critical reinforcement level

CLOSING

We appreciate the opportunity to be of service to you on this project. If you have any questions regarding the content of this letter, please do not hesitate to contact us.

Sincerely,

ENGEO Incorporated



Vlad Zasmolin, PE



Jeff Fippin, GE

vz/jaf/jf

Attachments: Selected References
Plan Set
Calculation Package

SELECTED REFERENCES

1. American Association of State Highway and Transportation Officials (AASHTO). 2020. AASHTO LRFD Bridge Design Specifications, 9th Edition.

2. American Concrete Institute (ACI). 2019. Building Code Requirements for Structural Concrete and Commentary (ACI 318-19).

3. American Concrete Institute ACI. 1997. Guide for the Design and Construction of Fixed Offshore Concrete Structures (ACI 357R-84).

4. American Society of Civil Engineers (ASCE). 2014. Seismic Design of Piers and Wharves (ASCE/COPRI 61-14).

5. ASTM International. Book of ASTM Standards.

    a. ASTM A615: Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement.
    b. ASTM A934: Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars.
    c. ASTM A1055: Standard Specification for Zinc and Epoxy Dual-Coated Steel Reinforcing Bars.
    d. ASTM A1064: Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete.
    e. ASTM C39: Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.
    f. ASTM C150: Standard Specification for Portland Cement.
    g. ASTM D6637: Standard Test Method for Determining Tensile Properties of Geogrids by the Single or Multi-Rib Tensile Method.
    h. ASTM D6638: Standard Test Method for Determining Connection Strength Between Geosynthetic Reinforcement and Segmental Concrete Units.

6. City of Berkeley.2023. Response to Qualifications (RFQ) for the Berkeley Water Transportation Pier Ferry Project, Berkeley, California. December 15, 2023.

7. Duncan, J.M., Wright, S.G., and Brandon, T.L. 2014. Soil Strength and Slope Stability, 2nd Edition.

8. ENGEO. 2025. Geotechnical Report, Berkeley Water Transportation Pier Ferry, Berkeley, California. December 16, 2024; Revised December 12, 2025. Project No. 25022.000.001.

9. Portland Cement Association (PCA). 2021. Guide to Lightweight Cellular Concrete.

10. Reinforced Earth. 2017. The Design of Mechanically Stabilized Earth Walls with Lightweight Cellular Concrete. Presented at 2017 KU CEAE Geotechnical Conference, Kansas, Missouri. November 9, 2017

PLAN SET

25022.000.001
December 12, 2025

GENERAL NOTES:

1. BASIS OF DESIGN

1.1 ENGEO; GEOTECHNICAL EXPLORATION, BERKELEY WATER TRANSPORTATION PIER FERRY, BERKELEY, CALIFORNIA; DECEMBER 12, 2025.

1.2 NCE; TOPOGRAPHIC SURVEY, BERKELEY WATER TRANSPORTATION PIER FERRY, BERKELEY, CALIFORNIA; SEPTEMBER 23, 2024.

1.3 AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO). 2020. AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 9th EDITION.

1.4 DATUM: CITY OF BERKELEY MEAN LOWER LOW WATER DATUM.

2. SYSTEM DESCRIPTION

2.1 THE MECHANICALLY STABILIZED EARTH (MSE) RETAINING WALL SYSTEM WILL CONSIST OF PRECAST CONCRETE FACING PANELS AND DISCRETE HIGH–ADHERENCE POLYMERIC SOIL REINFORCING STRIPS.

2.2 POLYMERIC SOIL REINFORCING STRIPS CONSIST OF DISCRETE BUNDLES OF CLOSELY PACKED HIGH STRENGTH POLYESTER FILAMENTS, LYING PARALLEL TO EACH OTHER, ENCASED IN A TOUGH AND DURABLE POLYETHYLENE COATING.

3. MATERIALS

3.1 THE REINFORCED BACKFILL SHALL BE CLASS I PERVIOUS LIGHTWEIGHT CELLULAR CONCRETE, AND SHALL BE REVIEWED AND APPROVED BY THE GEOTECHNICAL ENGINEER OF RECORD.

LIGHTWEIGHT CELLULAR CONCRETE PROPERTIES

MAXIMUM IN–PLACE UNIT WEIGHT, γ = 30 PCF

MINIMUM 28–DAY UNCONFINED COMPRESSIVE STRENGTH = 40 PSI

MINIMUM COEFFICIENT OF PERMEABILITY = 1 X 10^{-2} CM/S

3.4 PRECAST CONCRETE FACING PANELS SHALL BE CONSTRUCTED USING CONCRETE HAVING A MINIMUM COMPRESSIVE STRENGTH OF 5,000 POUNDS PER SQUARE INCH (PSI) AT 28 DAYS. CEMENT SHALL BE TYPE II/V CONFORMING TO ASTM C150. WATER CEMENT RATIO SHOULD NOT BE GREATER THAN 0.4.

3.5. REINFORCING STEEL SHALL BE ASTM 615 GRADE 60 DEFORMED REBAR, PLAIN GRADE 75 OR WELDED WIRE REINFORCEMENT (WWR) C CONFORMING TO ASTM A1064.

3.6 ALL REINFORCEMENT SHALL BE EPOXY COATED OR DUAL COATED PER PROJECT SPECIFICATIONS. EPOXY COATED BARS SHALL CONFORM TO ASTM A934 AND CALTRANS STANDARD SPECIFICATION 52–2.

3.7 POLYMERIC REINFORCEMENTS STRIPS MANUFACTURED FROM HIGH TENACITY, MULTIFILAMENT POLYESTER YARNS ALIGNED AND CO–EXTRUDED WITH LLDPE LINEAR LOW DENSITY POLYTHENE TO FROM POLYMERIC STRIPS. POLYMERIC REINFORCEMENT STRIPS SHALL BE "PARAWEB 2D 30" OR APPROVED EQUIVALENT (TULT = 6,744 LBS, WIDTH = 3.27 INCHES).

3.8 FILTER FABRIC SHALL BE NON–WOVEN NEEDLE PUNCHED POLYPROPYLENE GEOTEXTILE MEETING MINIMUM CRITERIA IN PROJECT SPECIFICATIONS (MIRAFI 180N, OR APPROVED EQUIVALENT).

3.9 DRAINAGE PIPES SHALL BE A 4–INCH DIAMETER PVC PERFORATED PIPE (SCHEDULE SDR 35) PIPE.

3.10 DRAINAGE MEDIUM SHALL CONSIST OF CLEAN, WASHED, CRUSHED ROCK MEETING THE GRADATION AND DURABILITY REQUIREMENTS IN THE PROJECT SPECIFICATIONS.

4. SITE PREPARATION

PRIOR TO CONSTRUCTION THE CONTRACTOR SHALL CLEAR AND GRADE THE FOUNDATION AND REINFORCED BACKFILL AREA, REMOVING TOP SOIL, BRUSH, SOD AND OTHER ORGANIC DELETERIOUS MATERIALS. ANY UNSUITABLE SOIL SHALL BE REMOVED AND REPLACED WITH COMPACTED BACKFILL MATERIAL TO PROJECT SPECIFICATIONS.

AN EVALUATION OF THE SUITABILITY OF THE FOUNDATION SOIL AND THE REQUIRED BEARING CAPACITY SHOULD BE PERFORMED BY THE OWNER TESTING AGENCY.

5. CONSTRUCTION

5.1 LEVELING PAD – PLACE LEAN CONCRETE, CAST IN–SITU AND TOWELED SMOOTH TO FORM A HORIZONTAL BASE FOR THE PANELS. THE LEVELING PAD SHOULD BE CONSTRUCTED WITHIN 1/8– INCH TOLERANCE OF THE SPECIFIED BOTTOM OF PANEL. THE LEVELING PAD SHALL HAVE A MINIMUM 28–DAY COMPRESSIVE STRENGTH OF 2,000 PSI.

5.2 PANEL POSITIONING – CHECK THAT ALL PANELS COMPLY WITH THE REQUIRED PANEL DIMENSIONS AND THE CORRECT NUMBER OF CONNECTIONS PRIOR TO PLACING.

5.3 PLACING GEOTEXTILE – INSTALL NON–WOVEN GEOTEXTILE TO FULLY ENCAPSULATE THE LIGHTWEIGHT CELLULAR CONCRETE. THE GEOTEXTILE MUST COVER ALL VERTICAL AND HORIZONTAL PANEL JOINTS AND CREATE A COMPLETE SEPARATION LAYER BETWEEN THE LIGHTWEIGHT CELLULAR CONCRETE AND THE RETAINED BACKFILL.

5.4 PLACING LIGHTWEIGHT CELLULAR CONCRETE – THE CONTRACTOR IS RESPONSIBLE FOR COMPLYING WITH THE MIXING AND PLACEMENT OF LIGHTWEIGHT CELLULAR CONCRETE PER SPECIFICATION SECTION 02304.

5.5 PLACING AND CONNECTION OF THE REINFORCEMENT LAYERS – STARTING FROM THE BACK OF THE PANELS, THE WIDTH OF THE REINFORCED BACKFILL SHOULD BE MEASURED AND MARKED WITH PINS OR WITH A STRING LINE. LIGHTWEIGHT CELLULAR CONCRETE SHALL BE PLACED TO THE ELEVATION OF EACH STRIP LEVEL AND ALLOWED TO CURE FOR A MINIMUM OF 24 HOURS. REINFORCEMENT STRIPS SHALL BE CONNECTED TO PANELS AFTER CURING.REFER TO POLYMERIC GEOSTRIP REINFORCEMENT MANUFACTURER CONSTRUCTION MANUAL FOR INSTALLATION GUIDELINES.

5.6 A 4–INCH DIAMETER PERFORATED PVC COLLECTOR PIPE SHALL BE INSTALLED BEHIND THE WALL. THE PIPE SHALL DISCHARGE TO THE BAY VIA SOLID PVC OUTLET PIPES SPACED AT 50 FEET ON CENTER. THE COLLECTOR PIPE SHALL DISCHARGE TO THE BAY VIA PVC OUTLETS SPACED AT 50 FEET ON CENTER.

5.7 SUBGRADE PREPARATION AND RETAINED BACKFILL – PREPARE THE FOUNDATION SUBGRADE LEFT AT GRADE OR IN AREAS TO RECIEVE FILL BY SCARIFYING TO A DEPTH OF 8–INCHES, MOISTURE CONDITIONING, AND RECOMPACTING. PLACE ALL NEW SOIL FILL IN HORIZONTAL LIFTS NOT EXCEEDING 12–INCHES (LOOSE).

BOTH THE PREPARED SUBGRADE AND THE NEW FILL SHALL BE MOISTURE CONDITIONED TO 2 PERCENTAGE ABOVE OPTIMUM AND COMPACTED TO A MINIMUM OF 90% OF THE MAXIMUM DRY DENSITY (ASTM D1557).

60% SUBMITTAL
DECEMBER 12, 2025

REVISION	MARK	DATE	DESCRIPTION	APPROVAL
B		12-06-2025	60% SUBMITTAL	JAF
A		06-27-2025	30% SUBMITTAL	JAF

PROJECT MANAGER: DATE

DEPICTION OF MONUMENTS: DATE
SURVEY PARTY CHIEF

SUBMITTED: DATE
SUPERVISING CIVIL ENGINEER R.C.E. EXP.

DESIGN VZ | DRAWN LL | CHECK JAF | AS BUILT

HORIZ. NO SCALE | VERT. NO SCALE | BOOK | DATE 06/27/25

0 1 2 3
FOR REDUCED PLANS – ORIGINAL SCALE IS IN INCHES

WATERSHED REVIEW: DATE

APPROVED: DATE
CITY ENGINEER R.C.E. EXP.

CITY OF BERKELEY
PARKS RECREATION & WATERFRONT
1947 CENTER ST., 5th FL, BERKELEY, CA 94704

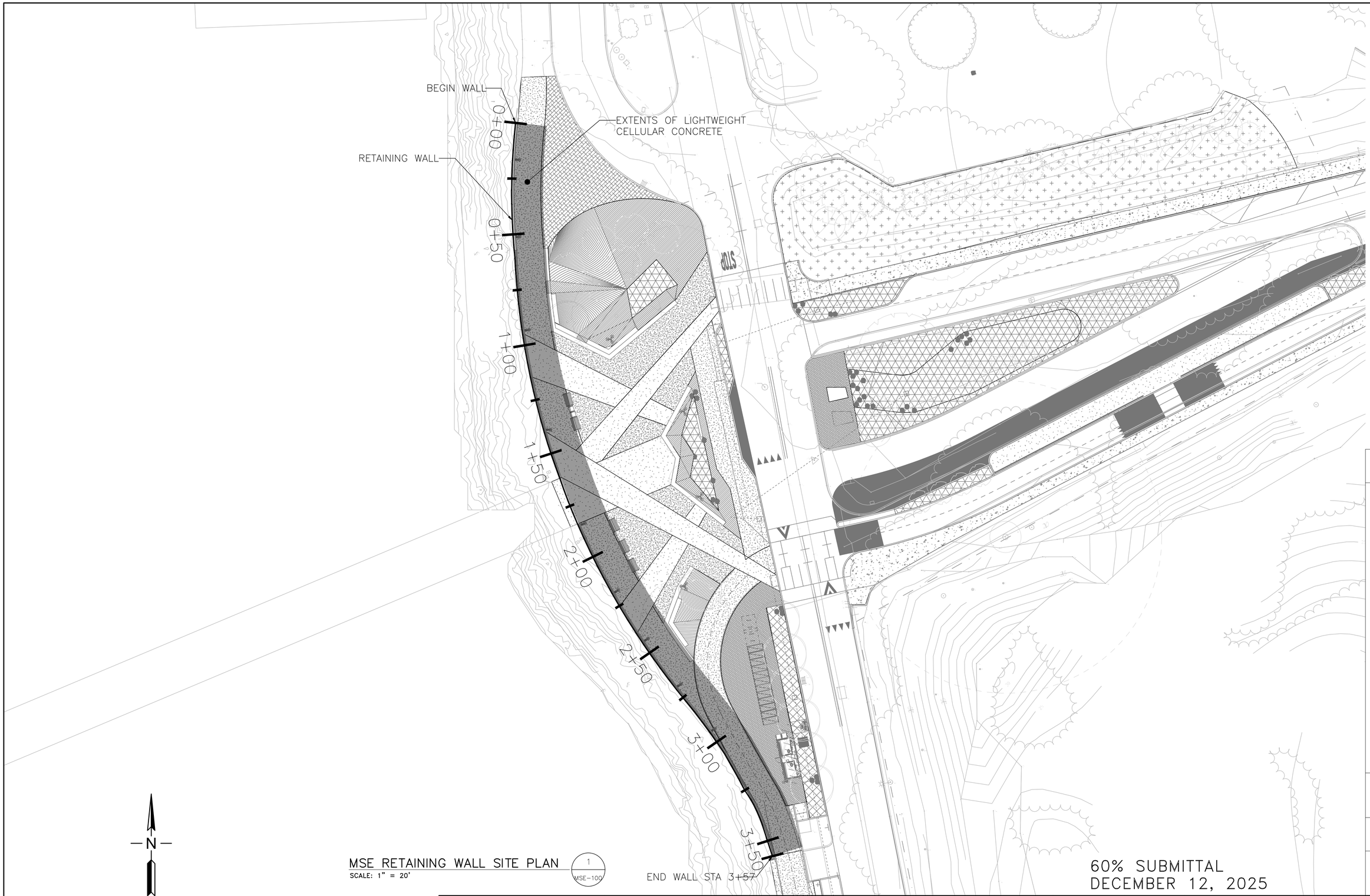
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BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT
CITY OF BERKELEY, ALAMEDA COUNTY, CALIFORNIA

MSE RETAINING WALL GENERAL NOTES

PLAN
FILE
MSE-001
SHEET OF

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MSE RETAINING WALL SITE PLAN (1 / MSE-100)

SCALE: 1" = 20'

60% SUBMITTAL
DECEMBER 12, 2025

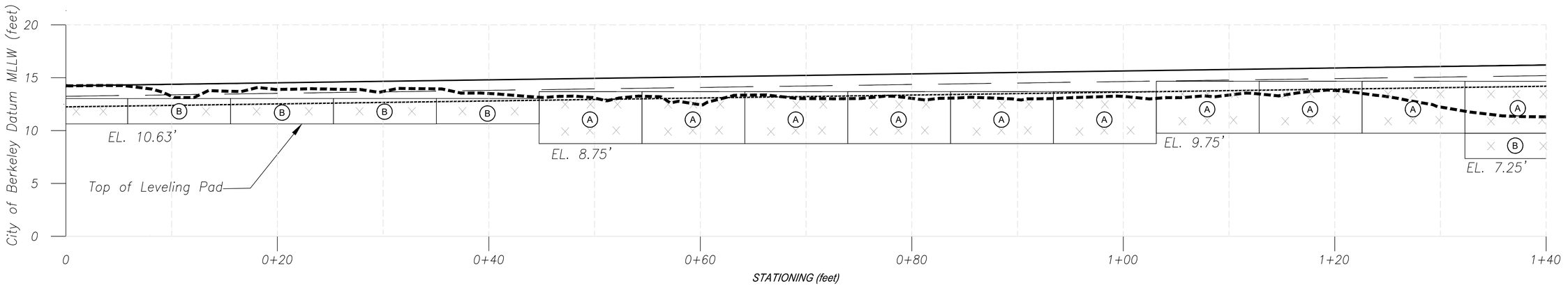
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0 1 2 3 FOR REDUCED PLANS – ORIGINAL SCALE IS IN INCHES	WATERSHED REVIEW: DATE	APPROVED: DATE	CHECK JAF	BOOK	MSE RETAINING WALL SITE PLAN	MSE-100
		CITY ENGINEER R.C.E. EXP.	AS BUILT	DATE 06/27/25	PARKS RECREATION & WATERFRONT 1947 CENTER ST., 5th FR, BERKELEY CA94704	SHEET ___ OF ___

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A		06-27-2025	30% SUBMITTAL

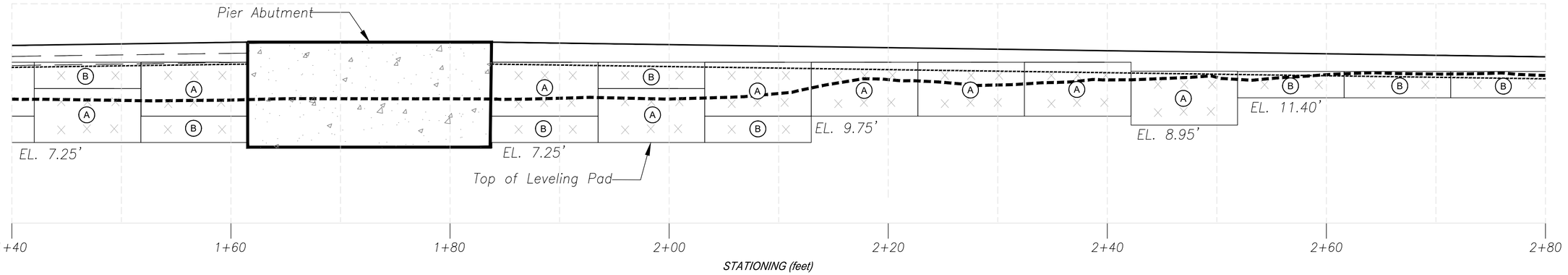
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APPROVAL

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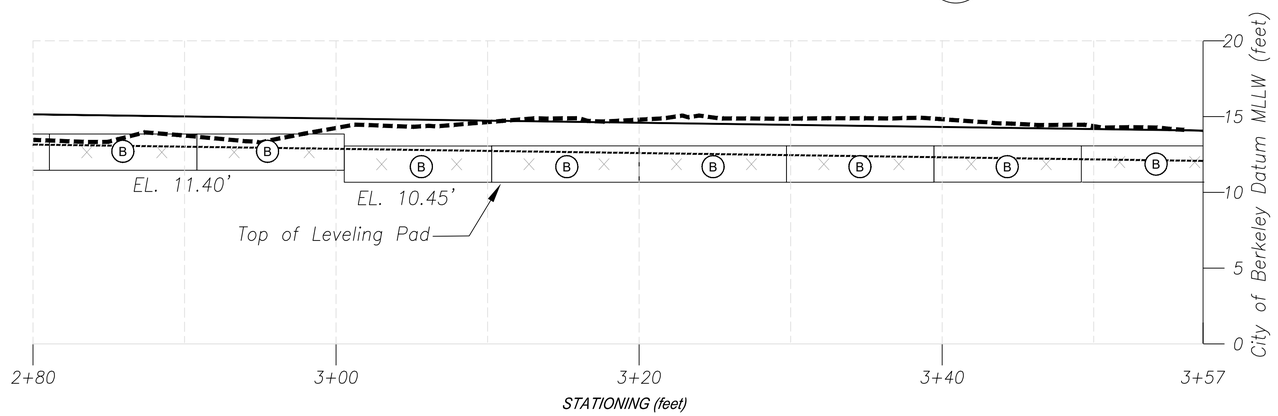
PROFILE ALONG WALL ALIGNMENT (BEGIN) 1 MSE-200

SCALE: 1" = 5'



PROFILE ALONG WALL ALIGNMENT (CONTINUED) 2 MSE-200

SCALE: 1" = 5'



PROFILE ALONG WALL ALIGNMENT (END) 3 MSE-200

SCALE: 1" = 5'

Legend

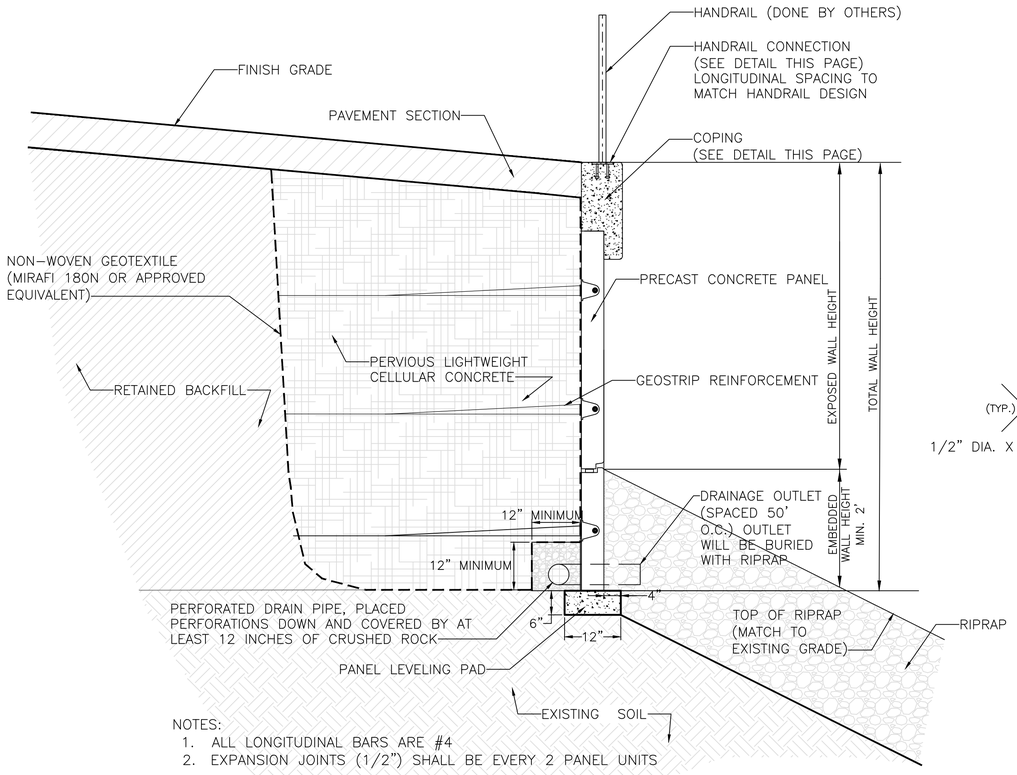
- Finished Grade at Top of Wall
- Existing Grade and Finished Grade at Bottom of Wall
- Bottom of Coping
- (B) Pannel Type (See Sheet MSE-300)
- Geostrip Connection (typical)

60% SUBMITTAL
DECEMBER 12, 2025

PROJECT MANAGER: DATE	DEPICTION OF MONUMENTS: DATE	SUBMITTED: DATE	DESIGN VZ	HORIZ. AS SHOWN	PARKS RECREATION & WATERFRONT 1947 CENTER ST., 5th FR, BERKELEY CA94704	ENGEO Expect Excellence	BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT CITY OF BERKELEY, ALAMEDA COUNTY, CALIFORNIA	PLAN
	SURVEY PARTY CHIEF	SUPERVISING CIVIL ENGINEER R.C.E. EXP.	DRAWN LL	VERT.			MSE RETAINING WALL ELEVATION SECTION	FILE
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		CITY ENGINEER R.C.E. EXP.	AS BUILT	DATE 06/27/25				SHEET OF

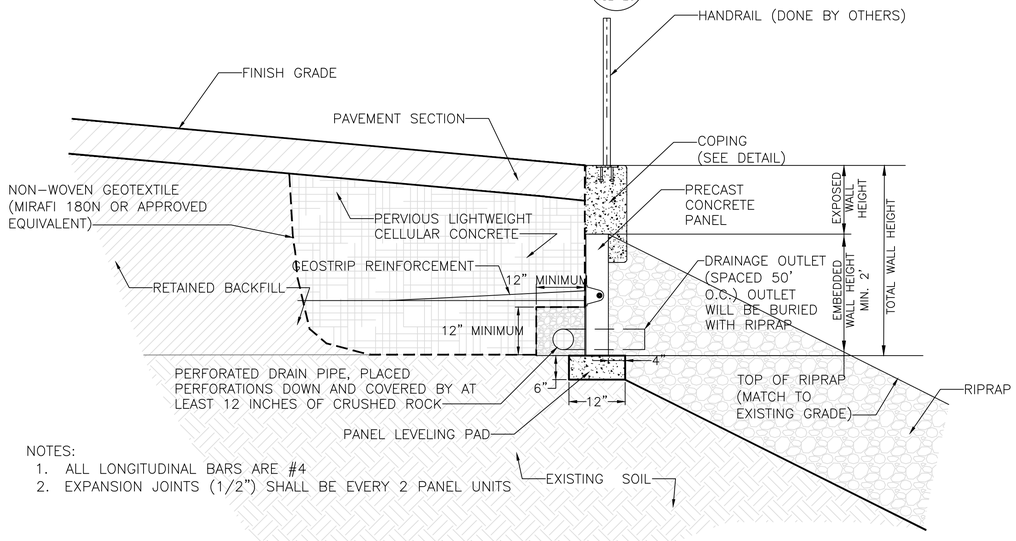
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A	06-27-2025	30% SUBMITTAL		JAF

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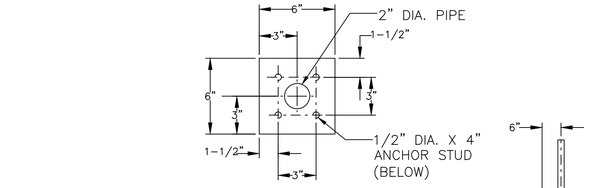
NOTES:
1. ALL LONGITUDINAL BARS ARE #4
2. EXPANSION JOINTS (1/2") SHALL BE EVERY 2 PANEL UNITS

TYPICAL 1–1/2 PANEL DETAIL
STATION 1+32 TO 1+62
STATION 1+84 TO 2+13
NOT TO SCALE — 1 / MSE–201

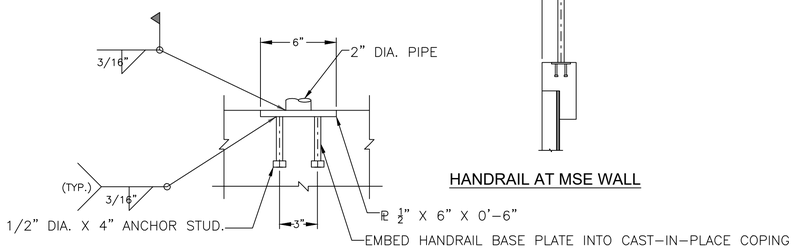


NOTES:
1. ALL LONGITUDINAL BARS ARE #4
2. EXPANSION JOINTS (1/2") SHALL BE EVERY 2 PANEL UNITS

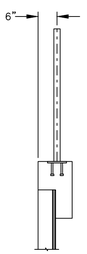
TYPICAL 1 PANEL DETAIL
STATION 0+45 TO 1+32
STATION 2+13 TO 2+52
NOT TO SCALE — 2 / MSE–201



PLAN AT HANDRAIL BASE PLATE

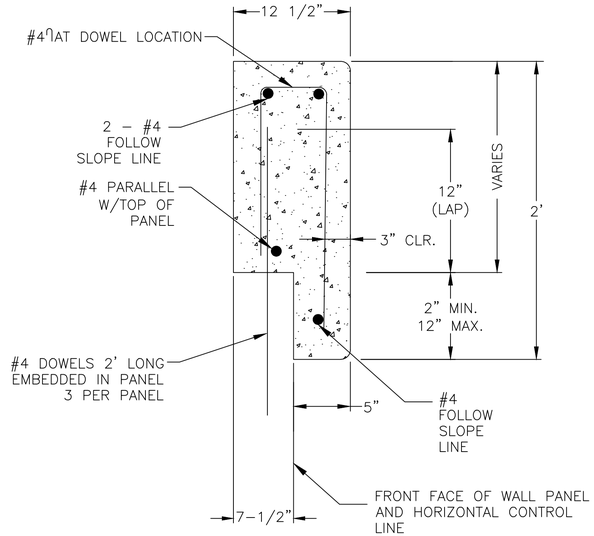


SECTION AT HANDRAIL BASE PLATE



HANDRAIL AT MSE WALL

HANDRAIL CONNECTION DETAILS
NOT TO SCALE — 4 / MSE–201



NOTES:
1. ALL LONGITUDINAL BARS ARE #4
2. EXPANSION JOINTS (1/2") SHALL BE EVERY 2 PANEL UNITS

COPING DETAIL
NOT TO SCALE — 5 / MSE–201

60% SUBMITTAL
DECEMBER 12, 2025

REVISION	MARK	DATE	DESCRIPTION	APPROVAL
B		12-05-2025	60% SUBMITTAL	JAF
A		06-27-2025	30% SUBMITTAL	JAF

PROJECT MANAGER: ____ DATE ____
DEPICTION OF MONUMENTS: ____ DATE ____ SURVEY PARTY CHIEF
WATERSHED REVIEW: ____ DATE ____
SUBMITTED: ____ DATE ____ SUPERVISING CIVIL ENGINEER — R.C.E. ____ EXP. ____
APPROVED: ____ DATE ____ CITY ENGINEER — R.C.E. ____ EXP. ____
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BOOK	
DATE	06/27/25

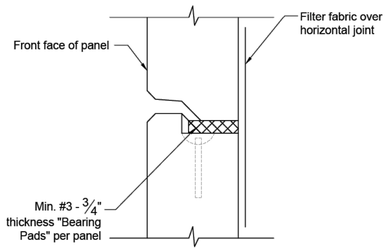
PARKS RECREATION & WATERFRONT 1947 CENTER ST., 5th FL BERKELEY CA94704

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BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT
CITY OF BERKELEY, ALAMEDA COUNTY, CALIFORNIA
TYPICAL CROSS SECTION

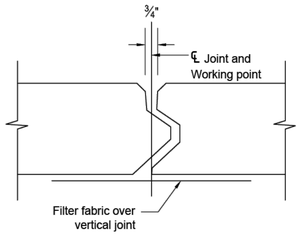
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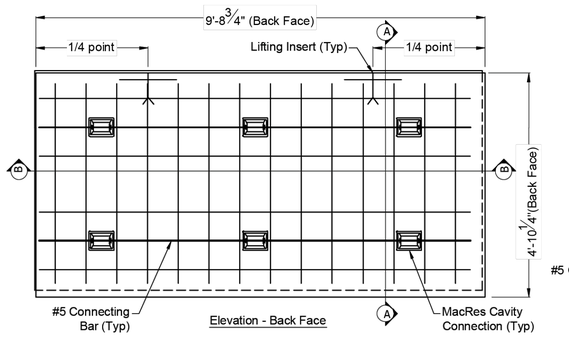
* BEARING PAD THICKNESS
For wall heights <30', 3/4" thick bearing pad
For wall heights >30', 1" thick bearing pad

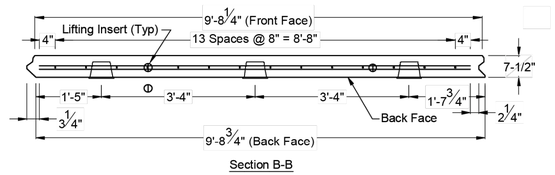
HORIZONTAL JOINT



VERTICAL JOINT

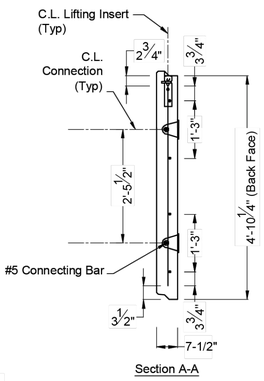
PANEL JOINT DETAIL 1 MSE-300
NOT TO SCALE



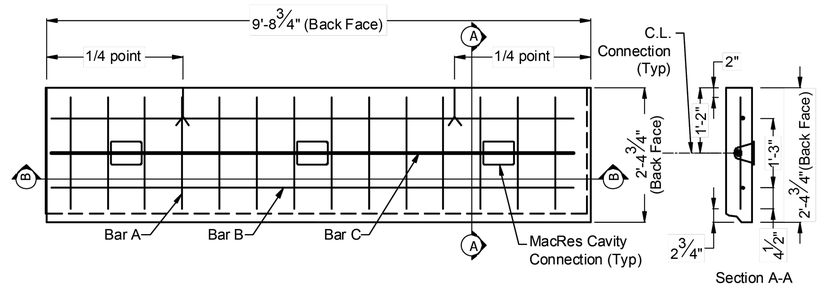


Section B-B

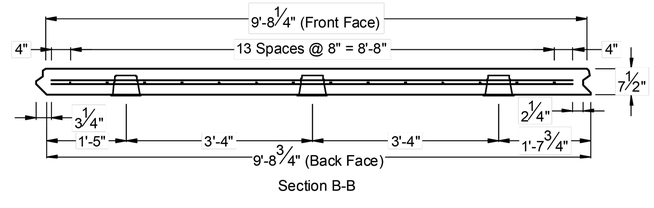
TYPICAL "A" PANEL 2 MSE-300
NOT TO SCALE



Section A-A

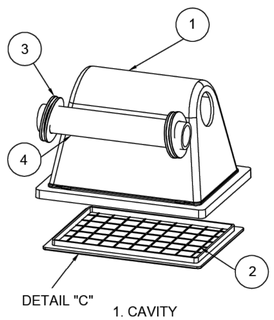


Section A-A



Section B-B

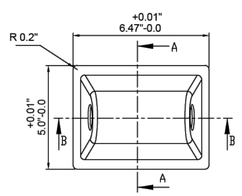
TYPICAL "B" PANEL 3 MSE-300
NOT TO SCALE

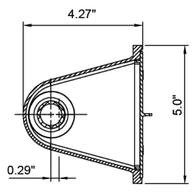


DETAIL "C"

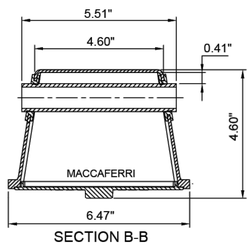
1. CAVITY
2. COVER FOR CAVITY
3. BUSH
4. SLEEVE

CONNECTION DETAIL





SECTION A-A



SECTION B-B

CONNECTION SECTIONS

PANEL TYPE	PANEL THICKNESS	MINIMUM/TYPICAL PANEL REINFORCEMENT
A	7 1/2" MIN.	VERTICAL BARS ARE #3 @ 8" O/C HORIZONTAL BARS ARE #4 @15" O/C
B	7 1/2" MIN.	VERTICAL BARS ARE #3 @ 8" O/C HORIZONTAL BARS ARE #4 @15" O/C

NOTES:
1. REBAR LAYOUT IS SHOWN ON DETAILS A AND B. WWR (ASTM A1064) MAY BE SUBSTITUTED, AT THE SAME SPACING, AS FOLLOWS:
#4 Rebar >> WWR Grade 60 W19.6
#4 Rebar >> WWR Grade 75 W15.7
#4 Rebar >> WWR Grade 80 W14.7
#3 Rebar >> WWR Grade 60 W11.1
#3 Rebar >> WWR Grade 75 W8.9
#3 Rebar >> WWR Grade 80 W8.3

2. ALL PANEL SHALL HAVE TWO BURKE 2-TON LIFTING ANCHORS.

3. ALL REINFORCEMENT SHALL BE EPOXY COATED OR DUAL COATED PER PROJECT SPECIFICATIONS.

PANEL REINFORCEMENT SCHEDULE 5 MSE-300
NOT TO SCALE

GEOSTRIP CONNECTION DETAIL 4 MSE-300
NOT TO SCALE

60% SUBMITTAL
DECEMBER 12, 2025

REVISION	MARK	DATE	DESCRIPTION	BY
B		12-06-2025	60% SUBMITTAL	JAF
A		06-27-2025	30% SUBMITTAL	JAF

APPROVAL

FILE NAME: G:\Drafting\PROJECTS_24000 to 25999_25022\WSE-WALL\60PercentSet\25022000001_MSE-300-121225.dwg LAYOUT NAME: MSE-300 PLOTTED: Friday, December 12, 2025 - 3:47pm USER: CCulbertson

PROJECT MANAGER: DATE
DEPICTION OF MONUMENTS: DATE
SURVEY PARTY CHIEF
SUBMITTED: DATE
SUPERVISING CIVIL ENGINEER R.C.E. EXP.
WATERSHED REVIEW: DATE
APPROVED: DATE
CITY ENGINEER R.C.E. EXP.
0 1 2 3
FOR REDUCED PLANS – ORIGINAL SCALE IS IN INCHES

DESIGN VZ
DRAWN LL
CHECK JAF
AS BUILT
HORIZ. NO SCALE
VERT. NO SCALE
BOOK
DATE 06/27/25

PARKS RECREATION & WATERFRONT
1947 CENTER ST., 5th FL. BERKELEY, CA 94704

ENGEO Expect Excellence

BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT
CITY OF BERKELEY, ALAMEDA COUNTY, CALIFORNIA
PANEL DETAILS

PLAN
FILE
MSE-300
SHEET OF

CALCULATION PACKAGE

25022.000.001
December 12, 2025

ENGEO
Expect Excellence



Berkeley Transportation Pier Ferry
Precast Panel Mechanically Stabilized Earth Seawall
ENGEO
12/12/2025
Software Version: RSWall 1.001

Project Summary

Project Name:	BERKELEY WATER TRANSPORTATION PIER FERRY
Analysis:	PRECAST PANEL MSE SEAWALL
Author:	VZ/JAF
Company:	ENGEO
Date Created:	12/12/2025

Project Settings

Units

Unit system:	Imperial, stress as psf

Wall Type

Wall type:	Segmental retaining wall

Design standard

Selected design standard:	AASHTO 2020
Base design standard:	AASHTO 2020

Segmental Wall Unit Properties

0+00 to 0+45 and 2+52 to 3+57

Height:	4.2 ft
Depth:	0.625 ft
Width:	357 ft
Center of gravity:	0.23 ft
Unit weight:	150 lbs/ft3

0+45 to 1+32 and 2+13 to 2+52

Height:	6.3 ft
Depth:	0.625 ft
Width:	357 ft
Center of gravity:	0.23 ft
Unit weight:	150 lbs/ft3

1+32 to 1+62 and 1+84 to 2+13

Height:	9.3 ft
Depth:	0.625 ft
Width:	357 ft
Center of gravity:	0.23 ft
Unit weight:	150 lbs/ft3

Soil Properties

Retained Backfill	
Unit weight:	125 lbs/ft3
Friction angle:	32°
Soil-structure friction angle:	21.3°
Long-term cohesion:	0 psf
Interface adhesion ratio:	0
Water confined:	No
Lightweight Cellular Concrete	
Unit weight:	50 lbs/ft3
Friction angle:	35°
Soil-structure friction angle:	23°
Long-term cohesion:	0 psf
Interface adhesion ratio:	0
Water confined:	No
Existing Fill	
Unit weight:	120 lbs/ft3
Friction angle:	0°
Soil-structure friction angle:	0°
Long-term cohesion:	500 psf
Interface adhesion ratio:	0
Water confined:	No
Rip Rap	
Unit weight:	135 lbs/ft3
Friction angle:	40°
Soil-structure friction angle:	26°
Long-term cohesion:	0 psf
Interface adhesion ratio:	0
Water confined:	No
Concrete Leveling Pad	
Unit weight:	150 lbs/ft3
Friction angle:	0°
Soil-structure friction angle:	0°
Long-term cohesion:	14400 psf
Interface adhesion ratio:	0
Water confined:	No

Reinforcement Properties

ParaWEB 2	
General	
Coverage:	33.2
Geosynthetic Properties	
Allowable tensile strength (static):	7455.61 lbs/ft
Allowable tensile strength (seismic):	10288.7418 lbs/ft
Creep factor:	1.38
Secant stiffness:	68521.7660918697 lbs/ft
Interaction Friction Factor Function	
F* at top:	2
F* at reference depth:	0.7
Reference depth:	20 ft
Alpha coefficient:	0.9
Interface sliding friction angle:	25.27°
Connection Properties Connection Strength Function (constant)	
Constant strength:	35073 lbs/ft

SEGMENT 1 (Stationing 0+00 to 0+45 and 2+52 to 3+57)

1. Strength Limit State

1.1. Summary

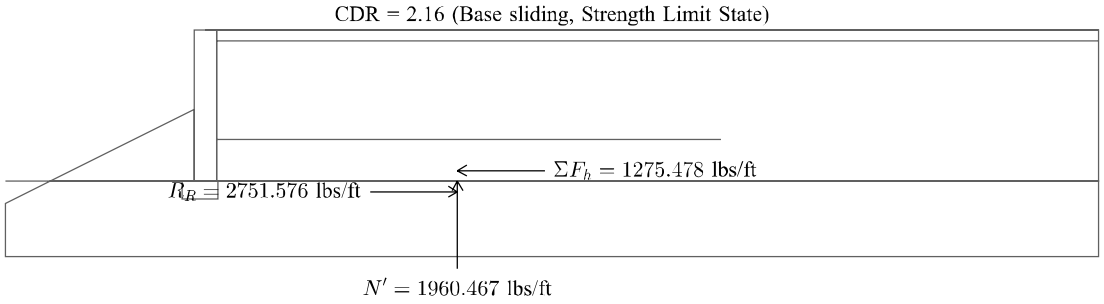
1.1.1. External stability - Strength Limit State

Failure mode	CDR
Base sliding	2.157
Overturning	2.405
Bearing	2.617

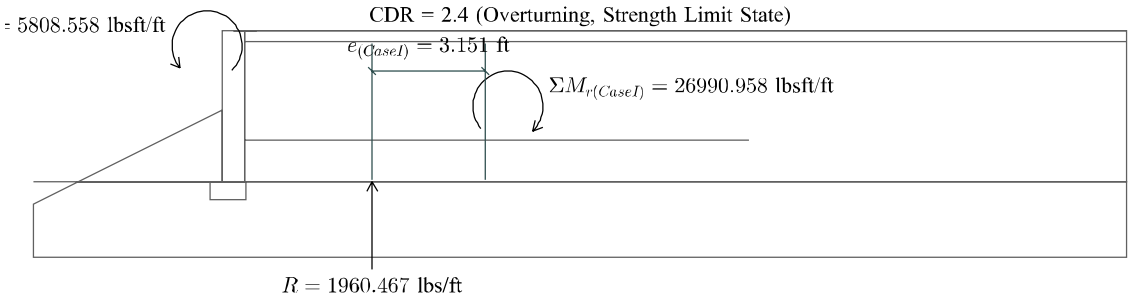
2.1.2. Reinforcement stability - Strength Limit State

Layer	Tensile strength	Connection	Pullout
1	2.19	2.19	4.139

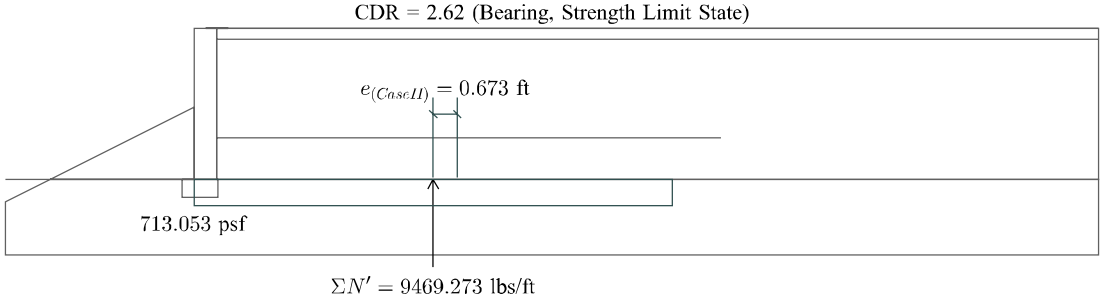
1.2. Base sliding



1.3. Overturning



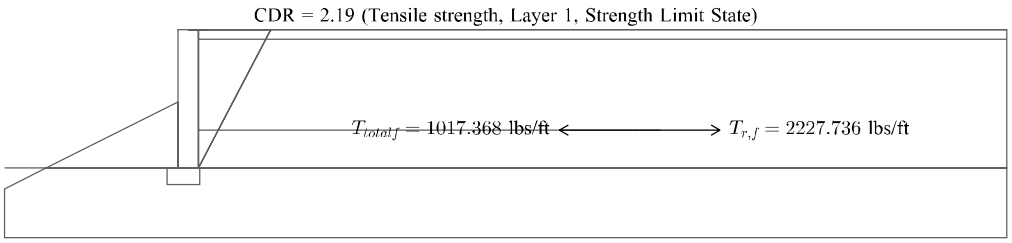
1.4. Bearing



1.5. Tensile strength

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

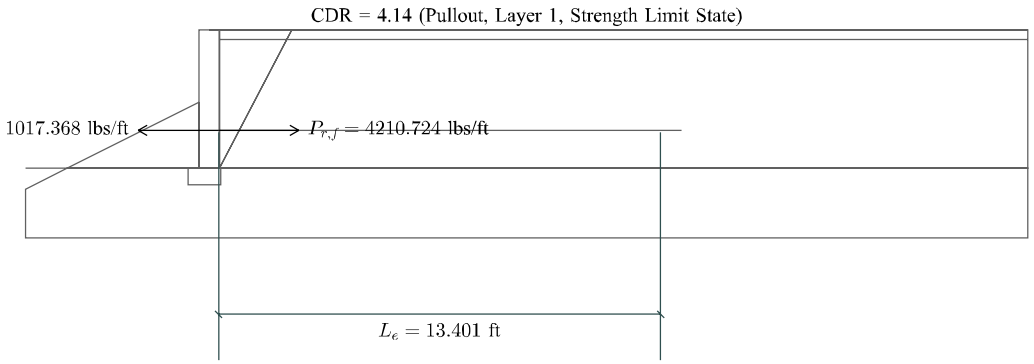
1.5.1. Tensile strength (layer 1)



1.6. Pullout

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

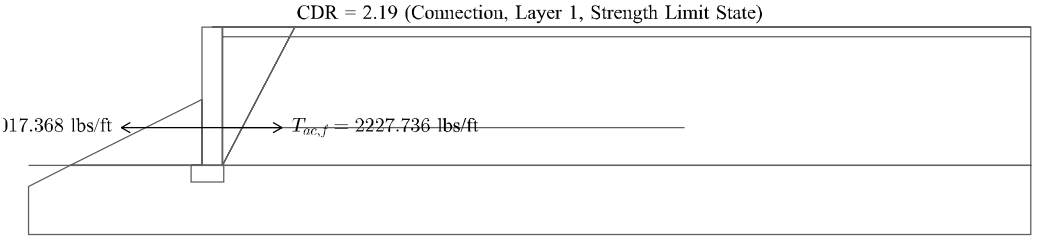
1.6.1. Pullout (layer 1)



1.7. Connection

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

1.7.1. Connection (layer 1)



2. Extreme Limit State

2.1. Summary

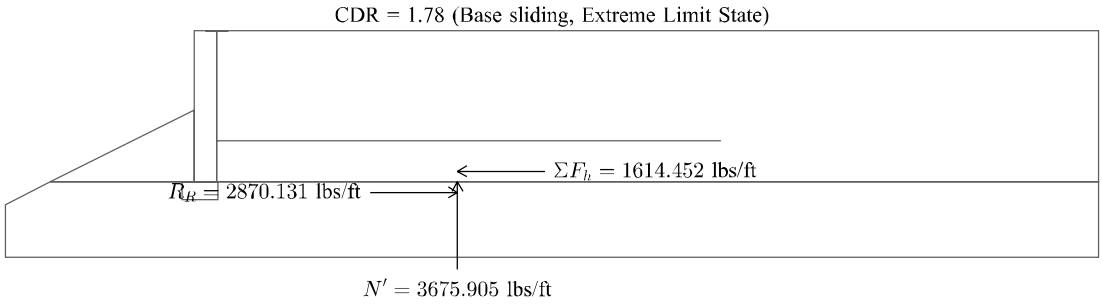
2.1.1. External stability - Extreme Limit State

Failure mode	CDR
Base sliding	1.778
Overturning	5.403
Bearing	8.373

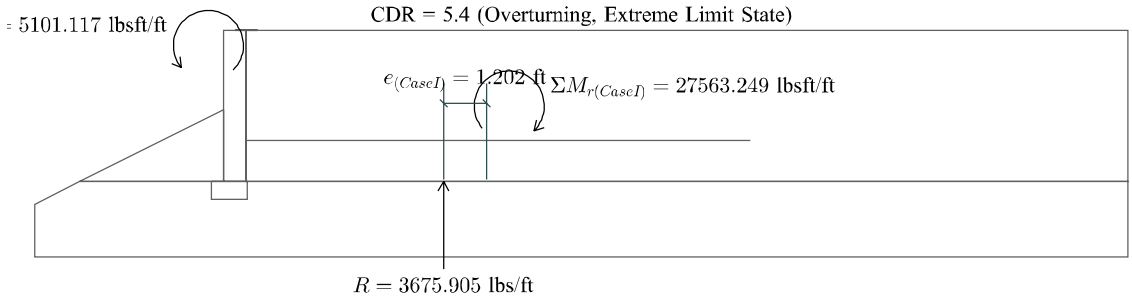
2.1.2. Reinforcement stability - Extreme Limit State

Layer	Tensile strength	Connection	Pullout
1	9.632	9.632	6.85

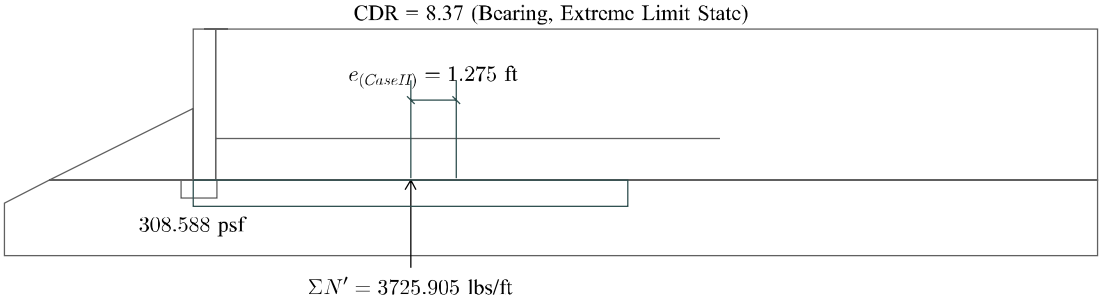
2.2. Base sliding



2.3. Overturning



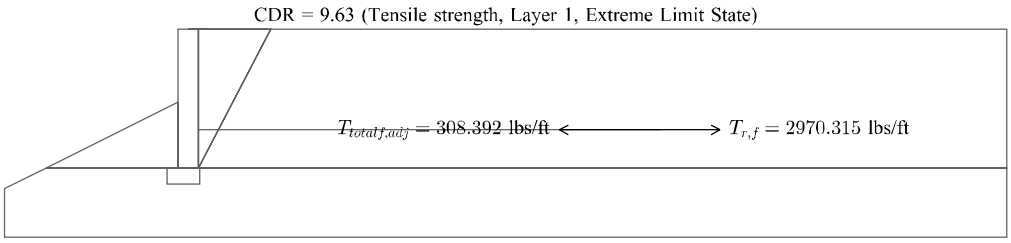
2.4. Bearing



2.5. Tensile strength

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

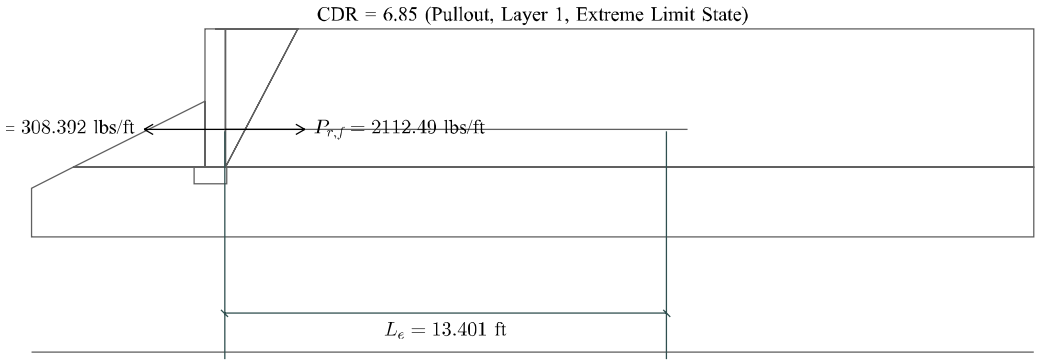
2.5.1. Tensile strength (layer 1)



2.6. Pullout

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

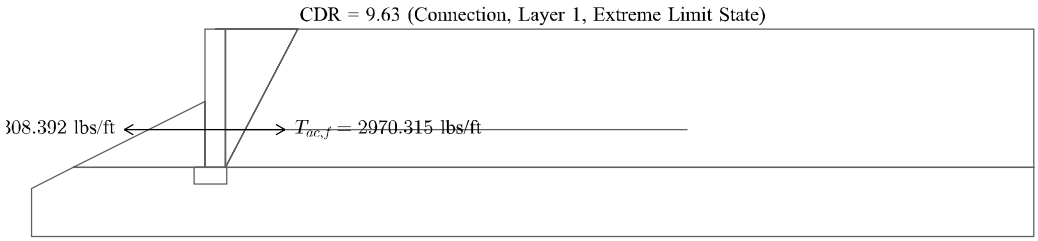
2.6.1. Pullout (layer 1)



2.7. Connection

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

2.7.1. Connection (layer 1)



SEGMENT 2 (Stationing 0+45 to 1+32 and 2+13 to 2+52)

1. Strength Limit State

1.1. Summary

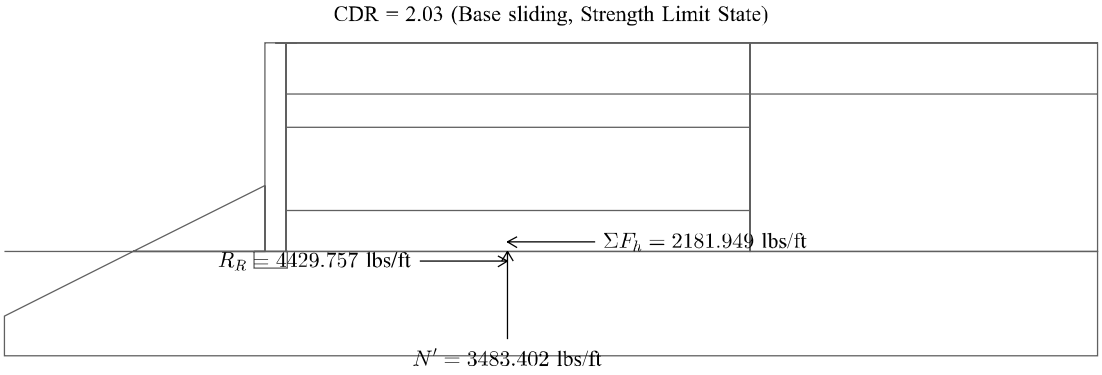
1.1.1. External stability - Strength Limit State

Failure mode	CDR
Base sliding	2.03
Overturning	1.934
Bearing	1.987

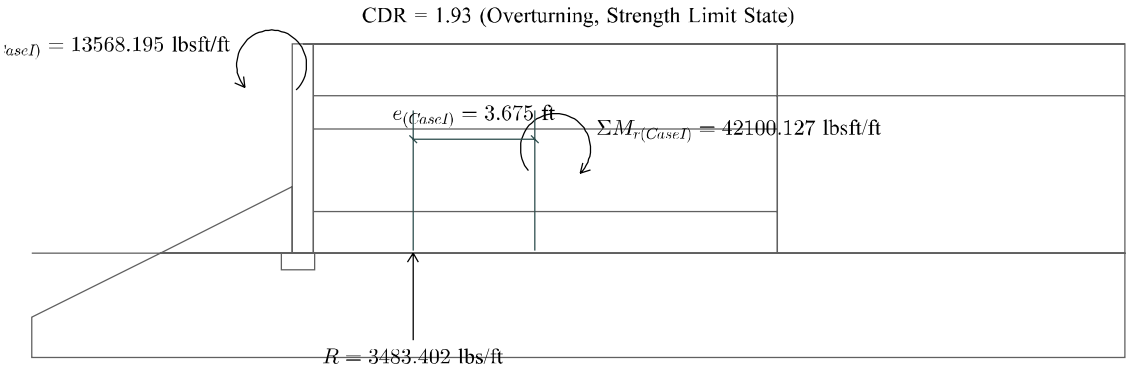
1.1.2. Reinforcement stability - Strength Limit State

Layer	Tensile strength	Connection	Pullout
1	3.156	3.156	8.011
2	2.931	2.931	2.971

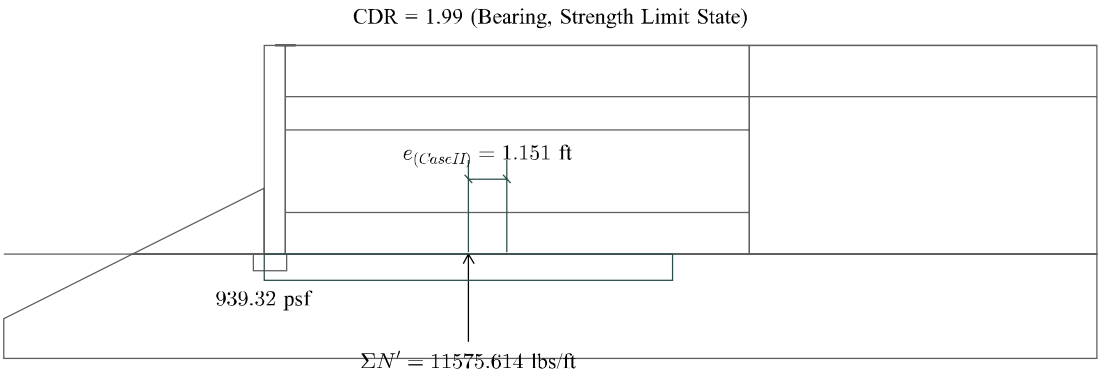
1.2. Base sliding



1.3. Overturning



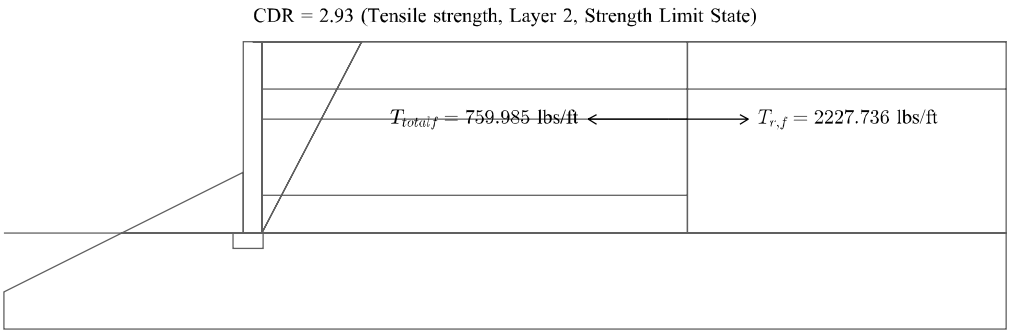
1.4. Bearing



1.5. Tensile strength

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

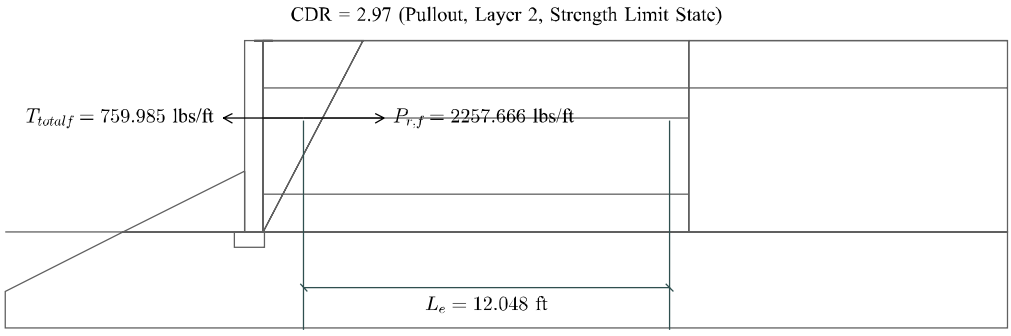
1.5.1. Tensile strength (layer 2)



1.6. Pullout

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

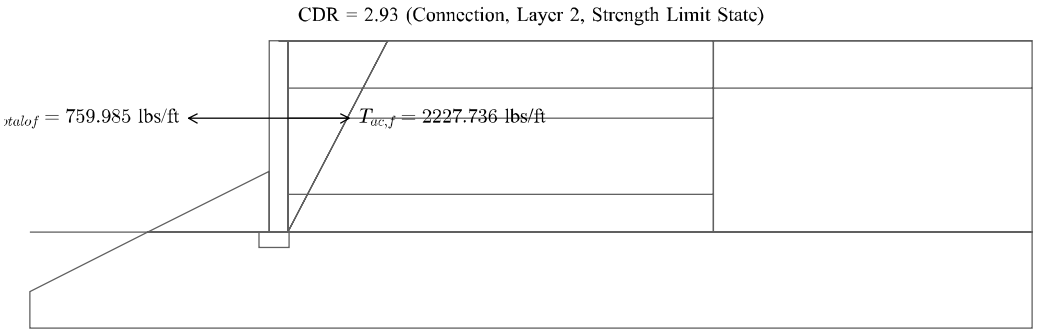
1.6.1. Pullout (layer 2)



1.7. Connection

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

1.7.1. Connection (layer 2)



2. Extreme Limit State

2.1. Summary

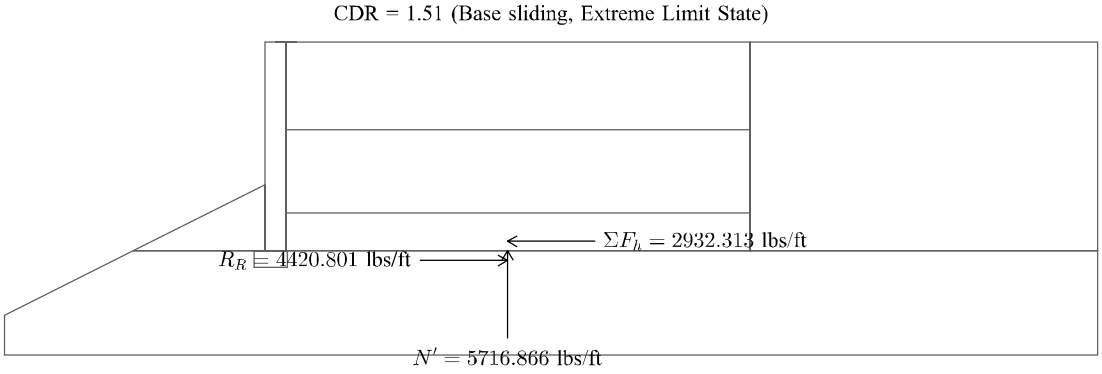
2.1.1. External stability - Extreme Limit State

Failure mode	CDR
Base sliding	1.508
Overturning	3.565
Bearing	4.956

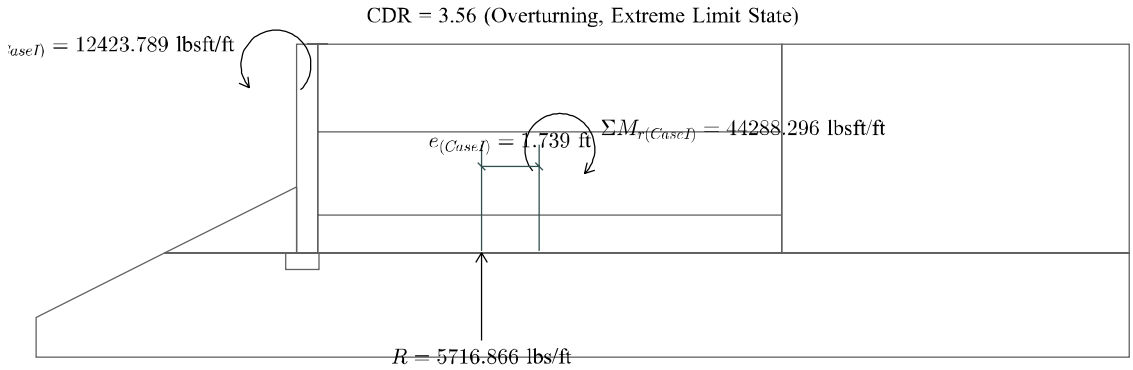
2.1.2. Reinforcement stability - Extreme Limit State

Layer	Tensile strength	Connection	Pullout
1	11.137	11.137	12.361
2	9.493	9.493	5.373

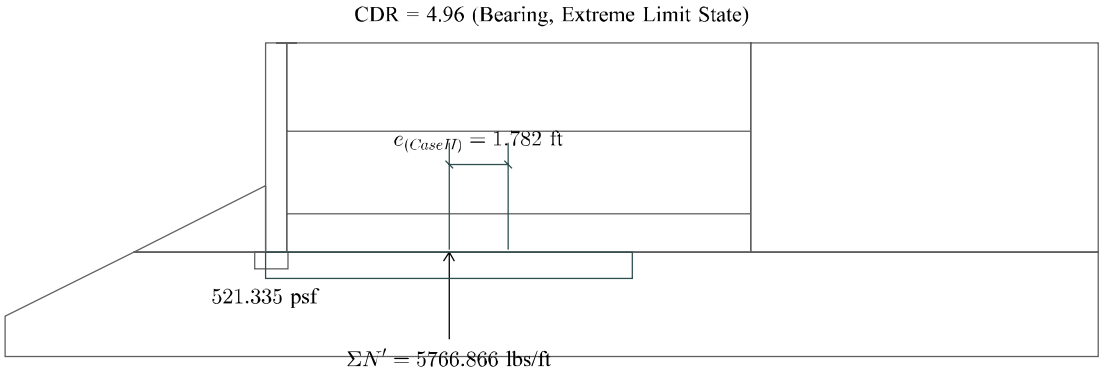
2.2. Base sliding



2.3. Overturning



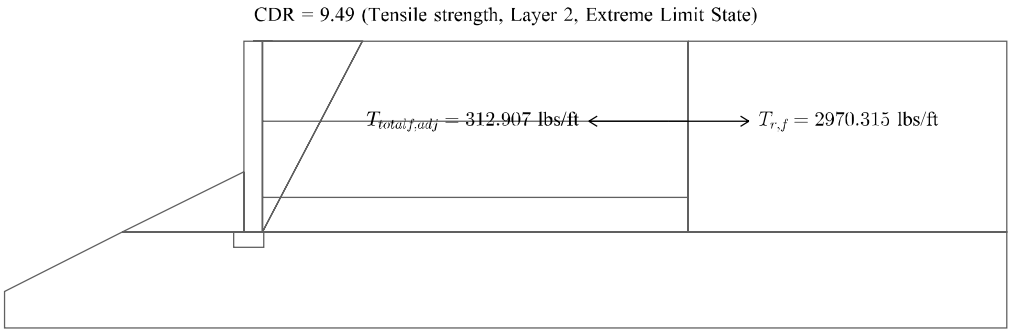
2.4. Bearing



2.5. Tensile strength

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

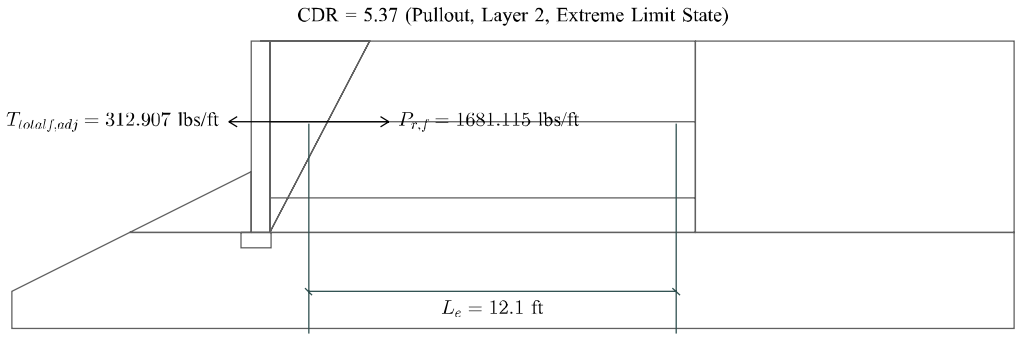
2.5.1. Tensile strength (layer 2)



2.6. Pullout

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

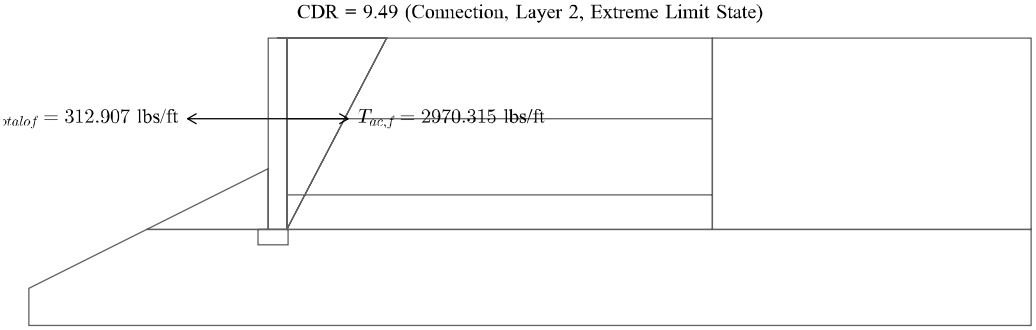
2.6.1. Pullout (layer 2)



2.7. Connection

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

2.7.1. Connection (layer 2)



SEGMENT 3 (Stationing 0+45 to 1+32 and 2+13 to 2+52)

1. Strength Limit State

1.1. Summary

1.1.1. External stability - Strength Limit State

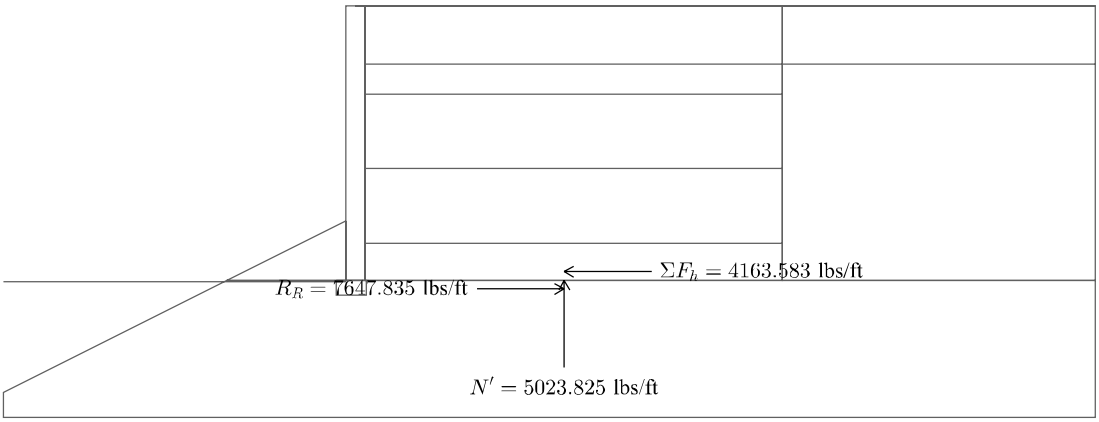
Failure mode	CDR
Base sliding	1.837
Overturning	1.235
Bearing	1.379

1.1.2. Reinforcement stability - Strength Limit State

Layer	Tensile strength	Connection	Pullout
1	2.313	2.313	8.538
2	3.092	3.092	7.269
3	2.631	2.631	2.591

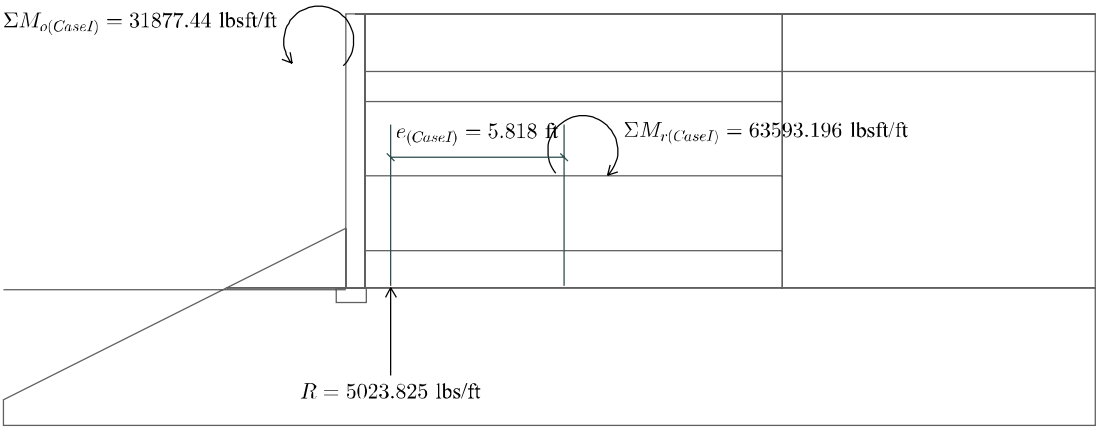
1.2. Base sliding

CDR = 1.84 (Base sliding, Strength Limit State)

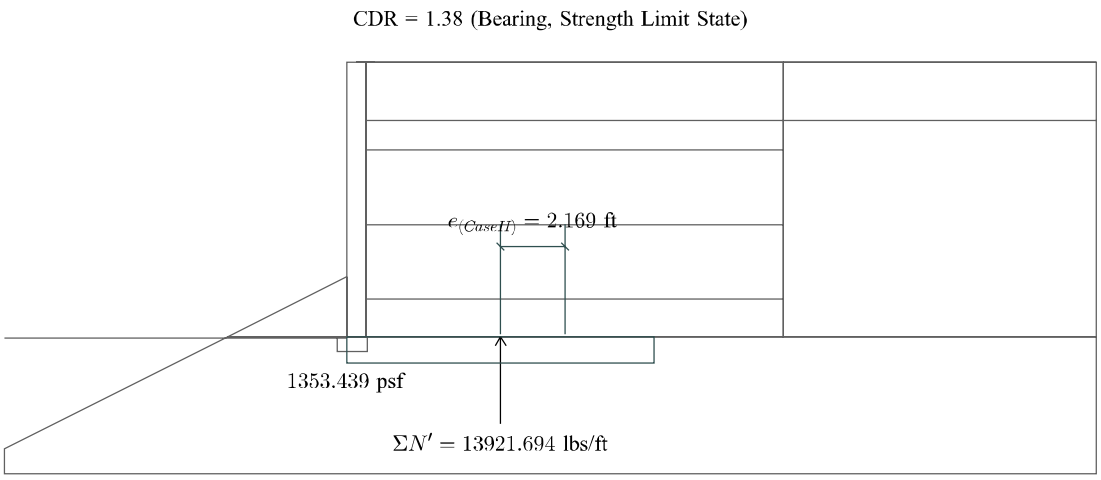


2.3. Overturning

CDR = 1.24 (Overturning, Strength Limit State)



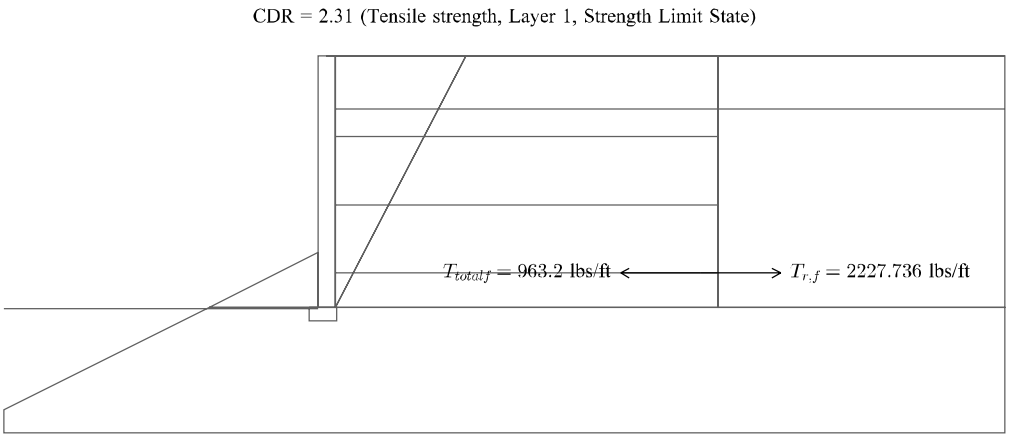
2.4. Bearing



2.5. Tensile strength

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

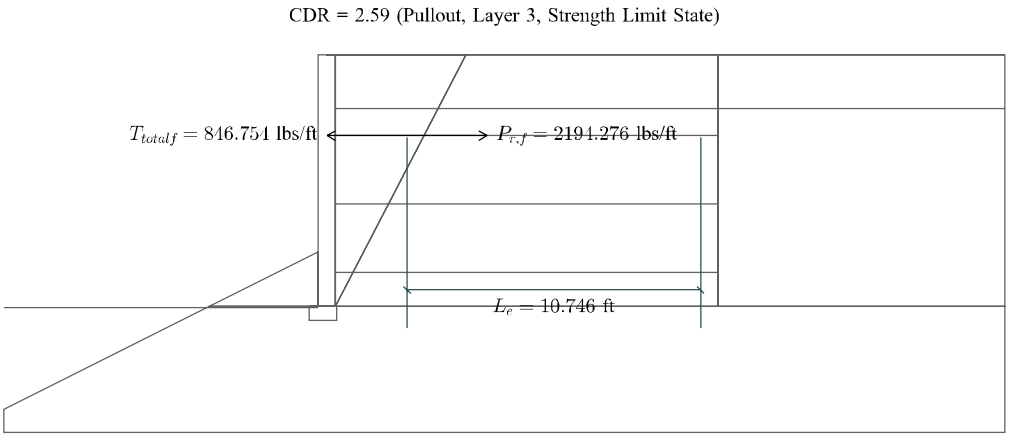
2.5.1. Tensile strength (layer 1)



2.6. Pullout

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

2.6.1. Pullout (layer 3)

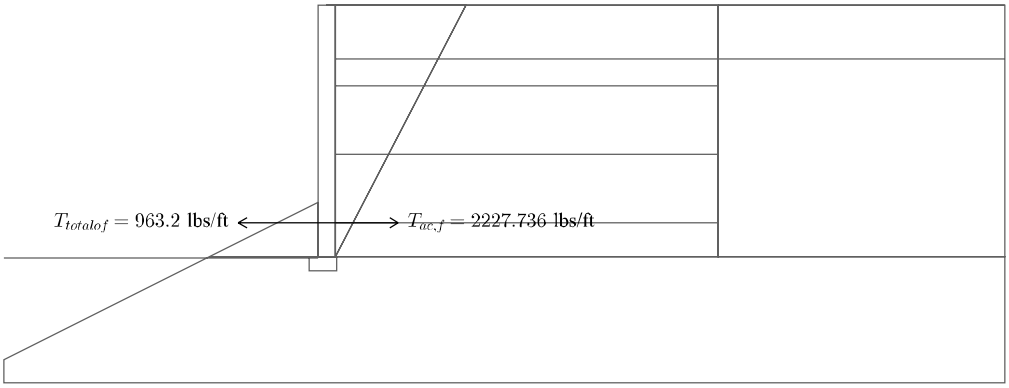


2.7. Connection

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

2.7.1. Connection (layer 1)

CDR = 2.31 (Connection, Layer 1, Strength Limit State)



3. Extreme Limit State

3.1. Summary

3.1.1. External stability - Extreme Limit State

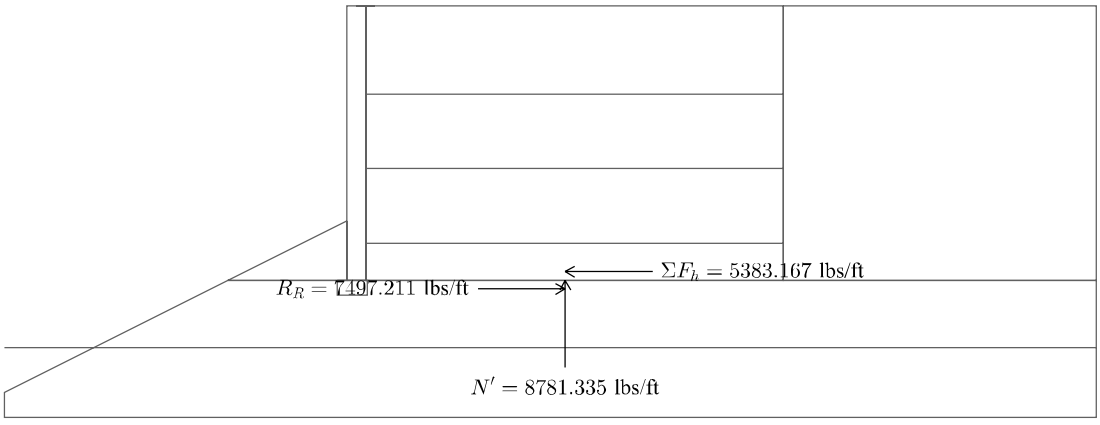
Failure mode	CDR
Base sliding	1.393
Overturning	2.391
Bearing	2.738

3.1.2. Reinforcement stability - Extreme Limit State

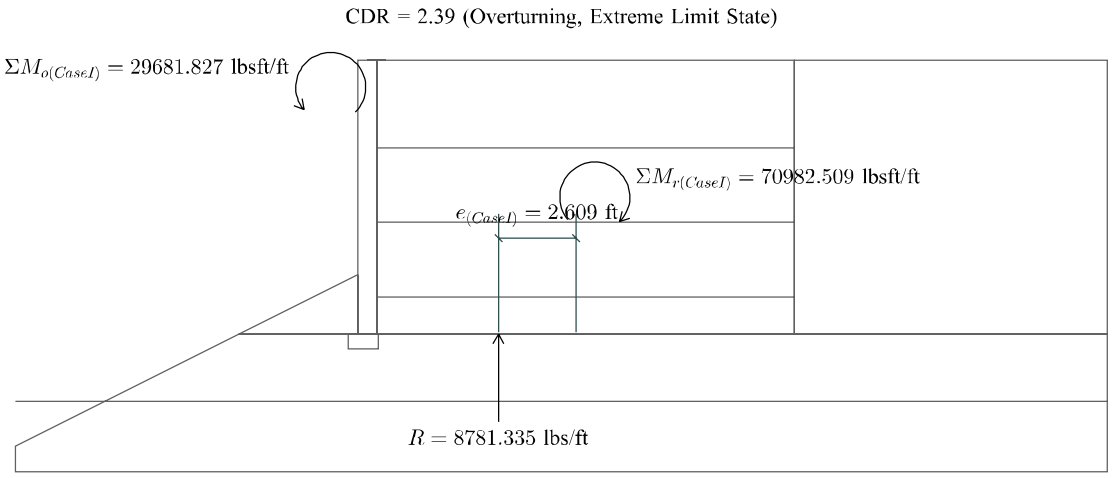
Layer	Tensile strength	Connection	Pullout
1	7.548	7.548	11.474
2	9.311	9.311	9.716
3	7.986	7.986	4.421

3.2. Base sliding

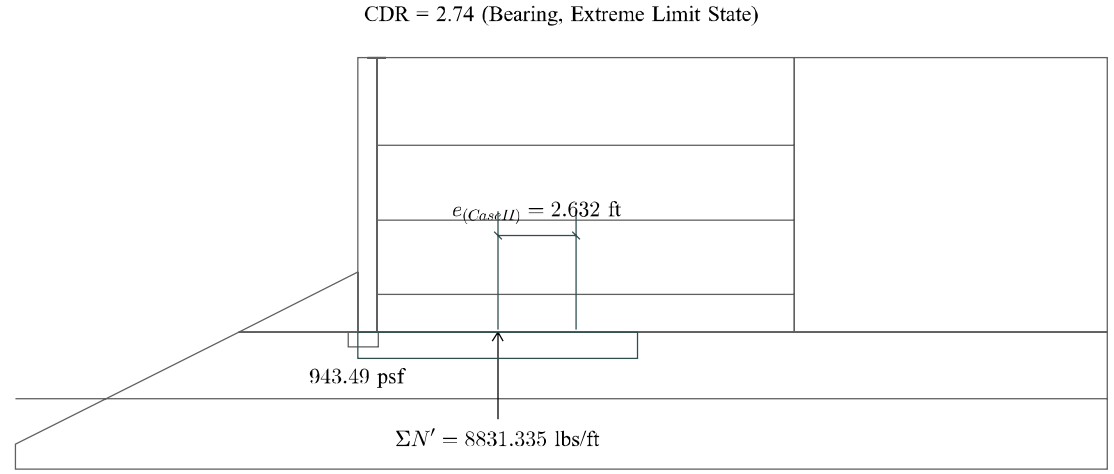
CDR = 1.39 (Base sliding, Extreme Limit State)



3.3. Overturning



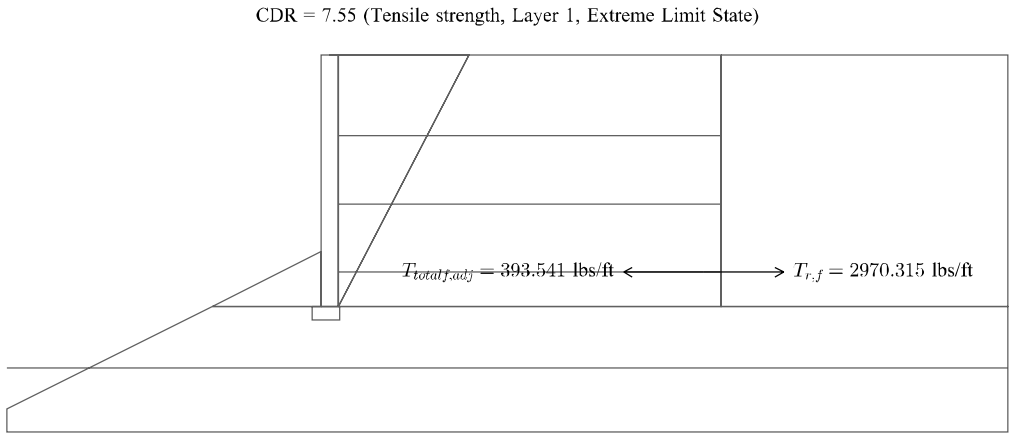
3.4. Bearing



3.5. Tensile strength

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

3.5.1. Tensile strength (layer 1)

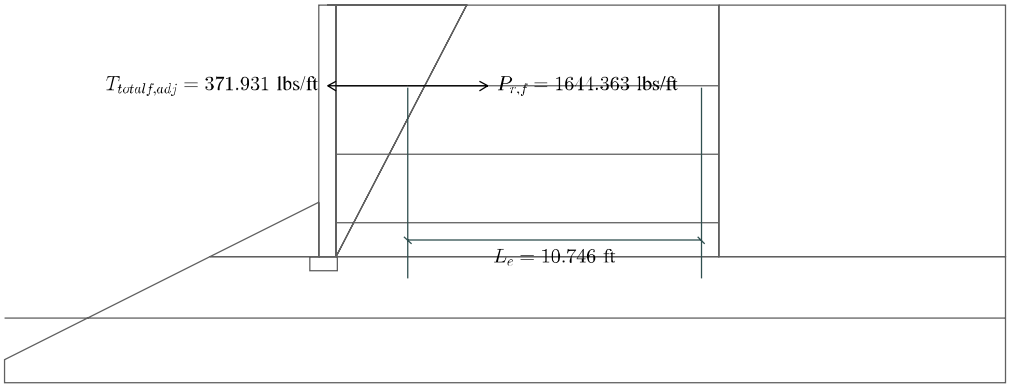


3.6. Pullout

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

3.6.1. Pullout (layer 3)

CDR = 4.42 (Pullout, Layer 3, Extreme Limit State)

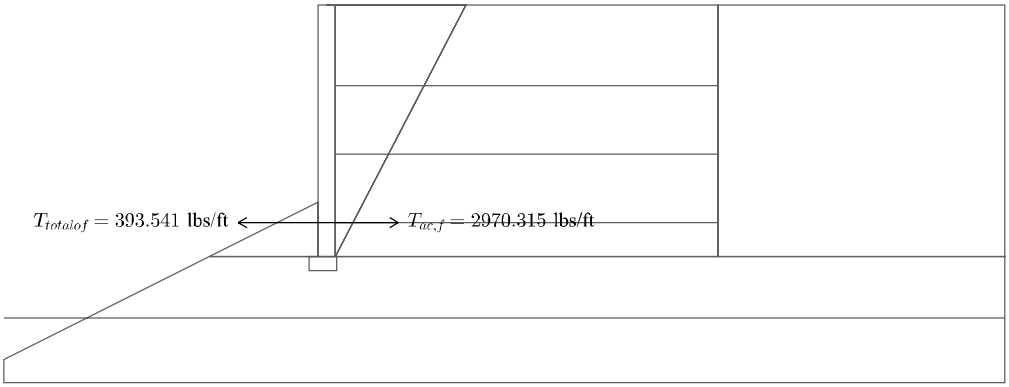


3.7. Connection

Note: Displaying result for critical layer only. See the summary section for a table of results for all layers, or enable display of all layers in the Display Options.

3.7.1. Connection (layer 1)

CDR = 7.55 (Connection, Layer 1, Extreme Limit State)





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APPENDIX B: VESSEL INFORMATION (PENDING)

DRAFT

APPENDIX C: PAVEMENT DESIGN

DRAFT

MEMORANDUM

Date:	December 8, 2025	Project Number:	1421.01.55
To:	James Connolly, PE, COWI North America, Inc.		
From:	NCE		
Subject:	Berkeley Water Transportation Pier Ferry Project – Pavement Design		

This memorandum presents the findings and recommendations for the pavement rehabilitation of University Avenue, Seawall Drive, and the parking lot at Seawall Drive which are part of the landside improvements for the Berkeley Water Transportation Pier Ferry (BWTPF) Project.

Background

One of NCE's responsibilities for the BWTPF project is to provide the pavement design. The geotechnical investigation, soil sampling, and soil laboratory testing and reporting has been provided by ENGEO, a subconsultant to COWI. NCE used the data and findings included in ENGEO's June 2025 Berkeley Water Transportation Pier Ferry Geotechnical Report to prepare the pavement design for the project.

The BWTPF project includes landside improvements to portions of the street network and parking lots at the Berkeley Marina. The improvement to the streets and pavements will be located along University Avenue from the roundabout at Marina Boulevard to the western terminus of University Avenue at Seawall Drive, Seawall Drive from the western terminus of University Avenue to the south end of Seawall Drive, and the paved path between the shoreline and the western edge of Seawall Drive, and will include the parking lot east of Seawall Drive. NCE is tasked to prepare the pavement design for the proposed transportation elements of the project, which include University Avenue, Seawall Drive, a section of Bay Trail between the shoreline, and the western edge of Seawall Drive and the parking lot east of Seawall Drive. University Avenue will be narrowed to fit an elevated cycle track along its southern University Avenue and Seawall Drive constitute approximately 0.5 centerline miles. The elevated cycle track along University Avenue and the section of Bay Trail constitute approximately 0.5 centerline miles. The parking lot constitutes approximately 72,000 square feet of permeable parking and bioretention, along with paved drive lanes.

Scope

To accomplish the pavement design portion of this project, NCE first performed a field visit to assess pavement conditions. NCE then performed pavement design using the 2018 Caltrans Highway Design Manual (HDM),

engineering judgment, and ENGEO's data and findings from their geotechnical investigation. The work performed and recommended designs are discussed in the following sections.

Field Investigation

Pavement Condition Survey

NCE conducted a pavement condition survey on University Avenue, Seawall Drive, the parking lot to the east of Seawall Drive and north of His Lordship's Restaurant, and the paved path along the western edge of Seawall Drive for the purpose of evaluating the existing pavement distresses and associated failure mechanisms.

1. Street Sections (University Avenue Travel lanes, University Avenue Terminus and Transit Area, Seawall Drive):
    a. University Avenue is in poor condition, exhibiting widespread alligator cracking, advanced block cracking, raveling, and weathering with localized areas of pumping and fatigue-related distortions, indicating the need for reconstruction. The wheelpaths exhibit depressions, pumping, and delamination of the existing asphalt concrete (AC) surface layer.
    b. Seawall Drive exhibits extensive raveling and weathering, high severity block cracking, and a failed longitudinal joint with intermittent patching. The loss of aggregate from the AC surface combined with widespread age-related cracking highlights the need for a reconstruction to provide a new structure able to sustain heavy loads and traffic planned for this area.
2. The parking lot (east of Seawall Drive and north of His Lordship's Restaurant) shows advanced stages of high severity alligator cracking with some depressions and potholes on the north side. The southern portion of the parking lot has high severity block cracking and weathering.

Representative images of these pavements is presented in Appendix A.

Pavement Coring

ENGEO, the geotechnical subconsultant on the COWI project team, performed a geotechnical investigation and pavement coring at various project locations. ENGEO performed geotechnical borings for the project and logged the pavement and aggregate base (AB) thicknesses at 13 locations related to pavement design for the project. ENGEO provided pavement cores on University Avenue, Seawall Drive and on the parking lot. The pavement coring and select lab testing results are summarized in Table 1 below. The pavement coring site plan and pavement coring logs are included in ENGEO's June 2025 Berkeley Water Transportation Pier Ferry Geotechnical Report. Additional test results are presented in ENGEO's report. NCE used the soil laboratory test results for the pavement design.

Table 1. Core information from ENGEO Geotechnical Report

CORE NUMBER	LOCATION	ASPHALT CONCRETE (AC) THICKNESS (inches)	AGGREGATE BASE (AB) THICKNESS (inches)	DEPTH TO PAVEMENT FABRIC* (inches)	MOISTURE CONTENT (%)	PLASTICITY INDEX	R-VALUE
1-C1	University Ave.	12	12	4¼	12.2	25	10
1-C2	University Ave.	9¼	12	2½	-	-	-
1-C3	University Ave.	4¼	12	2	14.3	20	9
1-C4	Pathway	2	8	None	10.9	17	-
1-C5	Pathway	3	6½	None	-	-	-
1-C6	Parking Lot	2½	9	None	-	18	17
1-C7/1-B4	Parking Lot	10½	6	None	-	-	-
1-C8	Parking Lot	4½	3	None	14.3	17	12
1-C9	Seawall Dr.	4¼	-	2½	-	-	-
1-CPT1/1-B2	University Ave.	4¼	5	None	-	-	-
1-CPT2	University Ave.	4¼	6	2¼	17.2	22	-
1-CPT3	Seawall Dr.	5	7	2¾	-	-	-
1-CPT4	Parking Lot	2	9	None	-	-	-
1-B2	Seawall Dr.	4¼	5	None	-	-	-
1-B3	Seawall Dr.	5	7	2¼	-	24	9
1-B5	Seawall Dr.	3½	7	None	-	-	-

* below ground

Pavement Design

Pavement was designed in accordance with the HDM Section 633.1 as well as engineering judgment. The HDM method, commonly referred to as the Hveem or R-value method, is an empirical method based on research and field experimentation by Caltrans. It provides for the calculation of pavement structural layer design thicknesses with design lives of 10 to 20 years and Traffic Indices (TI) ranging from 5 to 15 using subgrade R-values.

Traffic Index

NCE used the TI values from the previous Marina Streets Improvement project for University Avenue, along with TI estimates for the lower traffic routes. NCE estimated the 20-year design TI for each rehabilitation street and reached the values listed in Appendix B, Table 1 based on prior experience and usual traffic levels for each of the pavement sections. University Avenue and the terminus are expected to carry heavy bus traffic and the TI was

estimated to be 8.5, whereas Seawall Drive will have substantial car traffic with a TI of 7.0. The parking lot and bike paths (raised cycle track along University Avenue and Bay Trail) are assumed to have a TI of 5.0. These TIs will be confirmed with the City for the Final memorandum.

Conventional Pavement Reconstruction

Due to the relatively thin existing in-place pavement structures and the expected increases in use and traffic, reconstruction of the pavement sections is recommended. Attempting to rehabilitate the existing sections by milling the pavement and placing an AC overlay, keeping the existing aggregate base (AB), and supporting soil materials in place would be inadequate to handle future traffic loads. Traditional reconstruction by removing and replacing the AC surface and AB following removal of the existing pavement is one solution for these streets, but it would be costly. Design options are for traditional reconstruction are presented in Appendix B, Table 1.

Value Engineering Pavement Reconstruction

Throughout the design process, NCE explored value engineering options as efficient and cost-saving alternatives to traditional reconstruction or mill and overlay treatments. These value engineering options included full depth reclamation (FDR) and rubberized hot mix asphalt (RHMA). Design options are for value engineering FDR reconstruction are presented in Appendix B, Table 1.

Full Depth Reclamation (FDR)

FDR has been defined as a recycling method in which all or part of the AC pavement section and a predetermined depth of underlying materials are pulverized and treated to produce a stabilized base course. Stabilizing agents typically include portland cement and lime or a combination of the 2. The FDR process includes 1) pulverization, 2) addition and mixing of a stabilizing agent, 3) shaping of the mixed material, 4) compaction, and 5) placement of a surface or a wearing course such as an AC overlay or chip seal. If the in-place material is insufficient to provide an adequate depth for the treated base, new materials may be imported and included in the process. This recycling method is typical for depths ranging from 4 to 12 inches but can be used for depths of up to 19 inches.

The advantages of FDR include:

- Significant improvement to existing failing pavement structure
- Elimination of environmental and structural distresses of the AC pavement such as longitudinal and transverse cracking, reflective cracking, fatigue cracking, rutting, and potholes
- Restoration of pavement crown and slope
- Improvement to ride quality by removing irregularities and rough areas
- Accommodation of widening operations
- Low production and engineering costs
- Reduced environmental impact in terms of waste material

An FDR with cement stabilization (FDR-C) with AC overlay option has been presented for the project streets owing to the pervasive and severe nature of the distresses. This option minimizes construction in-haul and off-

haul and is likely much less costly, as well as "greener," than traditional reconstruction. FDR was recommended for University Avenue between the Frontage Road and Marina Boulevard as part of the Marina Streets Improvement project, and NCE recommends FDR for these streets.

Rubberized Hot Mix Asphalt (RHMA)

RHMA is asphalt material that includes recycled tires that have been ground up and added into the asphalt binder. The primary technique used today for combining recycled rubber and asphalt is the asphalt rubber process, also known as the "wet process." The wet process involves blending ground tire rubber and asphalt binder, either in a blending unit at the hot mix plant (asphalt rubber [AR] process), and then adding the blended mixture to the specified aggregate gradation to produce the final mix. Caltrans Standard Specification Sections 39 and 92 address the AR binder. AR is used in chip seals and gap graded (RHMA-G) and open graded (RHMA-O) mixes.

RHMA-G is generally used as overlays on pavements that are susceptible to reflective cracking. Numerous laboratory studies, field installations, and accelerated pavement testing programs, have identified advantages of properly designed and constructed AR and terminal blend systems. Such advantages include:

- Resistance to reflective cracking
- Reduction in pavement noise
- Improved driving safety
- Increased structural capacity
- Reduced environmental impact in terms of waste material

Since RHMA-G provides a higher resistance to reflective cracking than traditional AC, RHMA would be an appropriate option for streets with extensively cracked underlying layers. As structural cracks develop throughout the life of the pavement, they begin at the bottom of the surface layer and propagate upwards. Therefore, the use of RHMA-G would help minimize their impact at the surface.

NCE looked into the use of RHMA-G as it is a strong performing material and we frequently recommend its use. However, we do not recommend the use of RHMA-G in combination with FDR or for traditional reconstruction for this project. Since the FDR process will rebuild the supporting structure of the pavement and reflective cracking mitigation is not a concern for these streets, the need for RHMA-G is minimized. Similarly so with traditional reconstruction. Additionally, given the relatively low tonnage of asphalt material needed for this project, pricing of RHMA-G will likely be very high. If the city is interested in using this material, it could be used as a replacement for the top 2 inches of the AC overlay.

Recommended Rehabilitation Treatments

In addition to considering traditional design and value engineering design options, NCE also considered budget constraints, constructability, street condition, land use (i.e., bikes, transit, and commercial), grade constraints, and pavement defects (i.e., delamination, alligator/block cracking, pumping, raveling, and weathering). Considering these factors, NCE developed site appropriate and cost-effective reconstruction recommendations for the project.

Table 1 in Appendix B summarizes the recommended pavement reconstruction design options for the project. Two options are recommended: FDR-C and traditional reconstruction.

AC Reconstruction

This treatment is a form of structural reconstruction in which the existing AC and some subgrade is removed and replaced with a new AB layer and AC surfacing. This will result in a new structure with no distressed layers. Adding AB between the AC and subgrade provides structural support for vehicle traffic and resistance to subgrade movement or distortion resulting from moisture variation.

FDR-C Reconstruction

This treatment involves pulverizing and milling of the existing AC layer, followed by stabilizing the underlying aggregate base and subgrade layers with cement to create a strengthened base. This structure is then overlaid with an AC layer to provide a pavement with adequate thickness for the expected traffic loads. A stabilized subgrade also has the advantage of being more volumetrically stable and resistant to moisture cycling which mitigates against subgrade movement and reflective cracking in the pavement surface.

Construction Considerations

FDR-C

Previous experience at the Berkeley Marina has indicated that several precautions must be taken as part of the FDR-C construction process. Since the area on which these pavements reside is composed of fill, we anticipate issues with larger stones, bricks, pieces of concrete, etc. in the existing pavement structure that could cause problems with the pulverization. Typically, pieces larger than 3 inches must be removed from a site prior to FDR-C to avoid damaging equipment and to ensure the specified post-pulverization gradation is met. On the recent University Avenue project NCE had specified to “comb” thru the soil with a skeleton bucket to remove these oversized pieces, and this is recommended for these street sections as well.

Conventional AC Overlay

The following AC mix may be used for the conventional overlay: Type A AC mix, with ½-inch nominal maximum size aggregate (per Caltrans Standard Specification Section 39). The asphalt binder grade should be PG 64-10 (per Caltrans HDM). AC shall be spread and compacted in the appropriate number of layers needed, following maximum thickness guidelines in Section 39 of the Caltrans Standard Specifications.

Closure

We trust this memorandum provides the necessary information at this time. If you have further questions, please contact us at (510) 215-3620.

Appendix A

Pavement Condition Summary

Street Name: University Avenue





Distresses observed:

1) Widespread alligator cracking – medium and high severity
2) Block cracking – medium and high severity
3) Weathering – medium severity

Street Name: University Avenue Terminus and Transit Area





Distresses observed:

1) Wheelpath alligator cracking – medium and high severity
2) Block cracking – medium and high severity
3) Weathering – medium severity
4) Delamination and failed patches
5) Raveling – medium and high severity

Street Name: Seawall Drive





Distresses observed:

1) Wheelpath alligator cracking – medium and high severity
2) Block cracking – medium and high severity
3) Weathering – medium severity
4) Delamination and failed patches
5) Raveling – medium and high severity

Street Name: Parking Lot





Distresses observed:

1) Pervasive alligator cracking – medium and high severity
2) Block cracking – medium and high severity
3) Weathering – medium and high severity
4) Raveling – medium and high severity

Appendix B

Pavement Design Table

TABLE 1
Berkeley Water Transportation Pier Ferry Project
Draft Pavement Designs

Section	Street Name	Begin	End	Dir	Length (ft)	Width (ft)	Estimated Total Area (sy)	Lanes	Functional Class [1]	TI [2]	R-value	Existing AC Thickness (in)	Depth to Fabric (in)	AB Thickness (in)	Pavement Design HMA Reconstruction Option	Pavement Design FDR Reconstruction Option
1	Parking Lot - South (Parking Aisles only)	South End	350' North of Restaurant Structure	NA	-	-	-	1	Other	5.0	12	4.5	-	3.0	4.0 in HMA over 9.0 or 8.0 in AB	4.0 in HMA over 8.0 in FDR-C
2	Parking Lot - North (Parking Aisles only)	350' North of Restaurant Structure	North End of Parking Lot	NA	-	-	-	1	Other	5.0	17	2.5	-	9.0	4.0 in HMA over 8.0 in AB	4.0 in HMA over 8.0 in FDR-C
3	Seawall Drive	South End	University Ave	NB/SB	1,140	25	3,167	2	-	7.0	12	4.25	2.5	-	6.0 in HMA over 12.0 in AB	4.0 in over 12.0 in FDR-C
4	Seawall Drive Pathway (Bay Trail)	South End	University Ave	NB/SB	1,140	9	1,140	1	-	5.0	12	4.25	2.5	-	4.0 in HMA over 9.0 or 8.0 in AB	4.0 in HMA over 8.0 in FDR-C
5	University Ave Terminus and Transit	Seawall Drive	450' East of Seawall Dr	EB/WB	450	37	1,850	2	-	8.5	9	5.0	-	7.0	8.0 in HMA over 14.0 in AB	6.0 in HMA over 14.0 in FDR-C
6	University Ave Travel Lanes	450' East of Seawall Drive	Roundabout	EB/WB	1,380	37	5,673	2	-	8.5	9	4.25	-	12.0	8.0 in HMA over 14.0 in AB	6.0 in HMA over 14.0 in FDR-C

[1] Functional Class is based on the City's assigned classification in the City's StreetSaver database.

[2] The Traffic Index (TI) was estimated based on the traffic data provided by the City for the Marina Streets project in 2020 which will be verified for the final report. The TI strongly controls the pavement design recommendations for street overlay and reconstruction.

APPENDIX D: COASTAL STUDY

DRAFT

DECEMBER 2025
CITY OF BERKELEY

BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT

COASTAL STUDY

COWI

COWI

ADDRESS COWI North America
555 12th Street
Suite 1700
Oakland, CA 94607

DECEMBER 2025
CITY OF BERKELEY

BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT

COASTAL STUDY

PROJECT NO.

A272767

VERSION	DATE OF ISSUE	DESCRIPTION	PREPARED	CHECKED	APPROVED
A	27 June 2025	Draft for 30% Design	GAML / ARBV	BRLW	JRVS
B	12 December 2025	Final	GAML	BRLW	JRVS

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Executive summary

This report documents the results of the coastal engineering study conducted for the Berkeley Water Transportation Pier Ferry (BWTPF) Project. The study supports the design and planning of resilient ferry infrastructure in San Francisco Bay, incorporating assessments of wave and wind conditions, extreme water levels, sea level rise (SLR), wave-induced downtime, and sediment transport processes. The analysis aims to ensure long-term performance and adaptability of the terminal under current and future climate conditions.

Basis of study

The environmental setting was characterized through analysis of tidal datums, wind and wave climates, and current patterns in the vicinity of Berkeley Pier. NOAA tide gauge data and outputs from regional hydrodynamic models provided the foundation for defining water levels and currents. Wind data from both long-term regional (1996–2023) and short-term local stations were evaluated, with limitations identified in the representativeness of local wind data for offshore wave generation. Therefore, regional wind records and hindcast datasets were prioritized for driving wave models.

Water level conditions were established based on:

- NOAA tidal datums (1983–2001 epoch)00
- AECOM's 2016 updated extreme tide elevations for the Bay, and
- The 2024 California Sea Level Rise Guidance, using the Intermediate scenario and a 50-year project design life.

The adopted design still water level includes:

- 1% AEP storm tide elevation of 10.3 ft (Berkeley MLLW),
- A sea level rise allowance of 1.6 ft,
- Resulting in a design still water level of 11.9 ft (Berkeley MLLW).

The SLR vulnerability assessment evaluated structural performance under four future scenarios, considering both 2075 and 2100 planning horizons under Intermediate, Intermediate-High and High SLR trajectories.

Wave modeling and diffraction simulations

A two-tiered numerical modeling approach was used to simulate wave propagation and wave disturbance within the terminal:

- MIKE 21 SW was applied to generate a 27-year time series of nearshore wave conditions, incorporating NOAA hindcasts, buoy data, and spatially variable wind forcing.

- MIKE 21 BW, a Boussinesq-type diffraction model, simulated wave behavior within the terminal for three breakwater configurations (250 ft, 300 ft and 400 ft). The model produced wave disturbance coefficients at the berths and plaza for multiple wave directions and periods.

Model results confirm that the inclusion of a breakwater is essential for reducing wave agitation at the ferry berths and shoreline. The comparative analysis of breakwater lengths shows that increasing the structure from 300 to 400 feet yields only marginal improvements in wave disturbance and operational downtime. The 250-ft length alternative was also analyzed, it provides noticeably less sheltering, particularly at the outer berth, resulting in slightly higher agitation levels. The 300-ft breakwater configuration provides sufficient sheltering for both berths, minimizes wave exposure, and achieves nearly the same reduction in downtime as the longer alternative. This supports the selection of the 300-ft breakwater as the recommended option. Design wave heights for this configuration were extracted at critical locations for use in minimum structures elevation evaluations.

Structures elevation design and overtopping assessment

Minimum crest elevations for the breakwater, pier deck, and shoreline plaza were defined using extreme wave conditions, overtopping formulas from EurOtop (2018), and SLR allowances:

- Breakwater: +17.6 ft (Berkeley MLLW), meeting overtopping threshold of 0.1 l/s/m under design conditions.

- Pier Deck: Max green water levels from 14.2 to 15.2 ft (Berkeley MLLW) informed the deck elevation range.

- Plaza: Final elevation set at +16.5 ft (Berkeley MLLW), exceeding the minimum requirement of 15.8 ft to ensure safety under current conditions and future storm events.

A detailed vulnerability assessment confirmed that under all but the most extreme sea level rise condition (High Scenario), the plaza, pier and breakwater maintain functionality and public safety.

Wave-induced downtime assessment

Berth operability was evaluated using the 27-year wave hindcast data. Downtime was defined by significant wave heights exceeding 1.5 ft, consistent with operational criteria

established in consultation with San Francisco Bay Ferry and the City of Berkeley. Key results include:

- The no-breakwater layout experiences seasonal peaks in monthly average downtime, exceeding 2% in summer months.
- The 250 ft, 300 ft and 400 ft breakwaters reduce average monthly and peak monthly downtime to below 0.1% and 2%, respectively.

Sediment transport and maintenance dredging

A coupled wave–current–sediment transport model was developed using MIKE 21 to assess sediment deposition under existing and proposed dredged conditions. The model was calibrated against observed sedimentation in the Berkeley Marina Channel. Key findings include:

- Over a six-year simulation, the average annual sedimentation rate was:
  - 0.20 feet per year in the approach channel
  - 0.09 feet per year in the berth pockets
- Based on these rates, estimated intervals for maintenance dredging are:
  - Channel: every 10 to 15 years, depending on operational depth (-9 or -10 ft MLLW)
  - Berth pockets: every 22 to 33 years

Conclusions

This integrated coastal engineering study confirms that the proposed terminal layout, including a 300-foot breakwater and design elevations for the deck and plaza, meets operational and resilience objectives. The design accounts for future sea level rise, ensures safe public access under storm conditions, and minimizes long-term maintenance requirements. The adopted methodology reflects current coastal engineering best practices and California coastal planning guidance, offering a solid foundation for project implementation.

1 Introduction

The Berkeley Water Transportation Pier Ferry (BWTPF) Project is a major waterfront infrastructure initiative intended to enhance waterborne transit access along the San Francisco Bay. The project (BWTPF) aims to revitalize a significant piece of waterfront infrastructure in Berkeley, California, transforming the existing pier into a modern, dual-purpose ferry terminal and public pier.

The project involves the design and implementation of a new ferry terminal at the foot of Berkeley Pier, including associated breakwaters, floating berths, a trestle, and shoreline plaza features. Given the dynamic coastal environment and long-term exposure to climate change, this project requires a thorough understanding of site-specific coastal conditions.

This report presents the findings of a detailed coastal engineering study conducted to inform the terminal design and evaluate its long-term performance. The study applies best-practice numerical modeling and coastal analysis methods to assess environmental forcing factors, determine minimum structural elevations, and evaluate sediment transport dynamics. It also considers future sea level rise projections, operational reliability, and design resilience in alignment with the latest 2024 California sea level rise guidance. Key components of the study include:

- Characterization of environmental conditions at the project site.
- Long-term spectral wave modeling and high-resolution diffraction modeling to evaluate wave transformation across the project area.
- Development of design still water levels using extreme tide conditions and sea level rise allowances consistent with the 2024 California Sea Level Rise Guidance.
- Calculations of minimum crest and deck elevations for breakwaters, deck, and shoreline infrastructure using wave overtopping and runup criteria.
- Evaluation of ferry berth downtime due to wave and wind conditions based on long-term hindcast data.
- Hydrodynamic and sediment transport modeling to assess the impact of the proposed dredging layout on sediment deposition and maintenance requirements.

The modeling results and vulnerability assessments serve to verify that the proposed terminal layout is resilient under both current and projected future conditions, including sea level rise and extreme weather events. The study supports the selection of a 300-ft breakwater as the recommended configuration, balancing effective wave protection with efficient use of resources. The findings provide a technical basis for design decisions, ensuring the terminal infrastructure will deliver reliable service and remain resilient under current and projected future conditions.

2 Basis of study

This section outlines the environmental parameters that form the basis for the coastal study. These conditions influence the physical processes affecting coastal infrastructure and natural systems. They are important for accurate assessments and modeling. The data and methodologies used to characterize water levels, currents, winds, and waves are presented herein, providing the foundation for analyses of coastal dynamics and design conditions.

2.1 Water levels

2.1.1 Tidal datum

Water levels were retrieved from the Berkeley tidal gauge (ID 9414816), which is the closest station to the project site, and reference information provided by the National Oceanic and Atmospheric Administration (NOAA) benchmark sheet. Table 2-1 presents the water levels recorded at the station which form the basis for our project site. Tidal data is per NOAA averages based on the 1983-2001 tidal epoch.

Note: City of Berkeley MLLW vertical datum differs from NOAA MLLW. This project adopted the City of Berkeley datum and refers to it as “MLLW” throughout this document. The conversion between datums is presented in Appendix A (eTrac, 2024).

Table 2-1 Tidal datum at Berkeley station, CA (ID 9414816)

Datum (NOAA)	Value [ft NAVD88]	Value [ft-Berkeley MLLW]	Description
HAT	7.8	8.4	Highest Astronomical Tide
MHHW	6.2	6.8	Mean Higher-High Water
MHW	5.6	6.2	Mean High Water
MSL	3.4	4.0	Mean Sea Level
MLLW	0.1	0.7	Mean Lower-Low Water
NAVD88	0.0	0.6	North American Vertical Datum of 1988
LAT	-2.1	-1.6	Lowest Astronomical Tide

2.1.2 Extreme water levels

A study done by (AECOM, 2016) - San Francisco Bay Tidal Datums and Extreme Tides Study - updated and expanded the USACE 1984 study of tidal datums and extreme tides for San Francisco Bay. The study modelled and presented detailed extreme tide information for the entirety of the Bay shoreline. The model captures the Bay's hydrodynamic conditions at a scale and resolution that exceeded available tide gauge data.

Table 2-2 reproduces the extreme tide elevation for point ID 519, coinciding with the Berkeley Pier location.

Table 2-2 Extreme tide elevation – ID 519 (AECOM, 2016)

Return Period (years)	Extreme Tide Elevation [ft NAVD88]	Extreme Tide Elevation [ft MLLW]
1	7.4	8.0
10	8.5	9.0
50	9.3	9.9
100	9.7	10.3

The extreme water level considered for this project is 10.3 ft MLLW, with a return period of 100 years (1% Annual Exceedance Probability - AEP).

2.1.3 Sea Level Rise (SLR)

The BWTPF Feasibility Study (GHD, 2023) was based on the 2018 California Sea-Level Rise Guidance, which relied on probabilistic projections informed by the IPCC Fifth Assessment Report (AR5). Since then, scientific advancements, including improved climate models, better ice-sheet dynamics understanding, and refined regional projections, have prompted an update. The 2024 California Sea Level Rise Guidance (OPC, 2024) integrates findings from the IPCC Sixth Assessment Report (AR6) and the Special Report on the Ocean and Cryosphere in a Changing Climate (SROCC), offering deeper insights into drivers of SLR such as thermal expansion, glacier melt, and ocean circulation shifts.

The updated guidance recommends a stepwise process for incorporating SLR into planning, ensuring adaptation efforts are methodical and responsive to new science. For the BWTPF Project, defining SLR allowances is critical for determining minimum structural elevations for the breakwater, pier, and shoreline plaza, ensuring resilience and safety throughout the infrastructure's design life.

The BWTPF Project adopts a 50-year design life (2025–2075) and uses the Intermediate Scenario from the 2024 California Guidance as the baseline for design. This scenario represents a precautionary yet practical estimate, balancing risk tolerance and cost-effectiveness.

Tide gauge selection

Projections from the San Francisco tide gauge indicate higher relative sea level rise compared to Alameda, which supports the decision to use it for setting minimum structural elevations. This conservative approach accounts for regional variability and potential underestimation of localized impacts if a lower-projection gauge were used.

As shown in Table 2-3, the differences between projections from the two gauges can be significant. For the year 2100, San Francisco projections exceed those of Alameda by approximately 40% under the low-end scenario and by about 5% under the high-end scenario. These variations underscore the importance of selecting a tide gauge that supports risk-averse design decisions, particularly for long-term infrastructure.

Table 2-3 Differences for Sea Level Rise allowance (Year 2100) for San Francisco and Alameda tide gauges – relative to year 2000

Sea Level Rise Scenarios for Year 2100, in feet					
Tide Gauge	Low	Int-Low	Intermediate	Int-High	High
San Francisco	1.0	1.6	3.1	4.8	6.5
Alameda	0.6	1.2	2.8	4.4	6.2
Difference	-40%	-25%	-10%	-8%	-5%

The design sea level rise for the project, given a 50-year design life (2025-2075) and Intermediate risk scenario, is 1.6 feet.

By adopting this scenario and tide gauge, the BWTPF Project ensures that critical infrastructure remains functional and safe through its intended lifespan without reliance on future retrofits, while aligning with state and regional climate resilience policies.

A more detailed description of the allowance selection and vulnerability assessment under more extreme scenarios (Intermediate-High and High) is provided in Section 5.5 and Appendix B.

2.1.4 Still water level

The still water level at the project location is shown in Table 2-4. This elevation excludes waves impacts but includes astronomical tides, storm surge and sea level rise allowance for the design life of the project.

Table 2-4 Design still water level

Parameter	Elevation [ft NAVD88]	Elevation [ft-Berkeley MLLW]
Extreme Water Level (1% AEP)	9.7	10.3
Sea Level Rise Allowance	1.6	1.6
Design Still Water Level	11.3	11.9

2.2 Currents

Current speeds in the vicinity of the Berkeley Pier are generally low, with variability influenced by tidal cycles and local wind conditions. While direct measurements in the immediate project area are limited, regional datasets suggest that nearshore current speeds are typically below 1 knot under normal tidal conditions. A detailed assessment of current patterns and speeds, including spatial and temporal variability, is presented in Section 6 and Appendix D, where a hydrodynamic model is used to simulate fields across the study area as one of the inputs for the sediment transport model.

2.3 Wind

2.3.1 Long-term wind data

The San Francisco Bay region is characterized by varied topography and water bodies which cause significant variations in the wind speed and directionality inside and outside the Bay area. Figure 2-1 shows annual wind roses at four measurement stations where data are available from 1997-2023 and indicates that the wind speed and directionality in the approaches to the project site can vary considerably from the offshore region to the inside of San Francisco Bay.

While the offshore region winds reach up to 30 knots and are mainly from WNW-NW direction, wind speeds measured inside the San Francisco Bay are generally 10-15 knots with significant variation in the directionality, depending on the location of measurements.

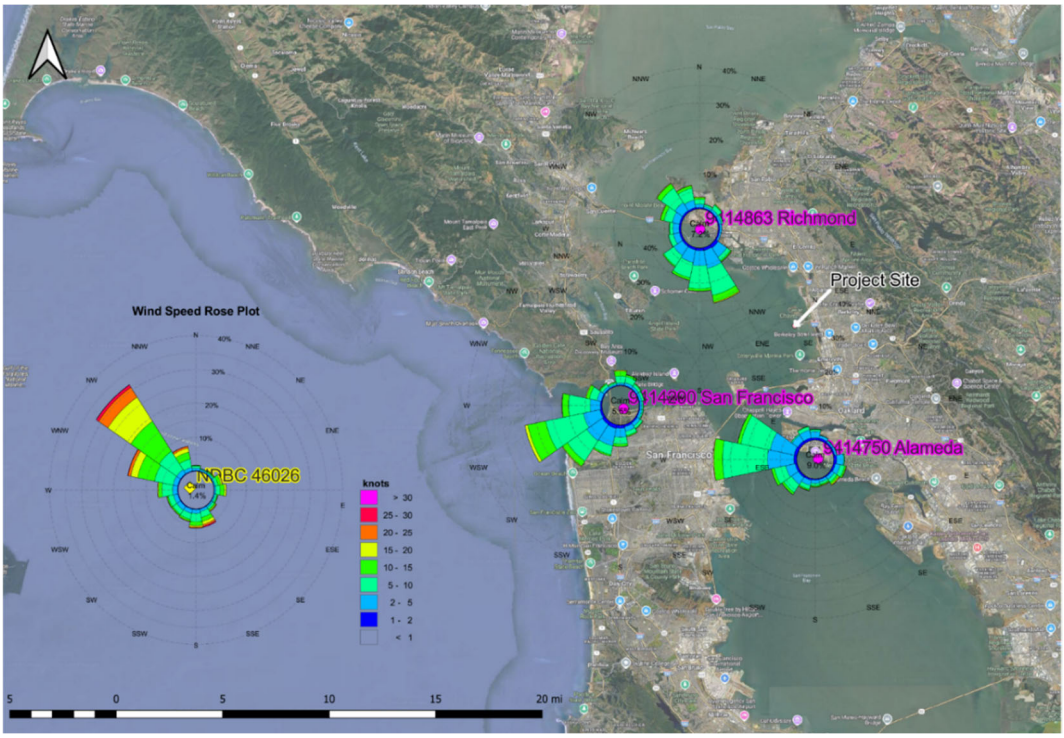


Figure 2-1 Wind roses from measured data in the vicinity of the project site.

Within San Francisco Bay, data from U.S. National Center for Environmental Information (NCEI)/ NOAA stations were analyzed. Station ID 9414290 – San Francisco is located west of the project site and Station ID 9414863 – Richmond is located north. Winds at San Francisco are seen to be predominately from SW-W directions (conforming to the geographical features of the area), whereas at Richmond, the winds are predominantly from S-SSE and from NW. At Station ID 9414750 – Alameda, located south of the project site, winds are from W-WNW directions.

Wind data from stations near the project site, shown in Figure 2-2, were also used to ascertain the variation in directionality near the project site.

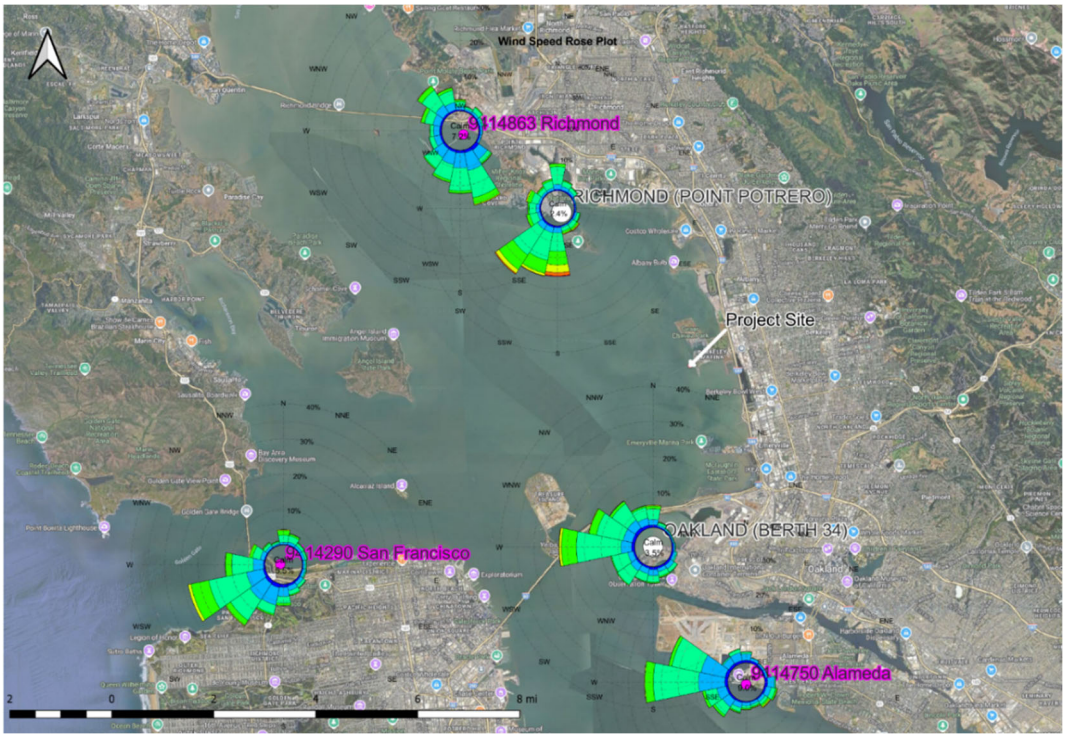


Figure 2-2 Wind roses from measured data within the San Francisco Bay near the project site.

2.3.2 Local wind data

Local wind data were available from two stations, as shown in Figure 2-3 and Table 2-5.

At the Berkeley Marina station, winds are distributed across a broad directional range, with dominant flows from the southwest to west sectors. Average wind speeds are generally moderate, mostly within the 3- to 12-knot range, and the wide directional spread indicates significant influence from local shoreline orientation, urban development, and terrain effects. This variability reflects the complex wind environment near the shoreline, where small-scale features can disrupt the broader regional wind pattern. The Berkeley Marina dataset spans less than six years (Jan 2014 to Dec 2019), which is insufficient for establishing robust climatological trends or for use in statistical analyses, especially in support of wave modeling.

The Reef Light station has prevailing winds from the west-southwest to southwest. Compared to Berkeley Marina, Reef Light experiences a greater frequency of stronger winds (particularly those exceeding 12 knots) and exhibits a narrower directional spread for prevailing winds. While this indicates better alignment with regional wind patterns, it is still influenced by its proximity to land and the limited fetch available at the measurement location. This dataset is also limited to about one year (Nov 2018 to Dec 2019), which restricts its reliability for long-term wind characterization and for

estimating extreme wind events. The annual wind rose for the Berkeley Marina (left) and Reef Light (right) stations are shown in Figure 2-3.

Table 2-5 Period of measurements for local wind data

Station	Period of Measurements
Berkeley Marina	Jan-2014 to Oct-2019
Reef Light	Nov-2018 to Dec-2019

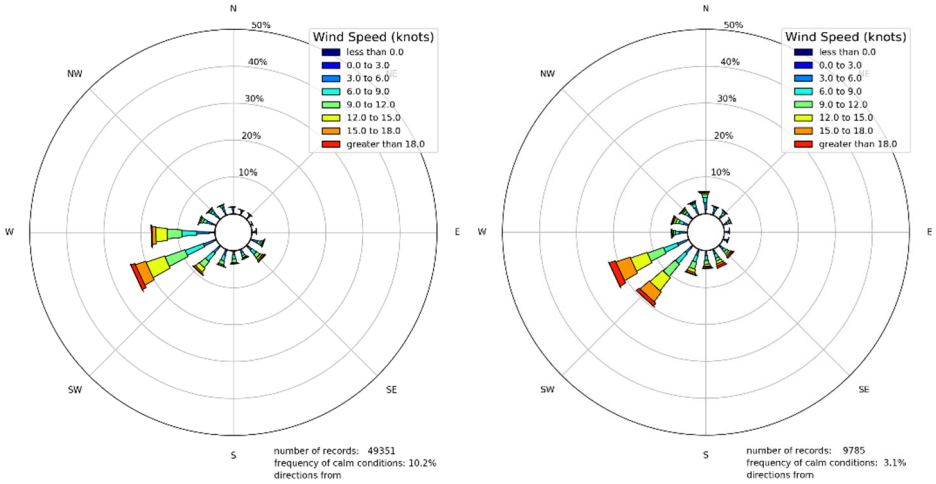


Figure 2-3 Annual wind rose for Berkeley Marina (left) and Reef Light (right).

Despite the differences between these two sites, neither station provides wind data that is fully representative of open-water conditions required for wind wave generation modeling. Both locations are constrained by limited fetch, short measurement durations, and proximity to complex coastal terrain, all of which reduce their suitability for driving wind wave models aimed at capturing large-scale wave dynamics. Their use should therefore be limited to local-scale assessments or for validating near-shore model behavior. In contrast, long-term offshore wind datasets should be prioritized for wave modeling and hindcasting.

2.3.3 Wind seasonality

Winds in the San Francisco Bay region show pronounced seasonal variability in both speed and direction, influenced by large-scale atmospheric circulation patterns and the region's complex topography.

The seasonal wind patterns were observed at long-term monitoring stations (San Francisco, Alameda, and Richmond) and short-term (Berkeley Marina and Reef Light).

Figure 2-4 to Figure 2-8 show the differences in wind roses between the month of June, the month of December, and the full year (annual) wind rose.

During the spring and summer months, the regional wind field is dominated by strong, persistent winds from the west to southwest, particularly at San Francisco and Alameda. These winds might be driven by the formation of inland thermal lows that enhance the onshore pressure along the California coast. Wind speeds during this period frequently exceed 12 knots, and in exposed areas such as San Francisco, can regularly reach or exceed 18 knots. This seasonal pattern is responsible for the most energetic wind forcing observed in the region and contributes substantially to wave generation and transport processes in the Bay and adjacent offshore areas.

Autumn and winter months are characterized by a weakening of the inland thermal gradient, leading to reduced wind speeds and greater directional variability. Across most stations, including local sites such as Berkeley Marina and Reef Light, wind roses show a broader spread during these months, with increased occurrences of southerly, southeasterly, and even northerly winds.

The Richmond station consistently deviates from the dominant west-to-southwest wind patterns observed at other locations. Across all seasons, Richmond displays a bimodal wind regime, with dominant winds from the S–SSE and N–NNW directions. This pattern is likely driven by the orientation of local topographic features and channeling effects within the northern Bay, highlighting the spatial heterogeneity of wind conditions across the region.

These seasonal trends have direct implications for wave modeling, coastal exposure assessments, and the planning of marine operations, and must be accounted for when evaluating wind-driven processes within the Bay and surrounding coastal waters.

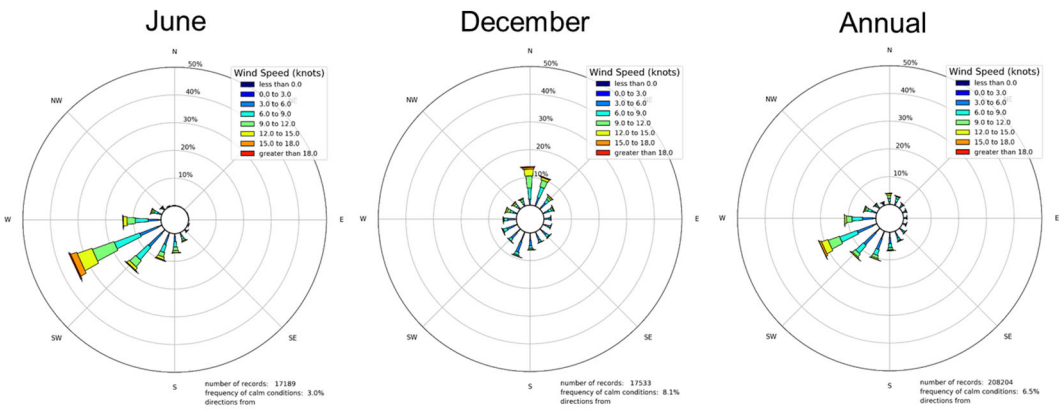


Figure 2-4 San Francisco Station wind roses (1997-2023) for June, December and the full year.

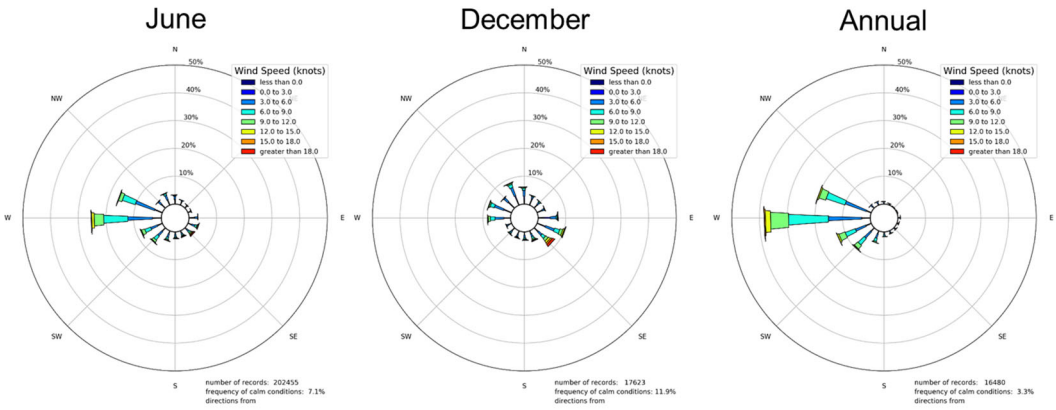


Figure 2-5 Alameda Station wind roses (1997-2023) for June, December and the full year.

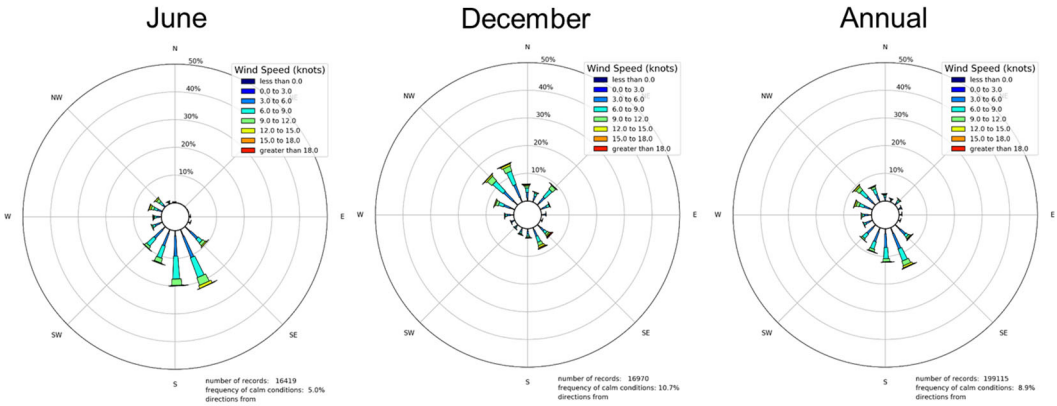


Figure 2-6 Richmond Station wind roses (1997-2023) for June, December and the full year.

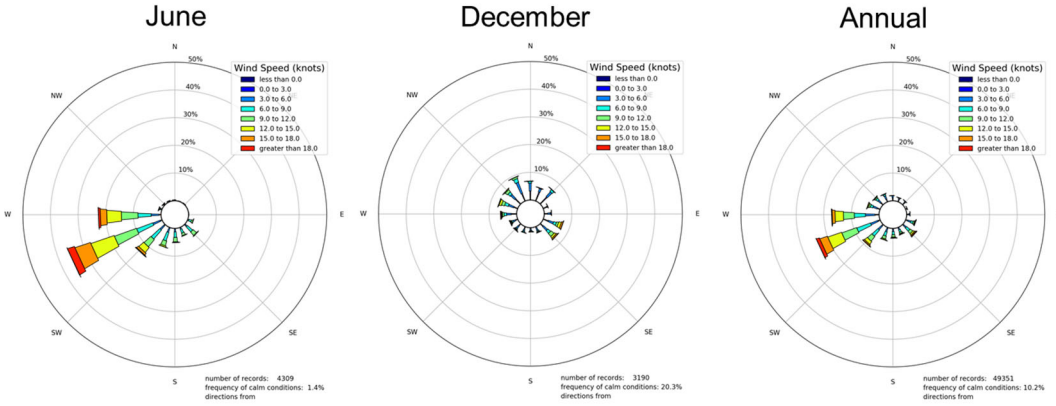


Figure 2-7 Berkeley Marina Station wind roses (2014-2019) for June, December and the full year.

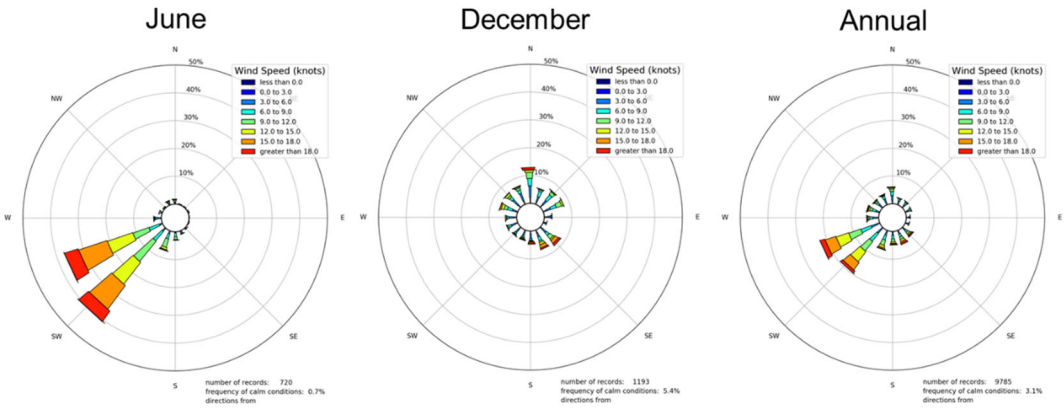


Figure 2-8 *Reef Light Station wind roses (2018-2019) for June, December and the full year.*

2.3.4 Extreme wind analysis

An extreme wind speed analysis was conducted using the 24-year dataset from the Alameda Station - Figure 2-1 and Figure 2-5. This station was selected for extreme analysis since it has the longest time series associated with the highest annual maxima. Weibull and Gumbel distributions were applied to the extreme values. Table 2-6 presents the estimated wind speeds for various return periods, along with the corresponding lower and upper bound values for the 95% confidence interval derived from the Weibull distribution (more conservative than Gumbel). It is important to note that the 100 and 200-year return period values carry lower confidence due to the limited length of the available time series.

Table 2-6 *Extreme wind analysis for Alameda station*

Return Period (years)	Extreme Wind Speed Central (Lower-Upper bounds) (knots)
10	35.0 (30.9 – 38.9)
25	37.9 (32.7 – 43.0)
50	39.8 (34.0 – 45.9)
100	42.0 (35.2 – 48.6)
200	43.7 (36.2 – 51.3)

2.4 Waves

Section 3 presents the methodology used to define the design wave conditions at the site.

3 Wave modeling

To define both operational and extreme wave conditions at the site, a two-tiered numerical modeling approach was adopted including a nearshore spectral wave model followed by a high-resolution diffraction model.

The nearshore wave model was used to simulate wave propagation and local wind-wave generation across the project area. The model accounts for key physical processes including wave growth due to wind forcing, refraction over variable bathymetry, shoaling, depth-induced breaking, and energy dissipation due to bottom friction. The resulting wave parameters at selected points were then used as inputs for the high-resolution diffraction model coefficients.

A diffraction model, based on the Boussinesq-type equations, was used to simulate wave transformations within the sheltered basin. This model accounts for complex phenomena including wave diffraction, reflection, nonlinear interactions, and partial wave blocking caused by structures. It was used to assess wave conditions at the berths, along the trestle, and at the plaza under the influence of the proposed breakwater.

3.1 Data basis

This section summarizes the metocean data sets used for this study, which include various data sets (hindcast, measurements) used for setting up the bathymetry and boundary conditions for numerical modeling.

3.1.1 NOAA WaveWatch III 30-year hindcast phase 2

NOAA WaveWatch III 30-year hindcast, provides wave climatology from 1979 to 2009. The wave model suite consists of global and regional nested grids. The parameters are available at 3-hour intervals. Spatially varying fields of significant wave height, peak wave period, and peak wave direction for total sea at 3-hour intervals were retrieved from the ‘West Coast of USA 4 minute (1/15 degrees)’ regional hindcast database.

3.1.2 NOAA WaveWatch III production hindcast

NOAA WaveWatch III production hindcast provides wave climatology from February 2005 to May 2019. The wave model suite consists of global and regional nested grids. The parameters are available at 3-hour intervals. Spatially varying fields of significant wave height, peak wave period, and peak wave direction for total sea at 3-hour intervals were retrieved from the ‘West Coast of USA 4 minute (1/15 degrees)’ regional hindcast database.

3.1.3 NOAA CO-OPS

NOAA's Center for Operational Oceanographic Products and Services (CO-OPS) is the authoritative source for accurate, reliable, and timely tides, water levels, currents, and other coastal oceanographic and meteorological information. CO-OPS maintains ocean observing infrastructure, including more than 200 permanent water level stations on the U.S. coasts and Great Lakes, an integrated system of real-time sensors concentrated in busy seaports, and temporary meters that collect observations for tidal current predictions.

Table 3-1 shows the stations, the data types and the periods for which data were extracted near the project site. Locations of the CO-OPS stations are shown in Figure 3-1.

Table 3-1 NOAA CO-OPS stations and data description

Station ID	Station Name	Data Type	Period
9414290	San Francisco	Wind	1996-2024
9414750	Alameda	Wind and Hourly Water Levels	1996-2024
9414863	Richmond	Wind	1996-2024

Tidal datums obtained at the three CO-OPS stations are provided in Table 3-2.

Table 3-2 Tidal datums at CO-OPS stations within the study region (in feet-MLLW).

Datum	Station 9414290 San Francisco	Station 9414750 Alameda	Station 9414763 Richmond
MHHW	5.8	6.6	6.0
MHW	5.2	6.0	5.4
MSL	3.1	3.4	3.3
MLW	1.1	1.1	1.1
MLLW	0.0	0.0	0.0
NAVD88	-0.1	0.2	0.0

3.1.4 NOAA NDBC

NOAA’s National Data Buoy Center (NDBC) maintains a network of marine observation and monitoring infrastructure which collect and disseminate in-situ, real-time, quality-controlled observations in the marine environment.

Table 3-3 shows the stations, the data types and the periods for which data were extracted. Locations of the NDBC stations are shown in Figure 3-1.

Table 3-3 NOAA NDBC stations and data description

Station ID	Station Name	Data Type	Period
46026	San Francisco Offshore	Wind, Significant Wave Height, Peak Wave Period, and Peak Wave Direction	1982-2024
46237	San Francisco Bar	Significant Wave Height, Peak Wave Period, and Peak Wave Direction	2007-2024

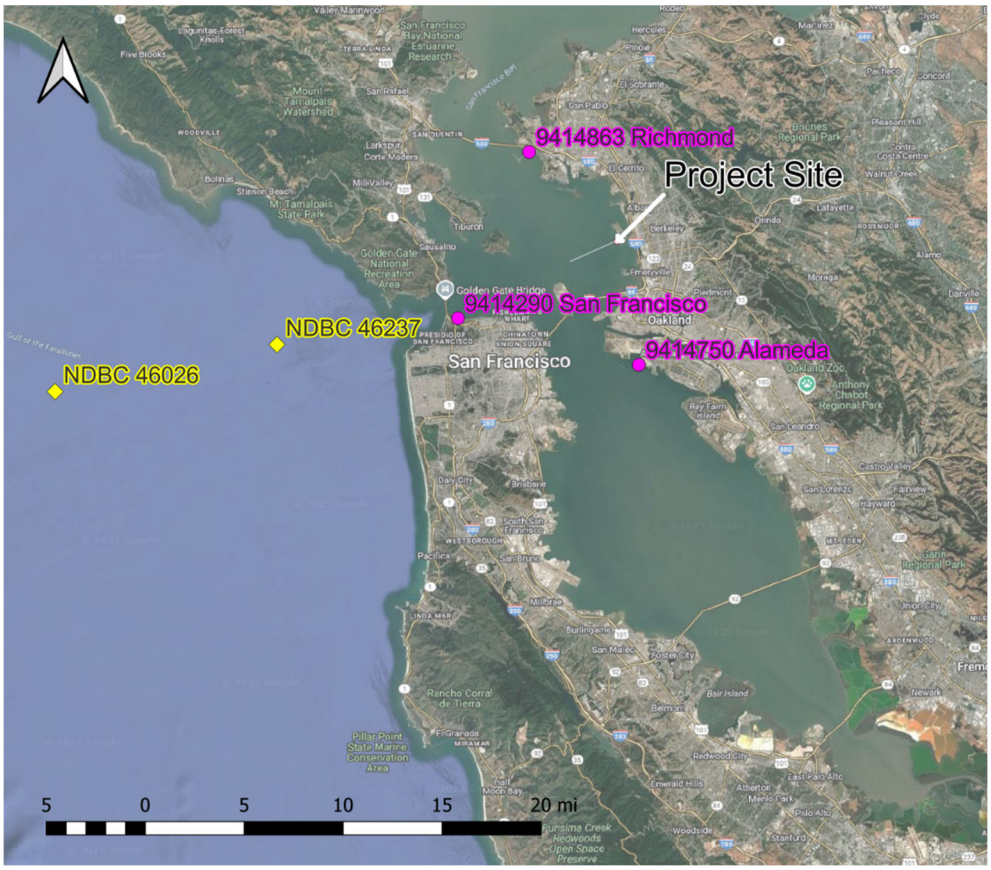


Figure 3-1 NOAA CO-OPS and NDBC wind, water levels, and wave measurement locations.

3.2 Bathymetry

The model bathymetry for the wave transformation study was created based on the USGS DEM for nearshore regions and survey data. Details of applied bathymetry data are given below:

› **USGS DEM** (2024): A digital bathymetric model with 1/3-arc-second (approximately 10-m) resolution, derived from diverse source data. The DEM covers the conterminous United States, Hawaii, Puerto Rico, other territorial

islands, and limited areas of Alaska. See Figure 3-2 for extent of coverage available within the model domain used in this study.

› **Local bathymetric survey** (2024): Bathymetry data for the site (Figure 3-3) was produced by eTrac Engineering Inc, a Woolpert Company for COWI. Survey data was collected on July 24, 25, 27 and 29, 2024 and plotted on August 22, 2024.

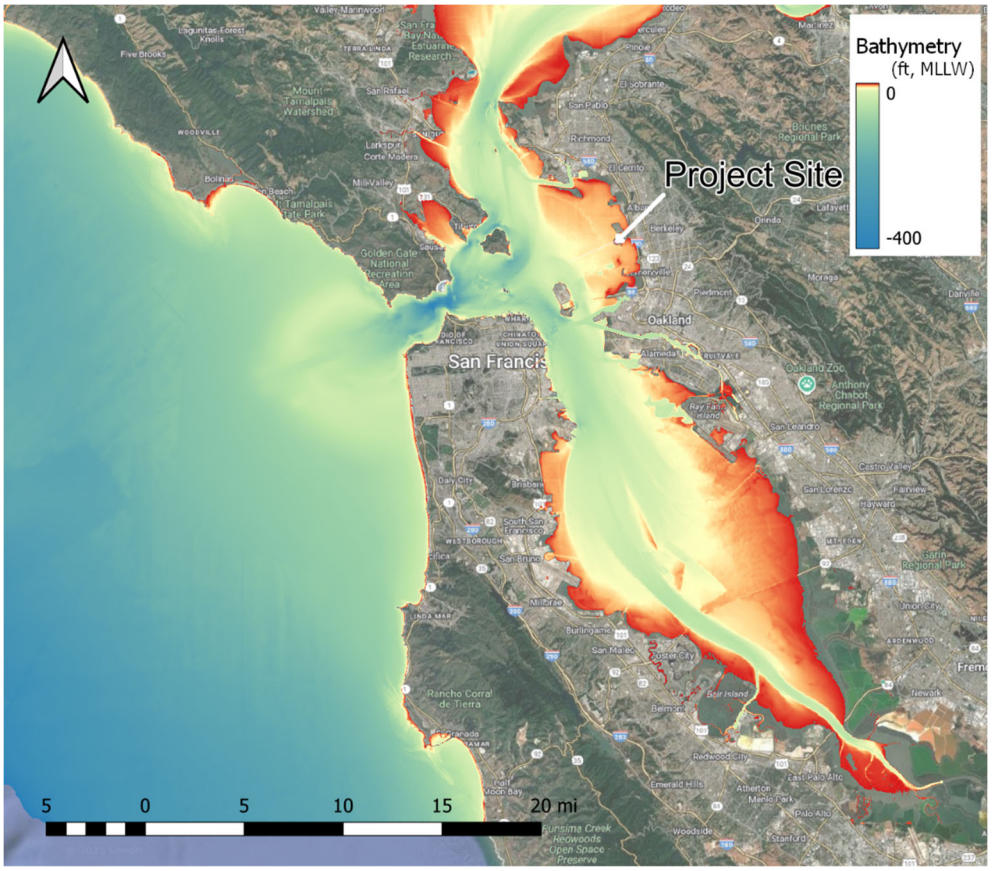


Figure 3-2 USGS DEM.

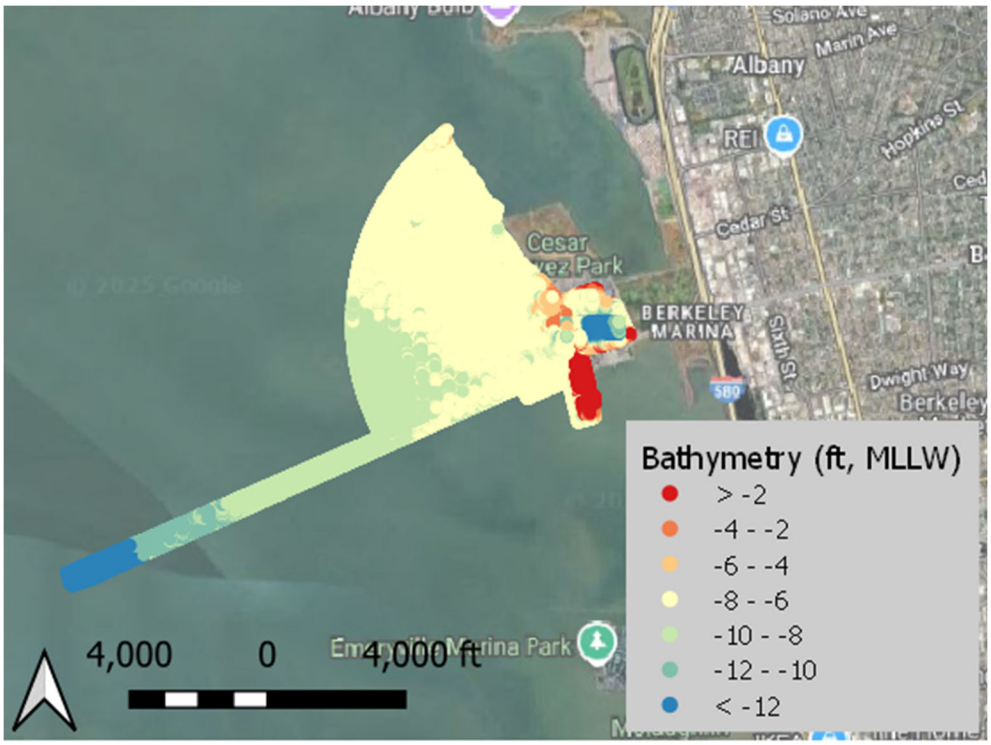


Figure 3-3 Local bathymetric survey from eTrac.

3.3 Nearshore wave modeling

A numerical nearshore wave model was conducted to obtain both operational and extreme wave conditions at locations in front of the breakwater as well as to derive the boundary conditions required as input for the tranquility (diffraction) model.

3.3.1 Model description

Numerical modeling of the wave conditions was performed using the Spectral Wave (SW) module of the comprehensive 2-dimensional MIKE 21 modeling suite from DHI, Denmark. The model is based on a flexible mesh, and hence very fine resolution bathymetries, where necessary, can be incorporated with this model.

MIKE 21 SW is a third-generation time-dependent spectral wind-wave model based on unstructured meshes. It simulates growth, propagation, decay, and transformation of wind generated waves and swells in offshore and coastal areas. The model includes wave growth by action of wind, interaction between waves with different frequencies and dissipation due to white capping. Furthermore, the model includes shallow water effects like refraction and shoaling due to varying depth and dissipation due to depth limited wave breaking and bottom friction.

The applied MIKE 21 SW model requires the following main inputs.

- Bathymetry of the area.
- Wind condition (wind speed and direction, or wind velocity components).
- Bottom friction parameters.
- Wave breaking parameters.
- Water level conditions.

3.3.2 Nearshore wave modeling methodology

The following methodology was adopted to develop a numerical wave model to simulate wave conditions for January 1997-December 2023:

- Bathymetry: Develop a mesh for a numerical wave model using various bathymetry data sets described in Section 3.2.
- Boundary conditions – waves:
  - Wave conditions for the January 1997 – December 2009 period were obtained from NOAA WaveWatch III 30-year Hindcast Phase 2 (Section 3.1.1).
  - Wave conditions for the January 2010 – December 2018 period were obtained from NOAA WaveWatch III Production Hindcast (Section 3.1.2).
  - Wave conditions for the January 2019 – December 2023 period were obtained from NOAA NDBC Buoy 46026 (Section 3.1.4).
- Boundary conditions – winds: Wind speed and direction time series from NOAA NDBC Buoy 46026, CO-OPS Stations 9414290, 9414750, and 9414863 were used to create of a spatial wind field mosaic using the extents shown in Figure 3-4. The mosaic thus created was used as wind forcing. See Section 2.3 for wind data and directionality analysis.
- Initial conditions – water levels: Hourly time series of measured water levels at NOAA CO-OPS Station 9414750 was used to specify water level variation.



Figure 3-4 Polygons used to create spatial wind field mosaic.

3.3.3 Model bathymetry

The model domain and mesh established for the wave transformation study is presented in Figure 3-5. The model domain includes the entire southern and central portions of San Francisco Bay and extends north up to San Pablo Bay (excluding). The offshore boundaries are located near the NDBC Buoy 46026.

The domain is divided into several zones with different triangular mesh resolutions. The spatial resolution of the mesh varies from an average element size of ~1-1.5 km in the region outside San Francisco Bay, transitioning gradually to a minimum of 150 m within the project site. Figure 3-6 shows the model bathymetry.

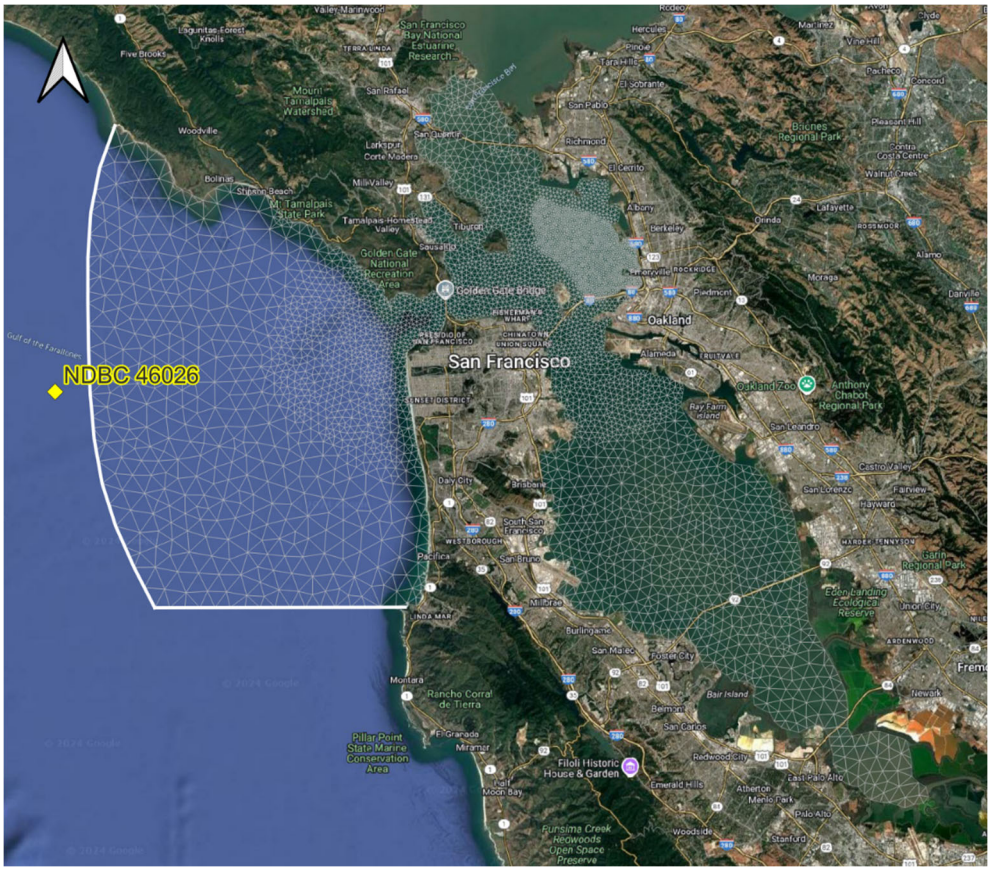


Figure 3-5 Model domain and mesh.

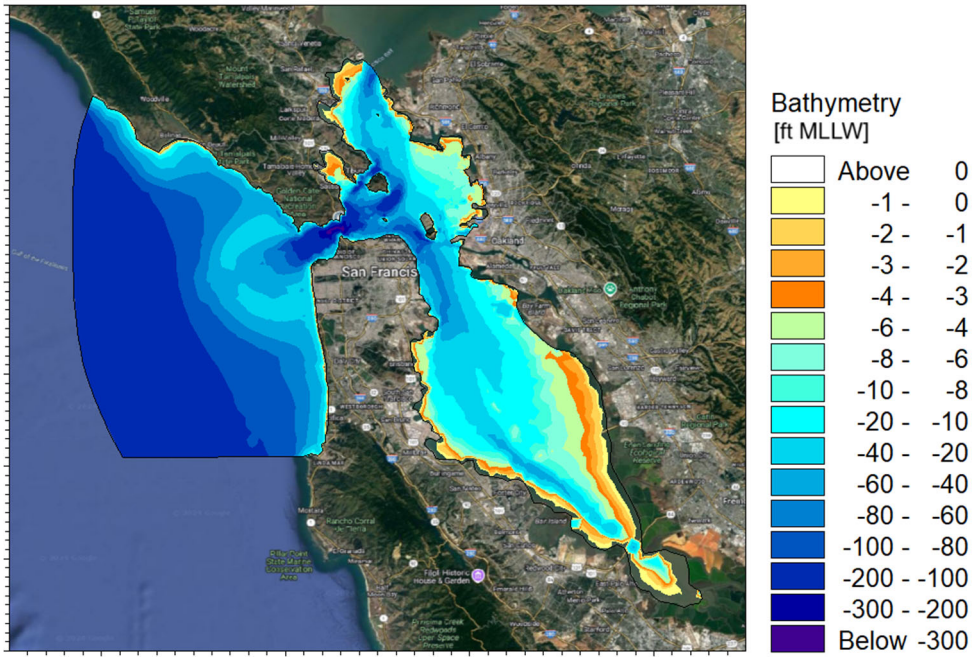


Figure 3-6 Model bathymetry.

3.3.4 Model validation

The numerical wave model was validated using measurements of significant wave height available at NDBC Station 46237. Figure 3-7 shows the quantile-quantile comparison of significant wave height between model results and measurements for a 4-year period between 2020 and 2023. A time series comparison of model results and measurements are shown in Figure 3-8 for a period between Oct 2022 and Jun 2023.

Figure 3-7 and Figure 3-8 indicate that the numerical wave model adequately represents the measured wave conditions at NDBC Station 46237. Significant wave heights generated from the model are on average slightly greater than the real-world observations. This adds a small amount of conservativeness to the model results and analysis.

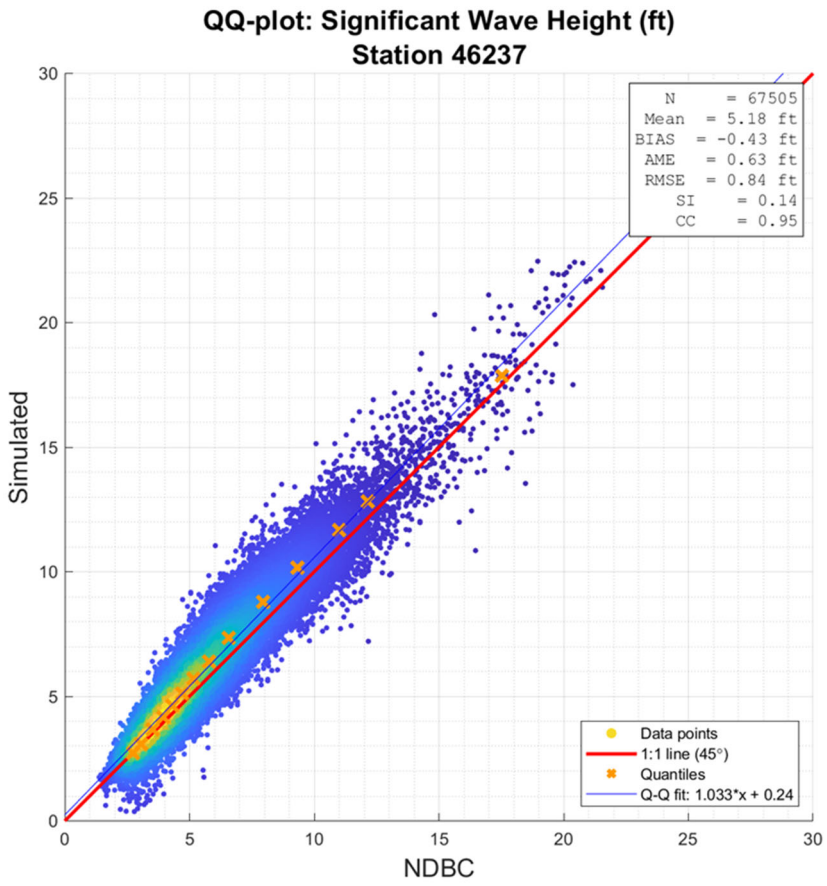


Figure 3-7 *Comparison of significant wave height between model results and measurements at NDBC Buoy 46237 for a 4-year period between 2020 and 2023.*

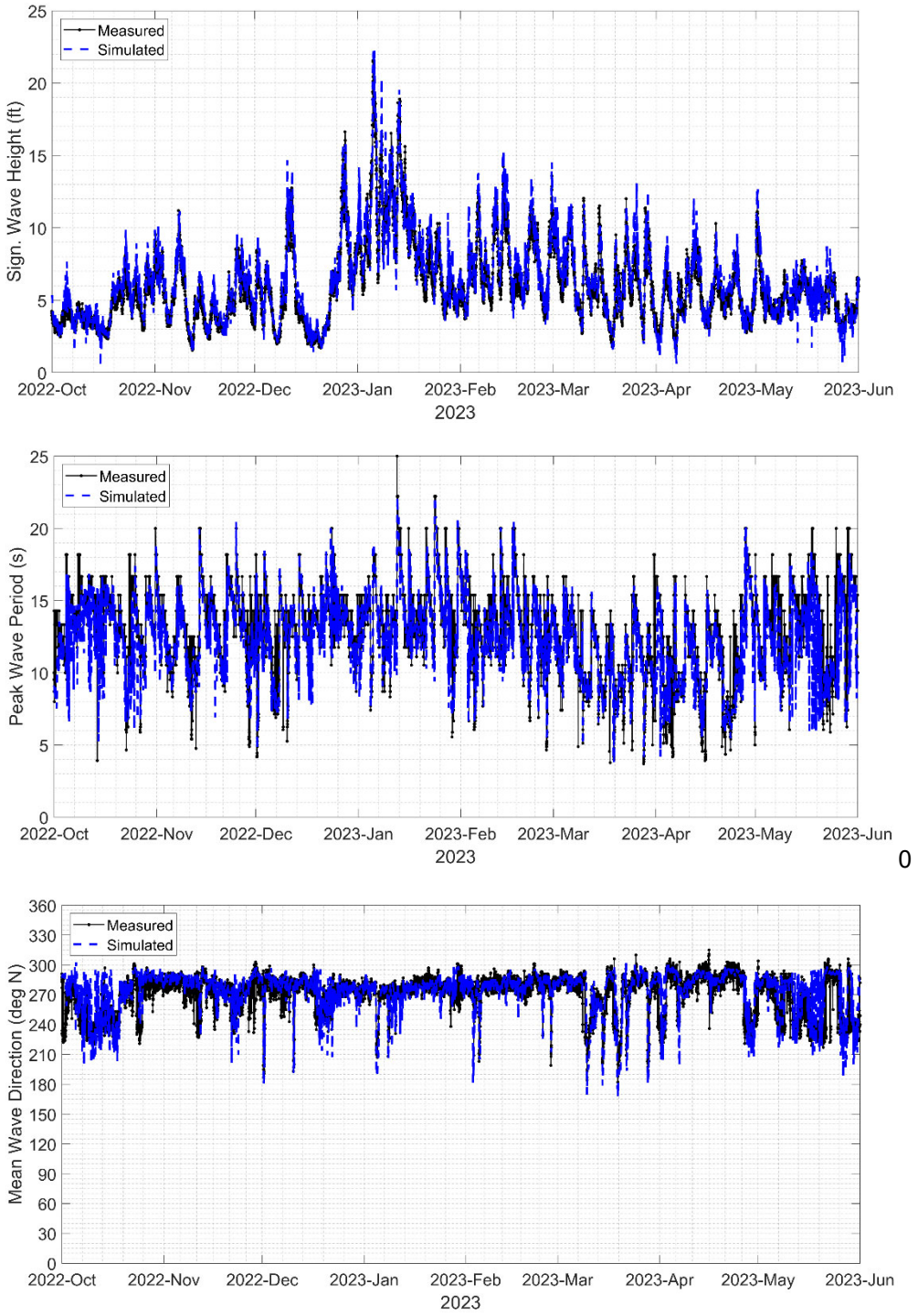


Figure 3-8 *Comparison of significant wave height (top), peak wave period (middle), and mean wave direction (bottom) between model results and measurements at NDBC Buoy 46237 for a period between Oct 2022 and Jun 2023.*

3.3.5 Model parameters

Table 3-4 contains a description of model parameters selected to obtain the comparison shown in the sections above.

Table 3-4 Model Parameters

Parameters		Value
Discretization	Spectral	Logarithmic: 26 frequencies $f_{j=0,1,2,..25} = 0.045*1.1^{j}$
	Directional	16 x 22.5°
Wind	Growth parameter	1.3
	Wave age tuning parameter	0.011
	Air-sea interaction type	Uncoupled
	Background Charnock parameter	0.0185
	Correction of friction velocity type	Simple cap
	Cap value	0.06
Wave Breaking		0.8
Bottom Friction	Nickuradse	0.02 m (0.8 in)
White capping	Cdis	1.33
	DELTA dis	0.5

3.3.6 Model results

The numerical wave model was simulated for a continuous period of 27-years (January 1997-December 2023) and was used to extract wave parameters at locations near the breakwater, see Figure 3-9. Example results from the wave model at BrkWtr-E output location are plotted in Figure 3-10, Figure 3-11 and Figure 3-12.

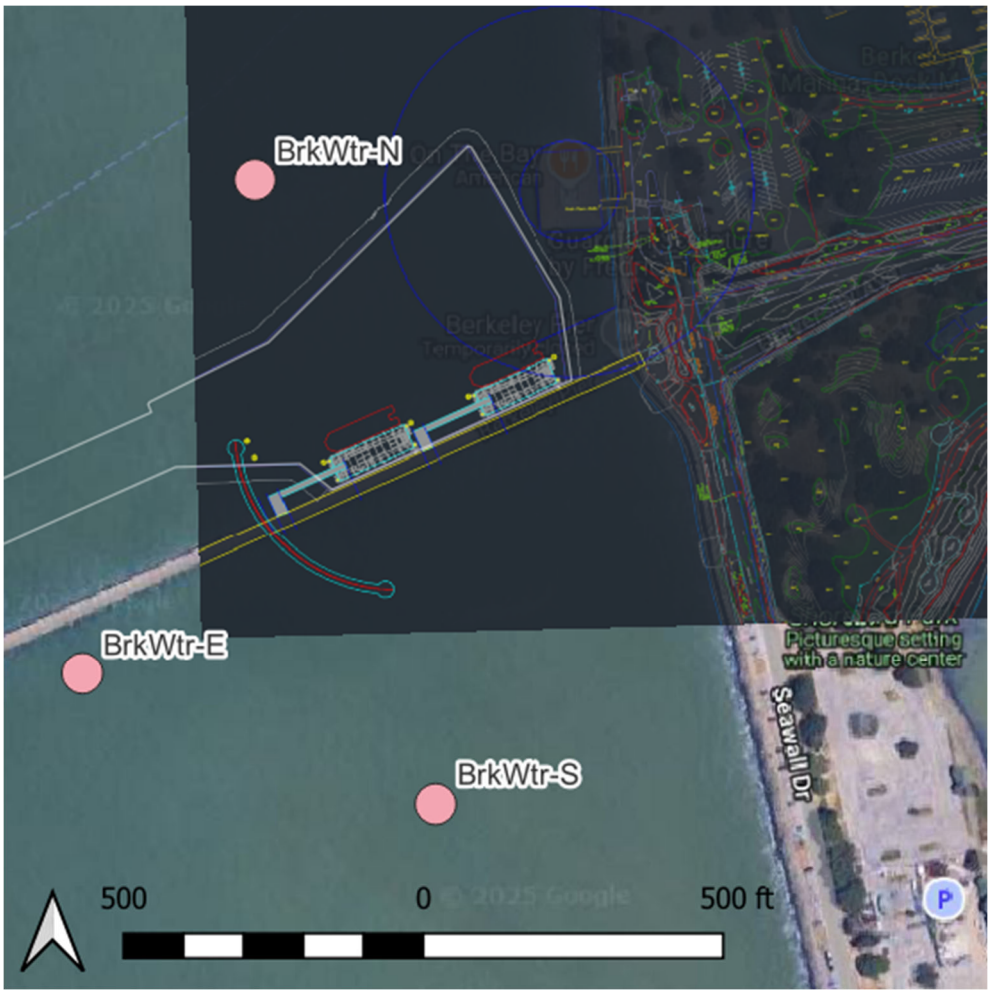


Figure 3-9 *Model output locations.*

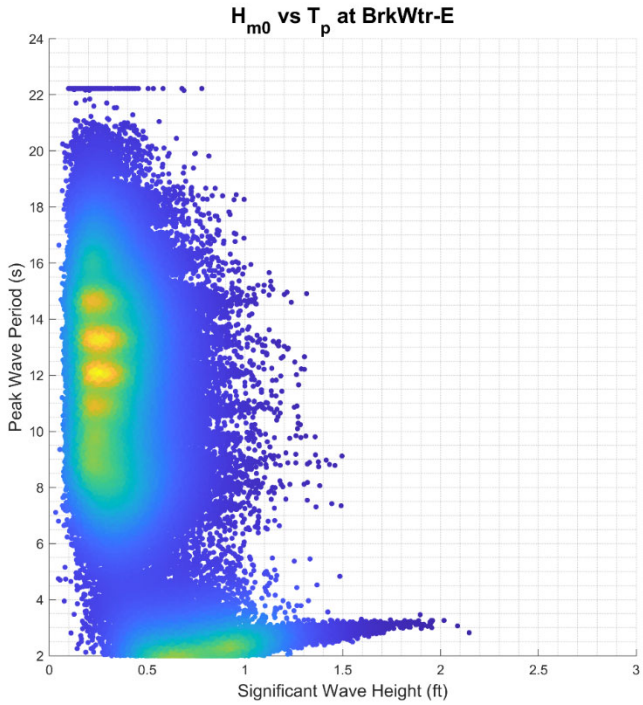


Figure 3-10 Scatter of significant wave height and peak wave period at BrkWtr-E location.

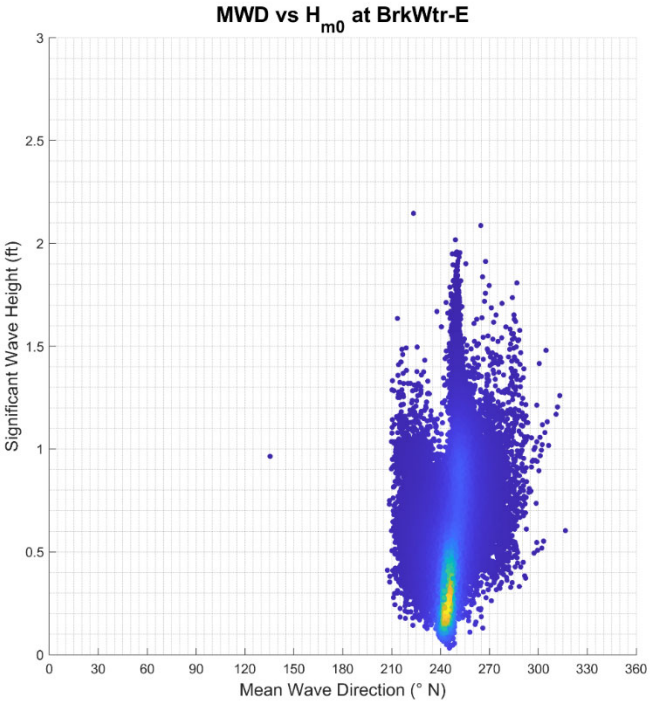


Figure 3-11 Scatter of significant wave height and mean wave direction at BrkWtr-E location.

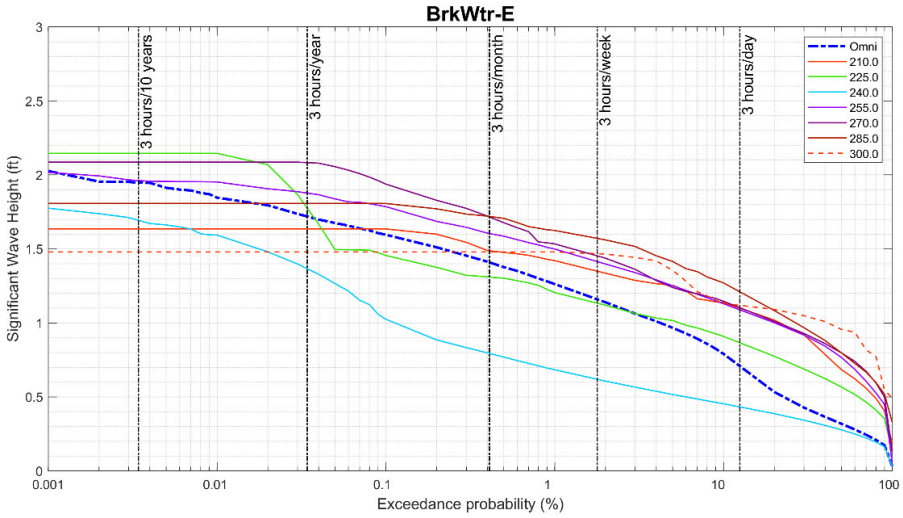


Figure 3-12 Directional exceedance of significant wave height at BrkWtr-E location.

3.3.7 Model sensitivity

The bathymetry in the vicinity of the project site, described in Section 2.5, is generally from -8 to -10 ft MLLW. As these are very shallow depths, the bottom friction model parameter can play a significant role in influencing the wave characteristics at the project site. In the absence of any measured data at the project site, model sensitivity to bottom friction parameter was studied by simulating extreme conditions with a lower bottom friction parameter of 0.005 m (0.2 in).

The extreme conditions were determined by performing a point-over-threshold analysis on wind speed at the three (3) CO-OPS stations near the project site. A similar analysis was also conducted on the offshore wave boundary conditions to obtain a set of extreme events for the sensitivity runs. Table 3-5 summarizes the threshold limits and the resulting number of events.

Table 3-5 Selection criteria for model sensitivity simulations

Quantity	Threshold	Number of Events
Wind speed at CO-OPS 9414290 San Francisco	20 knots	34
Wind speed at CO-OPS 9414750 Alameda	25 knots	36
Wind speed at CO-OPS 9414893 Richmond	22 knots	30
Significant Wave Height at Offshore Boundary	5 m (16.5 ft)	35

Figure 3-13 presents the increase in significant wave height at BrkWtr-E location with a lower bottom friction value and indicates that the median increase is 0.15 ft and maximum increase is 0.3 ft.

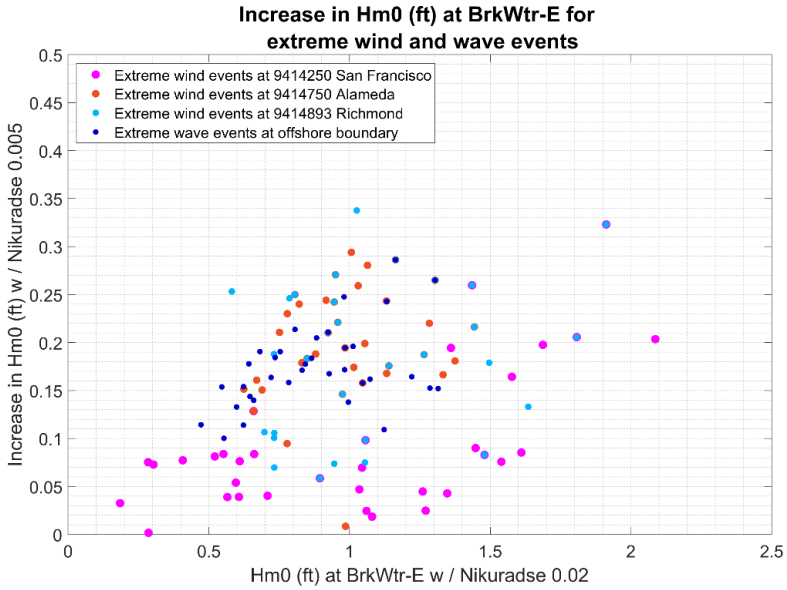


Figure 3-13 *Increase in significant wave height at BrkWtr-E location with a lower bottom friction value for different extreme events.*

In the absence of measured wave data at the project site, it could be considered to increase the extreme values calculated using model results time series by a maximum of 0.3 ft to mitigate any uncertainty due to bottom friction effects.

For the downtime analysis and design (extreme) wave conditions, a factor of 1.1 (10% increase in significant wave height) was applied to address model uncertainties and ensure robust decision-making.

3.3.8 Extreme wave analysis

An extreme wave analysis was conducted using the 24-year dataset from the wave model for an output point at the proposed breakwater location. Weibull and Gumbel distributions were applied to the extreme values. Table 3-6 presents the estimated wave heights for various return periods, along with the corresponding lower and upper bound values for 95% confidence derived from the Gumbel distribution (more conservative than Weibull and better fit).

The significant wave height from the upper bound of the 100-year return period (2.6 ft) was used as design wave condition for the breakwater.

Table 3-6 Extreme wave analysis at the proposed breakwater location

Return Period (years)	Extreme Wave Height Central (Lower-Upper bounds) (ft)
10	2.0 (1.9 – 2.2)
25	2.2 (2.0 – 2.4)
50	2.3 (2.0 – 2.5)
100	2.4 (2.1 – 2.6)
200	2.4 (2.1 – 2.7)

3.4 Wave diffraction modeling

To evaluate wave conditions within the sheltered basin, including at the berths, a diffraction modeling study was carried out to quantify wave disturbance in the lee of the proposed breakwater. Accurate assessment of wave conditions inside terminals is critical for ensuring safe and reliable operations, particularly under moderate to severe sea states.

A Boussinesq-type wave model, MIKE 21 BW, was used to simulate wave transformation processes such as diffraction, partial reflection, and nonlinear wave interactions within the terminal. These models are well-suited for resolving complex wave behavior in confined or semi-enclosed basins, including long-period motions and directional spreading effects. The model was applied to assess wave disturbance coefficients at key locations, including the berths, pier, and plaza. Results from this analysis are presented in the following sections.

3.4.1 Model Description

The numerical model applied for modeling of the wave propagation into the terminal was the Boussinesq Wave Module of DHI's MIKE 21 numerical model (MIKE 21 BW).

MIKE 21 BW is a Boussinesq type non-linear wave model, which simulates in the time domain the properties of propagation of irregular, directional waves into the harbor/terminal considering all important effects like shoaling, depth refraction, diffraction, bottom friction, non-linear wave-wave interaction, frequency and directional dispersion, partial and full reflection, and transmission through porous structures. Wave breaking is not included in the model.

The model requires the following input:

- Basic model parameters describing the extent of the model area, the grid spacing of the computational model grid, the time step and the duration of the simulation.
- A digitized bathymetry
- Incident wave conditions described by flux time series on the boundaries of the model area. Prior to simulation, these time series are generated on basis of specified wave spectra.
- Porosities (reflection and transmission coefficients) to describe the reflection characteristics for all structures and natural obstructions (breakwaters, quay walls, cliffs, beaches etc.) in the model area. The reflection is described by specification of the porosity of the nearest grid points to the reflective object. For impermeable objects, the porosity layer describing the reflection characteristics of a structure is backed by an impermeable land point.
- Description of so-called sponge layers, which are areas absorbing all wave energy propagating into the area (i.e., no reflection). Sponge layers are used to ensure that no unwanted and unnatural reflections occur in the model area (e.g., from the boundaries of the model).

The investigated layouts and the various input parameters applied in the simulations are described in the sections below.

Limitation of MIKE 21 BW

Depending on whether the deep-water correction terms are included in the solution of the Boussinesq-equations, the MIKE 21 BW numerical model is subject to different restrictions, as shown in Table 3-7.

Table 3-7 *Limitations of MIKE 21 BW when excluding and including deep-water correction terms in the computations.*

Deepwater terms *excluded*	Deepwater terms *included*
$\frac{h_{max}}{L_0} \leq 0.22$	$\frac{h_{max}}{L_0} \leq 0.60$
$\frac{L_{min}}{\Delta x} \geq 7$	$\frac{L_{min}}{\Delta x} \geq 7$
$\frac{T_{min}}{\Delta t} \geq 7$	$\frac{T_{min}}{\Delta t} \geq 32$
$\Delta t \leq \frac{\Delta x}{\sqrt{g h_{max}}}$	$\Delta t \leq \frac{\Delta x}{\sqrt{g h_{max}}}$

h_{max}	:	maximum water depth in the model area (m)
h_{min}	:	minimum water depth in the model area (m)
L_0	:	deep-water wavelength (m)
T_{min}	:	minimum wave period (s)
L_{min}	:	minimum wavelength (m) determined from T_{min} and h_{min}
Δx	:	grid spacing (m) in both x- and y-direction
Δt	:	time step (s)
g	:	gravitational acceleration (m/s^2)

Although the ferry terminal is in relatively shallow water, the prevalent waves in the area have very short periods. Hence, the simulations performed in this study were carried out with an h_{max}/L_0-ratio up to 0.50 for the shortest wave period. Therefore, it was necessary to include the deep-water correction terms in the computations.

3.4.2 Layout

The initial layout considered in the present study is shown in Figure 3-14, the breakwater length for the initial runs was 400 ft.

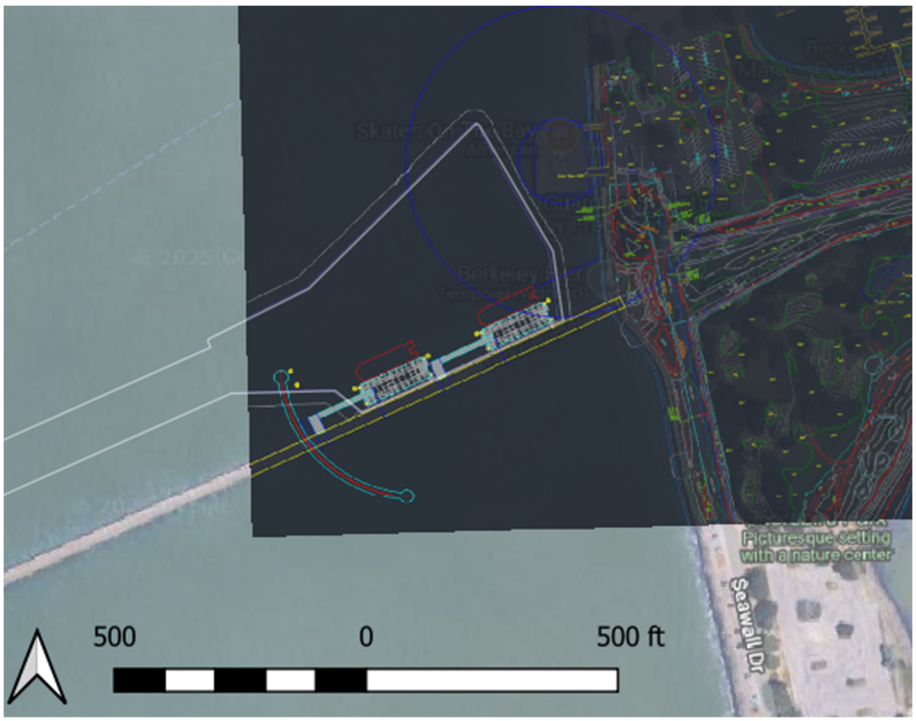


Figure 3-14 Project site showing the terminal layout used on the initial simulation runs.

3.4.3 Simulation matrix

The modeling was performed with an incoming significant wave height of $H_{m0,incoming}$=0.7 ft (0.2 m). A simulation matrix was prepared including four directions and six wave periods as presented in Table 3-8.

Table 3-8 Test matrix used for the wave disturbance simulations at the model boundary

Model Orientation	Wave Direction	Sign. Wave Height (ft)	Peak Wave Period (sec)
0°N	255°N	0.7	3s, 6s, 9s, 12s, 15s, 18s
	270°N		
315°N	210°N		
	225°N		
	240°N		
	300°N		

3.4.4 Model domain and bathymetry

Considering waves scattered in front of the breakwater (refer to Figure 3-11), wave directions at every 15° between 210°N and 270°N, and 300°N are considered for the present study. Since the wave direction varies, two model bathymetries were developed with orientation towards 0°N and 315°N. The purpose of having two orientations was to reduce the required extent of the model (saving computation time) and to make sure that the wave propagation in the model was generally perpendicular to the grid orientation as recommended for improved model performance.

The bathymetry generated with the grid orientation of 0°N is used for the wave directions of 255°N and 270°N, where the oblique mean wave direction relative to the model axes are ±10°-30° on either side. The bathymetry generated with an orientation of 315°N is used for simulating 210°N, 225°N, 240°N and 300°N. See Figure 3-15 and Figure 3-16 for the model domains used. Figure 3-17 shows the bathymetry within the model domains.

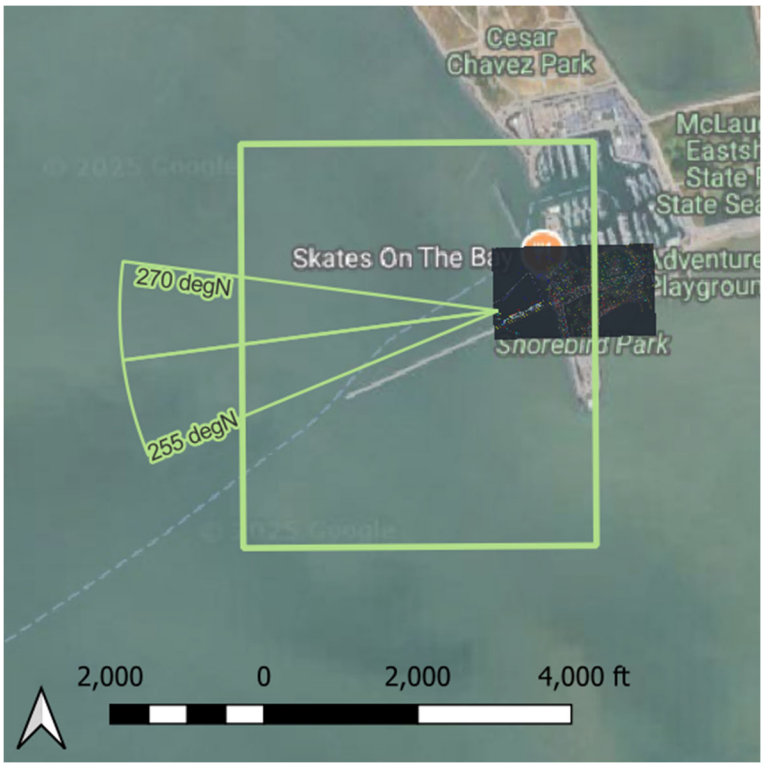


Figure 3-15 *BW model orientation 0°N used for incoming wave directions of 255°N and 270°N.*

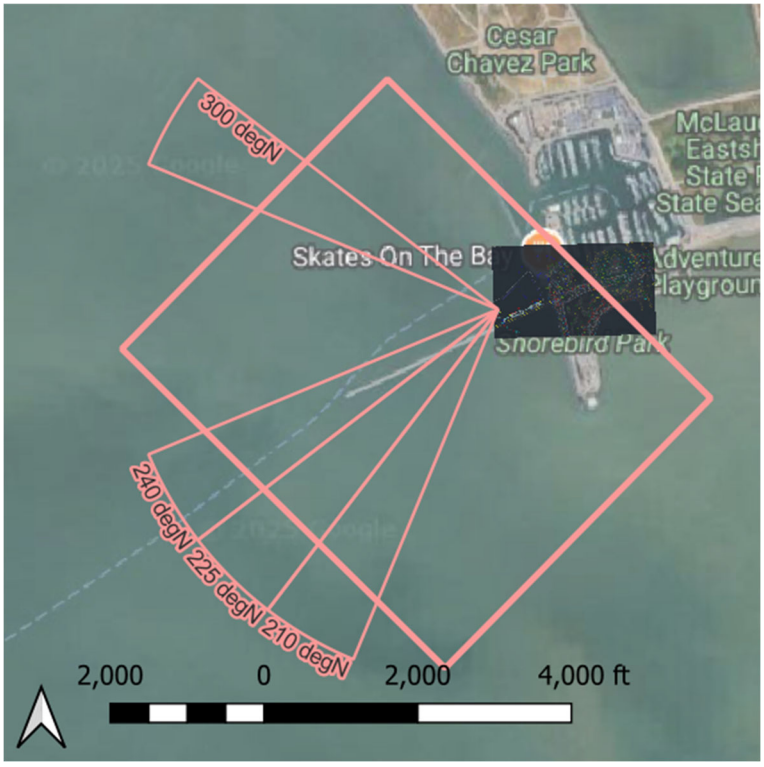


Figure 3-16 *BW model orientation 315°N used for incoming wave directions of 210°N, 225°N, 240°N, and 300°N.*

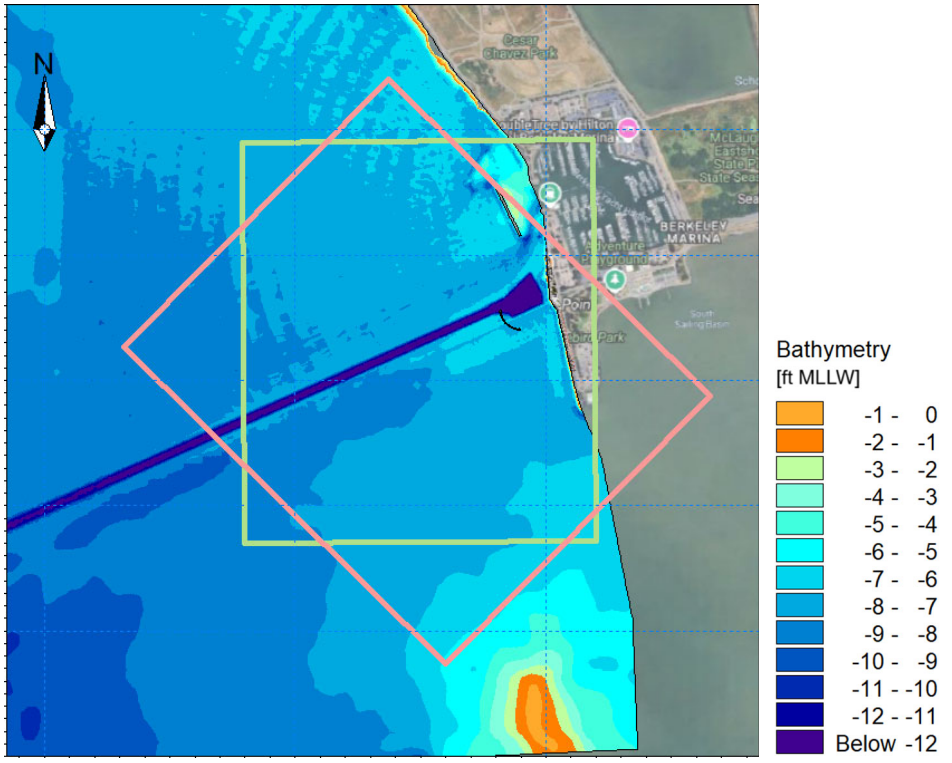


Figure 3-17 Bathymetry (ft MLLW) within the BW model domains.

As an example, the model bathymetry for model orientation of 0°N (see Figure 3-15 for box extent) is presented in Figure 3-18. In the model bathymetry, the horizontal datum is WGS1984/UTM 10N. A grid spacing of 2 m x 2 m was used for the model bathymetry.

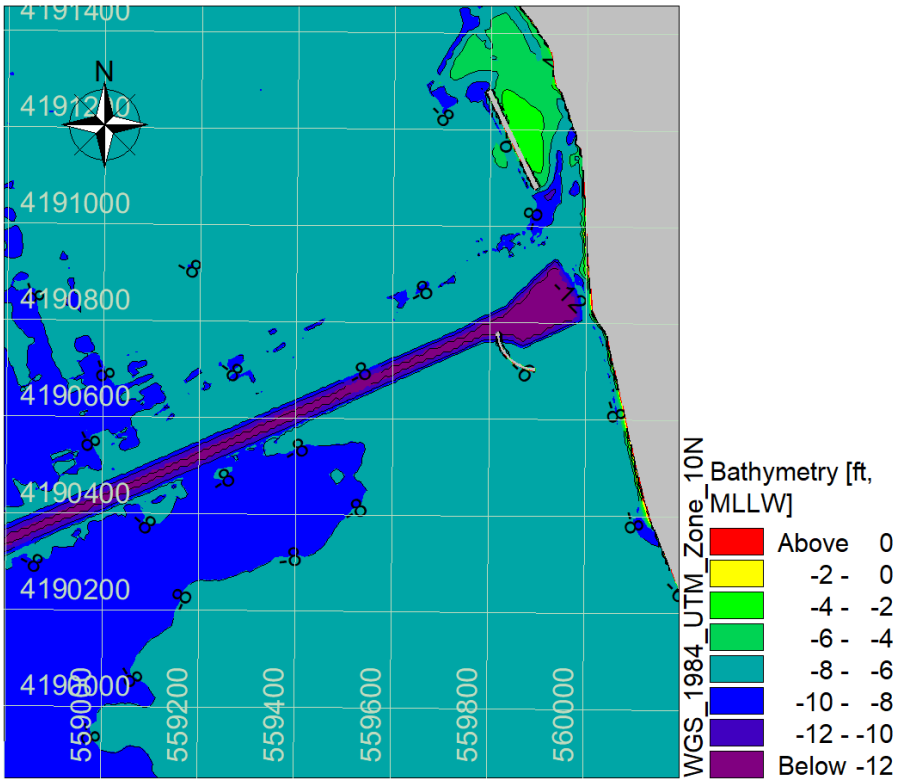


Figure 3-18 Bathymetry (ft MLLW) for model orientation = 0°N

3.4.5 Water level

All numerical simulations were carried out with two water levels: Mean Lower Low Water (MLLW) and Mean Higher High Water (MHHW). The MHHW level used for the diffraction (tranquility) simulations was determined by calculating the average MHHW at the three CO-OPS stations listed in Table 3-2. Thus, MHHW = 6.1 ft MLLW.

3.4.6 Minimum and maximum water depths

Due to the restrictions of the MIKE 21 BW model (see Table 3-7), the maximum water depth was set to h_{max}= 15 ft in all simulations and depths larger than h_{max} were reduced to h_{max}. This simplification of the bathymetry only has a very minor influence on the wave propagation and refraction due to the wavelength (L) to water depth (h_{max}) ratio.

Furthermore, to avoid and minimize model instability due to wave breaking, the minimum water depth was set to h_{min}= 3 ft in all simulations and depths shallower than h_{min} were increased to h_{min}.

3.4.7 Simulation duration and time step

To ensure fully developed wave patterns in the basin and reliable wave height statistics, the duration of each simulation generally corresponded to 30 minutes. The

last 20 minutes of each simulation were used for statistical analysis, i.e. determination of the significant wave height (H_{m0}) throughout the model area. The time step (Δt) applied in the simulations was 0.02 s.

3.4.8 Partial reflection and transmission

The reflective characteristics of various coastal structures in the model domain, such as rubble mounds, vertical walls, etc., is defined by porosity layers. The location of partial reflective and almost fully reflective structures is defined as shown in Figure 3-19. These are specified by the porosity values corresponding to the reflection characteristics of these structures. The assumed reflection characteristics are given in Table 3-9.

Table 3-9 Reflection coefficients specified in the model.

Feature / Structure	Reflection Coefficient
Revetment	0.40
Vertical Wall / Bulkhead	0.95

The porosity values applied in the model are calculated for these reflection coefficients considering actual water depths, wave height and period in front of the structure.

It must be noted that, as shown in Figure 3-19, the breakwater is specified to be highly reflective only at the ends. The middle segment is specified to be less reflective to minimize model instability, given the convex shape of the breakwater, the reflected waves radiate out in all directions causing instabilities near the wave boundaries.

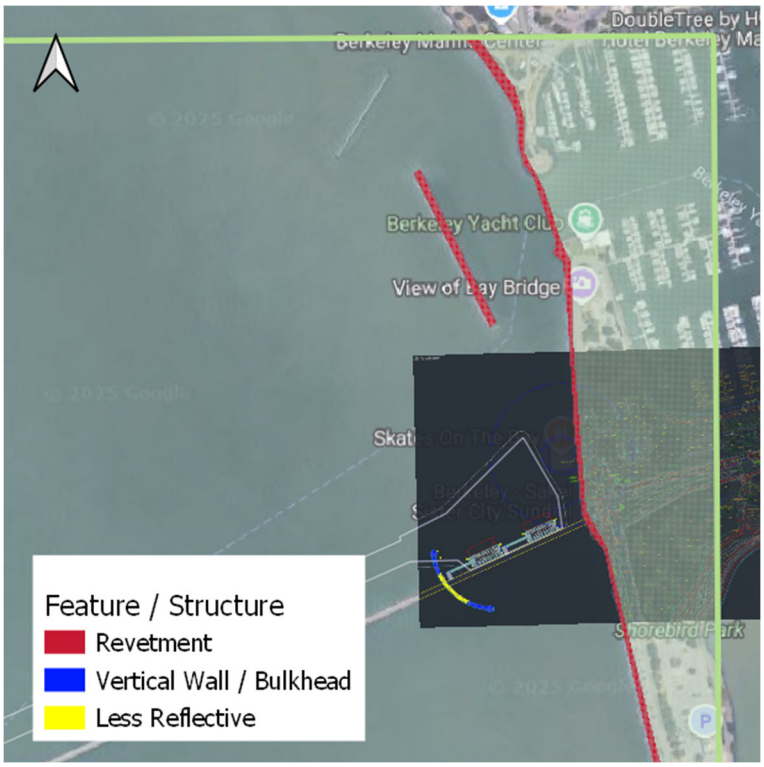


Figure 3-19 Location of almost fully reflecting vertical walls and partially reflecting revetments.

3.4.9 Sponge layer

The waves in the numerical model were generated along lines inside the model boundary, and since no reflection must occur from the boundaries of the numerical model, so-called sponge layers (layers which smoothly absorb all wave energy entering the layers) were introduced. Additionally, wave energy reaching the shoreline and areas within the model domain that do not influence waves at project site are also absorbed using "sponge" layers. This method is generally adopted in wave tranquility simulations to avoid wave reflection which is not useful in describing wave conditions within the terminal. The location of absorbing sponge layers is shown in Figure 3-20.

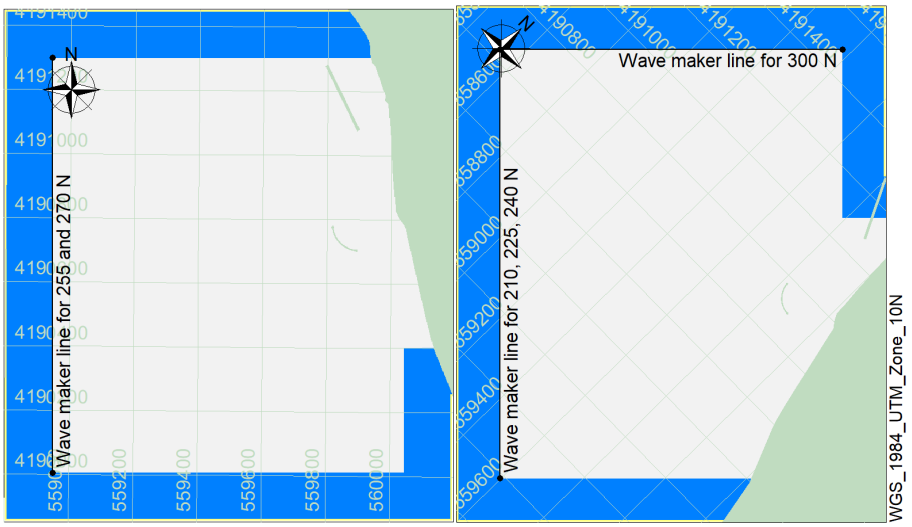


Figure 3-20 *Location of sponge layers (blue area) for wave absorption. Left: Model oriented towards 0°N. Right: Model oriented towards 315°N.*

3.4.10 Model wave generation

Waves are generated in the model domain by internal wave generation lines along the model boundaries. Irregular random waves were generated based on the JONSWAP spectrum for the input values of wave height (H_{m0}) and wave period (T_p). A maximum deviation of 20° from the mean wave direction is chosen for the simulations.

3.4.11 Model results

This section presents the results of the simulations. The purpose of the simulations is to document the wave disturbance at berth locations. The simulation matrix with the offshore wave conditions is presented in Table 3-8. The total duration of the simulations was 30 minutes, of which the last 20 minutes were used for determination of significant wave heights (H_{m0}) throughout the port.

An incident significant wave height of $H_{m0,incoming}$ = 0.2 m (approximately 0.7 ft) was applied in the simulations.

Results of the simulations are presented as wave disturbance coefficients. Schematic representation of the ferry terminal is given in Figure 3-21. The wave disturbance coefficients ($K = H_{m0}/H_{m0,incoming}$) have been calculated at the different berth locations, see Figure 3-21. For detailed analysis of the wave conditions at the two berths shown in Figure 3-21, average wave disturbance coefficients, K, were calculated in areas of 120 ft x 35 ft corresponding to the approximate vessel dimensions and are given in Table 3-10 for MLLW water level and in Table 3-11 for MHHW water level.

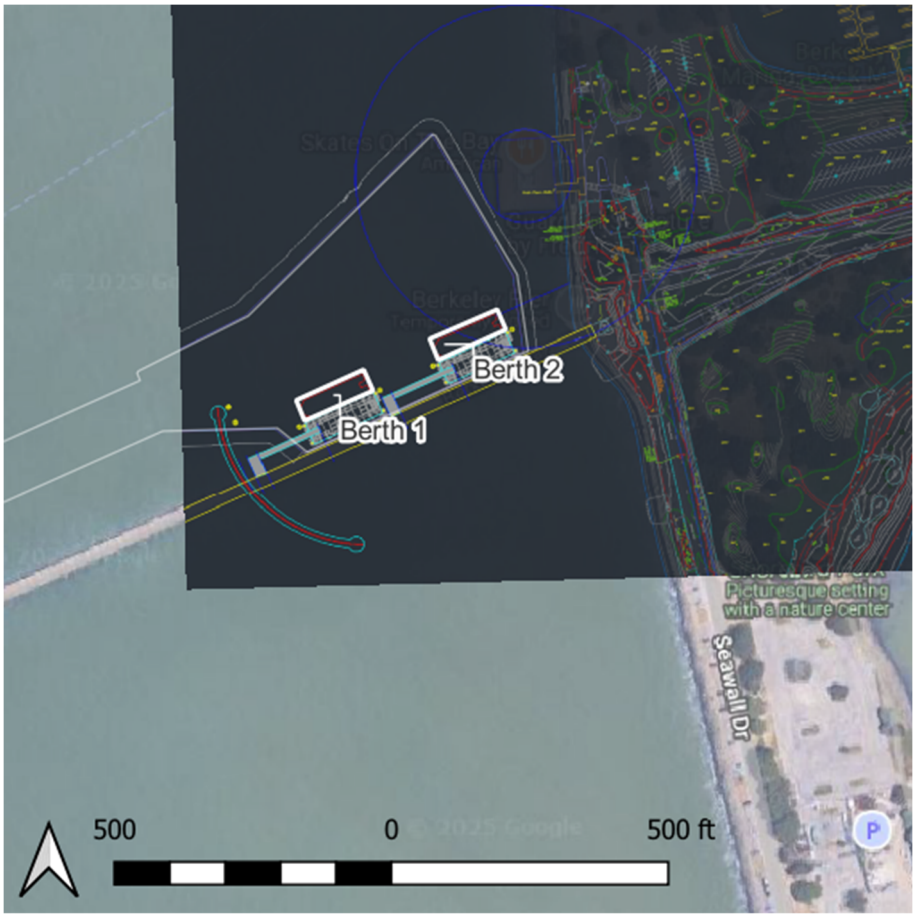


Figure 3-21 Areas within the ferry terminal used for mean wave disturbance calculation.

Table 3-10 Mean wave disturbance coefficients for MLLW simulations.

MWD (°N)	Tp (s)	Berth 1	Berth 2
210	3	0.35	0.51
210	6	0.41	0.46
210	9	0.41	0.42
210	12	0.41	0.43
210	15	0.38	0.41
210	18	0.38	0.40
225	3	0.36	0.36
225	6	0.35	0.35
225	9	0.35	0.35
225	12	0.33	0.33
225	15	0.31	0.31
225	18	0.29	0.34
240	3	0.32	0.25
240	6	0.29	0.26
240	9	0.26	0.26
240	12	0.27	0.25
240	15	0.25	0.26
240	18	0.24	0.25
255	3	0.23	0.31
255	6	0.17	0.18
255	9	0.17	0.19
255	12	0.19	0.20
255	15	0.20	0.22
255	18	0.20	0.22
270	3	0.54	0.78
270	6	0.32	0.51
270	9	0.28	0.46
270	12	0.29	0.45
270	15	0.30	0.44
270	18	0.28	0.41
300	3	1.04	1.05
300	6	1.02	1.14
300	9	1.02	1.13
300	12	0.94	1.09
300	15	0.86	1.00
300	18	0.85	0.97

Table 3-11 Mean wave disturbance coefficients for MHHW simulations.

MWD (°N)	Tp (s)	Berth 1	Berth 2
210	3	0.66	0.64
210	6	0.64	0.60
210	9	0.64	0.57
210	12	0.71	0.60
210	15	0.67	0.58
210	18	0.57	0.54
225	3	0.69	0.57
225	6	0.59	0.52
225	9	0.61	0.49
225	12	0.63	0.50
225	15	0.65	0.49
225	18	0.56	0.50
240	3	0.59	0.53
240	6	0.45	0.41
240	9	0.45	0.40
240	12	0.46	0.36
240	15	0.45	0.32
240	18	0.40	0.33
255	3	0.45	0.59
255	6	0.29	0.33
255	9	0.31	0.29
255	12	0.36	0.39
255	15	0.39	0.42
255	18	0.37	0.41
270	3	0.74	0.91
270	6	0.51	0.81
270	9	0.49	0.74
270	12	0.50	0.72
270	15	0.56	0.81
270	18	0.58	0.75
300	3	1.17	1.13
300	6	1.05	1.10
300	9	1.07	1.13
300	12	0.92	1.10
300	15	0.96	1.16
300	18	0.91	1.10

Wave disturbance coefficient maps for 240°N and 255°N, the two directions with most wave incidence for overall model domain and zoomed view of the terminal are presented in Appendix C.

3.4.12 Alternative breakwater lengths

In addition to model simulations as described above, wave tranquility was also assessed for alternative breakwater lengths of 300-ft and 250-ft. Figure 3-22 shows the layouts for the alternative breakwater lengths.

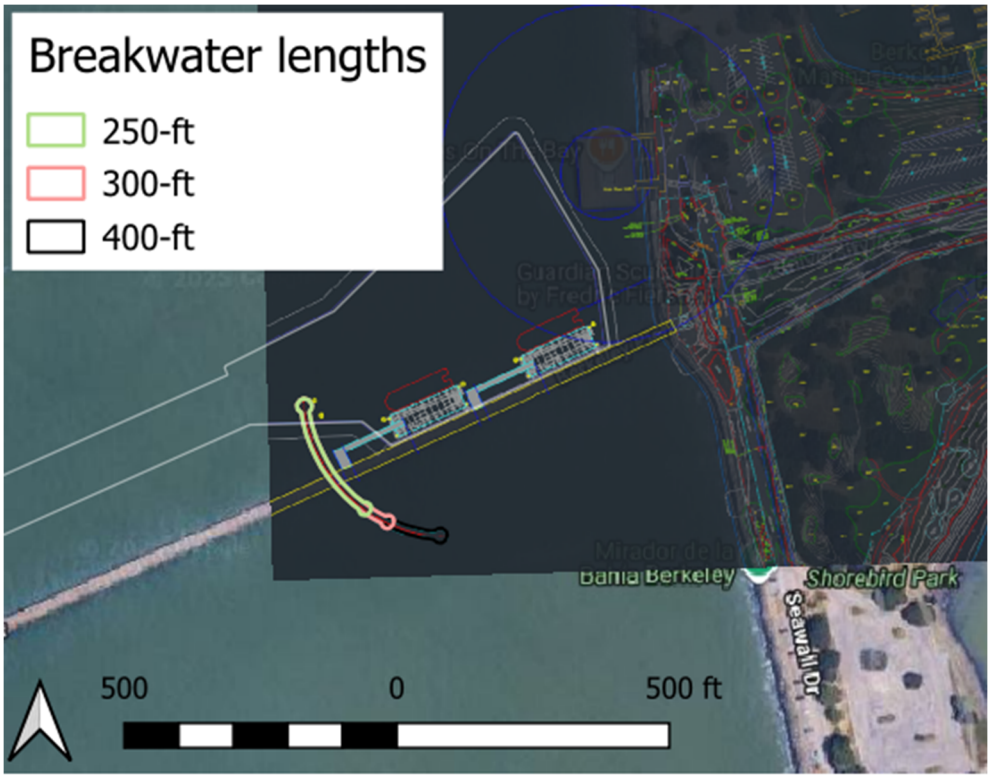


Figure 3-22 Layout for alternative breakwater lengths.

3.4.13 Model results for layout with 300-ft breakwater

Average wave disturbance coefficients, K (corresponding to the total wave energy), were calculated in berth areas and are given in Table 3-12 and Table 3-13 for the layout with 300-ft breakwater.

Table 3-12 Mean wave disturbance coefficients with 300-ft breakwater for MLLW simulations.

MWD (°N)	Tp (s)	Berth 1	Berth 2
210	3	0.42	0.74
210	6	0.46	0.61
210	9	0.46	0.54
210	12	0.45	0.58
210	15	0.42	0.54
210	18	0.43	0.50
225	3	0.40	0.53
225	6	0.39	0.44
225	9	0.39	0.43
225	12	0.40	0.42
225	15	0.37	0.40
225	18	0.35	0.41
240	3	0.31	0.30
240	6	0.30	0.28
240	9	0.29	0.27
240	12	0.28	0.30
240	15	0.25	0.28
240	18	0.24	0.25
255	3	0.23	0.30
255	6	0.18	0.17
255	9	0.20	0.18
255	12	0.21	0.21
255	15	0.20	0.22
255	18	0.21	0.21
270	3	0.54	0.78
270	6	0.33	0.50
270	9	0.28	0.45
270	12	0.28	0.44
270	15	0.30	0.45
270	18	0.29	0.41
300	3	1.04	1.04
300	6	1.02	1.13
300	9	1.00	1.13
300	12	0.92	1.06
300	15	0.85	0.98
300	18	0.87	0.96

Table 3-13 Mean wave disturbance coefficients with 300-ft breakwater for MHHW simulations.

MWD (°N)	Tp (s)	Berth 1	Berth 2
210	3	0.70	0.88
210	6	0.66	0.80
210	9	0.67	0.77
210	12	0.73	0.75
210	15	0.66	0.68
210	18	0.58	0.65
225	3	0.71	0.79
225	6	0.61	0.62
225	9	0.64	0.59
225	12	0.64	0.60
225	15	0.64	0.54
225	18	0.62	0.57
240	3	0.57	0.54
240	6	0.45	0.43
240	9	0.48	0.47
240	12	0.45	0.39
240	15	0.42	0.36
240	18	0.42	0.42
255	3	0.46	0.60
255	6	0.29	0.32
255	9	0.31	0.30
255	12	0.36	0.38
255	15	0.37	0.39
255	18	0.37	0.41
270	3	0.74	0.91
270	6	0.51	0.80
270	9	0.47	0.73
270	12	0.52	0.74
270	15	0.58	0.79
270	18	0.55	0.70
300	3	1.15	1.13
300	6	1.04	1.08
300	9	1.05	1.11
300	12	0.93	1.10
300	15	0.97	1.18
300	18	0.89	1.10

3.4.14 Model Results for Layout with 250-ft Breakwater

Average wave disturbance coefficients, K (corresponding to the total wave energy), were calculated in berth areas and are given in Table 3-14 and Table 3-15 for the layout with 250-ft breakwater.

Table 3-14 Mean wave disturbance coefficients with 250-ft breakwater for MLLW simulations.

MWD (°N)	Tp (s)	Berth 1	Berth 2
210	3	0.46	0.81
210	6	0.49	0.68
210	9	0.46	0.59
210	12	0.47	0.66
210	15	0.46	0.6
210	18	0.46	0.55
225	3	0.43	0.6
225	6	0.41	0.46
225	9	0.43	0.46
225	12	0.41	0.46
225	15	0.4	0.43
225	18	0.38	0.45
240	3	0.31	0.26
240	6	0.3	0.22
240	9	0.3	0.24
240	12	0.28	0.25
240	15	0.25	0.26
240	18	0.25	0.26
255	3	0.23	0.31
255	6	0.18	0.16
255	9	0.2	0.16
255	12	0.2	0.19
255	15	0.2	0.2
255	18	0.22	0.2
270	3	0.54	0.78
270	6	0.32	0.51
270	9	0.28	0.45
270	12	0.29	0.44
270	15	0.32	0.46
270	18	0.3	0.41
300	3	1.04	1.04
300	6	1.02	1.13
300	9	1.02	1.11
300	12	0.92	1.05
300	15	0.87	0.97
300	18	0.87	0.96

Table 3-15 Mean wave disturbance coefficients with 250-ft breakwater for MHHW simulations.

MWD (°N)	Tp (s)	Berth 1	Berth 2
210	3	0.66	0.88
210	6	0.65	0.86
210	9	0.67	0.82
210	12	0.73	0.77
210	15	0.69	0.69
210	18	0.62	0.68
225	3	0.62	0.79
225	6	0.55	0.63
225	9	0.61	0.6
225	12	0.64	0.59
225	15	0.61	0.56
225	18	0.63	0.6
240	3	0.6	0.47
240	6	0.46	0.39
240	9	0.47	0.43
240	12	0.45	0.37
240	15	0.43	0.37
240	18	0.5	0.44
255	3	0.47	0.61
255	6	0.29	0.32
255	9	0.3	0.3
255	12	0.33	0.36
255	15	0.33	0.38
255	18	0.36	0.41
270	3	0.74	0.91
270	6	0.51	0.81
270	9	0.47	0.73
270	12	0.53	0.74
270	15	0.55	0.77
270	18	0.48	0.67
300	3	1.16	1.13
300	6	1.05	1.08
300	9	1.03	1.1
300	12	0.94	1.11
300	15	0.97	1.18
300	18	0.89	1.09

3.4.15 Comparison of breakwater lengths

A review of the wave-disturbance plots in Appendix C and comparisons shown in Figure 3-23 (for the representative case of direction 255° (N), MHHW, and a 6-second period) indicate that shortening the breakwater has minimal influence on conditions at

Berth 1. The berth is sheltered close enough to the breakwater, so the reduction in the breakwater length does not meaningfully change the level of protection it receives.

Berth 2 is more exposed to incoming wave energy. The analyses show a modest increase in wave disturbance at this berth when the breakwater is shortened, a result of the reduced attenuation of waves for this more exposed location. The increase in wave exposure at Berth 2 is relatively small when reducing the breakwater from 400 ft to 300 ft, but becomes slightly more noticeable when the length is further reduced to 250 ft.

Based on these results, the 300 ft configuration provides the best balance between maintaining adequate sheltering and reducing structure length and is considered the optimal breakwater length for these priorities.

These findings, together with the operational downtime assessment, are used to determine the recommended breakwater length. The wave-disturbance results and operational performance will be considered together to identify the most appropriate design length for the breakwater.

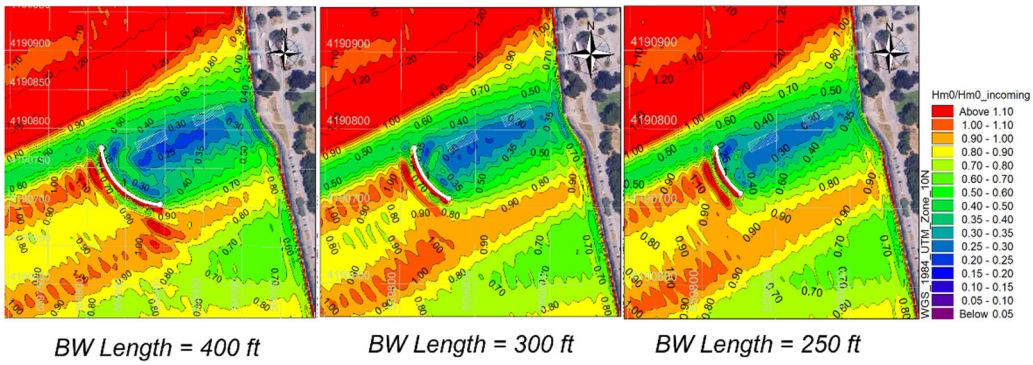


Figure 3-23 *Wave disturbance plot (Dir = 255°N at MHHW, Tp = 6 sec) for modeled breakwater lengths*

3.4.16 Wave time series in the ferry terminal

To establish the basis for calculating the wave statistics at the ferry terminal berths, see Figure 3-21, the results of the wave disturbance modeling were combined with the long-term wave conditions outside the port at point BrkWtr-E, see Figure 3-9. This was done by establishing transfer functions, describing the transformation of wave heights from point BrkWtr-E outside the port to a given area inside the port (based on the wave disturbance simulations), and then combining the transfer functions with the long-term time series of wave conditions at point BrkWtr-E. This provides a long-term time series of the wave heights at selected locations, for subsequent statistical analysis.

Time-series analyses of the wave conditions for each breakwater length were used to assess operational downtime and berth exposure, informing the final breakwater length, as described in Section 4.

3.4.17 Extreme diffraction wave model

The model was also run for extreme/design condition. The inputs consisted of the 100-year design wave conditions (Hs = 2.6 ft / Tp = 3.2 s) and the design water level (11.9 ft).Table 3-16 presents the results, for the 300-ft breakwater configuration, extracted at the approximate locations A (berth 1), B (between berths), C (berth 2) and D (between berth 2 and shoreline). These results were used to calculate deck and plaza minimum elevations and vulnerability to sea level rise.

Table 3-16 Significant wave heights offshore (input) and on the lee side of the breakwater under design conditions.

Location	H_s (ft)
Offshore (input)	2.6
A (berth 1)	1.8
B (between berths)	2.3
C (berth 2)	2.4
D (between berth 2 and shoreline plaza)	2.4

4 Downtime assessment

The objective of this downtime assessment is to evaluate potential weather-induced downtime frequency at the designated berth locations of the ferry terminal. Weather downtime refers to periods when operational activities at the berths are disrupted due to environmental conditions exceeding safety or operational thresholds. Specifically, this assessment quantifies the number of hours per year during which wave heights or wind speeds surpass these critical limits, rendering the berths temporarily inoperable.

The analysis focuses on understanding how local environmental conditions, including wind and wave climate, impact berth operability. This information will support a comprehensive overview of the site-specific weather constraints, providing a baseline for evaluating potential adjustments in operations or the physical layout of the terminal to minimize disruptions.

4.1 Downtime criteria

The threshold defining the upper limit of safe marine operations is a subjective quantity and it is best determined from local pilots and terminal operators experience.

In 2017, COWI conducted a downtime assessment for Mission Bay Ferry Landing (COWI, 2017) where COWI was provided with a chronological summary of cancelled service due to severe weather in 2016 and 2017. According to these data, the weather conditions that often create service issues for WETA involve winds above 20 knots (10.3 m/s). As part of the assessment, COWI compared the wave agitation at the ferry terminal locations for the specific days of winter in 2016 and 2017 when the operational terminals had to shut down due to bad weather conditions. The comparison reproduced the wave conditions that made the terminals inoperative, making it possible to set up an operational threshold for the Mission Bay Ferry Landing downtime assessment. A significant wave height downtime threshold of 1.5 feet was selected for that assessment.

Given that background, the following criteria have been agreed on with WETA and the City of Berkeley for this study.

- Wave height threshold: 1.5 feet
- Wind speed threshold: 20 knots (1-hour average)

4.2 Downtime due to waves

To assess the impact of wave action on operational reliability at the proposed terminal, a wave downtime analysis was conducted for two berths (B1 and B2) under three different layout configurations:

- No breakwater
- 250-foot breakwater
- 300-foot breakwater
- 400-foot breakwater

The analysis utilizes the downtime threshold of significant wave height greater than 1.5 feet.

Wave conditions were derived from a 27-year hindcast time series, as described in Section 3. This long-term dataset captures the variability of wave energy and supports a robust assessment of operability.

Downtime was evaluated on a monthly basis to investigate seasonal variations in exposure, informed by the wind seasonality analysis presented in Section 2.3.3. This approach provides insights into the expected operational performance of each breakwater alternative throughout the year.

The results quantify the potential improvement in berth downtime with the introduction of a breakwater and highlight the relative effectiveness of the 300-foot and 400-foot options in reducing wave-induced service disruptions.

Figure 4-1 presents the monthly average downtime calculated over a 27-year hindcast wave time series, representing the long-term average frequency with which the significant wave height exceeds the 1.5-foot threshold in each month.

The no-breakwater layout shows a clear seasonal pattern, with notable peaks in downtime during late spring and summer, particularly in June (2.1%) and July (1.6%). This reflects the influence of persistent southwesterly wind conditions during this period, which generates higher wave activity at the site.

In contrast, the breakwater configurations (250, 300 and 400 feet in length) result in reduced downtime across all months. The monthly average downtime for these scenarios remains consistently below 0.1%, demonstrating the impact of a breakwater on wave-induced operational disruptions.

The comparison between the breakwater alternatives indicates no significant difference in operational performance. The reduction in wave agitation and resulting downtime is nearly the same for the different configurations, indicating the additional breakwater length does not yield further meaningful improvement in operational reliability based on the established threshold for monthly averages.

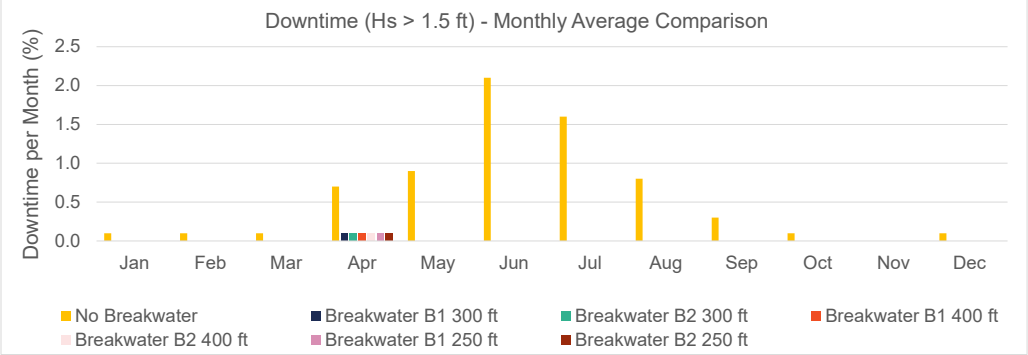


Figure 4-1 Monthly average wave downtime for different breakwater configurations.

In addition to the monthly averages, average monthly maxima were calculated to assess typical peak downtime conditions across the 27-year modeled time series. For each calendar month, the maximum downtime recorded in each individual year was extracted, and the average of those 27 yearly maxima was calculated. This approach provides a more representative estimate of the typical worst-case conditions experienced each month, rather than a single extreme value.

The results, shown in Figure 4-2, indicate that the no-breakwater configuration is subject to higher peak downtimes, averaging over 6% in June, and exceeding 4% in July and August. These values reflect recurring operational challenges during high-energy wave conditions typical of spring and summer months.

In contrast, the 250, 300 and 400-ft breakwater configurations show a significant reduction in average monthly maxima. Downtime remains below 2% for all months, including the more energetic periods of April through August. The difference between the three breakwater lengths downtime is minimal, reinforcing the earlier conclusion that a shorter breakwater is sufficient for mitigating wave-induced downtime to acceptable levels.

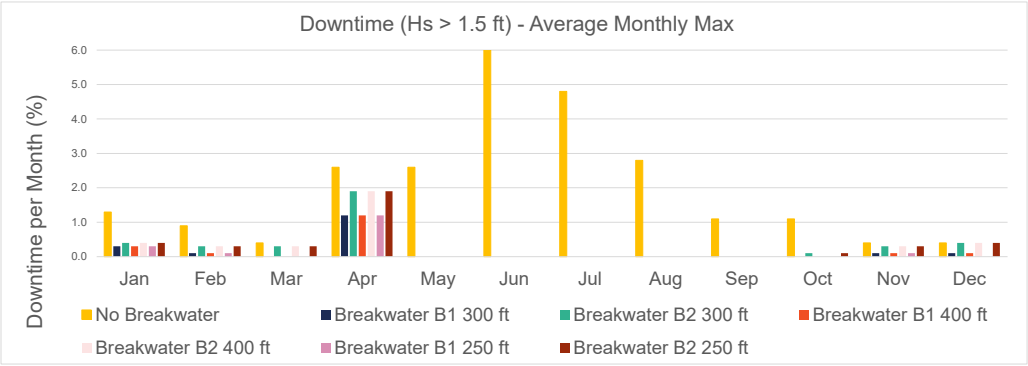


Figure 4-2 Average monthly maxima wave downtime for different breakwater configurations.

When the downtime results are considered alongside the wave-disturbance findings in Section 3.4.15, the 300-ft breakwater length is considered as the optimal option. It provides sufficient sheltering at both berths, avoids the increase in wave exposure

seen with the 250-ft configuration for berth 2, and performs nearly the same as the 400-ft breakwater in terms of downtime. Based on both the wave-disturbance and downtime assessments, the 300-ft breakwater is the recommended length.

5 Coastal structures minimum design elevations

5.1 Wave runup and overtopping

Runup and overtopping assessments were carried out in accordance with the methods and guidelines outlined in EurOtop (EurOtop, 2018), a standard for evaluating wave overtopping behavior on coastal structures. This approach ensures accuracy and consistency in determining the performance and safety of the breakwater under design wave conditions.

Runup was evaluated to estimate the maximum height that waves can reach on the breakwater structure. The calculation considered the incident design wave conditions, structure geometry, and breaker wave type.

The resulting 2% wave runup height was calculated, representing the height exceeded by 2% of waves. Overtopping was assessed to quantify the mean overtopping discharge and maximum overtopping wave volumes under design conditions. The calculations considered:

- Tolerable Overtopping Discharges: Determined based on the intended use of the area behind the breakwater, focusing on public access and recreation. EurOtop (EurOtop, 2018) provides guidelines for acceptable overtopping rates to ensure structural and public safety. It ensures the overtopping does not compromise the integrity of the breakwater, and limits discharge to prevent hazards such as excessive water flow or debris impacting recreational users.

- Influence of Foreshore: factored into account for energy dissipation before waves reach the structure.

For the purposes of design, a conservative overtopping threshold of 0.1 l/s/m (Mean Discharge Limits) was adopted. This value is consistent with guidance (EurOtop, 2018) for areas where impulsive wave action may occur and pedestrian safety is a primary concern. Limiting overtopping to this level ensures that wave impact, splash, and ponding are minimal and do not present hazards to recreational users or damage to infrastructure.

For vulnerability assessments, which explore conditions beyond the design horizon and under more extreme sea level rise scenarios, a more relaxed criterion was considered appropriate. In such cases, overtopping discharges up to 10–20 l/s/m may still be tolerable for pedestrian areas, particularly when wave heights are low ($H < 1.0$ m), the overtopping is non-impulsive, and the area is designed to drain quickly or remain safely accessible during intermittent overtopping events. These thresholds remain within the broader range of public safety criteria outlined in the guidelines (EurOtop, 2018) and provide flexibility for assessing long-term risk and performance under future conditions.

5.2 Breakwater crest elevation

The design of the breakwater crest elevation is critical as it directly influences the structure's ability to provide protection against waves, prevent overtopping, and ensure stability for the area it safeguards. Several factors were considered to arrive at the minimum crest elevation:

- Water Levels: including tidal range, storm surge, and sea level rise (Section 2.1).
- Wave Climate: the energy impacting the structure (Section 3).
- Wave Effects (Runup and Overtopping): to maintain functionality and prevent overtopping damage, the crest elevation must accommodate the wave runup height under design storm conditions. Acceptable overtopping rates for this design were considered based on categorizing the usage of the area behind the structure as recreational.
- Structural Considerations: the type of breakwater (vertical) affects the crest elevation due to wave energy dissipation and runup behavior.

5.2.1 Breakwater runup and overtopping

Table 5-1 summarizes the calculation results. The run up 2% formula applied a partial safety factor of one standard deviation. The minimum breakwater elevation is set at +17.6 ft Berkeley MLLW. This elevation is the run-up 2% elevation (calculated based on the 100-year wave height – Section 3.3.6) plus the design still water elevation needed to guarantee acceptable and safe overtopping discharge rates (<0.1 l/m/s).

Table 5-1 Wave effects and minimum breakwater crest elevation

Wave Run-up 2%	4.5 ft
Average Overtopping Discharge*	0.09 l/m/s
Minimum Breakwater Elevation	17 ft NAVD88 / 17.6 ft Berkeley MLLW

*Acceptable discharge at El. +17.56ft Berkeley MLLW

5.3 Deck elevation

The wave diffraction model developed for the project included a model run with the design still water level (11.9 ft MLLW) and the 100-year design incident wave height (2.6 ft). The incident waves vary along the trestle as the breakwater reduces the shelter towards the shoreline. Along the trestle, the significant wave heights at the deck for design conditions vary from 1.8 to 2.4 ft. The results for maximum green water elevation are presented in Table 5-2.

The maximum green water elevation is the highest vertical level reached by the crest of incoming waves during design conditions.

Table 5-2 Maximum green water elevation

Parameter	Elevation (ft MLLW Berkeley)
Design Still Water Level	11.9
100-year Max Crest Height	2.3-3.3
Max. Green Water Level	14.2-15.2

5.4 Plaza elevation

The minimum design elevation of the shoreline plaza was defined to ensure long-term public safety and avoid flooding under projected sea level rise and design wave conditions. The elevation was established based on a 50-year design life, with consideration for both hydrodynamic performance and public use requirements.

The following parameters were considered in determining the minimum recommended elevation:

- Design still water level, including sea level rise projections for the Intermediate Scenario - 2075 planning horizon.
- Wave transformation results for the shoreline located in the partially sheltered zone behind the breakwater. The considered wave height at the plaza, based on diffraction modeling and partial sheltering by the 300 ft-long breakwater, which reduces incident wave energy at the shoreline. The resulting design significant wave height at the plaza was estimated at 2.4 ft, and this value was used in calculating runup and overtopping under design storm conditions.
- Current shoreline revetment slope: assumed to be maintained at 1:3, extending approximately+11.5 ft to 12.5 ft, providing moderate energy dissipation and limits runup extent under typical storm conditions. Above the revetment, a seawall vertical structure was considered (containing the plaza fill), up to +16.5 ft (design elevation). The slope and rock material of the revetment were factored into the estimate of maximum wave runup using standard formulations.
- Public use considerations: the plaza is intended for frequent public access. The elevation was selected to ensure safe use without requiring closure due to minor flooding events.

Based on these parameters, a minimum elevation of +15.8 ft MLLW was defined to achieve overtopping rates close to zero under design conditions. This elevation was

deemed sufficient to avoid surface flooding and ensure safe pedestrian access, with negligible risk of discomfort or hazard during routine storm events.

However, to align with overall site grading, accessibility, drainage, and architectural integration, the final minimum design elevation (close to the plaza) was set at +16.5 ft MLLW and the maximum, +17.6 ft MLLW. This provides a generous safety buffer and ensures the plaza remains dry and fully operational not only under design conditions but also under more conservative sea level rise scenarios, as confirmed in the vulnerability assessments presented in the following section.

5.5 SLR vulnerability assessment

The breakwater, deck, and plaza elevation were established based on the minimum recommendations outlined in Sections 5.2, 5.3, 5.4, guided by the 2075 planning horizon and the Intermediate Scenario from the 2024 California Sea Level Rise Guidance. Table 5-3 summarizes these minimum elevations alongside the final design elevations selected by the project team to meet broader functional and operational requirements.

Table 5-3 Minimum and design elevations

Elevation (ft Berkeley MLLW)	Breakwater	Deck	Plaza
Minimum	17.6	14.2-15.2	15.8
Design	17.6	16.5-17.6	16.5

To evaluate the long-term resilience of these design elevations, a Sea Level Rise Vulnerability Assessment was conducted. The vulnerability assessment for the BWTPF Project evaluates the performance of key marine structures (breakwater, ferry pier deck, and shoreline plaza) under a range of future sea level rise scenarios extending beyond the 50-year design horizon. These scenarios include Intermediate, Intermediate-High, and High projections for both 2075 and 2100, as defined by the 2024 California Sea Level Rise Guidance. The assessment considered the effects of reduced freeboard, overtopping discharge, inundation risk, and public safety thresholds under increasingly severe conditions.

Results confirm that the infrastructure maintains full functionality and safety under the design condition, which corresponds to the Intermediate Scenario for 2075, and remains resilient under Intermediate-High conditions through 2100, with only minor overtopping and splash impacts. Under the High Scenario for 2100, overtopping and inundation exceed conservative thresholds, requiring operational restrictions and adaptive measures such as improved drainage and controlled public access. Despite these challenges, all components retain structural integrity, and the breakwater

continues to provide significant coastal protection even under extreme conditions. Appendix B presents the detailed assessment.

6 Hydrodynamic and sediment transport

The sediment transport study for the Berkeley Water Transportation Pier Ferry Project provides a detailed assessment of sedimentation dynamics and maintenance needs for the proposed ferry terminal and is presented in Appendix D. The analysis was conducted using a combination of hydrodynamic and sedimentation modeling, calibrated with historical and recent survey data from the Berkeley Marina Channel. The modeling focused on predicting sediment deposition rates within the approach channel and berth pockets, both of which are critical for ensuring navigational reliability and planning future dredging operations.

The results of the six-year simulation indicate that sedimentation rates are highest in the first year following dredging, particularly in the channel, and then gradually stabilize. Over the modeled period, the average annual sedimentation rate across the full dredged area was 0.15 feet, with the channel experiencing a higher average rate of 0.20 feet per year and the berth pockets a lower rate of 0.09 feet per year.

Based on these modeled sedimentation rates, the study estimates the frequency of maintenance dredging required to maintain operational depths in both the channel and berth pockets from the initial dredge elevation -12 ft. Two operational depths were considered: –10 and –9 ft. The estimated intervals until maintenance dredging is needed are presented in Table 6-1

Table 6-1 Estimated dredging intervals (years) for channel and berth pockets

Operational Depth (ft Berkeley MLLW)	Est. Years until Maintenance Dredging	
	Channel	Berth Pocket
-10	10	22
-9	15	33

Despite the robust modeling approach, the study acknowledges limitations. The sedimentation rates used for calibration are based on survey data from a narrow segment of the Berkeley Marina Channel, and there is a lack of site-specific suspended sediment concentration measurements, which introduces some uncertainty into the predictions. Additionally, the results are most reliable within the project footprint, and extrapolation beyond these areas is not recommended. The study also notes that the modeled breakwater length was later reduced from 400 feet to 300 feet, but this change is not expected to significantly affect sedimentation patterns in the navigation channel or berth areas.

In conclusion, the sediment transport analysis provides a reasonable and locally validated estimate of sedimentation rates and maintenance dredging needs for the

BWTPF project. The results support long-term planning and highlight the importance of ongoing monitoring and data collection to refine future predictions and maintenance strategies.

7 Conclusion

This study presents a comprehensive technical assessment confirming the feasibility and resilience of the Berkeley Water Transportation Pier Ferry Project. The integration of long-term environmental datasets, regional hindcast records, and advanced numerical modeling has established robust design parameters for water levels, wave conditions, wind, and currents specific to the Berkeley Marina area. These parameters have informed a series of simulations and analyses that characterize how wave energy, sediment transport, and operational constraints will interact with the proposed ferry terminal structures under a range of present and future conditions.

Wave modeling results demonstrate that the inclusion of a breakwater is essential for reducing wave agitation at the ferry berths and shoreline. The comparative analysis of breakwater lengths shows that increasing the structure from 300 to 400 feet yields only marginal improvements in wave disturbance and operational downtime. The 300-ft breakwater configuration provides sufficient sheltering for both berths, minimizes wave exposure, and achieves nearly the same reduction in downtime as the longer breakwater configuration.

Design elevations for the breakwater crest, terminal deck, and shoreline plaza were determined using overtopping thresholds and future sea level rise projections. The methodology follows EurOtop (EurOtop, 2018) criteria and incorporates a 1% annual exceedance probability storm tide event combined with sea level rise allowances consistent with the 2024 California Sea Level Rise Guidance. The recommended elevations ensure structural safety and accessibility under both current conditions and future scenarios extending to 2100. The vulnerability assessment confirms that these elevations are adequate for most of the evaluated sea level rise trajectories, with limited exceedances occurring under only the most extreme conditions.

The study also evaluated the operational reliability of ferry service by quantifying wave-induced berth downtime across 27 years of hindcast data. Without a breakwater, significant downtime occurs, particularly in summer months. The addition of either breakwater configuration results in a near-zero average monthly downtime and ensures that service reliability remains above typical thresholds throughout the year. The minimal difference in performance between the 300-ft and 400-ft breakwater options provides a clear argument in favor of the more efficient design.

Sediment transport modeling provides locally validated estimates of sedimentation rates and maintenance dredging needs for the approach channel and berth pockets. The six-year simulation indicates that sedimentation rates are highest in the first year following dredging and then stabilize, with average annual rates of 0.20 feet in the channel and 0.09 feet in the berth pockets. Based on these rates, maintenance dredging is expected to be required every 10 to 15 years for the channel and every 22 to 33 years for the berth pockets, depending on the operational depth maintained. These intervals support long-term planning and operational reliability, though ongoing

monitoring and data collection are recommended to refine future predictions and maintenance strategies.

Overall, this study provides a sound technical foundation for advancing the BWTPF project into permitting, detailed design, and construction. The recommended design is robust, cost-effective, and aligned with regional transportation and climate adaptation objectives. The methods applied reflect best practices in coastal engineering and support the long-term resilience and reliability of the ferry terminal infrastructure under both current and projected future coastal conditions.

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Appendix A – eTrac Memo, vertical datum conversion

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Memo

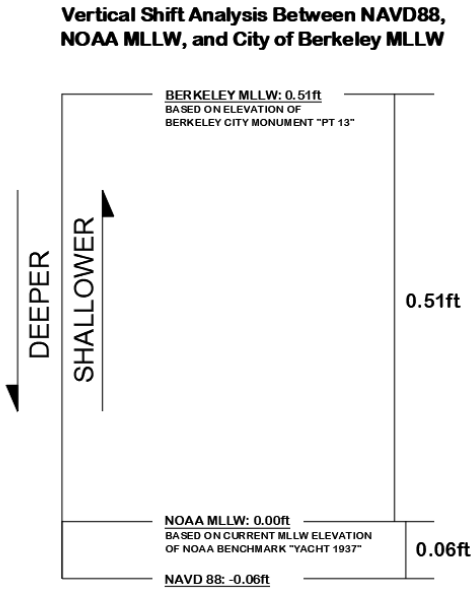
Title: Vertical Shift Analysis Between NOAA MLLW and City of Berkeley MLLW

Project: Berkeley Water Transportation Pier Ferry (BWTPH)

Dredging operations at the Berkeley Marina are based on the current MLLW elevation at NOAA benchmark “YACHT 1937”. The MLLW elevation of "YACHT 1937" is documented at 10.99ft. Using an eTrac GNSS base station, eTrac observed an elevation on this benchmark of 10.93ft NAVD88. This would result in a vertical shift of the 0.06ft shallower to bring the NAVD88 elevation to the MLLW elevation.

For the Berkeley Water Transportation Pier Ferry Project, the provided vertical control uses the Berkeley City Monument “Pt 13” with an MLLW elevation of 19.96ft. Using the same eTrac GNSS base station, eTrac observed an elevation at this benchmark of 19.39ft NAVD88. This would result in a vertical shift of 0.57ft shallower to bring the NAVD88 elevation to the MLLW elevation.

Based on the eTrac observations at “YACHT 1937” and “PT 13”, the difference between the two NAVD88-to-MLLW shifts is 0.51ft. To adjust data based on “YACHT 1937” MLLW to bring it to “PT 13” MLLW, the data would be vertically adjusted shallower by 0.51ft.



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Appendix B Sea Level Rise risk assessment

DECEMBER 2025

BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT

SEA LEVEL RISE RISK ASSESSMENT

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DECEMBER 2025

BERKELEY WATER PIER FERRY (BWTPF) PROJECT

SEA LEVEL RISE RISK ASSESSMENT

PROJECT NO.

A272767

VERSION	DATE OF ISSUE	DESCRIPTION	PREPARED	CHECKED	APPROVED
Final	12/12/2025	Sea Level Rise Risk Assessment	GAML	BRLW	JMC/ JRVS

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Attachment A - Regional Shoreline Adaptation Plan Maps for Project Vicinity

EXECUTIVE SUMMARY

This executive summary describes the sea level rise scenarios, design inputs, and risk assessment results for the BWTPF project along the Berkeley waterfront. It focuses on functional performance of the breakwater, ferry pier deck, and shoreline plaza through the 50-year design life (to 2075) and under end-of-century conditions (2100).

Highlights

- Design life and scenario: intermediate SLR scenario with allowance of 1.6 ft by year 2075 (project life).
- Still water and total water levels: Design SWL = 11.9 ft MLLW (Extreme tide 10.3 ft + SLR 1.6 ft). Structure minimum elevations were set using TWL (SWL + wave runup/green water).
- Performance through 2100: Under intermediate scenario (SLR 3.1 ft), all elements remain functional; overtopping increases but stays within tolerable ranges for managed public areas.
- High scenario stress test: Under high scenario (SLR 6.5 ft by 2100), plaza and deck are inundated; breakwater experiences overtopping far above pedestrian safety thresholds, requiring access restrictions and adaptive measures.

Inputs and Scenarios

- Nearest tide gauge used: San Francisco; Alameda was evaluated for comparison
- Vertical datum: Berkeley MLLW (MLLW = NAVD88 + 0.57 ft)
- Extreme Water Level: 100-yr (1% AEP) = 10.3 ft MLLW
- Design SWL: 11.9 ft MLLW (10.3 ft MLLW + 1.6 ft SLR)
- Design SLR allowance: 1.6 ft (Intermediate scenario, 2075)
- SLR scenarios considered for vulnerability assessment: Intermediate, Intermediate-High, High
- SLR projections (selected years) 2075: 1.6/2.6/3.6 (Int/Int-High/High); 2100: 3.1/4.8/6.5 (Int/Int-High/High)
- Structures assessed: Breakwater, Fixed Pier Deck, Shoreline Plaza

Results Summary

Number	Scenario	Year	SLR (ft)	SWL (ft MLLW)	Breakwater Status	Deck Status	Plaza Status
0 (Design)	Intermediate	2075	1.6	11.9	OK	OK	OK
1	Intermediate	2100	3.1	13.4	OK	OK	OK
2	Intermediate-High	2075	2.6	12.9	OK	OK	OK
3	Intermediate-High	2100	4.8	15.1	OT	OT	OT
4	High	2075	3.6	13.9	OT	OT	OT
5	High	2100	6.5	16.8	OT	Inundation	Inundation

OK: Element is above the design water elevation and will not be subjected to flooding during the scenario considered (overtopping < 1 l/m/s)

OT: Element will be subjected to overtopping during the scenario considered

1 Introduction

This report presents a comprehensive Sea Level Rise (SLR) Risk Assessment for the Berkeley Water Transportation Pier Ferry (BWTPF) Project, a transformative infrastructure initiative located along the Berkeley waterfront in California. The project intends to revitalize the existing Berkeley Pier by converting it into a modern, dual-purpose facility that functions both as a public ferry terminal and a recreational pier. This redevelopment is a critical component of the city's broader vision to enhance regional mobility, improve climate resilience, and expand public access to the shoreline. Figure 1-1 shows the location of the BWTPF Project within the regional shoreline context.

As part of the permitting process, the San Francisco Bay Conservation and Development Commission (BCDC) requires that all shoreline projects evaluate flood risk using the 100-year storm event, incorporating the best available projections of future sea level rise. In alignment with this requirement, the BWTPF SLR Risk Assessment applies the latest guidance from the 2024 State of California Sea-Level Rise Guidance by the Ocean Protection Council (OPC, 2024) and the BCDC Climate Change Policy Guidance (BCDC, 2025b). These frameworks emphasize a precautionary, science-based approach to infrastructure planning in the face of accelerating climate change.

This assessment evaluates the vulnerability of key project components, including the breakwater, fixed pier, floating structures, gangway, and shoreline plaza, under a range of sea level rise scenarios and planning horizons. It also identifies design elevations and performance thresholds that ensure the infrastructure remains functional, safe, and adaptable through the end of design life and beyond. The analysis incorporates both still water and total water level projections, considers overtopping and inundation risks, and provides recommendations for long-term resilience and adaptive capacity.

Given the project's 50-year design life, the assessment applies the Intermediate Sea Level Rise Scenario as the baseline for establishing design elevations and evaluating long-term performance. Additional analyses under the Intermediate-High and High scenarios for both 2075 and 2100 were completed to understand how the project may perform under more extreme future conditions. These evaluations provide a forward-looking basis for determining the elevations and adaptive capacity needed to ensure that the breakwater, pier, floating structures, and shoreline plaza continue to function safely and reliably through the design horizon and into later-century conditions.

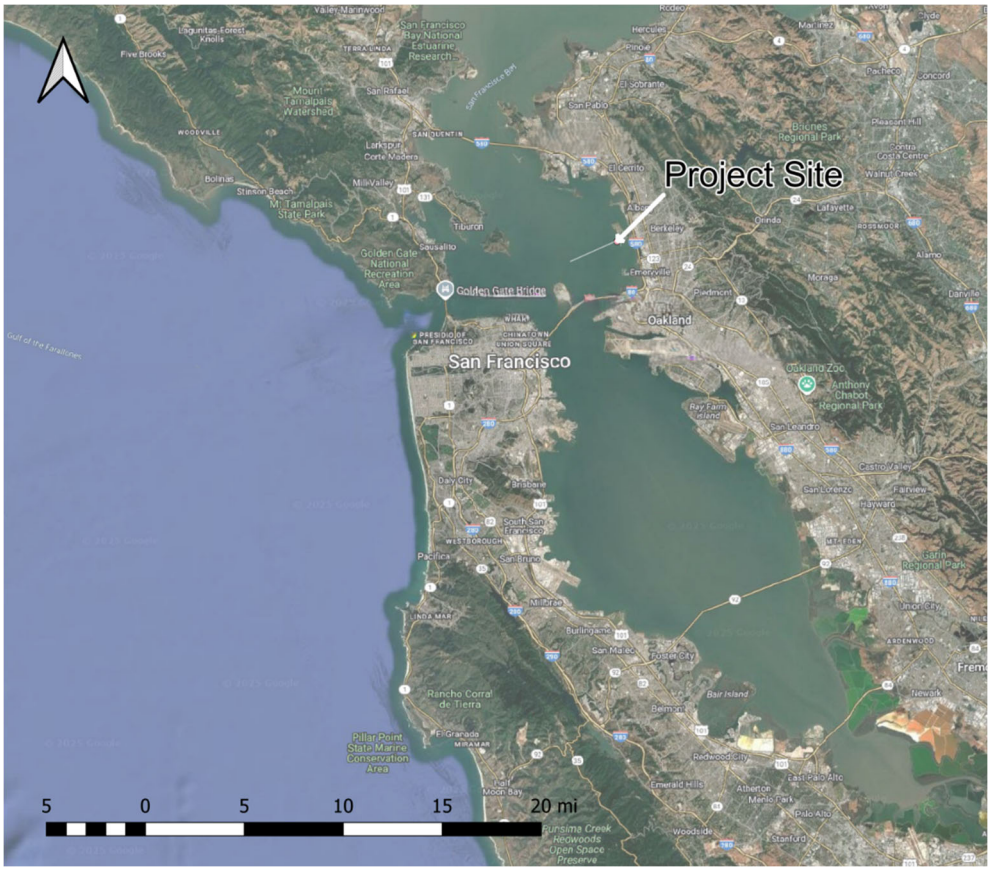


Figure 1-1: Project site

2 Current policies

The development of this assessment was guided by recommendations outlined in a set of planning and policy documents that collectively inform current approaches to sea level rise adaptation in the region.

- State of California Sea Level Rise Guidance, 2024 Science & Policy Update (OPC, 2024)
- BCDC Regional Shoreline Adaptation Plan, December 2024 (BCDC, 2024)
- BCDC San Francisco Bay Plan, March 2025 (BCDC, 2025a)
- BCDC San Francisco Bay Plan Climate change Policy Guidance, May 2025 (BCDC, 2025b)

The approach to defining sea level rise allowances for projects in California has evolved over recent years, reflecting ongoing advancements in scientific understanding and policy development. Recognizing the accelerating pace of climate change and its impacts on coastal regions, the state has continuously refined its approach to ensure that planning efforts are scientifically robust, precautionary, and adaptable to evolving climate projections.

The probabilistic approach used in the feasibility study was informed by data from the Intergovernmental Panel on Climate Change Fifth Assessment Report (IPCC, 2014), which evaluated sea level rise projections based on various Representative Concentration Pathways (RCPs). These pathways represented different greenhouse gas emissions scenarios and their corresponding impacts on global climate systems, including sea level changes.

There have been advancements in sea level rise science since then, including refinements in climate models, improved ice-sheet dynamics understanding, and better regional projections. These scientific advancements compelled a reassessment and update of the guidance. The 2024 California Sea Level Rise Guidance (OPC, 2024) integrates these updates, drawing from the most recent IPCC's Sixth Assessment Report (AR6) - (IPCC M.-D. V., 2021) and the Special Report on the Ocean and Cryosphere in a Changing Climate (SROCC) - (IPCC H.-O. P.-D., 2019). These reports provide deeper and updated insights into the primary drivers of sea level rise, such as the thermal expansion of seawater, the melting of glaciers and ice sheets, and shifts in ocean circulation patterns.

Building on this scientific foundation, the San Francisco Bay Conservation and Development Commission (BCDC) incorporated the state-defined projections into its

climate adaptation policies and planning frameworks. These projections serve as the baseline for evaluating shoreline vulnerability and guiding long-term adaptation strategies throughout the Bay Area. BCDC's updated Climate Change Policy Guidance (2025), the Regional Shoreline Adaptation Plan (2024), and the San Francisco Bay Plan (2025) all reference these projections to ensure consistency in how sea level rise is addressed in permitting, planning, and regional coordination.

By adopting the state's projections, BCDC has established a unified approach that supports coherent adaptation planning across jurisdictions.

2.1 Scientific advancements in the California 2024 guidance

One of the most notable advancements in the 2024 guidance is the shift from an exclusive reliance on probabilistic projections to a more comprehensive approach that incorporates a range of emissions pathways. This shift acknowledges the uncertainties associated with ice-sheet dynamics and tail-risk events, such as the potential collapse of the West Antarctic Ice Sheet, which could contribute to more extreme sea level rise scenarios. By spanning multiple emissions scenarios, the 2024 guidance enables decision-makers to better anticipate a range of potential outcomes and implement precautionary adaptation measures.

Another major update is the inclusion of localized assessments of vertical land motion, which were not as comprehensively addressed in the 2018 guidance. Vertical land motion, which includes both uplift and subsidence, can significantly affect relative sea level changes at specific locations. The 2024 guidance incorporates detailed localized projections for each of the 14 NOAA coastal tide gauges in California, offering site-specific information for planners. This improvement is particularly relevant for regions like the San Francisco Bay Area, where additional tide gauges in Alameda and Port Chicago provide refined local data.

The guidance also incorporates the latest methodologies for high-resolution coastal modeling, integrating new observational data from satellite altimetry, GPS, and advanced coastal monitoring programs. These data sources enhance the accuracy of sea level rise projections and improve the capacity to model storm surge, tidal influences, and wave dynamics in conjunction with rising sea levels. Additionally, the guidance accounts for compound flood risks, which consider the interaction of sea level rise with extreme precipitation events and riverine flooding.

2.2 Sea Level Rise planning framework

The BCDC Climate Change Policy Guidance recommends a stepwise process for incorporating sea level rise scenarios into planning and decision-making. This framework ensures that adaptation efforts are methodical, iterative, and responsive to new scientific findings. The steps include:

1 Identifying the Nearest Tide Gauge (Section 4)

2 Evaluating the Planning or Project Time Horizon (Section 5)

3 Choosing Multiple Sea Level Scenarios for Vulnerability Assessment and Evaluate Project Risk Tolerance (Section 6)

4 Conducting Risk Assessment (Section 7 and 8.2)

5 Exploring Adaptation Options (Section 8.3)

6 Preparing Adaptative Management Plan

For the Berkeley Water Transportation Pier Ferry Project, the definition of sea level rise allowance is critical in determining the minimum structural elevations required for the various components of the project. This includes ensuring that the elevations of the breakwater, pier, and shoreline plaza meet the anticipated future sea level rise conditions. The purpose of incorporating sea level rise allowances is to build in resilience to potential future conditions, ensuring the longevity and safety of the infrastructure over its designed lifespan.

To define these allowances, this study will follow a structured approach outlined in the 2024 California Sea Level Rise Guidance and the recent 2025 San Francisco Bay Plan Climate Change policies. By following this methodology, the project aims to minimize future risks while maintaining operational integrity based on the latest scientific knowledge.

3 Definitions and project water elevations

3.1 Water elevation definitions

The following definitions, related to water elevations and components of flood hazards, are used in the subsequent sections.

- SWEL: Still-Water Elevation is the water elevation that results from the superposition of astronomical tide, storm surge, and tsunami (if applicable) over an observation period. It does not include the effects of wave action (dynamic effect) and therefore constitutes a static water surface elevation that remains constant for minutes to hours. If the SWEL exceeds a certain shoreline elevation, the area behind it will experience extreme flooding.

- TWL: Total Water Level represents the maximum elevation the water reaches as the result of superimposing the static still-water elevation (SWEL) with the dynamic effects such as wind-waves, swell, and related wave runup. The TWL is reached for a few seconds at a time, but it is repeated during a storm event. If TWL exceeds a certain shoreline or structure elevation, the area behind it will be subject to overtopping, but the magnitude of flooding will vary.

The distinction between SWEL and TWL is important to note, particularly along coastal areas, because a shoreline exceeded by the SWEL constitutes an inundation or large-scale flooding scenario whereas embankments exceeded by TWL constitute an overtopping scenario that may only cause short-term nuisance flooding.

3.2 Project tidal elevations

3.2.1 Vertical datum

This project adopted the City of Berkeley vertical datum and refers to it as “MLLW” throughout this document. The Berkeley MLLW datum is 0.57 ft shallower than the NAVD88 vertical datum (+0.57’ NAVD88). The referenced elevations are given in feet-Berkeley MLLW (MLLW), unless noted otherwise.

3.2.2 Still Water Elevation (SWEL)

The San Francisco Bay Tidal Datums and Extreme Tides Study (AECOM, 2016) updated the tidal datums and extreme tides for the San Francisco Bay. The study modeled and presented detailed extreme tide information for the entirety of the Bay

shoreline. The model captures the Bay's hydrodynamic at a scale and resolution that exceeded available tide gauge data.

Table 3-1 reproduces the extreme tide elevation for point ID 519, coinciding with the Berkeley Pier. The provided elevations exclude waves impacts but include astronomical tides and storm surge.

The extreme water level for current conditions is 10.3 ft MLLW, with a return period of 100 years (1% Annual Exceedance Probability - AEP).

Table 3-1: Extreme Tide Elevations at Point ID 519 (AECOM, 2016)

Return Period (years)	Extreme Tide Elevation [ft-NAVD88]	Extreme Tide Elevation [ft-MLLW]
1	7.4	8.0
10	8.5	9.0
50	9.3	9.9
100	9.7	10.3

The design still water level at the project location is shown in Table 3-2. This elevation excludes waves impacts but includes astronomical tides, storm surges and sea level rise allowance for the design life of the project.

Table 3-2: Design Still Water Level

Parameter	Elevation [ft-NAVD88]	Elevation [ft- MLLW]
Extreme Water Level (1% AEP)	9.7	10.3
Sea Level Rise Allowance	1.6	1.6
Design Still Water Level	11.3	11.9

3.2.3 Total Water Level (TWL)

The TWL is evaluated for each structure, determining the minimum structure elevation used to evaluate the sea level rise risk in Section 8.2. The TWL was determined using combined extreme conditions (1% AEP storm surge and 1% AEP wave heights) adding a conservative margin to support the protection strategy and define minimum structure elevations.

3.2.4 Sea Level Rise projection

For the BWTPF Project, the projected sea level rise proposed by the California Natural Resources Agency and the Ocean Protection Council on “State of California Sea-Level Rise Guidance: 2024 Update” (OPC, 2024) was used.

Steps 1 through 4 of the California 2024 guidance were followed to define the design sea level rise allowance. The project does not consider a phased approach for adaptation, so steps 5 and 6 were not considered. These steps are described from Section 4 to 8 of this document.

4 Identifying the nearest tide gauge

The San Francisco and Alameda tide gauges are the nearest monitoring stations to the BWTFP site. Although the Alameda gauge is geographically closer, the San Francisco gauge was selected for use in this study as a more conservative basis for sea level rise projections. This choice is aligned with precautionary planning principles, ensuring that infrastructure design is robust under higher-end risk scenarios.

As shown in Table 4-1, the differences between projections from the two gauges can be significant. For the year 2100, San Francisco projections exceed those of Alameda by approximately 40% under the low-end scenario and by about 5% under the high-end scenario. These variations underscore the importance of selecting a tide gauge that supports risk-averse design decisions, particularly for long-term infrastructure. See Section 6 for definitions of sea level rise scenarios referenced in Table 4-1.

Table 4-1: Differences for Sea Level Rise Allowance (Year 2100) for San Francisco and Alameda Tide Gauges – relative to year 2000

Sea Level Rise Scenarios for Year 2100, in feet					
Tide Gauge	Low	Int-Low	Intermediate	Int-High	High
San Francisco	1.0	1.6	3.1	4.8	6.5
Alameda	0.6	1.2	2.8	4.4	6.2
Difference	-40%	-25%	-10%	-8%	-5%

5 Evaluating project time horizon

For the BWTPF, a design life of 50 years has been established, placing its planning horizon around the year 2075. According to the 2024 California Sea Level Rise Guidance, the time horizon is a critical determinant in selecting appropriate sea level rise values, as it directly influences design assumptions and risk tolerance. Given the absence of a phased adaptation strategy in this project, it is essential to incorporate a sea level rise projection that accounts for the full 50-year operational lifespan from the outset. This ensures that all structural elements are adequately elevated and resilient to anticipated sea level conditions throughout their service life, without reliance on future upgrades or retrofits. In addition to designing for 2075 conditions, the project will also include a vulnerability assessment for the year 2100 to evaluate longer-term exposure and inform future planning considerations, especially considering evolving climate scenarios.

This dual-focus strategy aligns with best practices in climate resilience by balancing present-day investment with future risk awareness.

6 Sea Level Rise scenarios and risk tolerance

California's 2024 Sea Level Rise Guidance outlines five distinct sea level rise scenarios: Low, Intermediate-Low, Intermediate, Intermediate-High and High. Each scenario is defined and labeled according to the target value of global mean sea level (GMSL) rise in 2100:

- Low (0.3 m or 1.0 ft by 2100)
- Intermediate-Low (0.5 m or 1.6 ft by 2100)
- Intermediate (1.0 m or 3.3 ft by 2100)
- Intermediate-High (1.5 m or 4.9 ft by 2100)
- High (2.0 m or 6.6 ft by 2100)

These scenarios are based on updated global climate projections, including findings from the IPCC Sixth Assessment Report, and reflect a range of possible future conditions driven by varying levels of greenhouse gas emissions, rates of ice sheet melt, and global policy action. Each scenario provides a different trajectory for sea level rise, with the Low and Intermediate-Low Scenarios reflecting highly optimistic futures requiring accelerated decarbonization and large-scale carbon removal technologies. In contrast, the High and Extreme Scenarios are considered more precautionary, particularly in the context of high warming levels and low-confidence but potentially high-impact ice sheet loss mechanisms.

The selection of a scenario for planning or design purposes depends on multiple factors, including the project's design life, the criticality of the asset, the level of acceptable risk, and the socio-economic and environmental consequences of over- or under-estimating future conditions. For most applications with medium to high risk aversion, the guidance recommends evaluating at least the Intermediate and Intermediate-High Scenarios. The High Scenario may be included when the consequences of failure are severe or when a worst-case analysis is needed. Conversely, the Low and Intermediate-Low Scenarios are generally not recommended for planning or design, as they are unlikely to be achieved under current global climate trajectories without transformative change.

For the BWTPF Project, the Intermediate Scenario from the 2024 California Sea Level Rise Guidance was selected to establish sea level rise design values. This scenario provides a reasonable and practical estimate for projected sea level rise by mid-to-late century, aligning with the project's 50-year design life.

California Sea-Level Rise Guidance (OPC, 2024) and San Francisco Bay Climate Change Policy (BCDC, 2025b) states that the Intermediate Scenario is recommended for sea level rise projections up to and including 2050. It also states that the Intermediate-High Scenario may be appropriate for multiuse paths and public access that are part of a transportation network or provide services that the public relies on. While the Intermediate Scenario is used for defining minimum elevations and structural thresholds, a broader risk assessment will also be conducted using the Intermediate-High and High Scenarios to account for more extreme, yet plausible, sea level rise outcomes and ensure long-term resilience of the project components.

The median value of sea level rise scenarios provided by the 2024 California Guideline (OPC, 2024) incorporate the local estimate of vertical land motion. Across all locations, the long-term rate of past vertical land motion is assumed to persist into the future, and future relative sea level rise is adjusted accordingly. This average rate of vertical land motion is near zero (0.1mm or 0.003ft uplift per year) at the project site.

7 Conducting risk assessments

For each marine structure included in the BWTPF Project (specifically the deck, breakwater, and plaza) a recommended minimum elevation will be determined based on a design water level that incorporates sea level rise projections from the Intermediate Scenario and a 50-year design life. This recommended elevation will serve as a baseline to ensure the structures are appropriately adapted to anticipated future conditions.

The final design elevations established by the project team, are informed by the minimum recommended values and consider additional factors such as constructability, aesthetics, operational requirements, and stakeholder input.

The structure-specific sea level rise risk assessment will evaluate potential risks under the Intermediate-High and High scenarios and extend the analysis beyond the 50-year design horizon, through the year 2100. Table 7-1 shows the sea level rise allowance used in design and vulnerability assessment for the marine structures.

Table 7-1 Risk Assessment Conditions

Condition	Scenario	Year	SLR Allowance (ft)	Differences from Design (ft)
Design	Intermediate	2075	1.6	0.0
Assessment 1	Intermediate	2100	3.1	1.5
Assessment 2	Intermediate- High	2075	2.6	1.0
Assessment 3	Intermediate- High	2100	4.8	3.2
Assessment 4	High	2075	3.6	2.0
Assessment 5	High	2100	6.5	4.9

8 Project design features and analysis

The project involves the design and implementation of a new ferry terminal at the foot of the existing Berkeley Pier, consisting of 1) Breakwater, 2) Fixed Pier, 3) Floats with Guide Piles, 4) Gangway, and 5) Shoreline Plaza, see Figure 8-1. Among these elements, the float, guide piles, and gangway were excluded from the sea level rise risk analysis. The rationale for their exclusion is detailed in the following points.

- Float and Guide Piles: the float is able to move vertically along the piles with the tides. The float is not vulnerable to flooding as it is able to rise and lower with tides and the top of the guide piles is set at an elevation above the TWL for all years considered.

- Gangway: the gangway connects the float and the fixed pier. The gangway is an aluminum structure hinged at the fixed pier end at elevation 17.6 ft and has rollers at the float end allowing it to rise and lower with the float. The lowest elevation of the gangway under flooding is the same as the fixed pier and has the same expected exposure to flooding, that will be estimated in the following sections.

A summary of the sea level rise evaluation for the remaining project components is presented in the subsequent sections

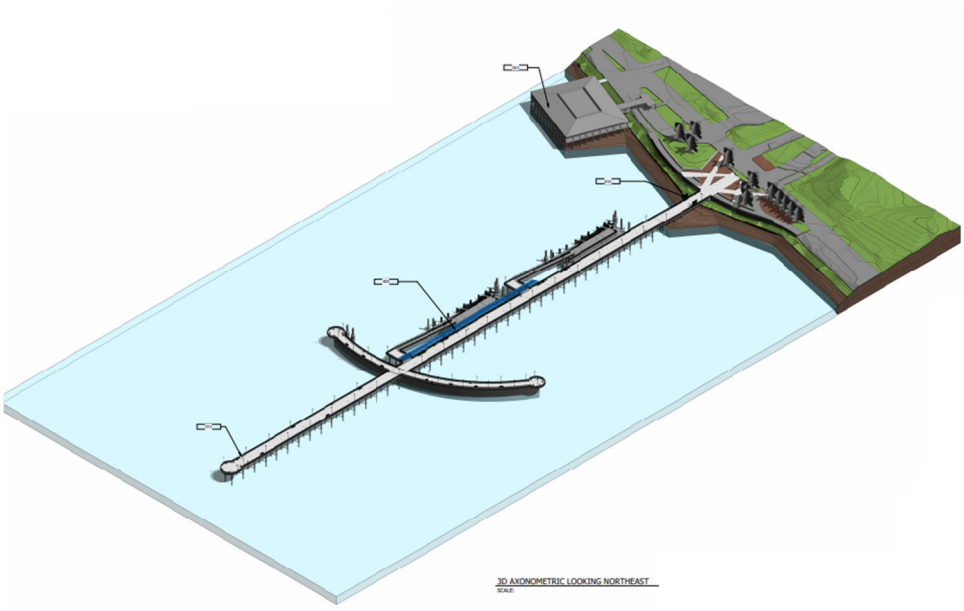


Figure 8-1: Berkeley Pier – Axonometric View (30% Submittal)

8.1 Sea Level Rise projections

The sea level rise projections are provided in Table 8-1. These sea level rise scenarios and projections for San Francisco are also adopted by FEMA/NOAA which coincide with the California 2024 Guidance (OPC, 2024). Both recommendations follow the same methodology from the most recent IPCC's Sixth Assessment Report (AR6) and the Special Report on the Ocean and Cryosphere in a Changing Climate.

The design sea level rise allowance for the project, based on a 50-year design life (2025–2075) and the Intermediate risk scenario, is 1.6 feet. Risk assessments were conducted for the scenarios highlighted in the table, specifically for the end of the project life in 2075 and the end-of-century condition in 2100. While the guidance from BCDC (BCDC, 2025b) recommends including a mid-century (2050) analysis, this study incorporates the end-of-design-life scenario, which is considered more conservative than the 2050 projection.

Table 8-1: Scenarios and Projection FEMA/NOAA and California Coastal Commission Sea Level Rise Policy Guidance 2024 Update and Design Interpolation for 2075

	Sea Level Rise Scenarios and Projection [feet]			
Year	Intermediate-Low	Intermediate	Intermediate-High	High
2050 (Mid-Century)	0.6	0.8	1.0	1.3
2070	1.0	1.4	2.2	2.9
2075* (Project Life)	1.1	1.6	2.6	3.5
2080	1.2	1.8	3.0	4.1
2100 (End of Century)	1.6	3.1	4.8	6.5

*Interpolation

8.2 Sea Level Rise risk assessment

The breakwater, deck, and plaza elevations were established based on the minimum recommendations from the 2075 planning horizon and the Intermediate Scenario from the 2024 California Sea Level Rise Guidance. This minimum elevation is the Total Water Level and includes the wave impacts on each structure added to the Still Design Water Level (11.9 ft MLLW).

- Breakwater: the minimum elevation includes an allowance for runup and acceptable overtopping on a vertical wall.
- Deck: green water elevation was included based on the modelled incident design wave along the deck. It is important to note that the deck is designed to withstand the wave loads without damage, to even higher than the design wave loads.
- Plaza: the wave impact (runup and acceptable overtopping) considering the design wave height at the toe of the revetment (partially protected by the breakwater) was added to the still water level.

Table 8-2 summarizes these minimum elevations for the design life (50 years) of the project, considering the Intermediate SLR scenario. The final design elevations are also included in the table. These final elevations were selected by the project team to meet broader functional and operational requirements. (COWI, 2025) provides details of the calculations for each minimum elevation structure including the overtopping volumes.

Figure 8-2 presents the typical breakwater cross-section, highlighting the crest elevation set at 17.6 ft MLLW. Figure 8-3 shows the corresponding pier section, where the elevation gradually decreases from 17.6 ft MLLW down to 16.5 ft MLLW at the connection with the plaza.

Table 8-2 Minimum and design elevations

Elevation (ft Berkeley MLLW)	Breakwater	Deck	Plaza
Minimum	17.6	15.2	15.8
Design	17.6	16.5-17.6*	16.5

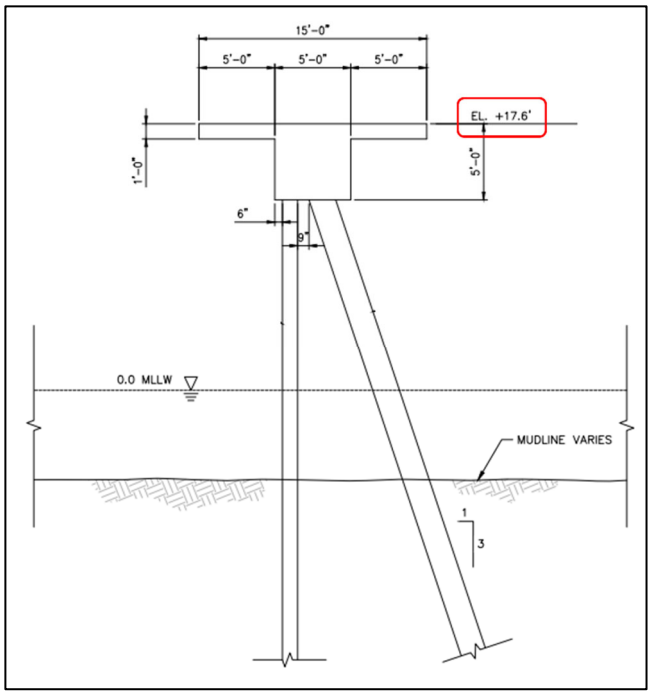


Figure 8-2: Breakwater section

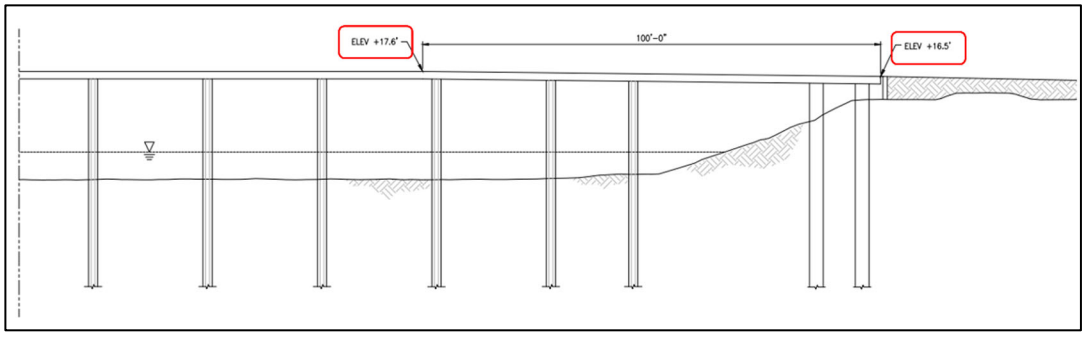


Figure 8-3: Pier and Plaza Elevation

To evaluate the long-term resilience of these design elevations, a Sea Level Rise Risk Assessment was conducted. The assessment considered the impact of future conditions beyond the 50-year design life and more conservative sea level rise scenarios. These conditions include projections for the years 2075 and 2100, under Intermediate, Intermediate-High and High scenarios.

The mean overtopping discharge rate is a key indicator of safety and hydraulic performance, particularly for coastal infrastructure adjacent to publicly accessible areas. It provides a practical measure of the volume of water expected to pass over the crest of a structure during storm events and is used to assess whether overtopping

remains within acceptable thresholds for pedestrian safety, user comfort, and structural serviceability.

In this assessment, the mean overtopping discharge rate will be evaluated for each sea level rise assessment scenario to determine whether design criteria are exceeded, to identify potential consequences for public use and structural integrity, and to support a risk-based understanding of when and where adaptive measures may be necessary. This approach ensures a consistent, safety-focused evaluation of long-term performance under evolving sea level and wave conditions.

The design threshold for overtopping volume, followed a conservative approach, by establishing mean overtopping volume to be less than 0.1 l/s/m. For this assessment, the criteria is stretched to the recommended range of 10-20 l/s/m for people at seawall and wave height 3.2 ft (EurOtop, 2018).

The following sections summarize the anticipated performance of the breakwater, deck, and plaza under each assessed sea level rise scenario. The analysis focuses on the ability of each component to continue functioning safely and effectively in the face of rising water levels and associated elevated wave impacts.

All the vertical elevations in this report are referenced to datum Berkeley MLLW.

8.2.1 Assessment 1: intermediate scenario – year 2100

The SLR allowance for year 2100 in an intermediate scenario (median) is 3.1 ft. When applied to the current 1% AEP extreme water level of 10.3 ft, this results in a projected design still water level of 13.4 ft (see Table 8-3). At this elevated water level, key shoreline and marine infrastructure elements, including the breakwater, ferry pier deck, and shoreline plaza, were reassessed to evaluate their functional and safety performance. The following subsections summarize the expected impacts of this sea level rise on each feature, focusing on crest freeboard reduction, wave overtopping behavior, wave slamming potential, and exposure to flooding or drainage challenges.

Table 8-3 Design Still Water Level for Assessment 1 – Intermediate Scenario, Year 2100

Parameter	Elevation
Extreme Water Level (1% AEP)	10.3 ft-MLLW
Sea Level Rise 2100	3.1 ft
Design Still Water Level	13.4 ft-MLLW

Breakwater: Under Assessment 1, the projected sea level rise of 3.1 ft increases the resulting mean overtopping discharge to approximately 1.0 l/s/m.

This value is near the upper threshold of 1.0 l/s/m, commonly referenced in (EurOtop, 2018) for safe pedestrian conditions in areas where wave action is non-impulsive and wave heights are moderate. Although this value exceeds the original design criterion of 0.1 l/s/m, selected as a conservative threshold for public safety behind a vertical wall, the condition remains within the broader acceptable range of 10–20 l/s/m identified in (EurOtop, 2018) for low wave conditions (H<3.3 ft) and well-managed pedestrian environments.

The breakwater remains functionally and operationally viable under this scenario, with overtopping discharges that exceed the original conservative design target but still fall within acceptable tolerances for safe public use.

Deck: Under Assessment 1, the projected sea level rise increases to 3.1 ft, which is 1.5 ft higher than the design SLR allowance used to establish deck elevation. This results in a revised still water level of approximately 13.4 ft MLLW, compared to the original 11.9 ft design level.

The combined water level (maximum green water elevation) under Assessment 1 is expected to increase to approximately from 15.7 to 16.7 ft, assuming the same wave height remains applicable.

The design deck elevation of 16.5 to 17.5 ft MLLW might be exceeded at some sections of the deck (closer to shore) during the design storm event. The structural integrity and operational function of the platform are therefore expected to be preserved through 2100 under the Intermediate Scenario, since platform components are designed to withstand these wave loads.

Plaza: Under Assessment 1 (Intermediate Scenario – Year 2100), the average overtopping discharge at the plaza is approximately 0.3 l/s/m, based on the design elevation of 16.5 ft MLLW. While this value exceed the conservative threshold of 0.1 l/s/m typically used for pedestrian areas, it remains below the broader upper limit of 10–20 l/s/m identified in (EurOtop, 2018) as tolerable for public areas where wave conditions are non-impulsive and the site includes adequate drainage and splash management.

At this discharge level, minor surface wetting and intermittent splash may occur during storm events or extreme tides, but public safety is not expected to be compromised, and the area would remain generally accessible and functional.

The plaza continues to perform within acceptable limits under Assessment 1 conditions. This result indicates that the structure is resilient through 2100 under the Intermediate Scenario, and no immediate adaptation measures are required beyond ongoing monitoring and operational readiness.

8.2.2 Assessment 2: intermediate-high scenario – year 2075

This assessment considers a projected sea level rise of 2.6 ft by the year 2075, consistent with the Intermediate-High Scenario from the 2024 California Sea Level Rise Guidance. This value represents a 1.0 ft increase above the design sea level rise allowance and reflects a more conservative planning trajectory that accounts for accelerated ice melt and higher-end global emissions outcomes.

The resulting increase in still water levels affects the performance of all coastal infrastructure elements, particularly in terms of reduced freeboard, increased overtopping, and greater exposure to flooding impacts. While the structures were not originally designed for this higher scenario, their vulnerability under these conditions provides important insight into operational limits and informs management considerations.

The following items describe the expected performance of the breakwater, ferry pier deck, and shoreline plaza under this scenario, including overtopping behavior, wave impact risks, and implications for public access and infrastructure resilience.

Table 8-4 Design Still Water Level for Assessment 2 – Intermediate Scenario, Year 2100

Parameter	Elevation
Extreme Water Level (1% AEP)	10.3 ft-MLLW
Sea Level Rise 2100	2.6 ft
Design Still Water Level	12.9 ft-MLLW

Breakwater: Under Assessment 2 (Intermediate-High Scenario – Year 2075), the projected sea level rise allowance increases to 2.6 ft, resulting in a still water level of 12.9 ft MLLW, approximately 1.0 ft above the design condition. This reduces the freeboard at the breakwater crest increasing the calculated mean overtopping discharge to approximately 0.5 l/s/m.

While this discharge exceeds the conservative design threshold of 0.1 l/s/m, it remains well within the tolerable range of 10–20 l/s/m for public areas under non-impulsive wave conditions, as described in the guidelines (EurOtop, 2018). Given the vertical structure and the wave climate characterized by design wave height of 2.6 ft, overtopping at this level is not expected to pose a safety risk to pedestrians.

In conclusion, the breakwater remains functional and safe under this scenario, with overtopping discharge levels that, while exceeding design targets, are still considered tolerable and manageable for the intended public use behind the structure.

Deck: Under Assessment 2, the projected sea level rise allowance increases to 2.6 ft, which represents a 1.0 ft rise above the design condition. This elevates the still water level to approximately 12.9 ft, compared to the original design level of 11.9 ft. The green water elevation could reach 16.2 ft.

Compared to the minimum deck design elevation of 16.5 ft, this results in a residual clearance of 0.3 ft, which remains within a safe operational range. The platform still provides sufficient clearance to maintain operational safety and performance under most storm conditions.

Assessment 2 conditions result in a reduction in freeboard compared to the design case but do not compromise the deck's performance.

Plaza: A sea level rise allowance of 2.6 ft is considered, 1.0 ft above the project's design scenario. The shoreline plaza, requiring a minimum elevation of 15.8 ft based on design calculations, was designed to an elevation of 16.5 ft, providing an additional margin of safety to support long-term usability, drainage efficiency, and integration with adjacent site elements.

Under this assessment condition, the calculated mean overtopping discharge is close to zero, indicating that the existing crest elevation and shoreline protection features continue to effectively prevent wave overtopping, even with the higher still water level. This suggests that no flooding or splash impact is expected during typical storm or tidal events through 2075, and the plaza can remain fully functional and safe for public access under these conditions.

This performance reinforces the effectiveness of the elevated design elevation and the current revetment slope in managing both sea level rise and storm wave activity. It also reflects the conservative design approach taken during planning, which successfully accommodates more extreme projections without the need for immediate adaptation.

8.2.3 Assessment 3: intermediate-high scenario – Year 2100

This assessment corresponds to the Intermediate-High Scenario for the year 2100, with a projected sea level rise of 4.8 ft (Table 8-5). This value is 3.2 ft higher than the design allowance and reflects a high-end trajectory that includes faster-than-expected ice sheet loss and limited success in global emissions reductions.

The significant increase in water levels reduces the available freeboard for marine structures and changes wave interaction behavior during extreme tide and storm conditions. This assessment tests the long-term resilience of the breakwater, deck, and shoreline plaza under higher exposure.

Table 8-5 Design Still Water Level for Assessment 3 – Intermediate-High Scenario, Year 2100

Parameter	Elevation
Extreme Water Level (1% AEP)	10.3 ft-MLLW
Sea Level Rise 2100	4.8 ft
Design Still Water Level	15.1 ft-MLLW

Breakwater: With a projected still water level of 15.1 ft MLLW, the vertical breakwater's crest freeboard is reduced, limiting its capacity to contain wave energy. Under this condition, the mean overtopping discharge increases to approximately 10 l/s/m.

While this discharge surpasses the original design criteria, it still falls within the range of 10–20 l/s/m identified in the guidelines (EurOtop, 2018) as conditionally acceptable for pedestrian areas, particularly when wave conditions are non-impulsive, as is the case for this site (wave heights below 3.3 ft). (EurOtop, 2018) notes that at these discharge levels, public safety can still be maintained if the space behind the structure is designed or managed to handle occasional overtopping, such as splash-tolerant paving, passive drainage, and limited occupancy during adverse conditions.

Despite the exceedance of the initial design target, the breakwater continues to provide valuable coastal protection and risk reduction. The structure maintains its wave energy dissipation capacity, prevents direct storm wave attack, and significantly reduces the frequency and severity of flooding compared to an unprotected condition.

Deck: The projected sea level rise reaches 4.8 ft, representing a 3.2 ft increase over the design SLR allowance. This results in a still water level of approximately 15.1 ft, still under the design deck elevation range of 16.5-17.6 ft.

The green water elevation exceeds the deck elevation. However, the deck can resist wave loads without damage, including loads exceeding the design wave conditions, as wave loading was not a governing factor in its design.

Plaza: Under Assessment 3 conditions, the projected sea level rise reaches 4.8 ft, increasing the still water level and wave impact potential. At the design elevation of 16.5 ft MLLW, the calculated mean overtopping discharge at the plaza is less than 15 l/s/m.

This value exceeds the original conservative design target of near-zero overtopping but remains within the broader range of 10–20 l/s/m identified by (EurOtop, 2018) as conditionally tolerable for public areas when wave conditions are non-impulsive and infrastructure is designed to manage occasional surface flooding. Given the sheltered

wave height, overtopping at this level would likely result in intermittent splashing, surface wetting, and temporary accumulation of water, particularly during high tides or storm events.

While this may reduce the comfort of usability of the plaza during extreme conditions, it does not pose a critical hazard, and the underlying structure (including the revetment, seawall, and surface grading) will remain intact. The surface drainage system, if properly maintained, would be expected to accommodate these volumes without compromising the long-term functionality of the space.

8.2.4 Assessment 4: high scenario – year 2075

This assessment corresponds to the High Scenario for the year 2075, with a projected sea level rise of 3.6 ft (see Table 8-6). This value is 2.0 ft higher than the SLR design allowance.

Table 8-6 Design Still Water Level for Assessment 4 – High Scenario, Year 2075

Parameter	Elevation
Extreme Water Level (1% AEP)	10.3 ft-MLLW
Sea Level Rise 2100	3.6 ft
Design Still Water Level	13.9 ft-MLLW

Breakwater: With a projected still water level of 13.9 ft MLLW, the vertical breakwater's crest freeboard is reduced, also reducing its capacity to contain wave energy. Under this condition, the mean overtopping discharge increased to approximately 3.0 l/s/m.

While this discharge surpasses the original design criteria, it still falls under the range of 10–20 l/s/m identified in the guidelines (EurOtop, 2018) as conditionally acceptable for pedestrian areas, particularly when wave conditions are non-impulsive, as is the case for this site (wave heights below 3.3 ft). EurOtop (EurOtop, 2018) notes that at these discharge levels, public safety can still be maintained if the space behind the structure is designed or managed to handle occasional overtopping, such as splash-tolerant paving, passive drainage, and limited occupancy during adverse conditions.

Despite the reduction in crest freeboard, the vertical breakwater remains structurally sound and fully capable of withstanding the projected still water level of 13.9 ft MLLW. While overtopping volumes are minimal and not expected to compromise the integrity of the breakwater, it is advisable to implement precautionary measures, particularly during high tide or storm conditions. Limiting pedestrian access to the breakwater,

especially in areas with restricted visibility or no clear line of sight to the water, would enhance public safety without necessitating structural modifications.

Deck: Under Assessment 4, the projected sea level rise allowance increases to 3.6 ft, which represents a 2.0 ft rise above the design condition. This elevates the still water level to approximately 13.9 ft, still under the design deck elevation range of 16.5-17.6 ft MLLW.

Although the calculated green water elevation exceeds the deck height, the deck can withstand the associated wave forces, even for conditions more severe than those used for design. Wave loading did not control the deck design, and no damage is expected under these scenarios

Plaza: The shoreline plaza, originally defined with a minimum elevation of 15.8 ft, was constructed to a design elevation of 16.5 ft, providing an additional margin of safety to support long-term usability, drainage efficiency, and integration with adjacent site elements.

Under this assessment condition, the calculated mean overtopping discharge is slightly higher than the design conditions, but still under the design threshold of 1.0 l/m/s, indicating that the existing crest elevation and shoreline protection features continue to effectively prevent wave overtopping, even with the higher still water level. This suggests that no flooding or splash impact is expected during typical storm or tidal events through 2075 in this SLR scenario, and the plaza can remain fully functional and safe for public access under these conditions.

This performance reinforces the effectiveness of the elevated design elevation and the current revetment slope in managing both sea level rise and storm wave activity. It also reflects the conservative design approach taken during planning, which successfully accommodates more extreme projections without the need for immediate adaptation.

8.2.5 Assessment 5: high scenario – year 2100

This assessment corresponds to the High Scenario for the year 2100, with a projected sea level rise of 6.5 ft (see Table 8-7). This value is 4.9 ft higher than the design allowance and reflects the long-term implications of accelerated climate change. Such a significant increase in sea level would substantially reduce the vertical clearance of coastal infrastructure and challenge the assumptions made during initial design phases.

The significant increase in water levels reduces the available freeboard for marine structures and changes wave interaction behavior, particularly during extreme tide and storm conditions. This assessment tests the long-term resilience of the breakwater,

deck, and shoreline plaza under high-exposure, high-impact scenarios. The results identify critical performance thresholds under climate extremes. This assessment includes a high emission scenario beyond the design life of the project. One consequence of this extreme rise in water level is the increase in the design wave height. However, for this assessment, the original design wave height was retained, as the results already indicate elevated levels of overtopping and flooding of the designed structures.

Table 8-7 Design Still Water Level for Assessment 5 – High Scenario, Year 2100

Parameter	Elevation
Extreme Water Level (1% AEP)	10.3 ft-MLLW
Sea Level Rise 2100	6.5 ft
Design Still Water Level	16.8 ft-MLLW

Breakwater: With a projected still water level of 16.8 ft MLLW, the vertical breakwater's crest freeboard is reduced, limiting its capacity to contain wave energy. Under this condition, the mean overtopping discharge increases to over 75.0 l/s/m.

This discharge surpasses the original design criteria and falls beyond the range of 10–20 l/s/m identified in the guidelines (EurOtop, 2018) as conditionally acceptable for pedestrian areas. EurOtop (EurOtop, 2018) notes that at these discharge levels, public safety cannot be maintained. The area behind the breakwater should not be used by pedestrians under these storm conditions.

Despite this exceedance, the breakwater continues to provide valuable coastal protection. The structure maintains its wave energy dissipation capacity, prevents direct storm wave attack, and significantly reduces the frequency and severity of flooding compared to an unprotected condition.

Past the breakwater design life, particularly during high tides and storm events, overtopping at this magnitude will cause temporary splash flooding, surface water accumulation, and no accessibility in the recreational area behind the breakwater. However, with adaptations, such as elevated hardscape edges, improved drainage, and controlled public access during extreme conditions, the space can remain safe and functional for some of its intended use.

Deck: The projected sea level rise reaches 6.5 ft, representing a 4.9 ft increase over the design SLR allowance. This results in a still water level of approximately 16.8 ft, already exceeding the minimum design deck elevation of 16.5 ft.

Assessment 5 clearly indicates that the deck will experience unacceptable flooding conditions for safe public use. However, it is important to note that the deck was designed with robust structural integrity and is fully capable of withstanding periodic inundation without sustaining structural damage. Public access should be restricted during these extreme conditions, especially in areas lacking clear visibility or escape routes.

Plaza: Under Assessment 5 conditions, the projected sea level rise of 6.5 ft results in a still water level of approximately 16.8 ft MLLW, which exceeds the Plaza's design elevation of 16.5 ft MLLW. This elevation leads to inundation of the Plaza during high tides and storm events, increasing the potential for wave impact and crest erosion. The combination of elevated water levels and wave action may destabilize the crest and erode the land behind it, compromising the long-term integrity of the surrounding landscape and infrastructure.

While the Plaza may retain some structural resilience, it is not safe for pedestrian use under these conditions. The risk of splash flooding, slippery surfaces, and unpredictable wave overtopping would present a hazard to public safety.

8.2.6 Assessment summary

This section provides a consolidated overview of the performance of key project components (Breakwater, Ferry Pier Deck, and Shoreline Plaza) under six sea level rise scenarios spanning from the project design condition (Intermediate Scenario, Year 2075) to the High Scenario for Year 2100. Table 8-8 shows a summary of the SLR conditions considered in this assessment.

Table 8-8 Summary of SLR Risk Assessment Conditions

Assessment	Scenario	Year	SLR Allowance (ft)	SWL (ft MLLW)
0 - Design	Intermediate	2075	1.6	11.9
1	Intermediate	2100	3.1	13.4
2	Intermediate-High	2075	2.6	12.9
3	Intermediate-High	2100	4.8	15.1
4	High	2075	3.6	13.9
5	High	2100	6.5	16.8

The assessments demonstrate that the infrastructure has been thoughtfully designed to withstand a wide range of future conditions, with most components maintaining functionality and safety well beyond their original design thresholds.

- Breakwater: Performs reliably under all intermediate-high scenarios and continues to offer substantial coastal protection even under high sea level rise conditions. Under the High Scenario for Year 2100, overtopping volumes exceed safe limits for pedestrian areas, requiring restricted access and adaptive design measures.

- Ferry Pier Deck: Maintains structural integrity across all scenarios. Even under elevated water levels (including inundation), the deck remains resilient, with only the most extreme conditions requiring access restrictions.

- Shoreline Plaza: Remains functional and safe under all but the most extreme scenario. Inundation and erosion risks emerge under the High Scenario for Year 2100.

Table 8-9 summarizes the flooding potential and operational status of each component under each scenario. Overall, the results affirm the robustness of the design and its capacity to accommodate evolving climate conditions with minimal disruption.

Table 8-9: Summary of Flooding Potential of Berkeley Pier During Design Conditions (elevations in ft MLLW)

Assessment Number	**0 - Design**	**1**	**2**	**3**	**4**	**5**
Scenario	Intermediate	Intermediate	Intermediate-High	Intermediate-High	High	High
Sea Level Rise Allowance (ft)	1.6	3.1	2.6	4.8	3.6	6.5
Year	2075	2100	2075	2100	2075	2100
SWL Elevation (ft MLLW)	11.9	13.4	12.9	15.1	13.9	16.8
Breakwater (EL +17.6 ft)	**OK**	**OK**	**OK**	**OT**	**OT**	**OT**
Pier (EL +16.5 to +17.6 ft)	**OK**	**OK**	**OK**	**OT**	**OT**	**I**
Shoreline Plaza (EL +16.5 ft)	**OK**	**OK**	**OK**	**OT**	**OK**	**I**

Notes:

OK	Element is above the Design Water elevation and will not be subjected to flooding during the scenario considered (overtopping < 1 L/m/s)
OT	Element will be subjected to overtopping during the scenario considered
I	Element will be subjected to inundation during the scenario considered

8.3 Resiliency and adaptability

The BWTPF Project was designed with emphasis on resiliency and adaptability to ensure long-term performance under evolving SLR conditions. The marine infrastructure components (breakwater, ferry pier deck, and shoreline plaza) were evaluated across a range of SLR scenarios extending through the year 2100, including Intermediate, Intermediate-High, and High projections from the 2024 California Sea Level Rise Guidance.

The project's design elevations were selected to meet the minimum requirements for the 2075 planning horizon but also to provide additional freeboard and structural robustness to accommodate more extreme future conditions. As demonstrated in Section 8.2, the breakwater, deck, and plaza maintain acceptable performance under all scenarios up to the Intermediate-High projection for 2100 and continue to offer partial functionality even under the High Scenario for 2100.The resiliency features include:

- Elevated design thresholds that exceed minimum recommendations, providing buffer capacity against overtopping and inundation.
- Material and structural specifications that support durability under splash, wave impact, and intermittent flooding.
- Passive drainage and splash-tolerant surfaces that mitigate the effects of overtopping in pedestrian areas.

Adaptability for future conditions

While the project does not incorporate a phased adaptation strategy, its design allows for future interventions to extend functionality and safety. These include:

- Operational adaptations, such as temporary access restrictions during extreme tide or storm events.
- Infrastructure enhancements, including elevated hardscape edges, reinforced drainage systems, and splash barriers.
- Monitoring and maintenance protocols to track performance and implement responsive measures as conditions evolve.

The breakwater, for example, remains structurally sound even under overtopping rates exceeding safe pedestrian thresholds in the High Scenario for 2100. Adaptive management, such as limiting access and improving drainage, can preserve its protective function without requiring full reconstruction.

Similarly, the ferry pier deck and shoreline plaza retain structural integrity under inundation conditions, allowing for managed use and potential retrofits to maintain public safety and usability.

Alignment with regional guidance

The resiliency and adaptability strategy aligns with the principles outlined in the BCDC Climate Change Policy Guidance (BCDC, 2025b), the Regional Shoreline Adaptation Plan (BCDC, 2024), and the OPC's 2024 Sea Level Rise Guidance (OPC, 2024). These frameworks emphasize precautionary planning, flexible design, and iterative adaptation to support long-term infrastructure viability in the face of climate uncertainty.

8.4 Project vicinity

Attachment A provides the Regional Shoreline Adaptation Plan (RSAP) Maps for the area of the Berkeley Water Transportation Pier Ferry Project. The following information can be determined from the maps:

- Intermediate Scenario Map: Surface flooding is shown within the shoreline plaza area of the project. As noted in earlier sections of this report, this area will be elevated to +16.5 ft MLLW. The maps also indicate inundation in portions of the adjacent land and at the Berkeley Marina.
- Intermediate-High Scenario Map: Flooding is identified along the shoreline immediately south of the project site and north of the peninsula, near the Berkeley Marina.
- High Scenario: Similar to the Intermediate-High Scenario, surface flooding is observed in the same shoreline section south of the project and north of the peninsula, at the Berkeley Marina.

The RSAP is a regional planning framework developed to address sea level rise across the San Francisco Bay. Under Senate Bill 272 (SB 272), local governments in the Bay Area are required to develop Subregional Shoreline Adaptation Plans. The RSAP Atlas is a supporting tool that helps jurisdictions meet these requirements by providing region-wide data on coastal flood hazards, community vulnerability, existing conditions, strategic regional priorities, and suitability for nature-based adaptation.

The maps referenced in this report were sourced directly from the RSAP Atlas tool. It is recommended that users exercise caution when interpreting these maps, particularly because the Intermediate-High and High scenarios show very similar flooding extents in the project vicinity.

9 Conclusions

The Berkeley Water Transportation Pier Ferry Project has been evaluated for its resilience and adaptability to future sea level rise (SLR) conditions, using the latest guidance from the State of California and regional policy frameworks. The assessment considered a range of scenarios, including Intermediate, Intermediate-High, and High projections, to understand how the breakwater, ferry pier deck, and shoreline plaza would perform through the project's 50-year design life (through 2075) and beyond (up to the year 2100).

Under the baseline (design) Intermediate scenario, which incorporates a sea level rise allowance of 1.6 feet for the year 2075, all major infrastructure components are expected to remain fully functional and safe. The breakwater, deck, and plaza are designed with conservative elevation thresholds, ensuring that they are positioned above the projected still water level and can withstand the combined effects of tides, storm surges, and wave action. Even as sea level rise projections increase for the end-of-century condition (SLR of 3.1 feet by 2100 under the Intermediate scenario), the structures continue to perform reliably. Overtopping rates rise modestly but remain within tolerable limits for managed public spaces, and the deck and plaza maintain their operational integrity.

For the Intermediate-High scenario, with SLR allowances reaching 2.6 feet by 2075 and 4.8 feet by 2100, the breakwater and plaza experience increased overtopping, but these rates are still manageable with appropriate drainage and operational protocols. The deck retains sufficient clearance and structural strength, even as freeboard is reduced.

In the High scenario, representing the most severe projections with SLR allowances of 3.6 feet by 2075 and 6.5 feet by 2100, the infrastructure faces challenges. The breakwater is subjected to overtopping volumes that exceed pedestrian safety thresholds, and both the deck and plaza are at risk of inundation during extreme tide and storm events. Under these conditions, public access would need to be restricted, and adaptive measures such as enhanced drainage, splash-resistant surfaces, and revised operational protocols would be required to maintain safety and usability.

The BWTPF Project's design reflects a precautionary approach, with elevated thresholds and materials that provide buffer capacity against overtopping and flooding. The assessment confirms that the breakwater continues to offer coastal protection, the deck maintains its structural integrity, and the plaza remains usable with minor adaptive measures, even under the most extreme scenarios considered. These conclusions highlight the project's alignment with best practices in adaptive management and its readiness to meet the challenges of sea level rise and climate uncertainty through the end of the century.

10 References

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Attachment A - Regional Shoreline Adaptation Plan Maps for Project Vicinity

rsap_atlas_map



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0 0.05 0.1 0.2 mi

0 0.07 0.15 0.3 km

Cities & Census Designated Places (Analysis) (CALFire 2022)

3.1 ft (2100 Intermediate) SLR Scenario (BCDC 2024)

Existing Bay

Surface Flooding (e.g. Tidal Inundation, Storm Surge, Emergent Groundwater)

Shallow Groundwater (0-9 ft below ground)

BCDC, USGS, ART, Sources: Esri, TomTom, Garmin, FAO, NOAA, USGS, © OpenStreetMap contributors, and the GIS User Community

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Cities & Census Designated Places (Analysis) (CALFire 2022)

4.9 ft (2100 Intermediate-High) SLR Scenario (BCDC 2024)

Existing Bay

Surface Flooding (e.g. Tidal Inundation, Storm Surge, Emergent Groundwater)

Shallow Groundwater (0-9 ft below ground)

BCDC, USGS, ART, Sources: Esri, TomTom, Garmin, FAO, NOAA, USGS, © OpenStreetMap contributors, and the GIS User Community

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Cities & Census Designated Places (Analysis) (CALFire 2022)

6.6 ft (2100 High) SLR Scenario (BCDC 2024)

Existing Bay

Surface Flooding (e.g. Tidal Inundation, Storm Surge, Emergent Groundwater)

Shallow Groundwater (0-9 ft below ground)

BCDC, USGS, ART, Sources: Esri, TomTom, Garmin, FAO, NOAA, USGS, © OpenStreetMap contributors, and the GIS User Community

Appendix C – Wave disturbance maps

Overall wave disturbance maps – 400 ft breakwater length

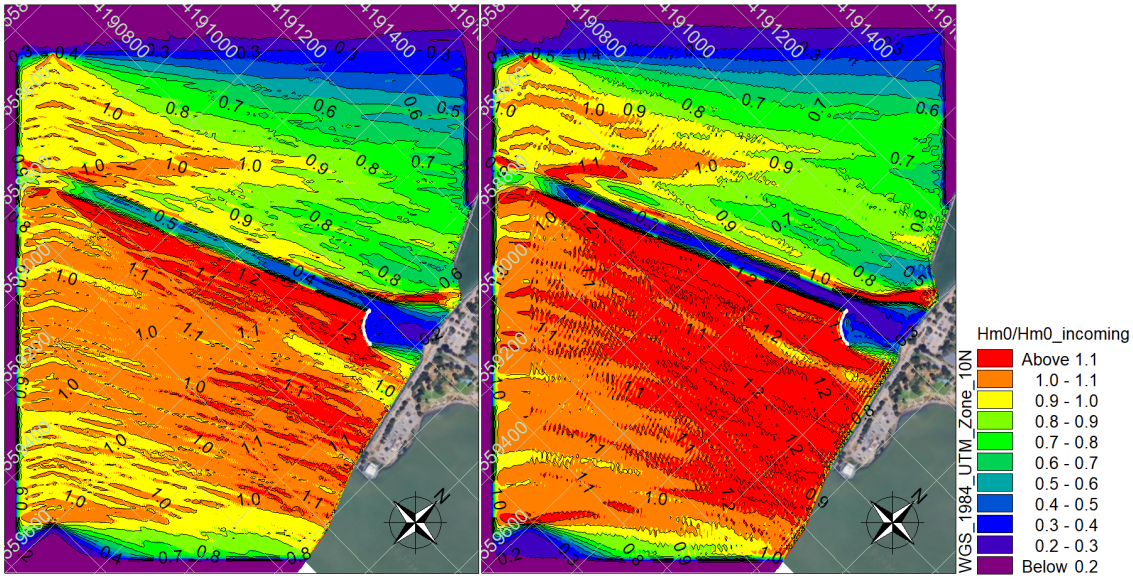


Figure C- 1 *Wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.*

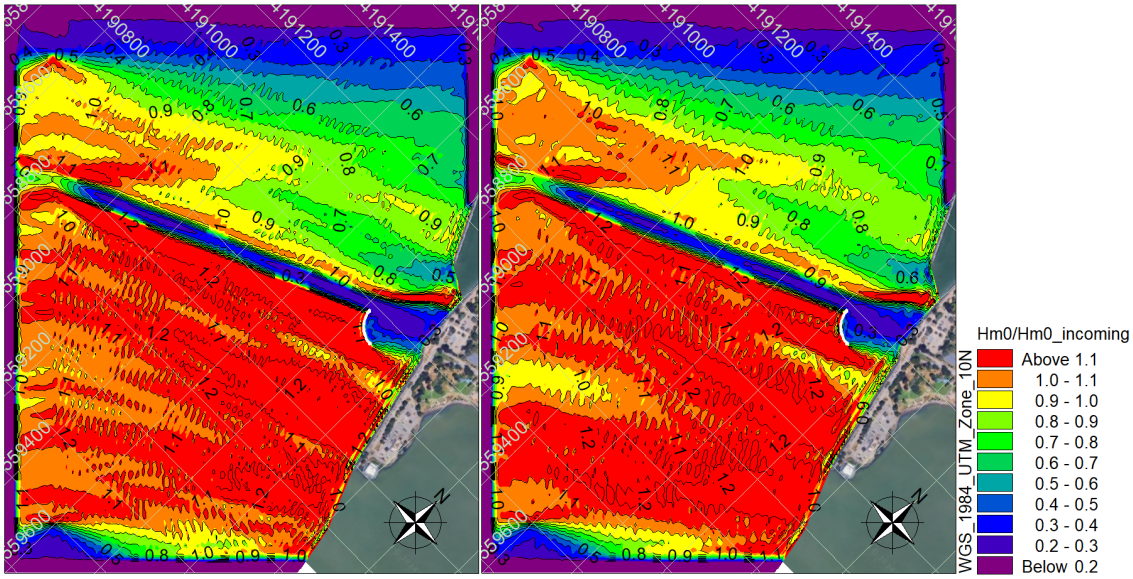


Figure C- 2 *Wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 se, 400 ft breakwater length.*

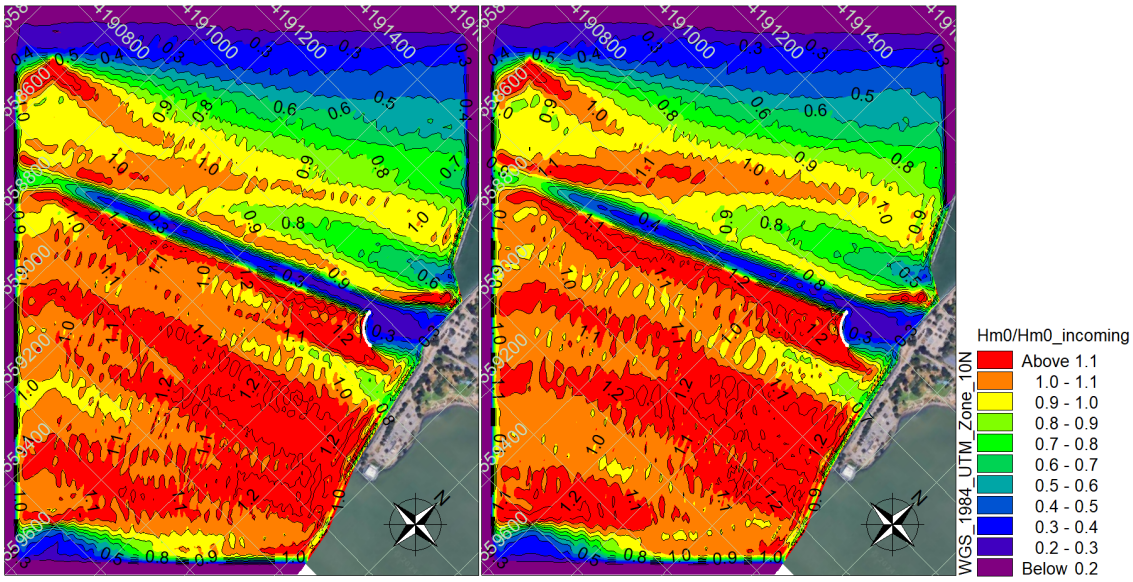


Figure C- 3 *Wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.*

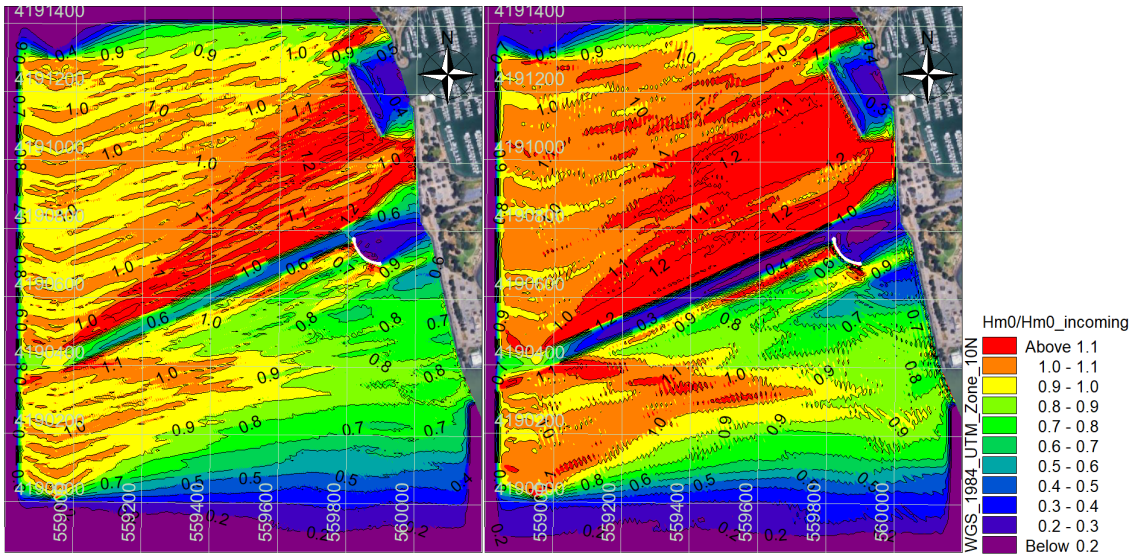


Figure C- 4 *Wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.*

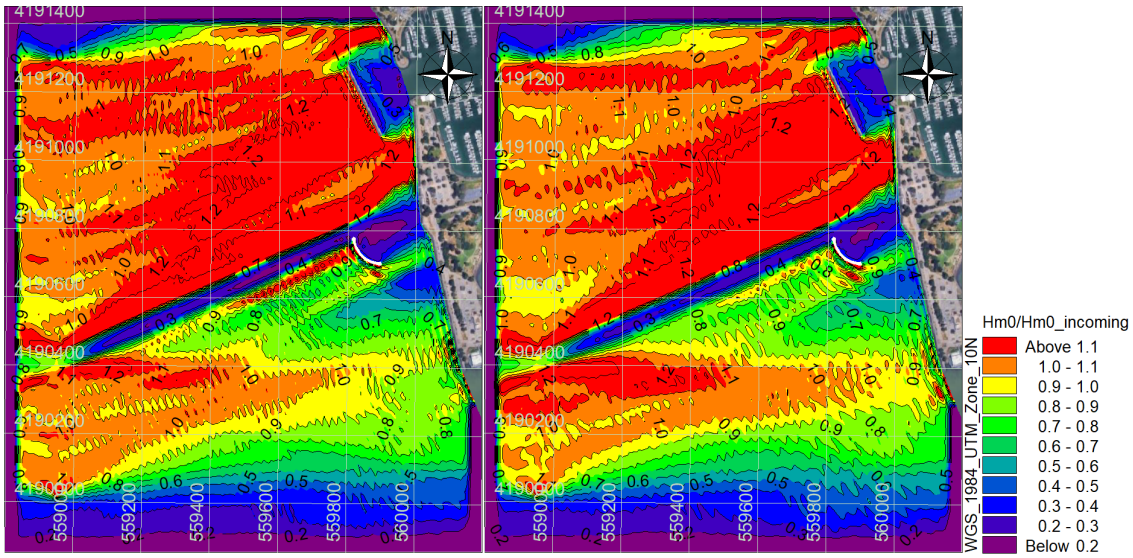


Figure C- 5 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.

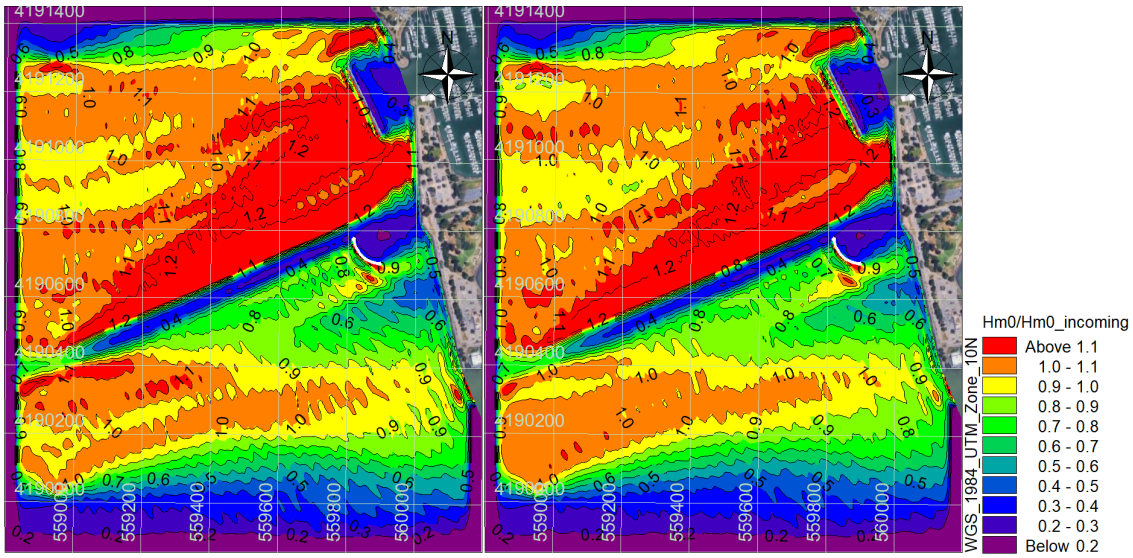


Figure C- 6 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.

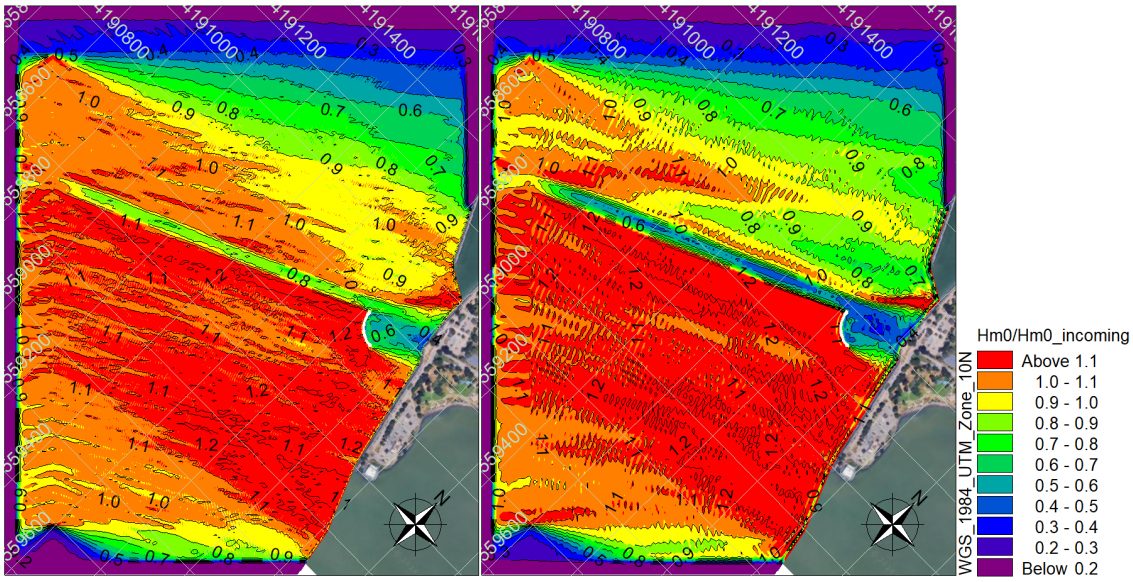


Figure C- 7 *Wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.*

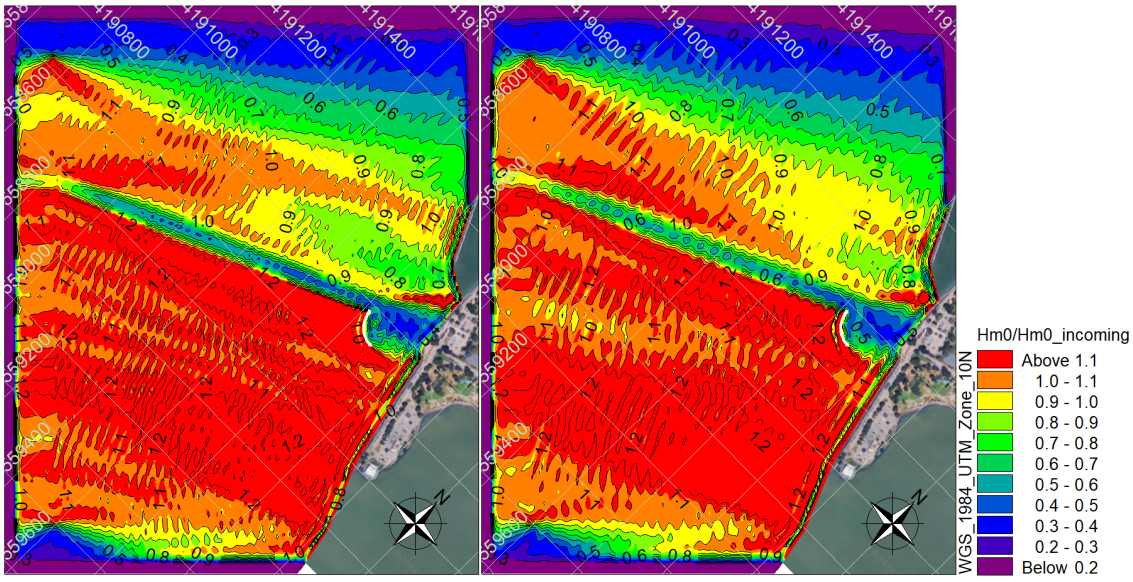


Figure C- 8 *Wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.*

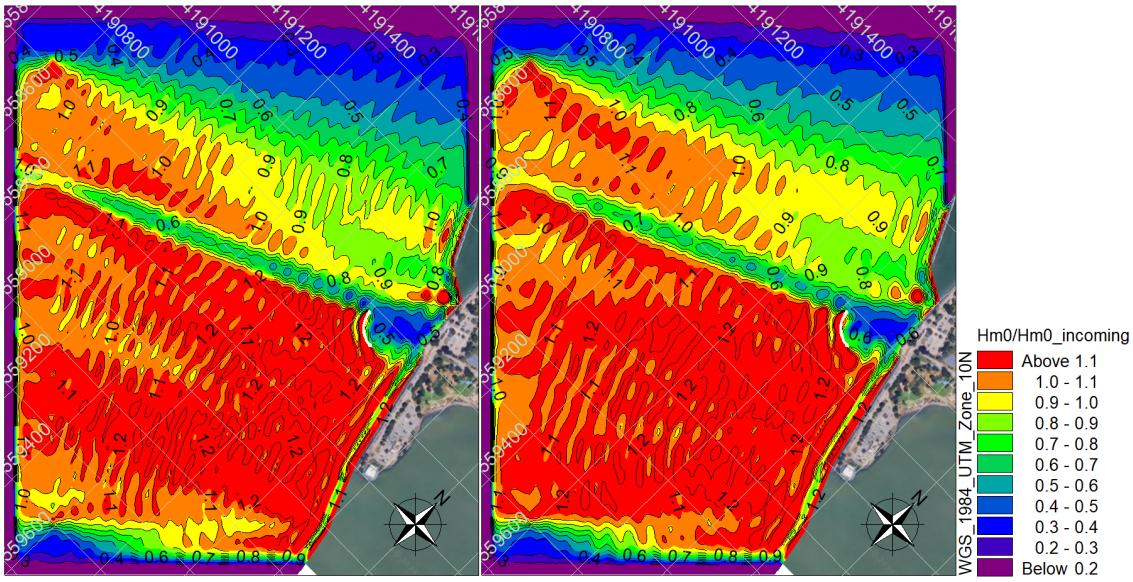


Figure C- 9 *Wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.*

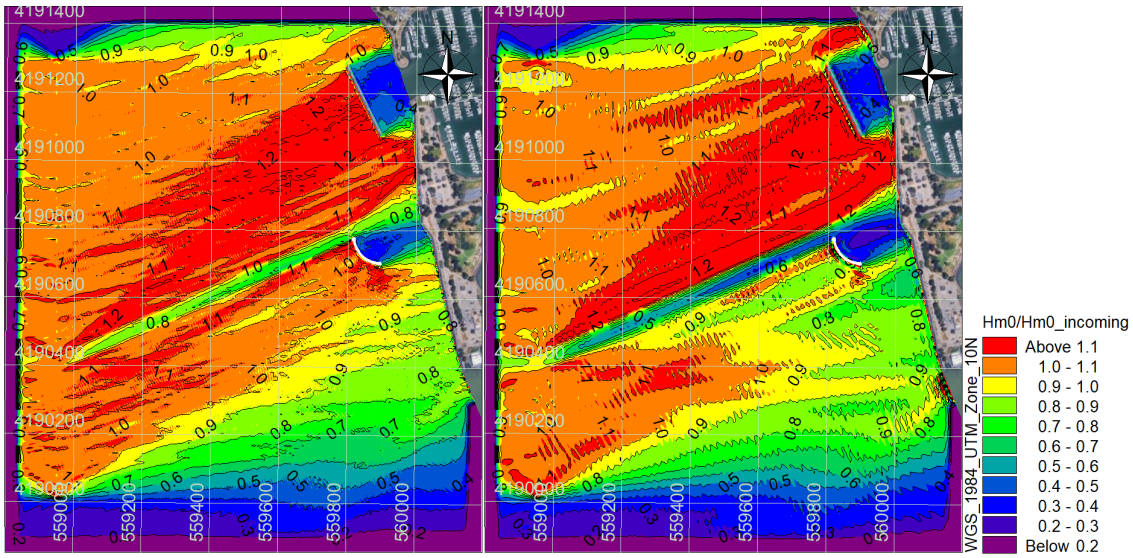


Figure C- 10 *Wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.*

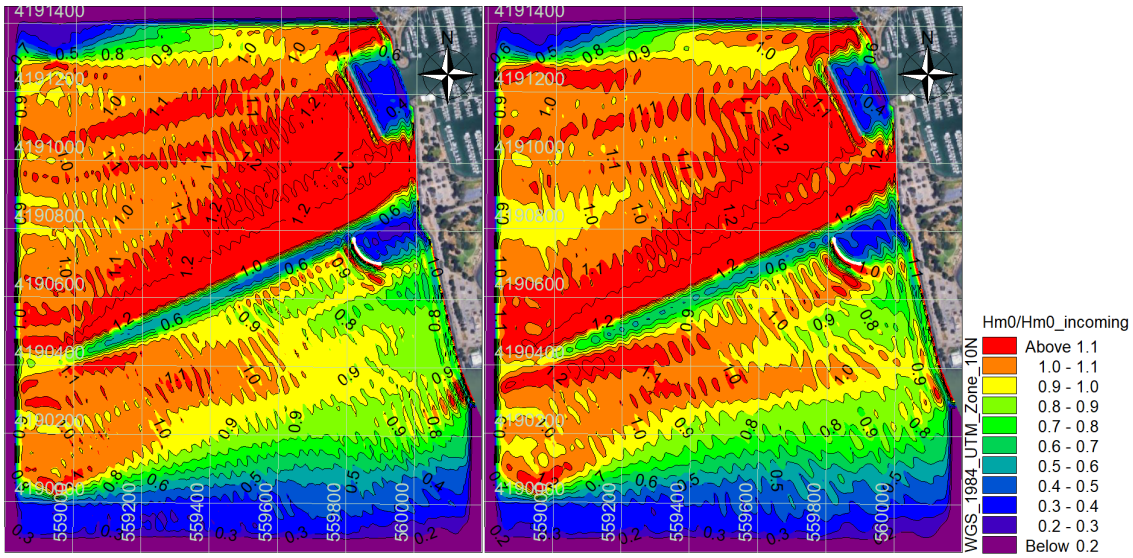


Figure C- 11 Wave disturbance plot for wave direction = 255°N at MHHW.
Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.

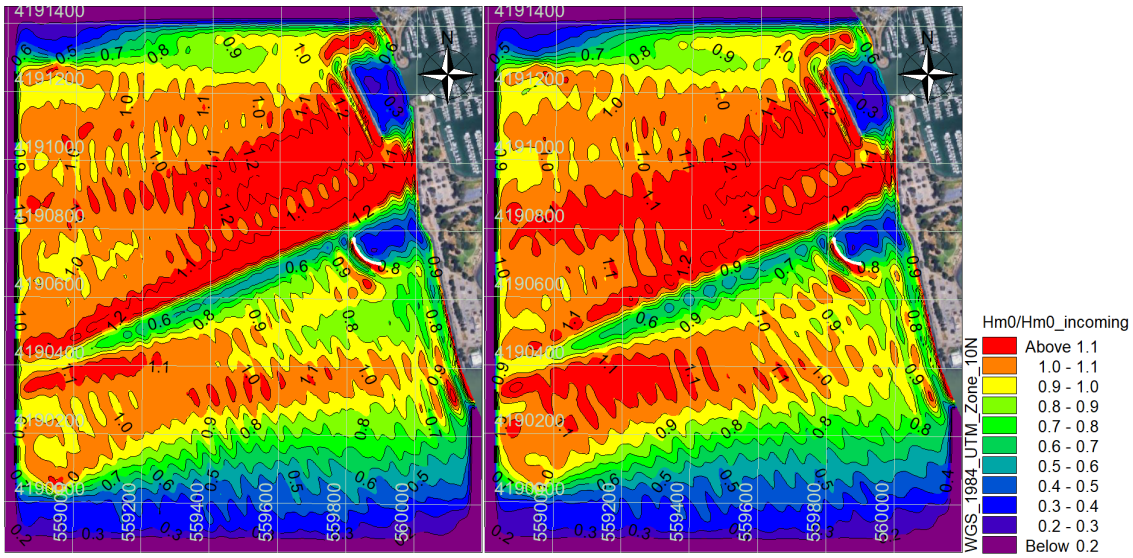


Figure C- 12 Wave disturbance plot for wave direction = 255°N at MHHW.
Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.

Ferry terminal wave disturbance maps – 400 ft breakwater length

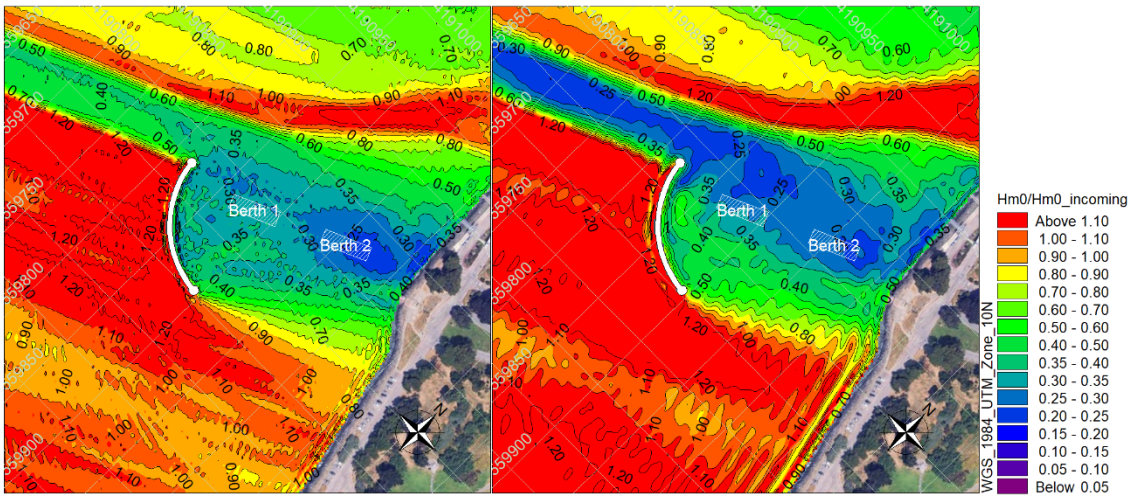


Figure C- 13 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.*

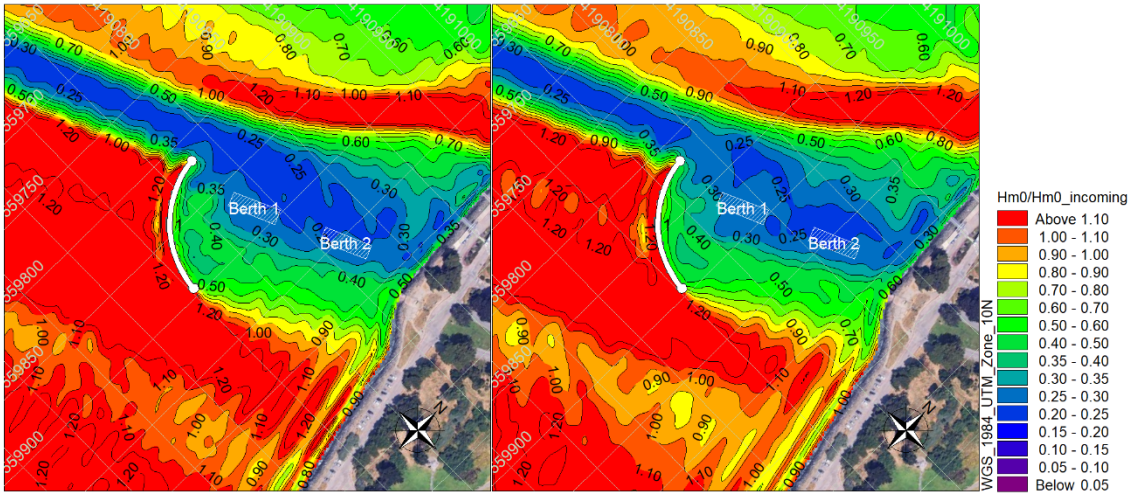


Figure C- 14 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.*

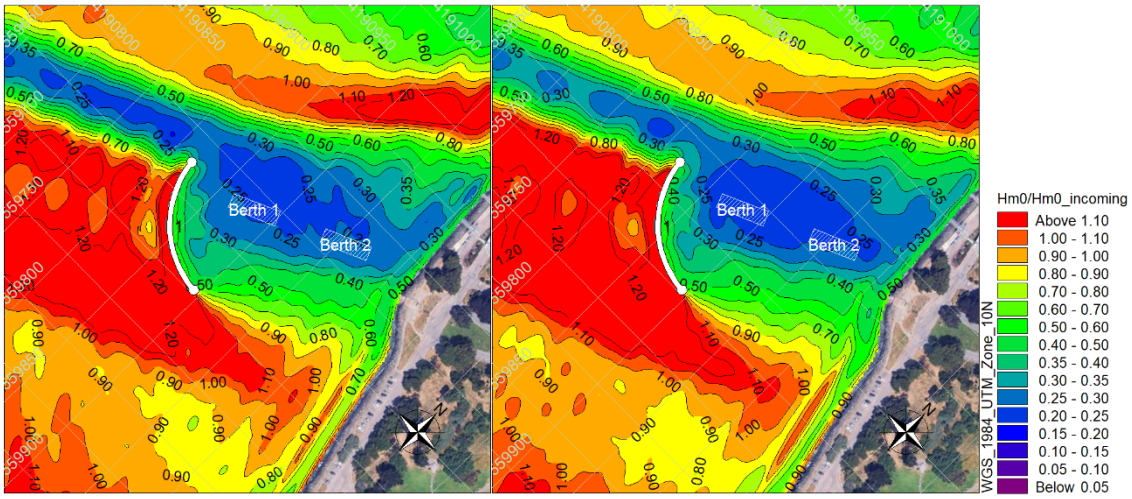


Figure C- 15 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.*

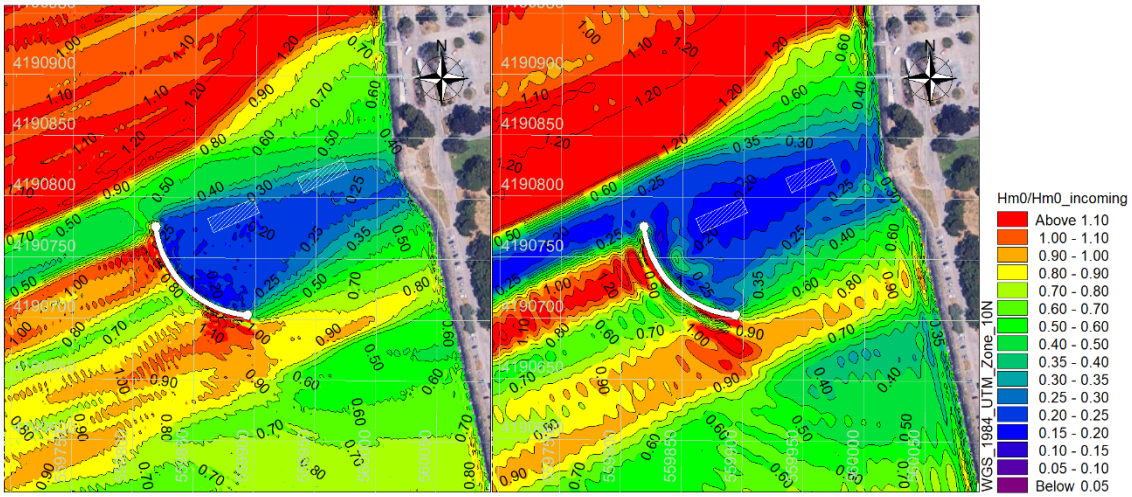


Figure C- 16 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.*

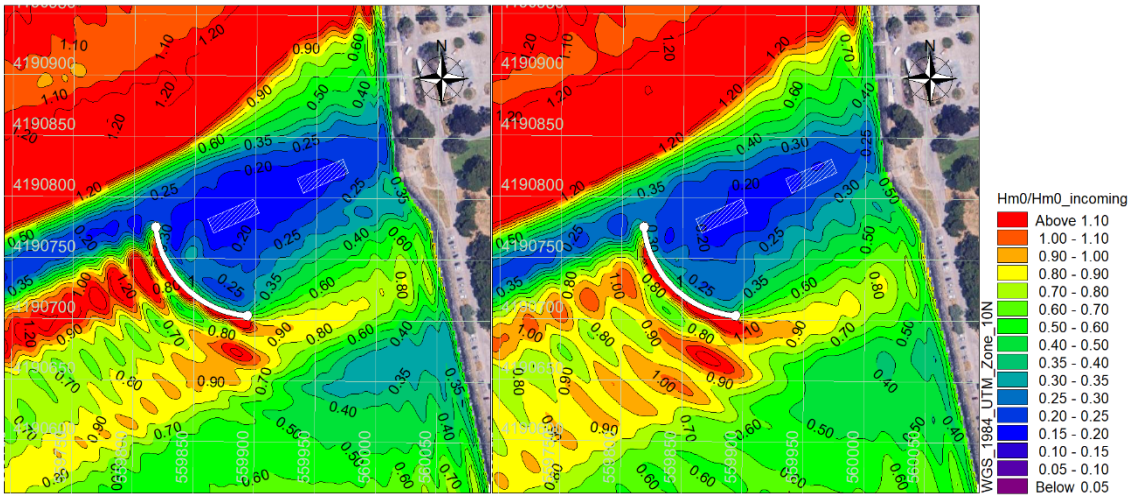


Figure C- 17 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.*

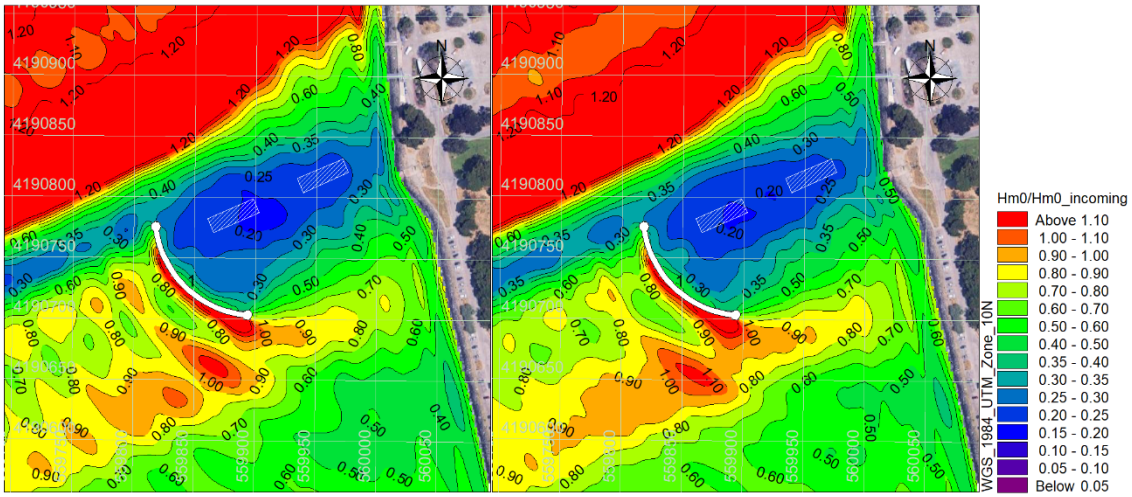


Figure C- 18 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.*

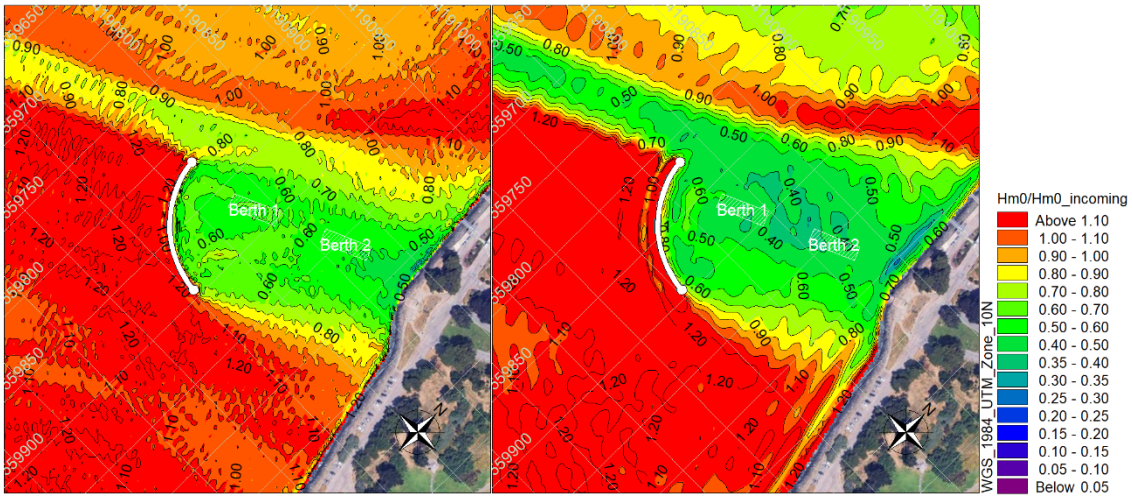


Figure C- 19 Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.

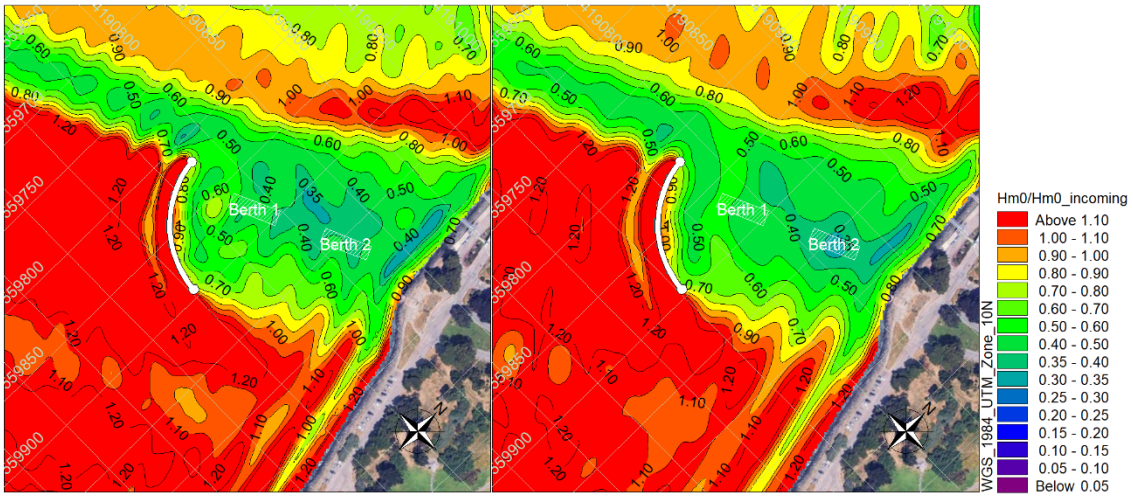


Figure C- 20 Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.

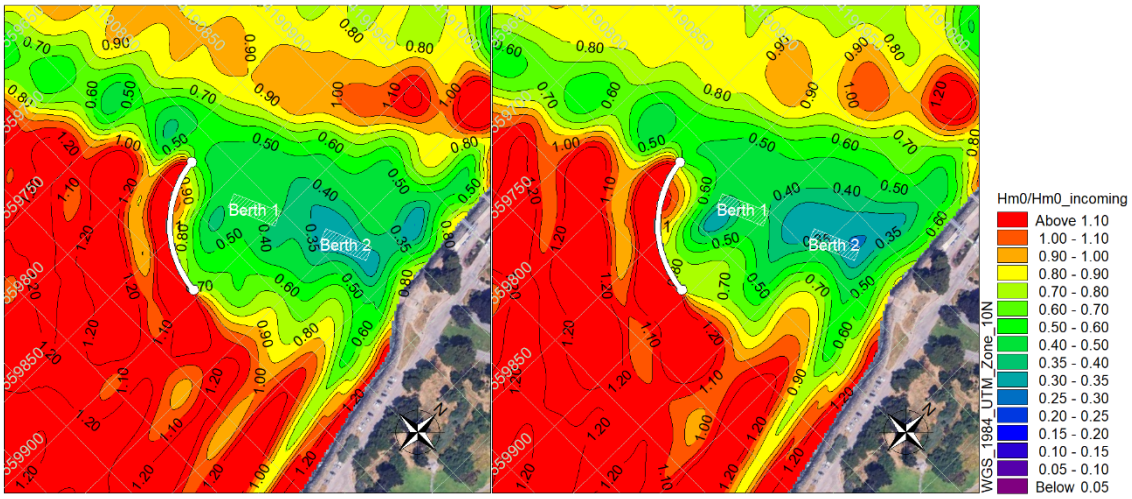


Figure C- 21 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.*

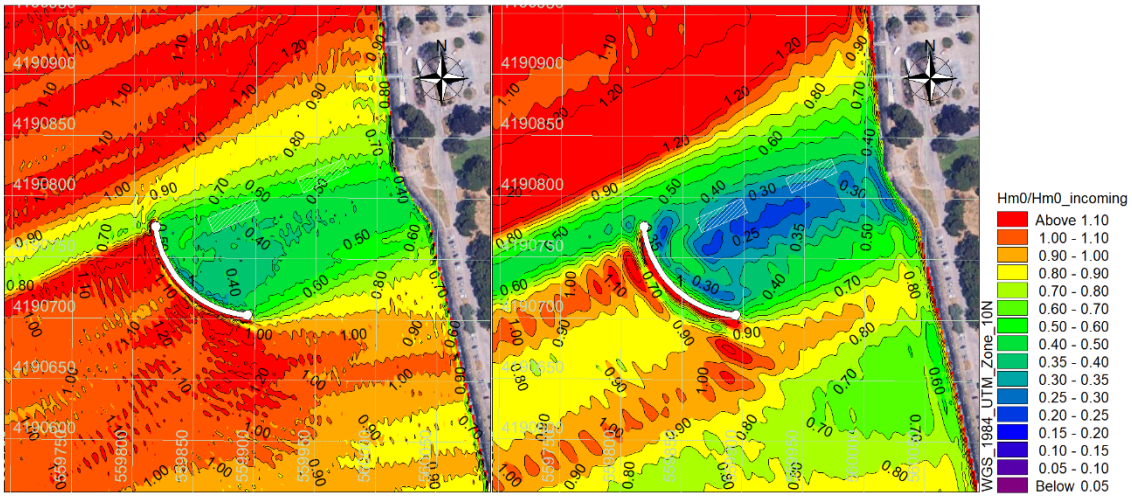


Figure C- 22 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.*

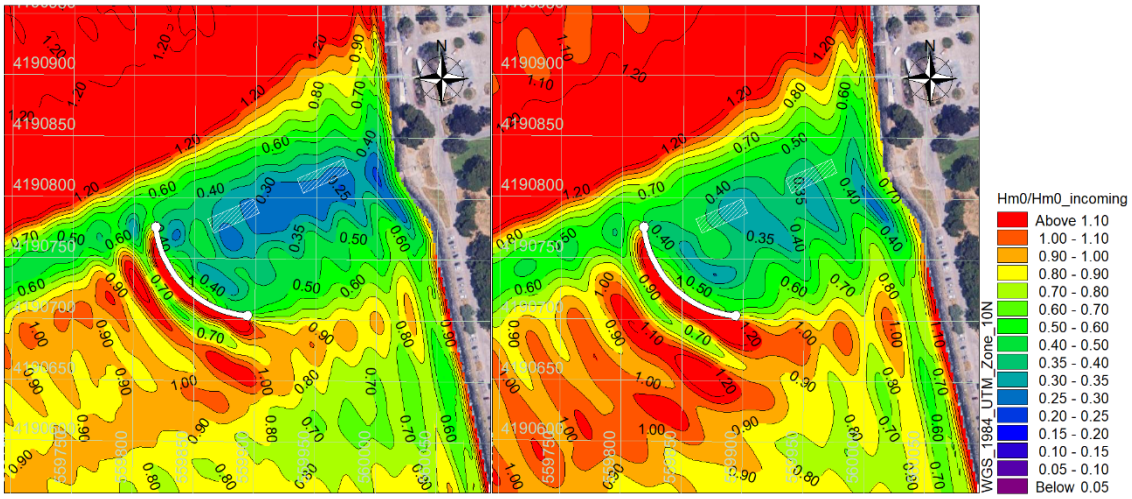


Figure C- 23 Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.

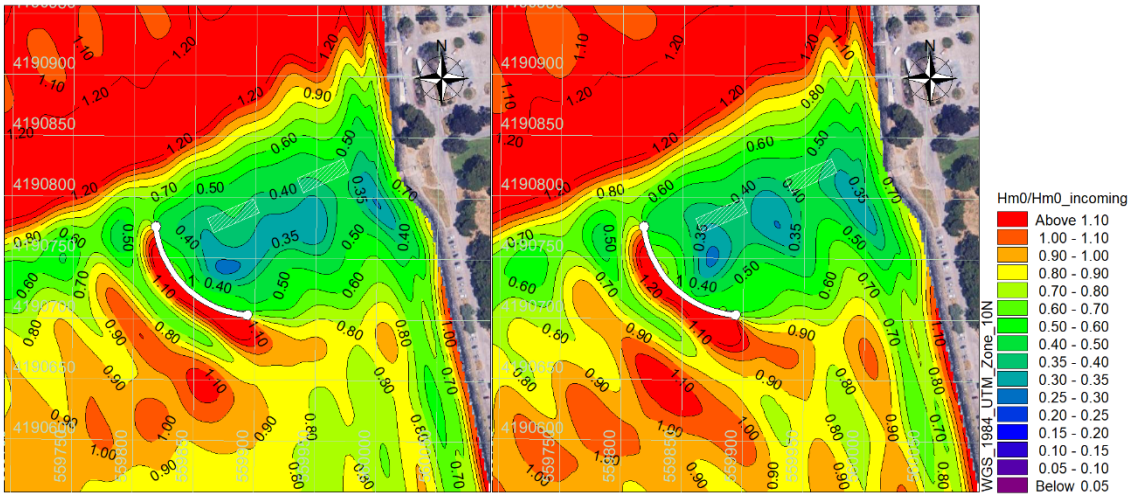


Figure C- 24 loseup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.

Overall wave disturbance maps – 300 ft breakwater length

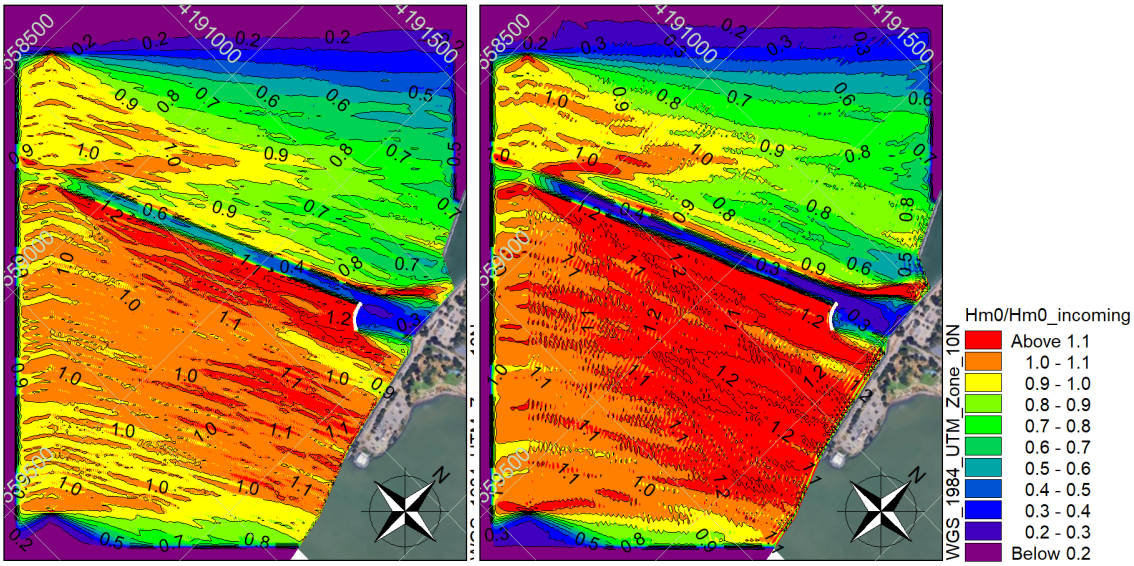


Figure C- 25 *Wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length*

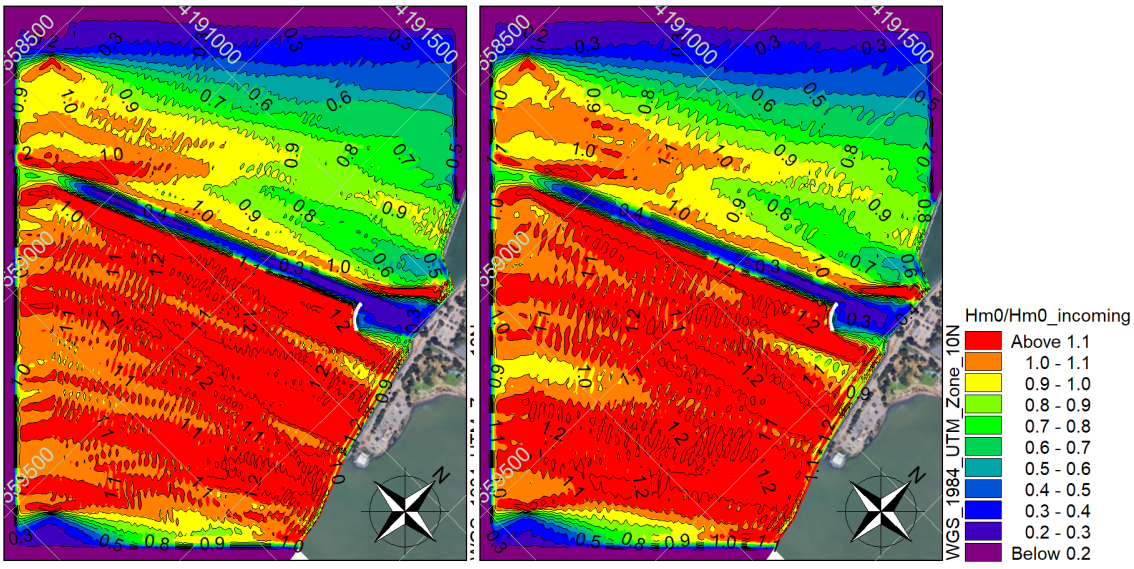


Figure C- 26 *Wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length*

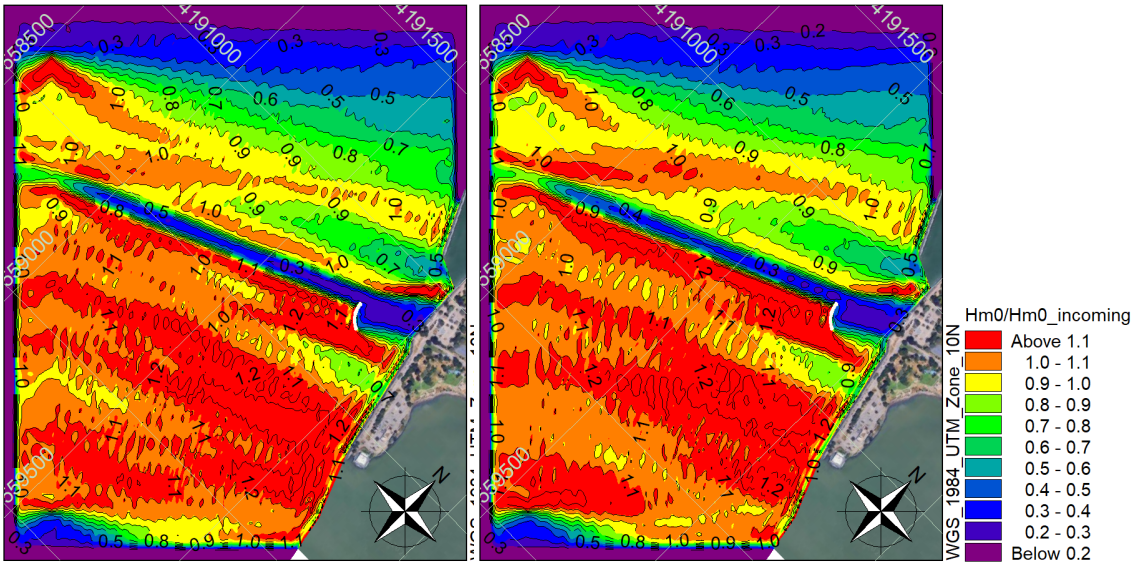


Figure C- 27 *Wave disturbance plot for wave direction = 240°N at MLLW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length

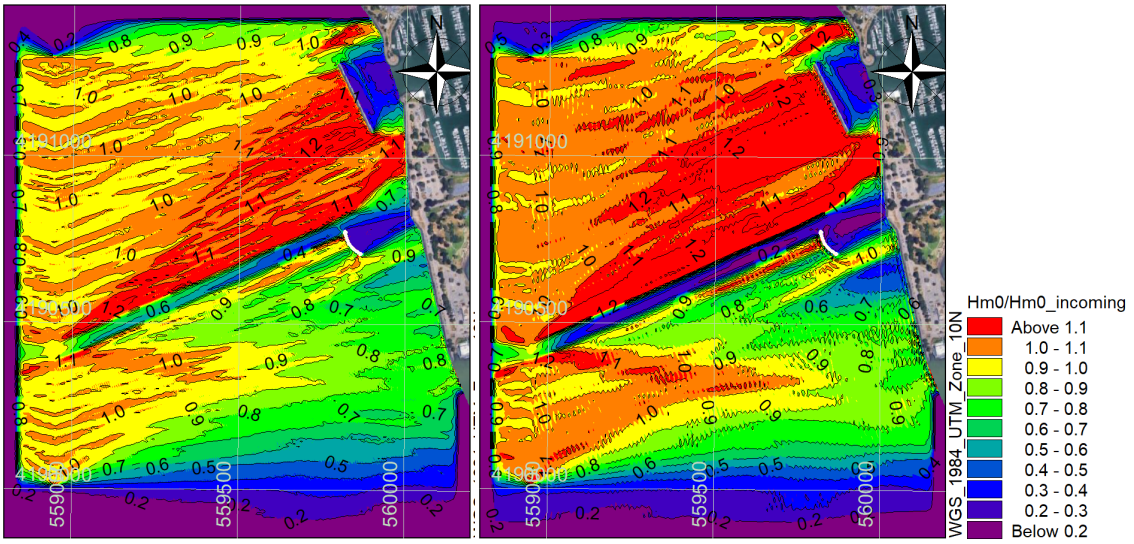


Figure C- 28 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length

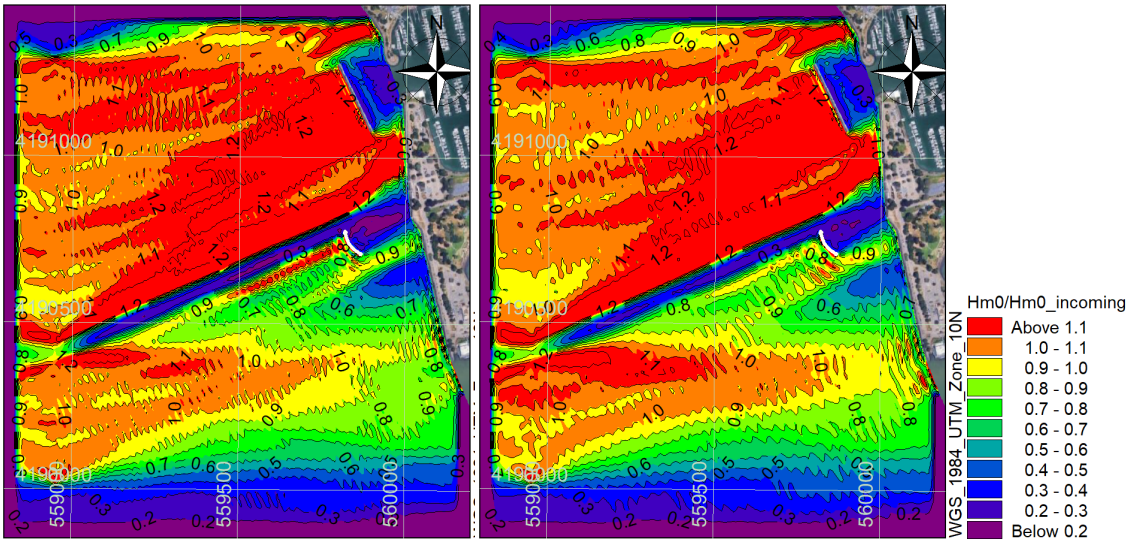


Figure C- 29 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length

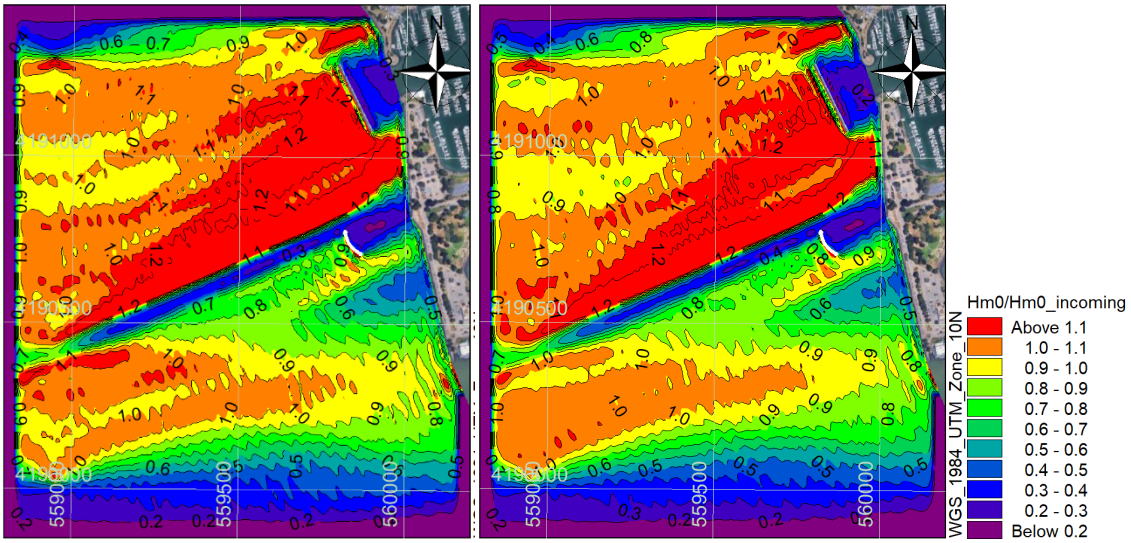


Figure C- 30 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length

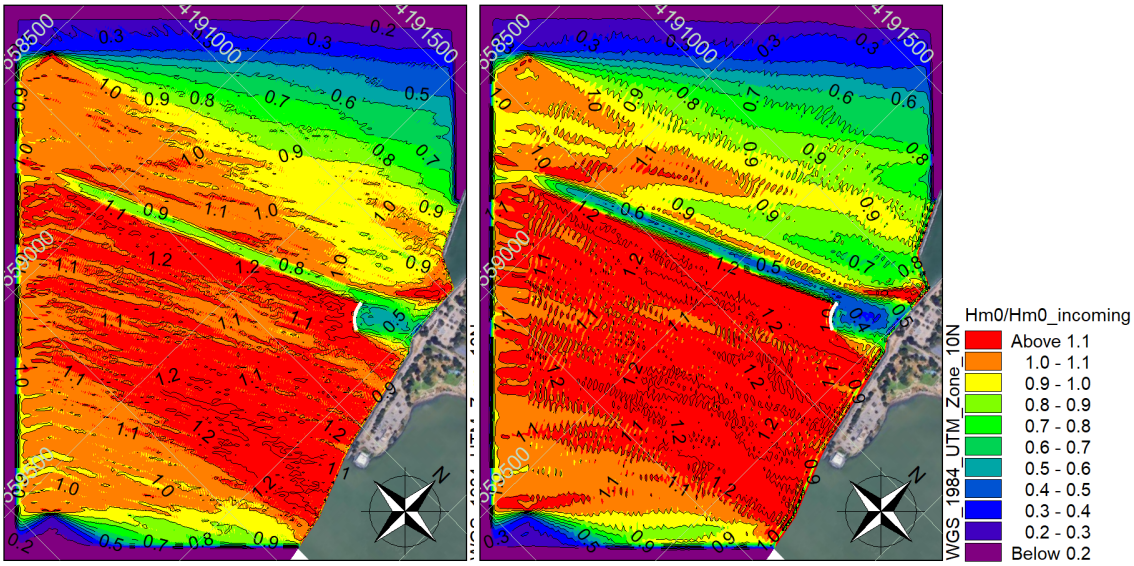


Figure C- 31 *Wave disturbance plot for wave direction = 240°N at MHHW.*
Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length

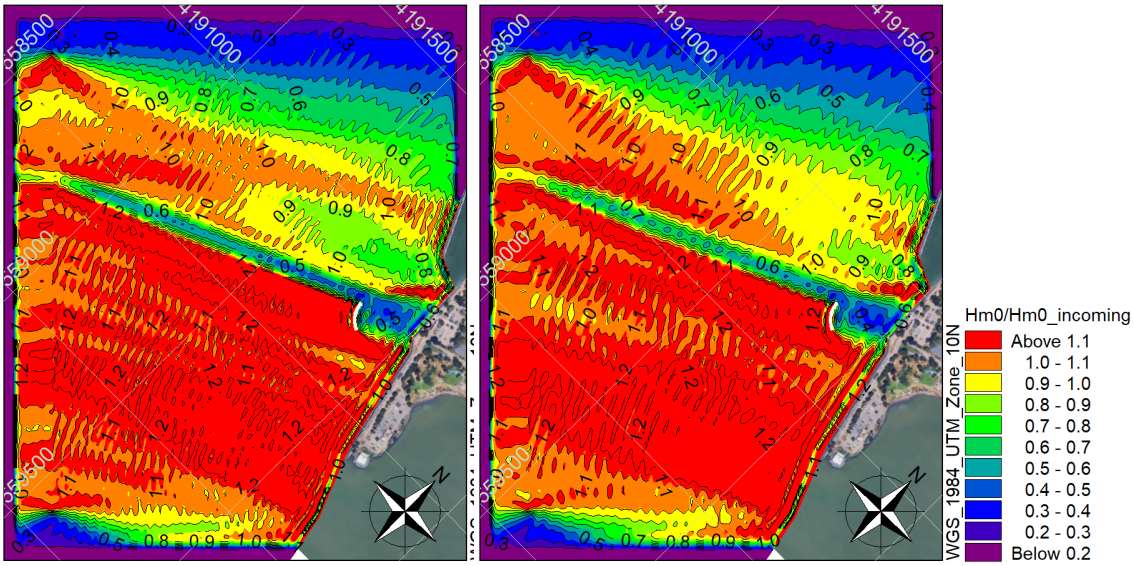


Figure C- 32 *Wave disturbance plot for wave direction = 240°N at MHHW.*
Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length

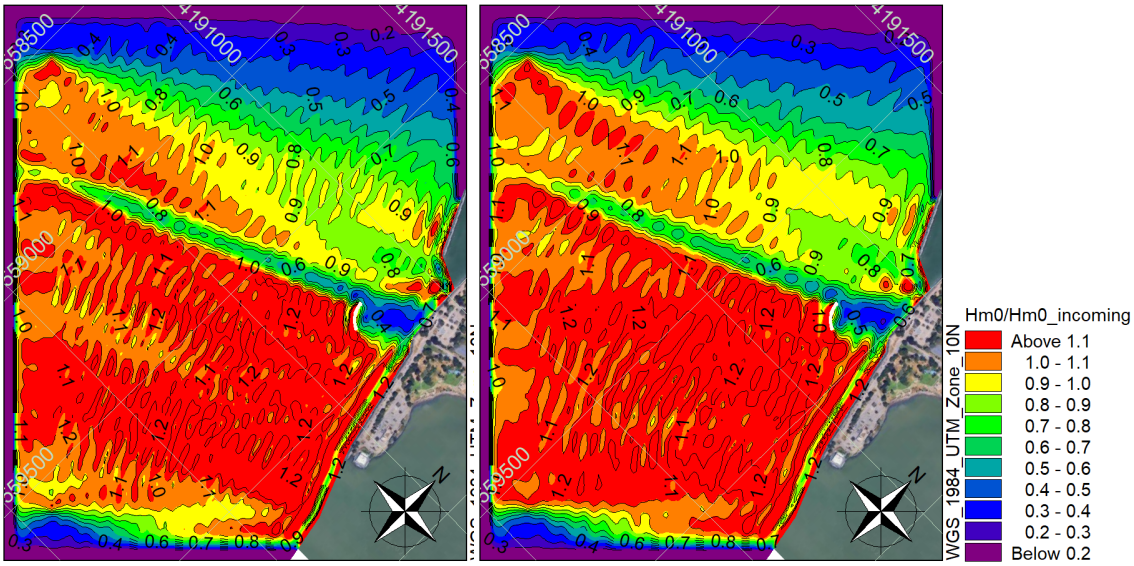


Figure C- 33 *Wave disturbance plot for wave direction = 240°N at MHHW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length

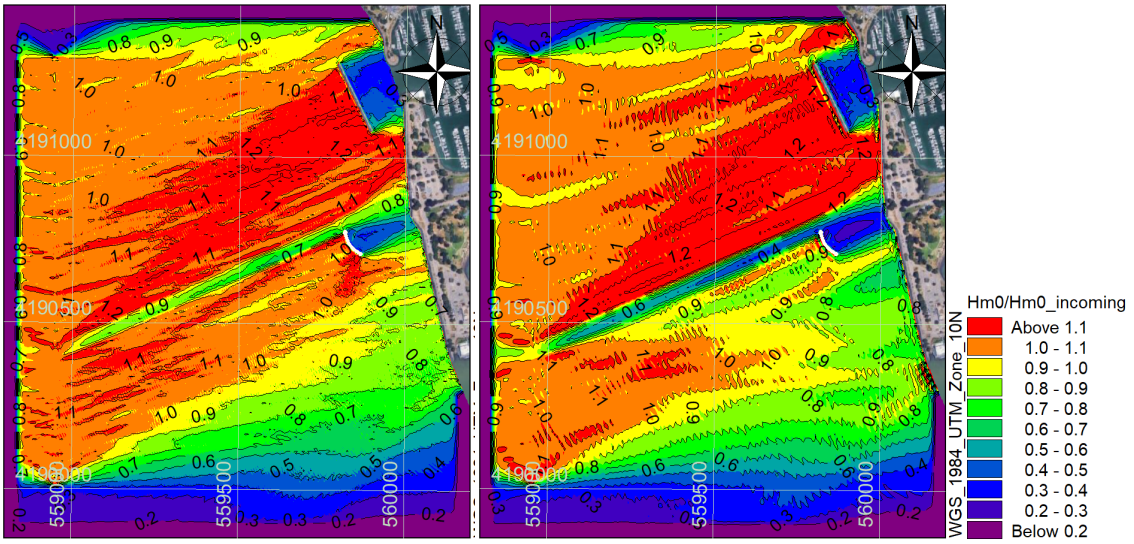


Figure C- 34 *Wave disturbance plot for wave direction = 255°N at MHHW.*
Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length

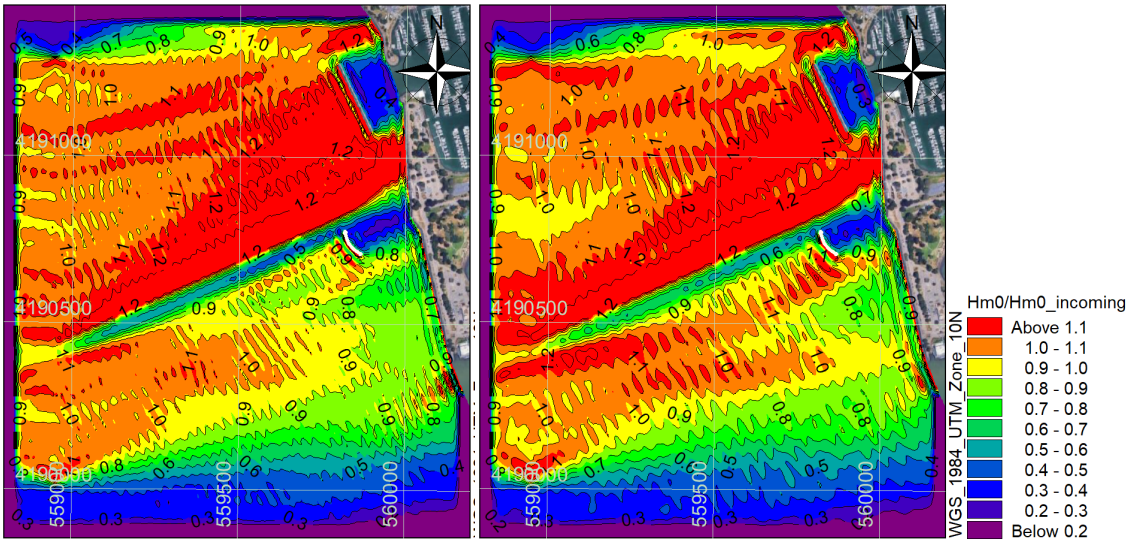


Figure C- 35 *Wave disturbance plot for wave direction = 255°N at MHHW.*
Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length

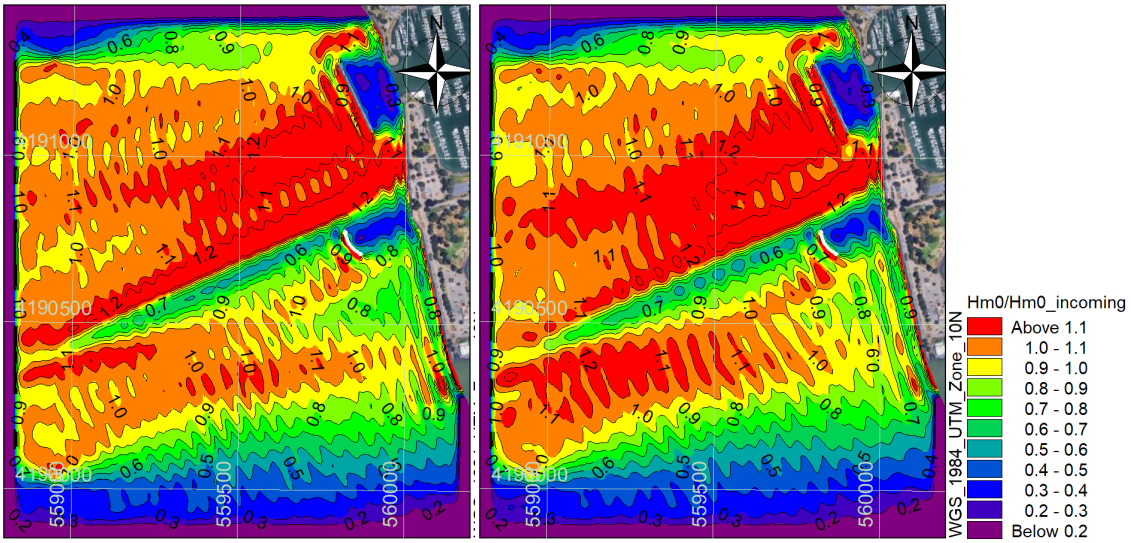


Figure C- 36 *Wave disturbance plot for wave direction = 255°N at MHHW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length

Ferry terminal wave disturbance maps – 300 ft breakwater length

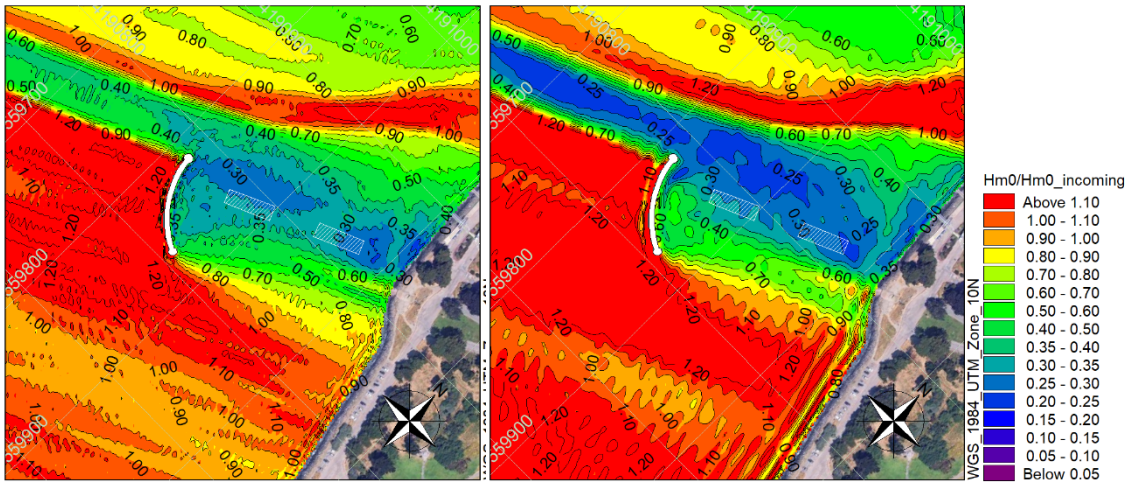


Figure C- 37 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length*

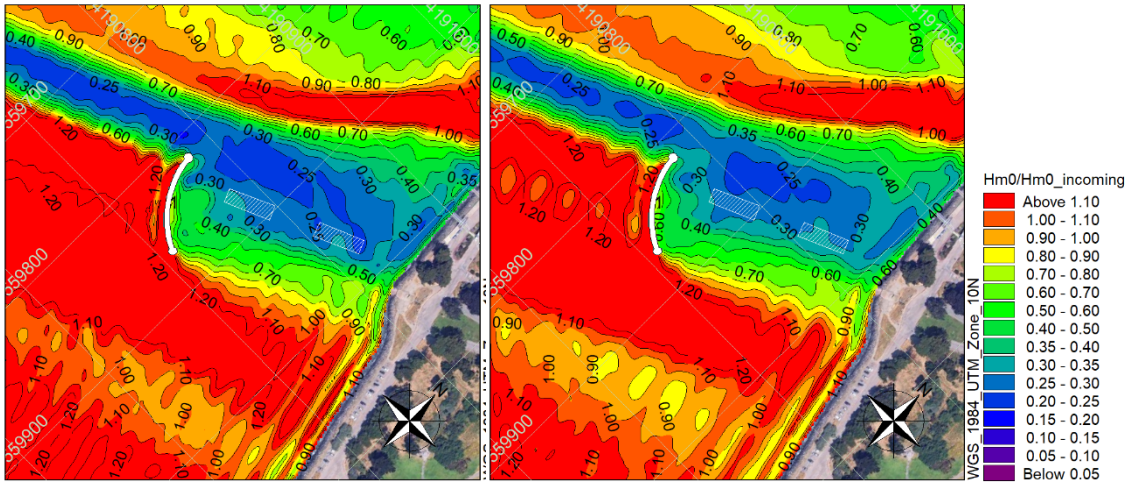


Figure C- 38 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length*

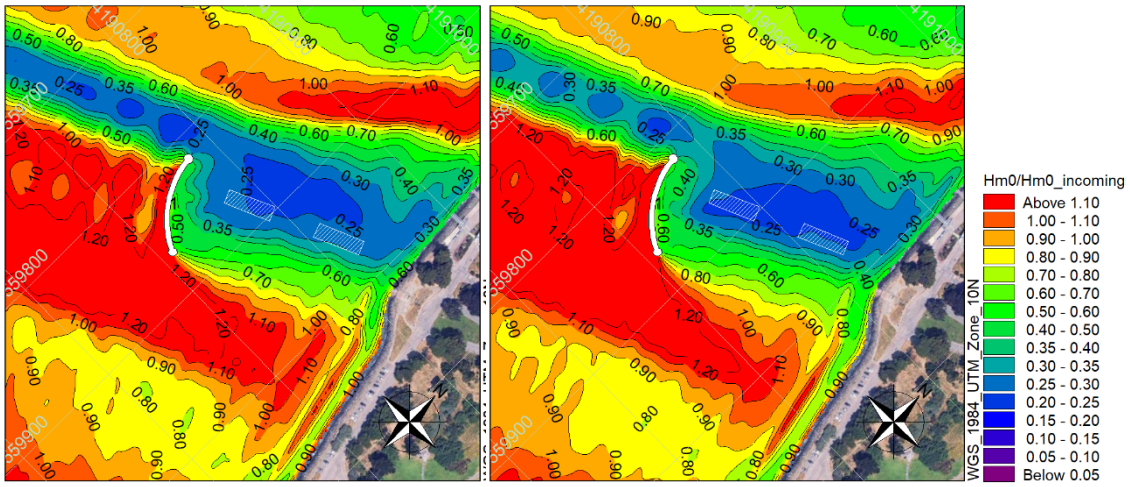


Figure C- 39 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length*

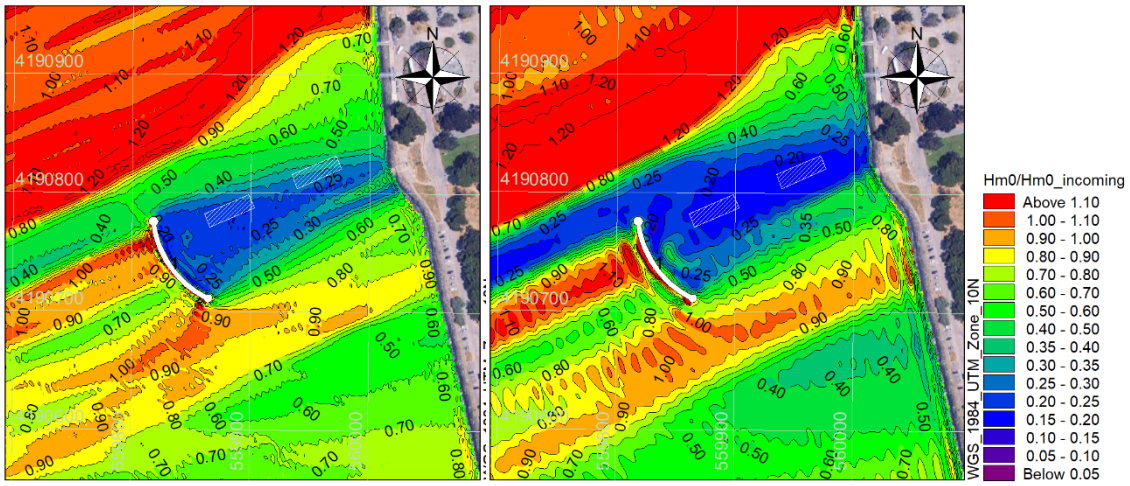


Figure C- 40 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length*

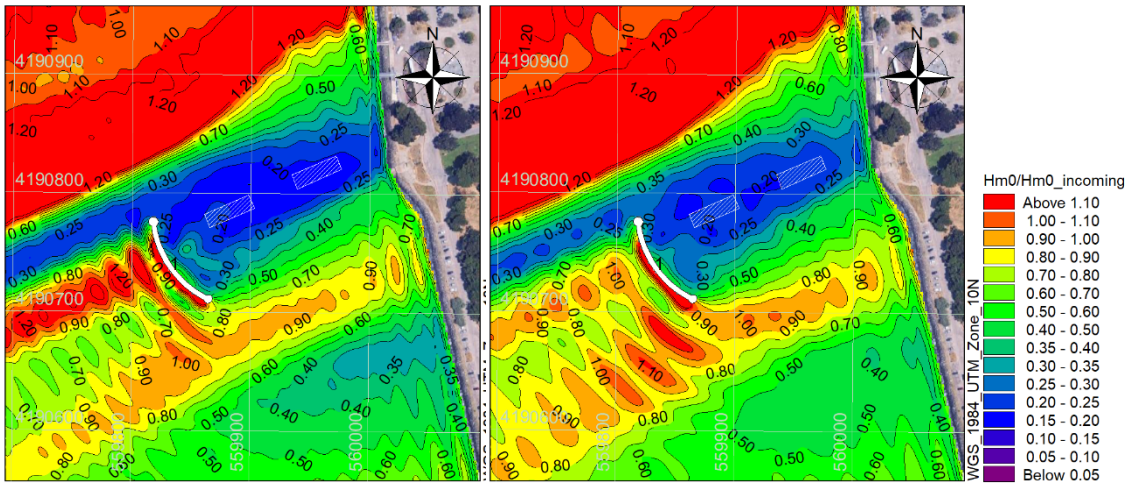


Figure C- 41 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length*

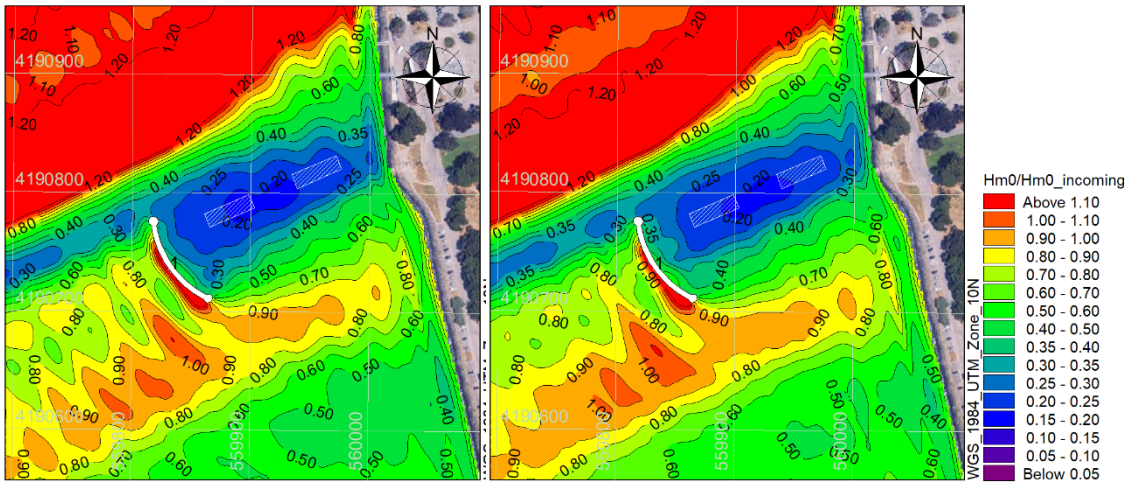


Figure C- 42 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length*

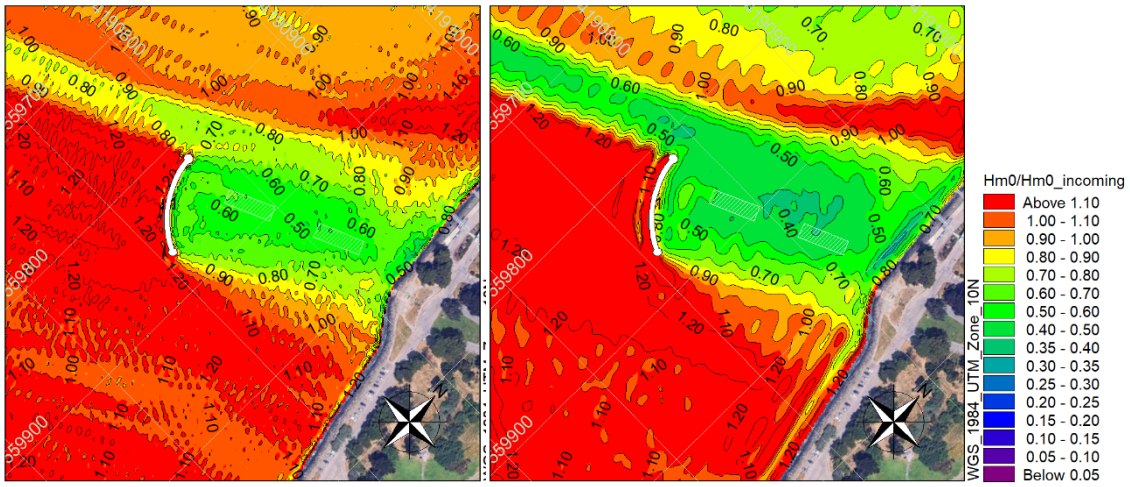


Figure C- 43 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length*

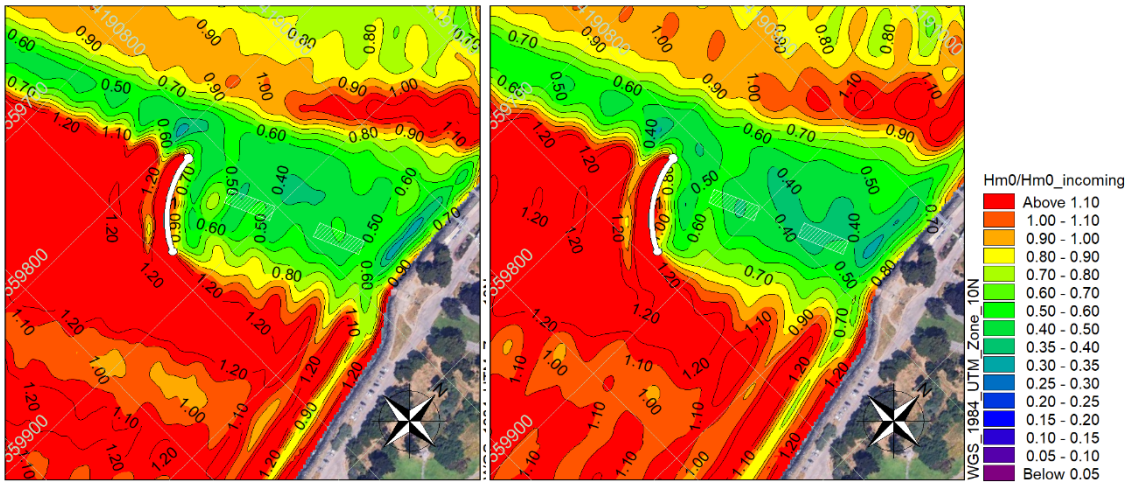


Figure C- 44 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length*

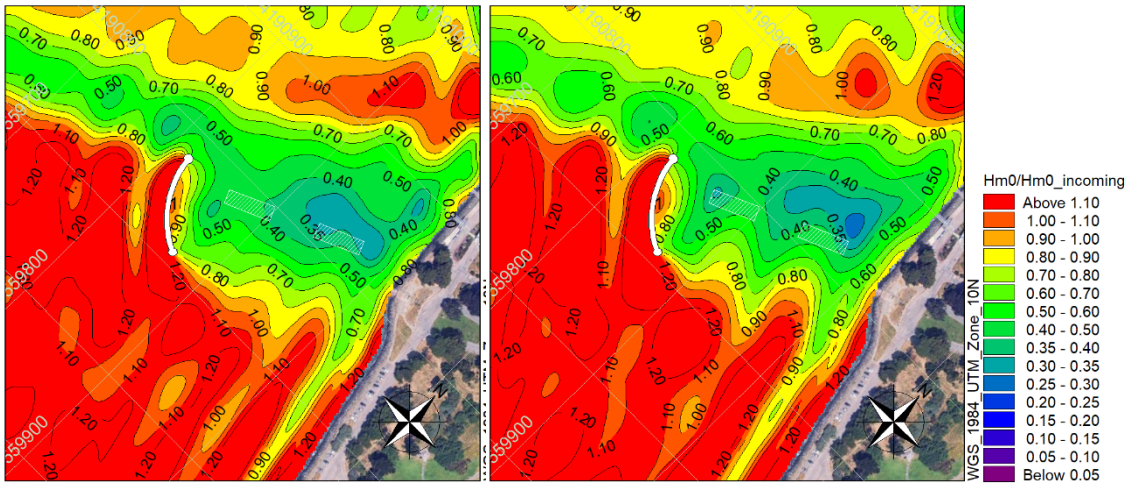


Figure C- 45 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length*

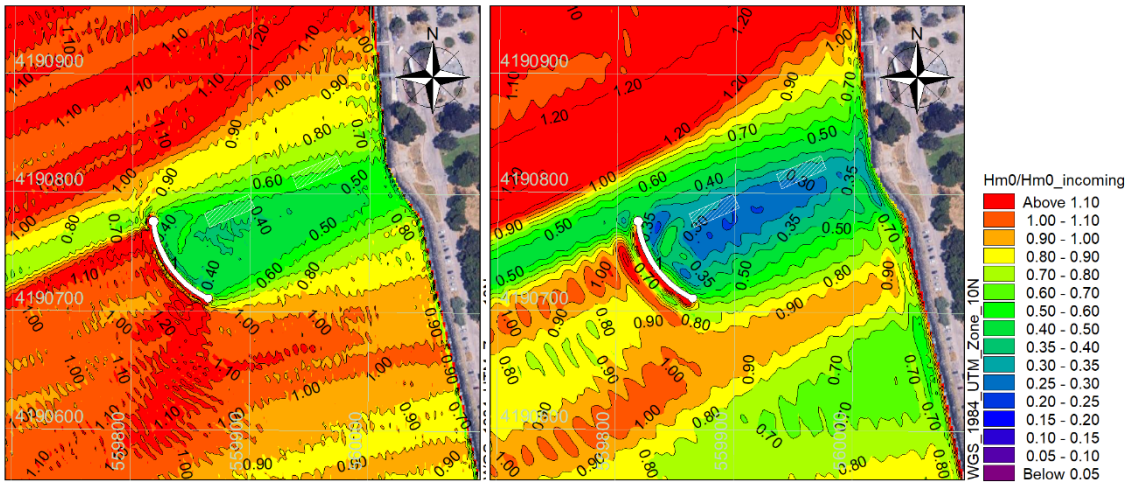


Figure C- 46 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length*

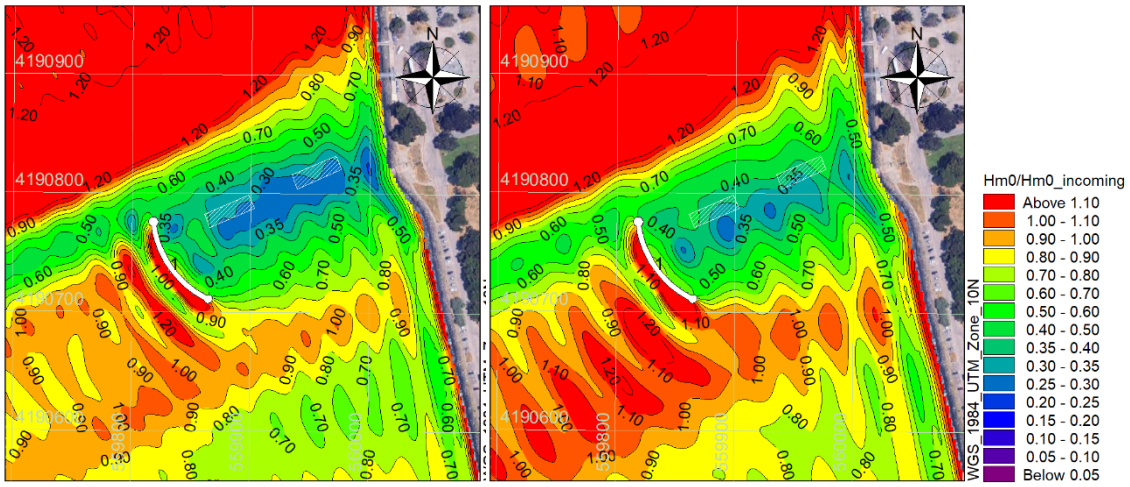


Figure C- 47 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length*

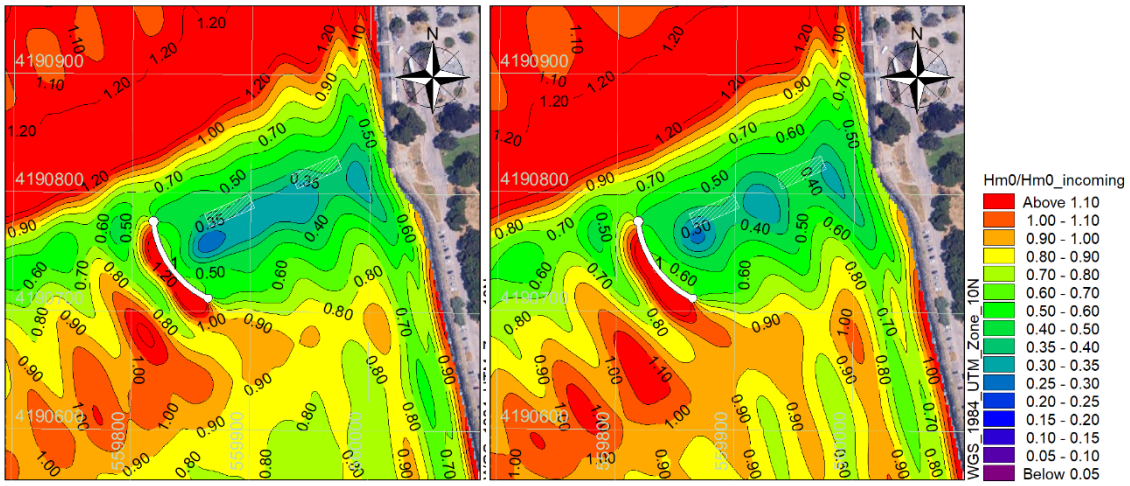


Figure C- 48 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length*

Overall wave disturbance maps – 250 ft breakwater length

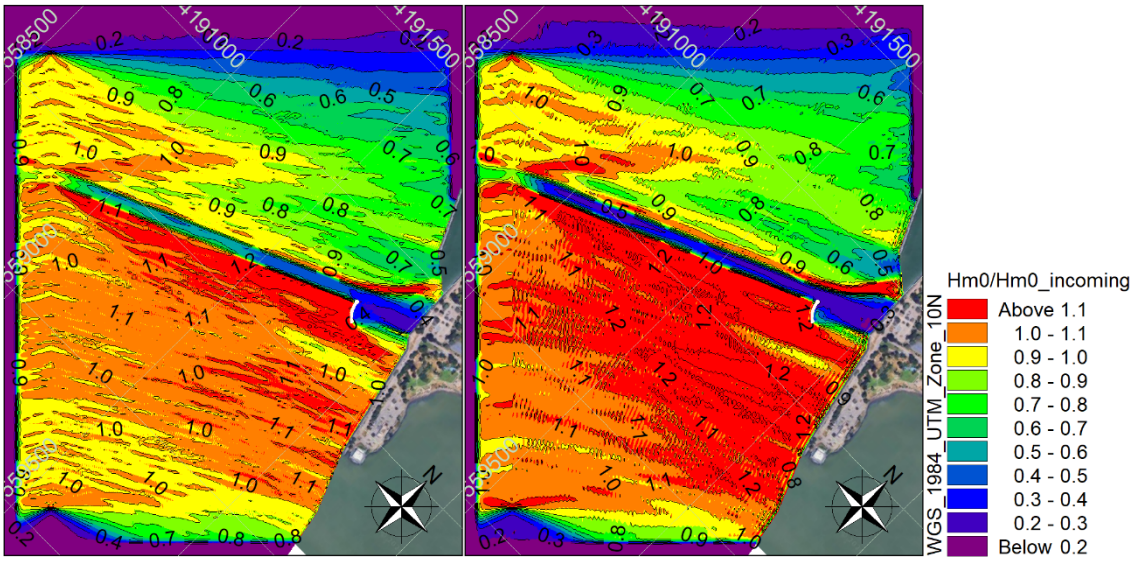


Figure C- 49 *Wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length*

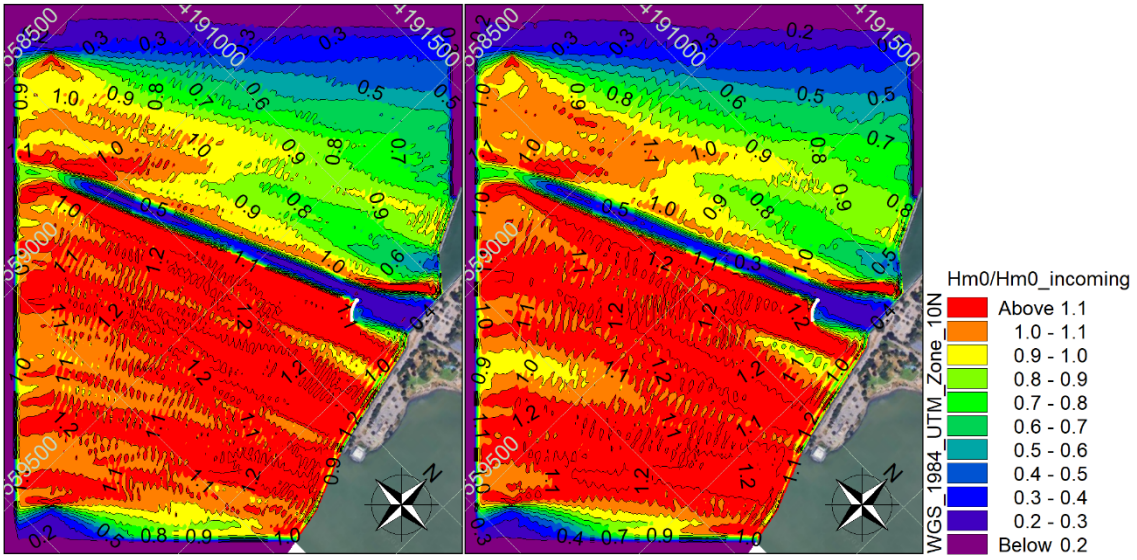


Figure C- 50 *Wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length*

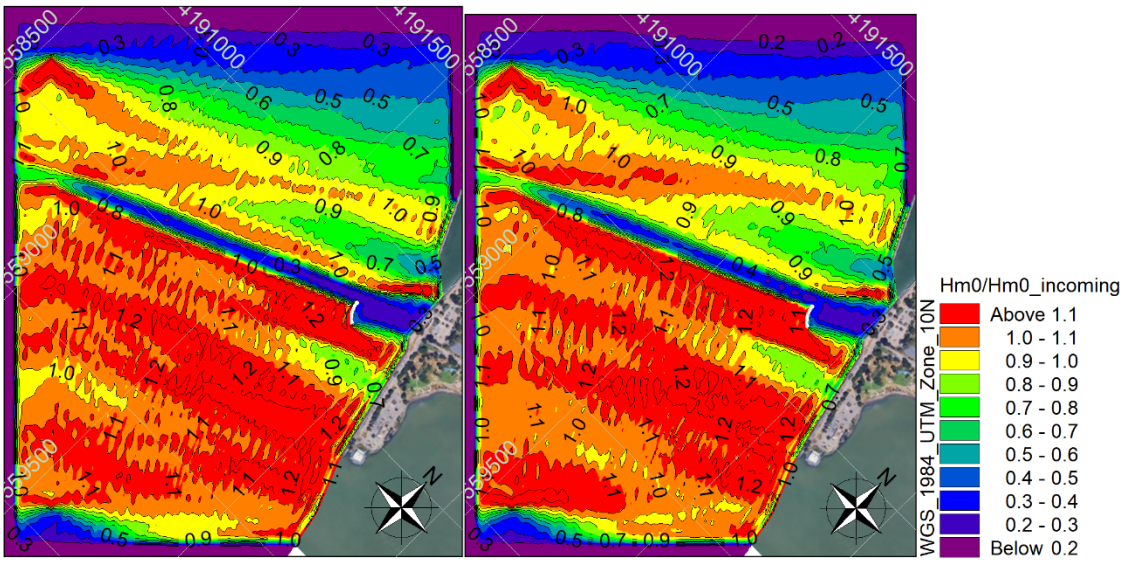


Figure C- 51 *Wave disturbance plot for wave direction = 240°N at MLLW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length

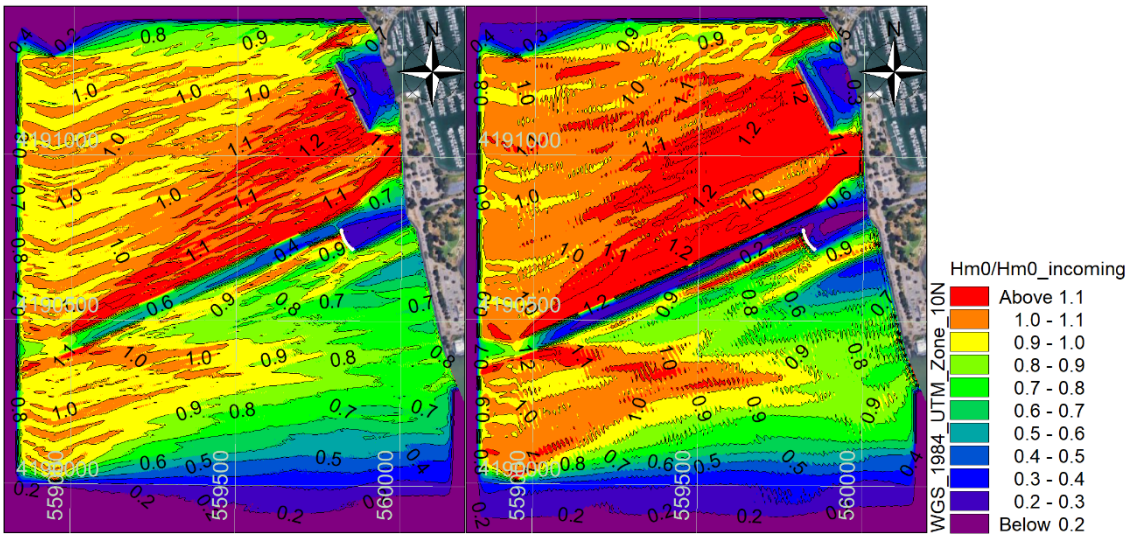


Figure C- 52 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length

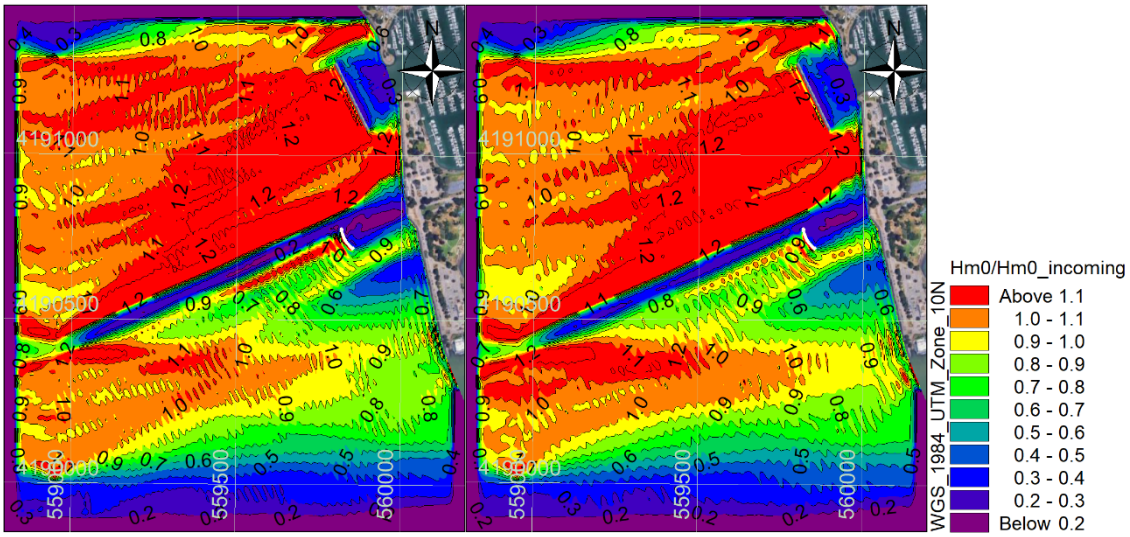


Figure C- 53 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length

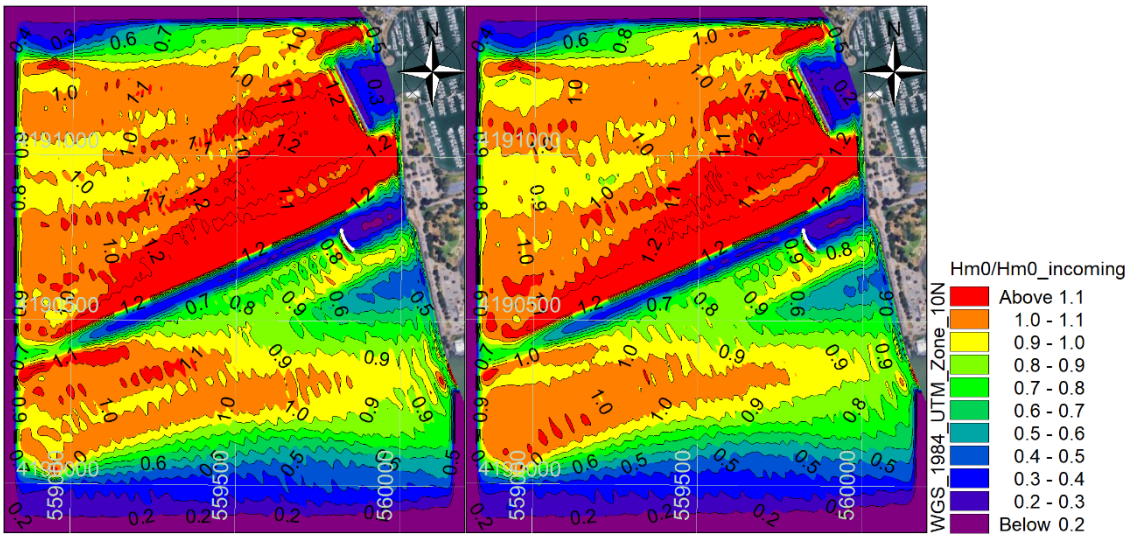


Figure C- 54 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length

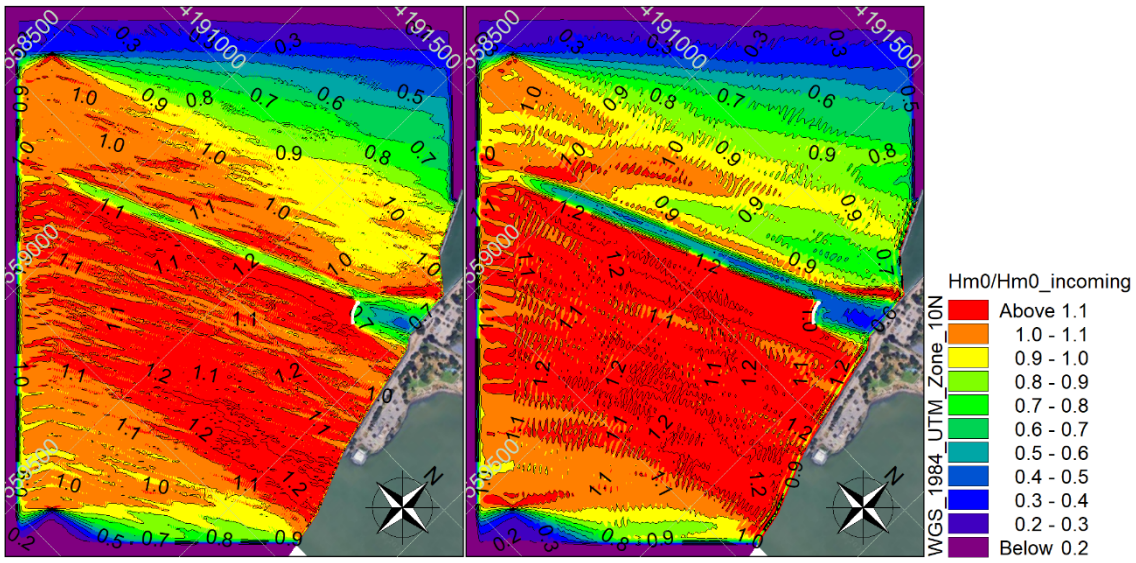


Figure C- 55 *Wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length*

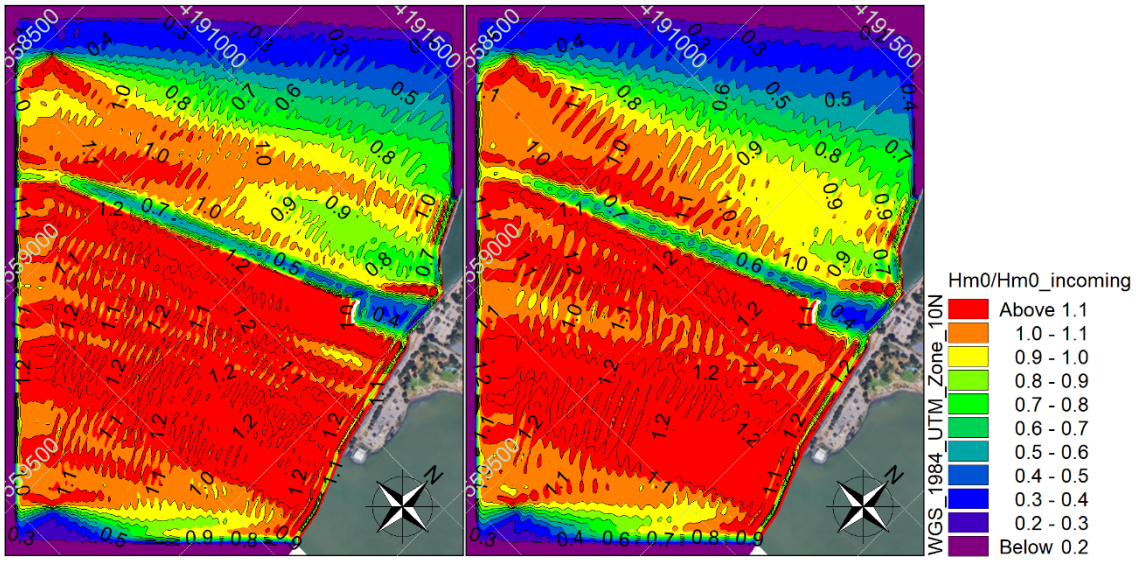


Figure C- 56 *Wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length*

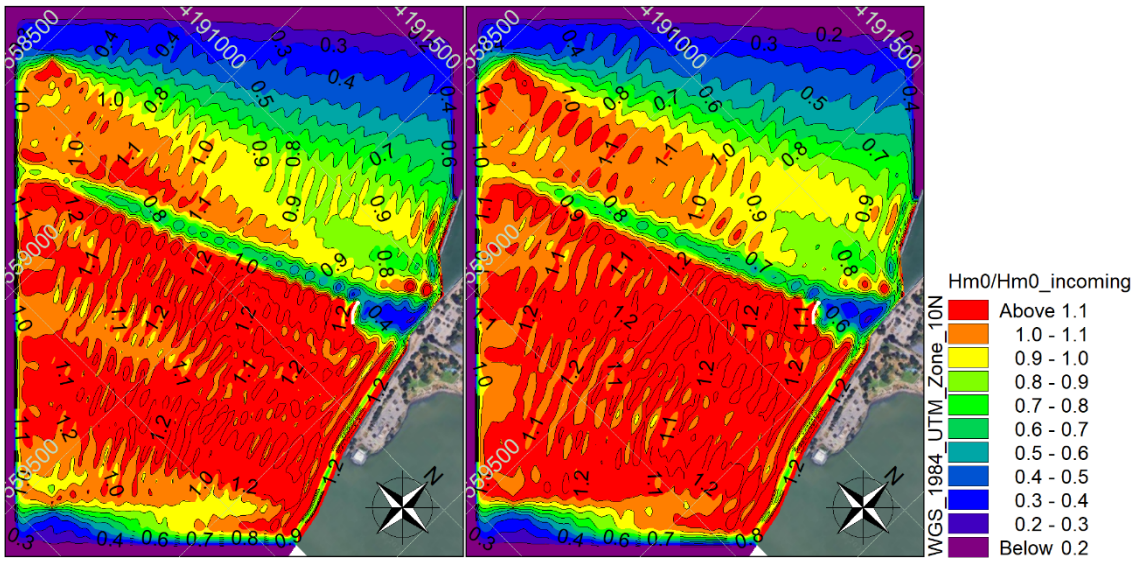


Figure C- 57 *Wave disturbance plot for wave direction = 240°N at MHHW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length

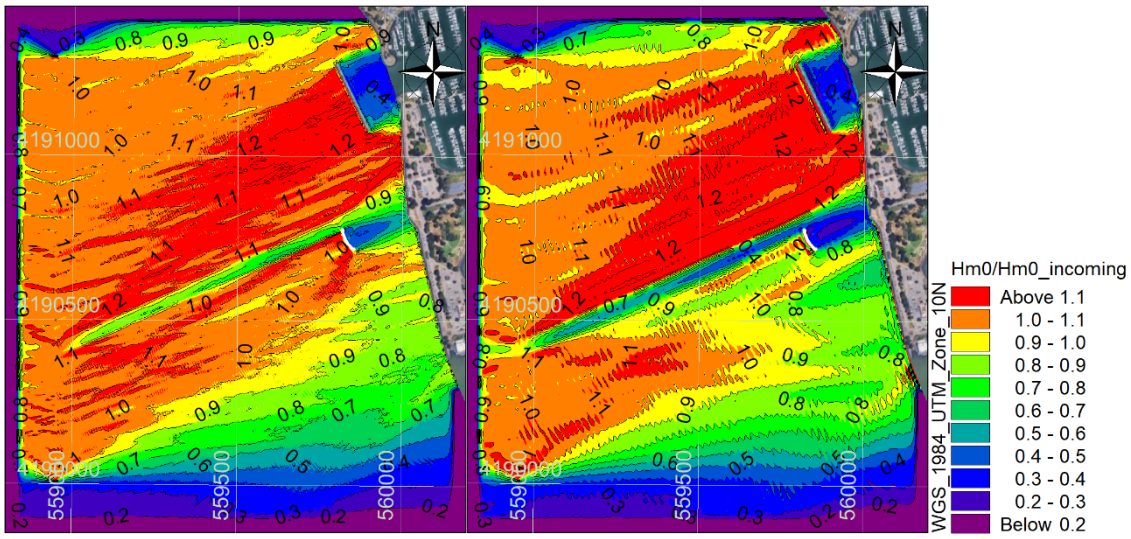


Figure C- 58 *Wave disturbance plot for wave direction = 255°N at MHHW.*
Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length

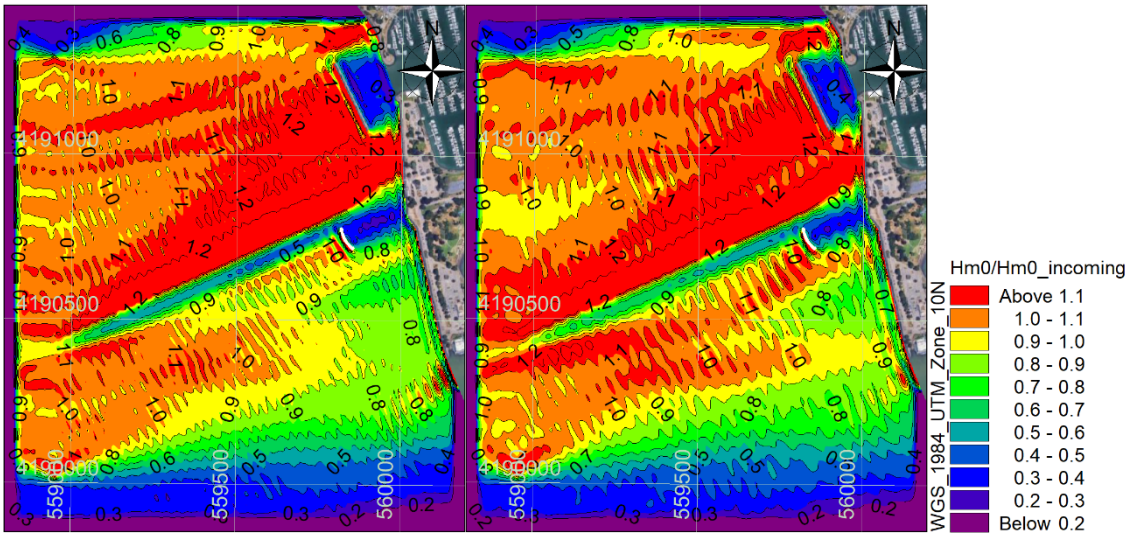


Figure C- 59 *Wave disturbance plot for wave direction = 255°N at MHHW.*
Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length

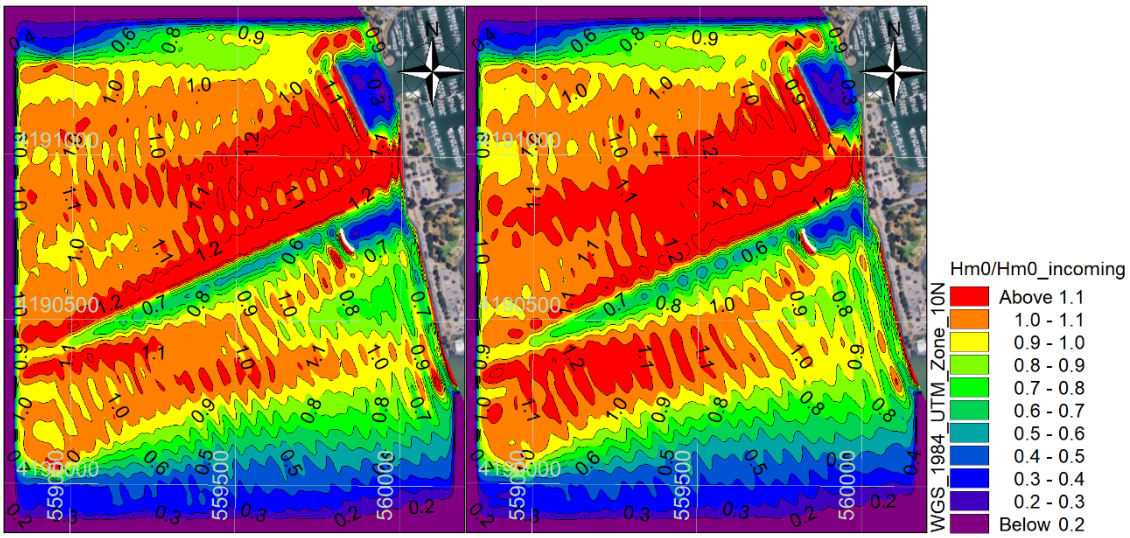


Figure C- 60 *Wave disturbance plot for wave direction = 255°N at MHHW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length

Ferry terminal wave disturbance maps – 250 ft breakwater length

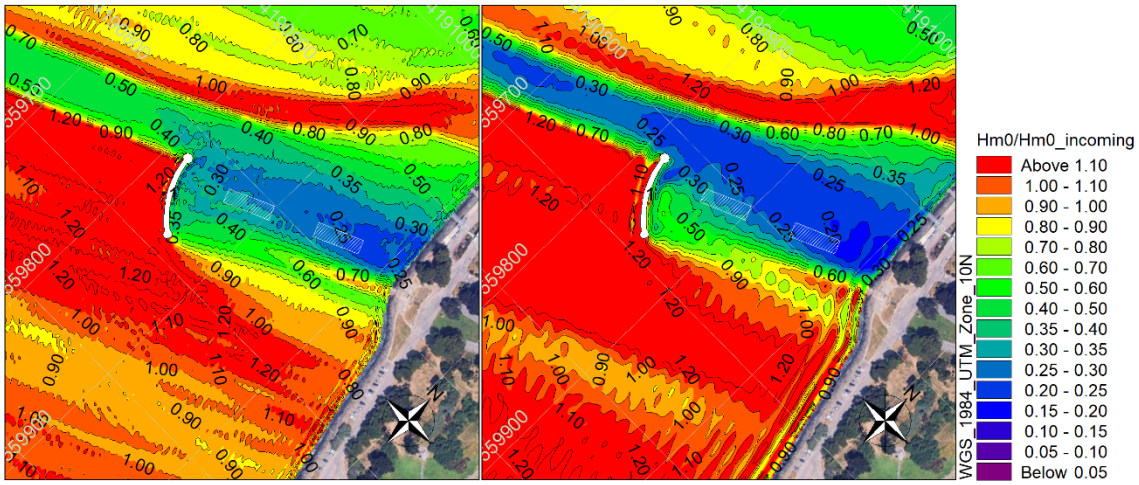


Figure C- 61 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length*

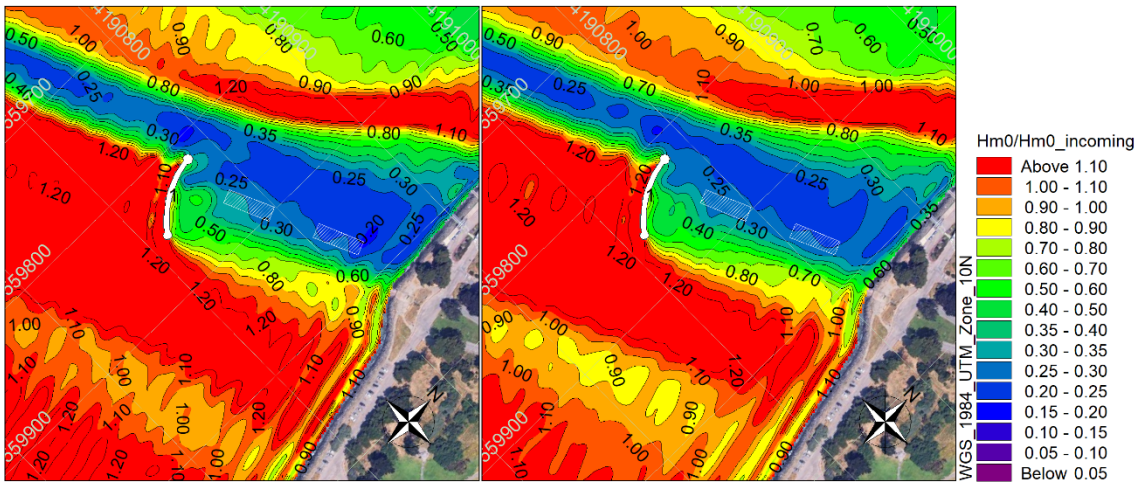


Figure C- 62 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length*

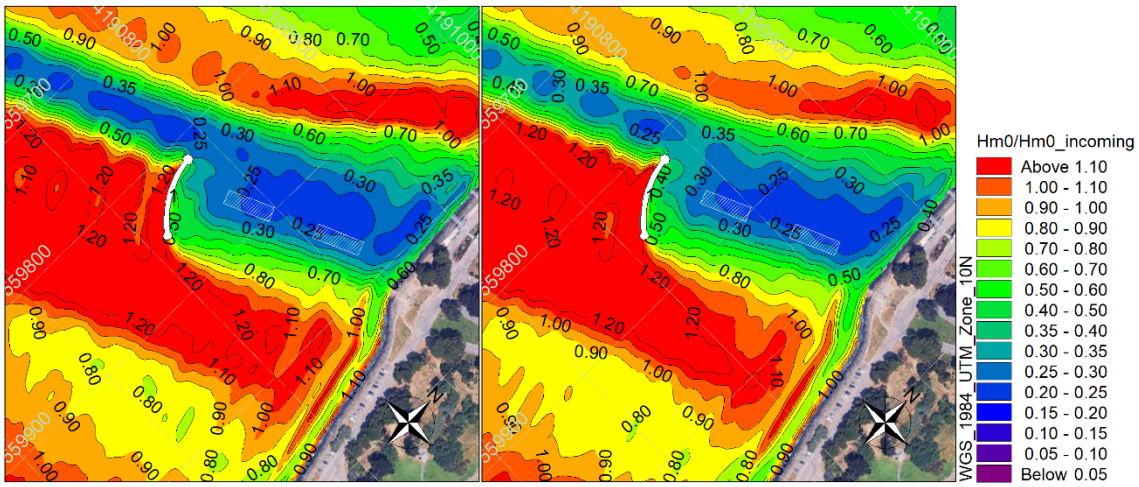


Figure C- 63 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length*

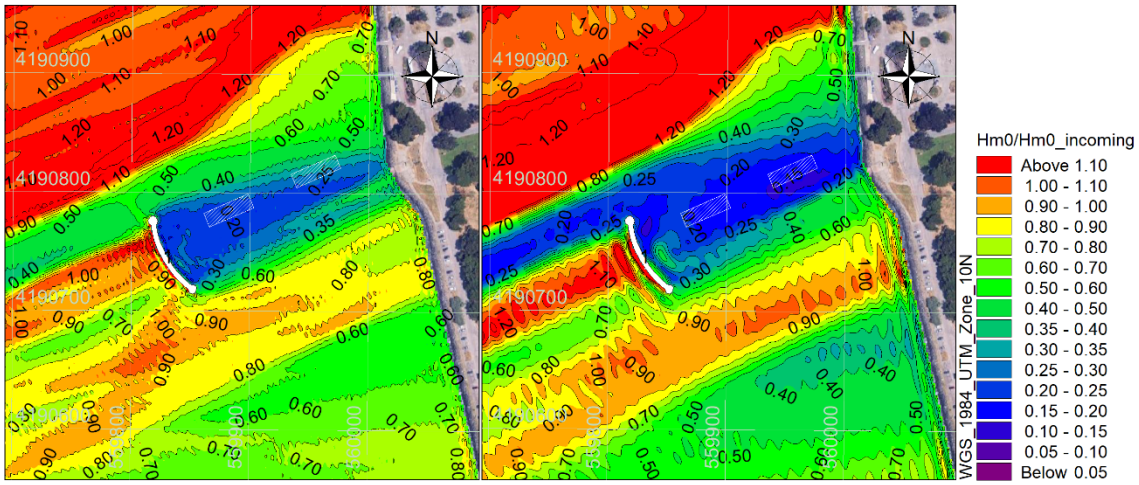


Figure C- 64 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length*

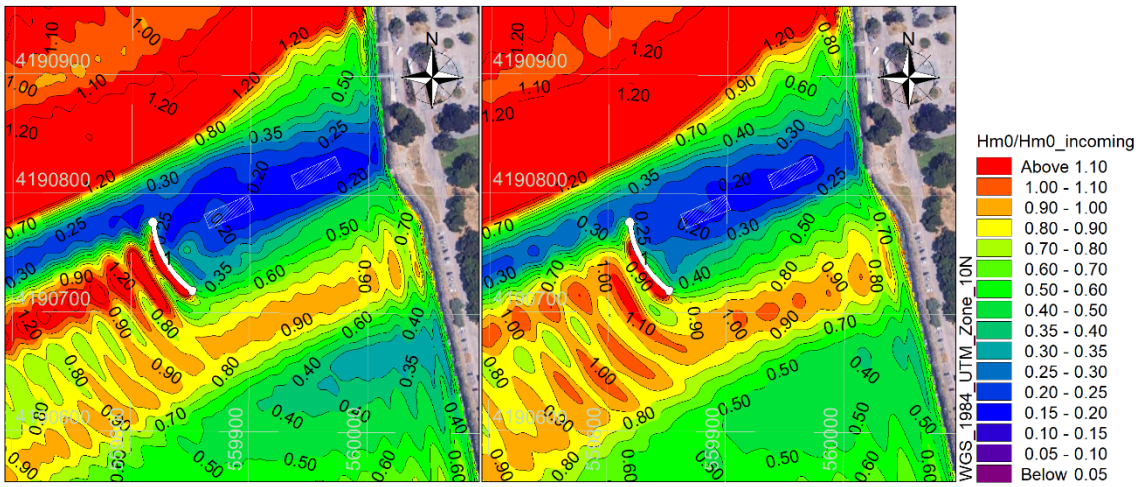


Figure C- 65 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length*

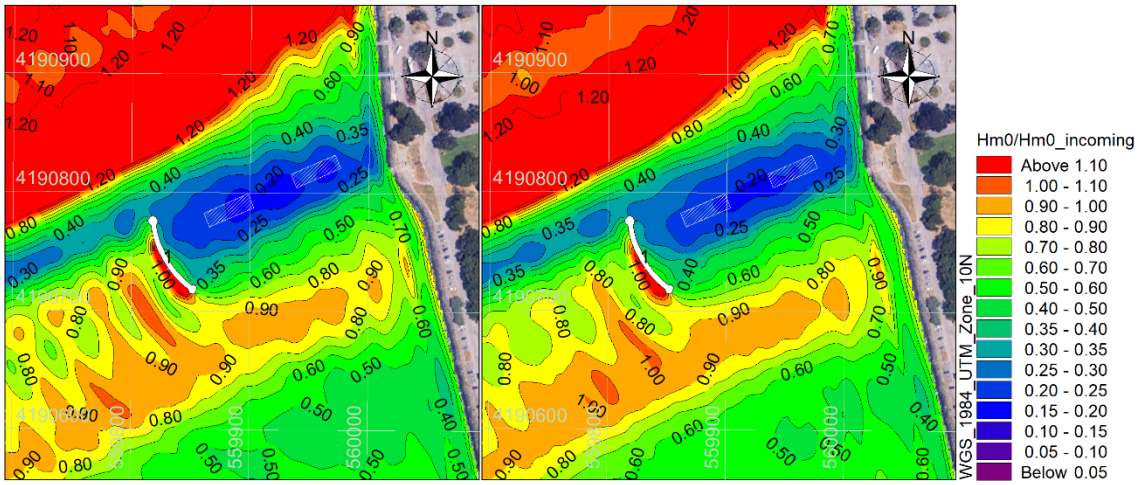


Figure C- 66 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length*

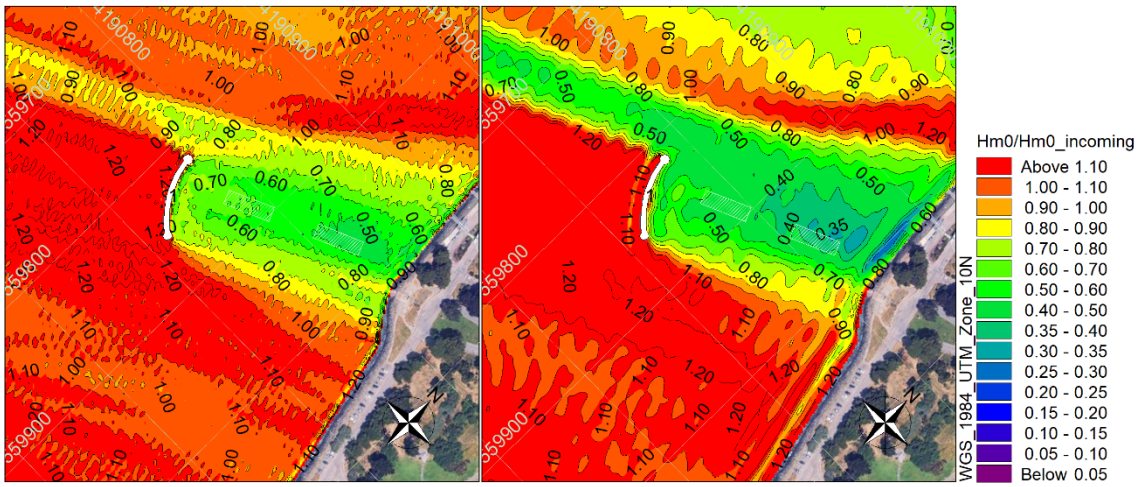


Figure C- 67 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length*

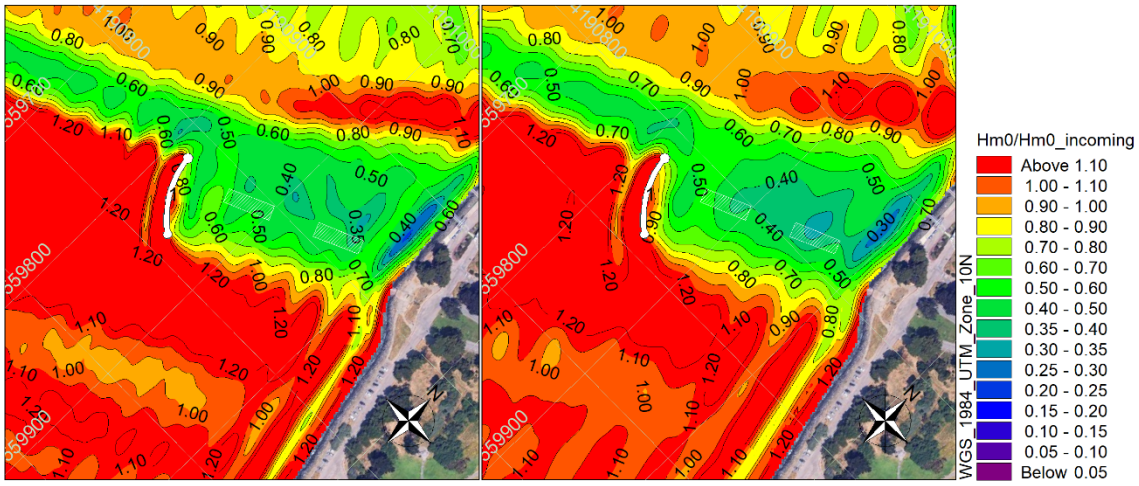


Figure C- 68 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length*

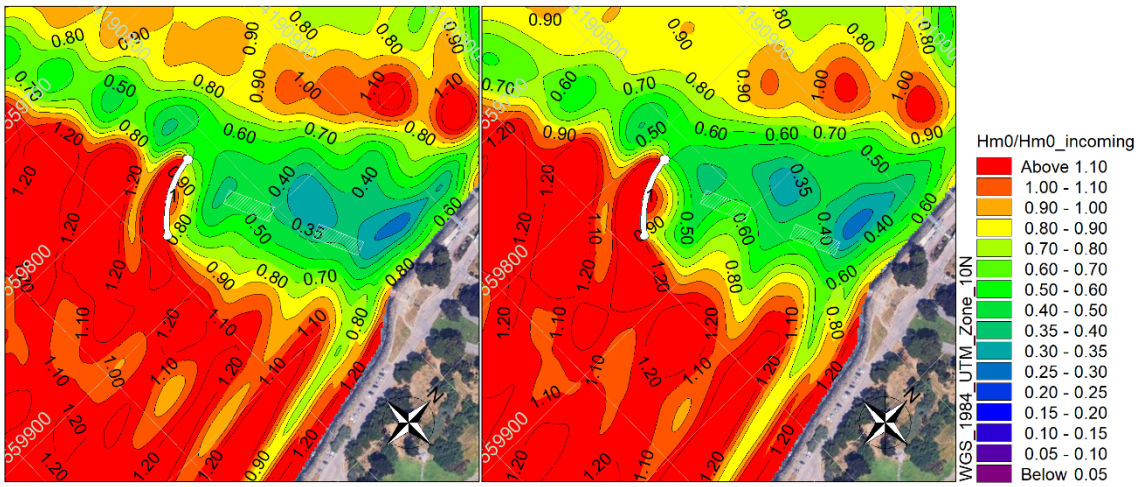


Figure C- 69 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length*

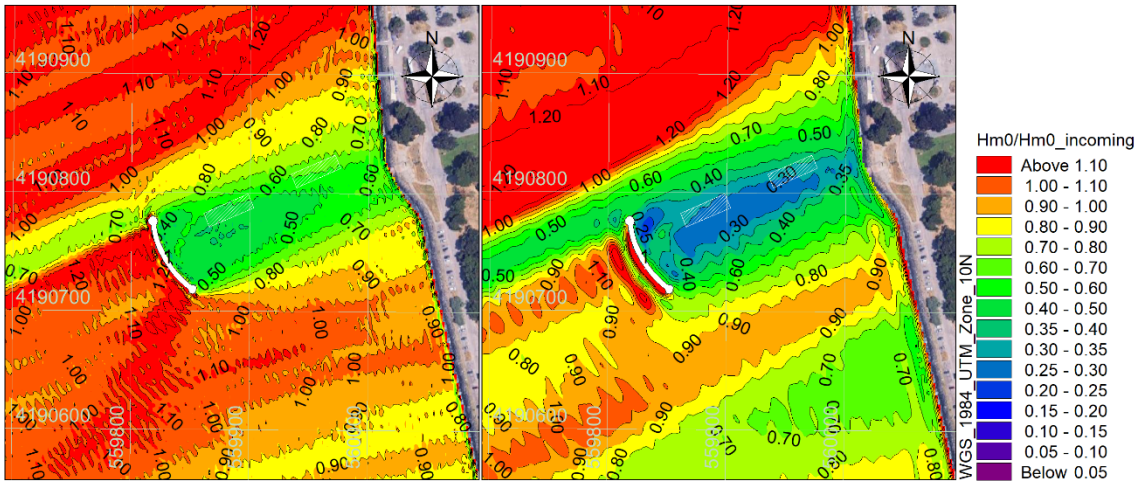


Figure C- 70 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length*

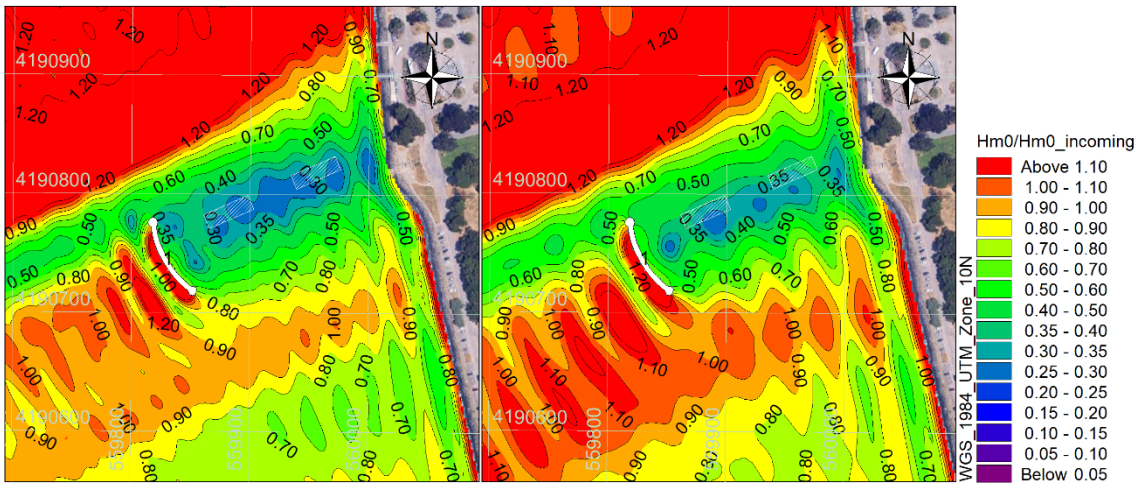


Figure C- 71 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length*

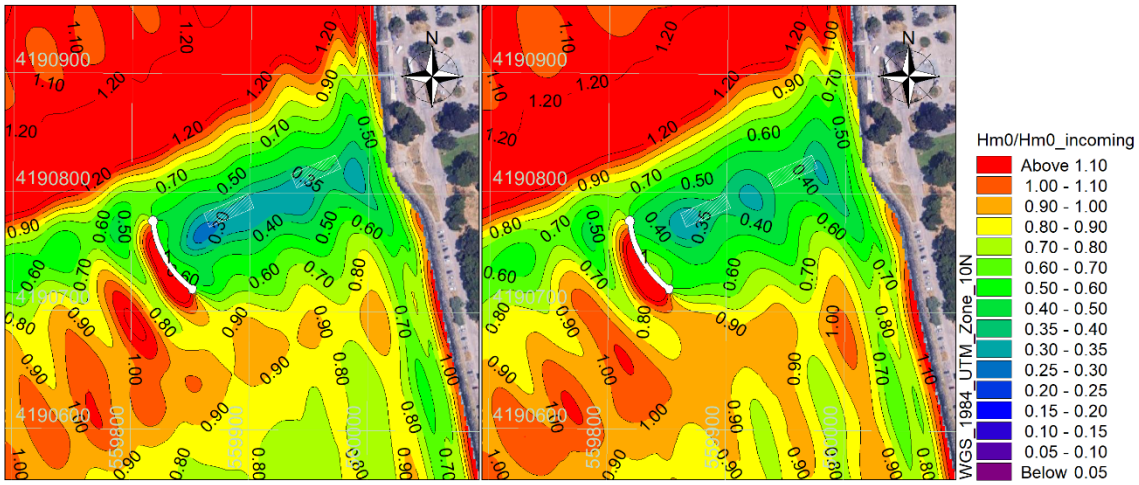


Figure C- 72 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length*

Appendix D - Sediment transport study

DECEMBER 2025
CITY OF BERKELEY

BERKELY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT

SEDIMENT TRANSPORT STUDY

COWI

DECEMBER 2025
CITY OF BERKELEY

BERKELY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT

SEDIMENT TRANSPORT STUDY

ADDRESS COWI North America
555 12th Street
Suite 1700
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PROJECT NO.
A272767

VERSION	DATE OF ISSUE	DESCRIPTION	PREPARED	CHECKED	APPROVED
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1 Introduction

The Berkeley Water Transportation Pier Ferry (BWTPF) Project is a major waterfront infrastructure initiative intended to enhance waterborne transit access along the San Francisco Bay (Figure 1-1). The project aims to revitalize a significant piece of waterfront infrastructure in Berkeley, California, transforming the existing pier into a modern, dual-purpose ferry terminal and public pier.

The project involves the design and implementation of a new ferry terminal at the foot of Berkeley Pier, including associated breakwaters, floating berths, a trestle, and shoreline plaza features. The concept layout is shown in Figure 1-2.

The sediment transport analysis was carried out for a 400 ft breakwater configuration, before design was optimized to a 300 ft breakwater length. The 100 ft reduction does not meaningfully change sedimentation patterns in the navigation channel. Modeled sedimentation within the berth and turning basin remains minimal, indicating that the results are also representative for the 300 ft breakwater layout and provide a reasonable approximation of sediment transport behaviour given the model and data limitations.

As part of the design process, COWI carried out hydrodynamic and sedimentation modeling to assess currents, wave-driven transport, and long-term sediment deposition within the dredged areas. The study indicates that sedimentation at the site is generally low, consistent with historical USACE surveys in the Berkeley Marina Channel, where long-term rates average approximately 0.07 ft per year.

For the BWTPF dredged areas, the six-year simulations show average infilling of approximately 0.20 ft per year in the approach channel and 0.09 ft per year within the berth pockets. Based on the modeled rates, maintenance dredging in the channel is expected approximately every 10 to 15 years depending on the selected operational depth (–10 ft or –9 ft MLLW), while the more sheltered berth pockets would require dredging roughly every 22 to 33 years. These findings align with post-dredge survey data collected in 2024–2025 and provide confidence in the localized predictions within the project footprint, while underscoring the importance of continued monitoring and future refinement of site-specific sediment inputs. This document summarizes the hydrodynamic and sedimentation modeling methodology, results, and implications for long-term maintenance planning for the BWTPF project.



Figure 1-1 BWTPF Location

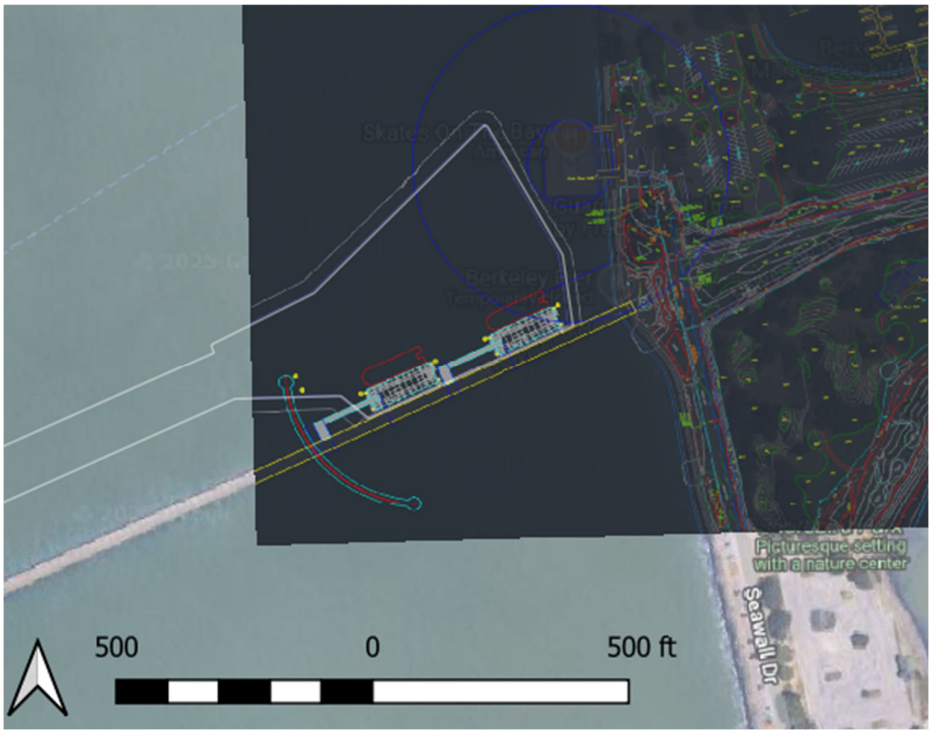


Figure 1-2 BWTPF Concept Layout

2 Data basis

This chapter summarizes the metocean data sets used for this study, which include various data sets (hindcast, measurements) used for setting up the bathymetry and boundary conditions for the numerical modeling.

2.1 NOAA CO-OPS

NOAA's Center for Operational Oceanographic Products and Services (CO-OPS) is the authoritative source for accurate, reliable, and timely tides, water levels, currents, and other coastal oceanographic and meteorological information. CO-OPS maintains ocean observing infrastructure, including more than 200 permanent water level stations on the U.S. coasts and Great Lakes, an integrated system of real-time sensors concentrated in busy seaports, and temporary meters that collect observations for tidal current predictions.

Table 2-1 shows the stations, the data types and the periods for which data were extracted. Location of the CO-OPS stations are shown in Figure 2-1.

Table 2-1 NOAA CO-OPS Stations and Data Description

Station ID	Station Name	Data Type	Period
9414290	San Francisco	Wind and Hourly Water Levels	1996-2024
9414750	Alameda	Wind and Hourly Water Levels	1996-2024
9414863	Richmond	Wind and Hourly Water Levels	1996-2024
s08010	Southampton Shoal Channel LB6	Currents	Aug 2023
s09010	Oakland Outer Harbor LB3	Currents	Aug 2023

Tidal datums obtained at the three CO-OPS stations are provided in Table 2-2.

Table 2-2 Tidal Datums at CO-OPS stations within the study area (ft MLLW).

Datum	Station 9414290 San Francisco	Station 9414750 Alameda	Station 9414763 Richmond
MHHW	5.8	6.6	6.0
MHW	5.2	6.0	5.4
MSL	3.1	3.4	3.3
MLW	1.1	1.1	1.1
MLLW	0.0	0.0	0.0
NAVD88	-0.1	0.2	0.0

2.2 NOAA NDBC

NOAA’s National Data Buoy Center (NDBC) maintains a network of marine observation and monitoring infrastructure which collect and disseminate in-situ, real-time, quality-controlled observations in the marine environment.

Table 2-3 shows the stations, the data types and the periods for which data were extracted. Location of the NDBC station is shown in Figure 2-1.

Table 2-3 NOAA NDBC Station and Data Description

Station ID	Station Name	Data Type	Period
46026	San Francisco Offshore	Wind	1982-2024

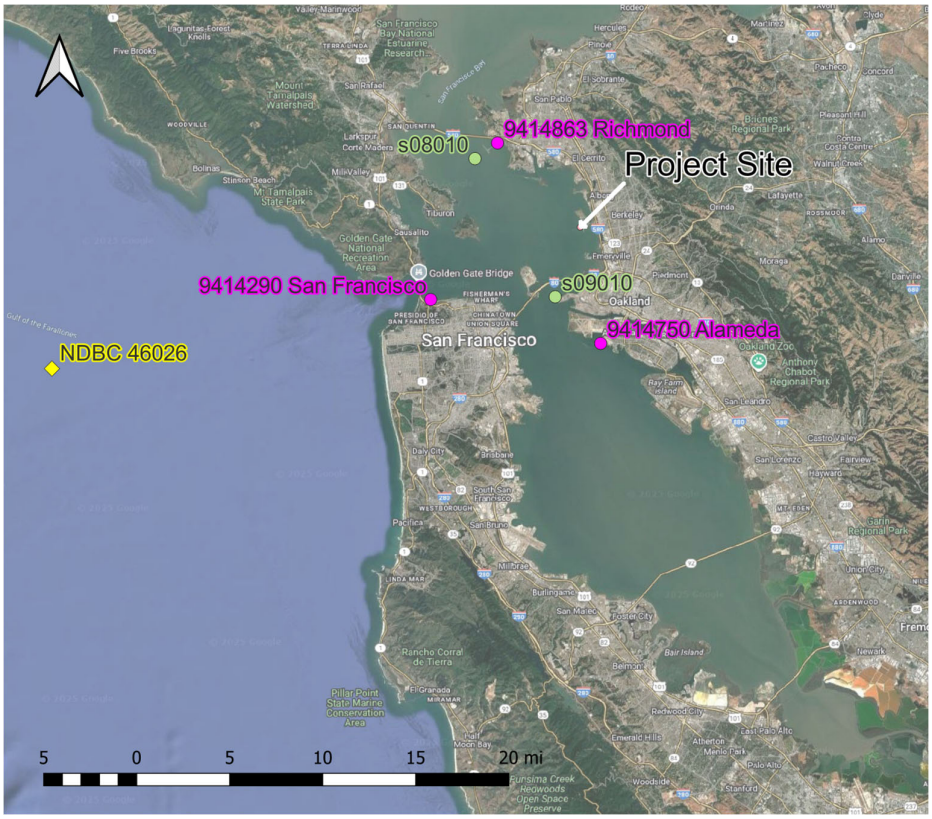


Figure 2-1 Wind, water levels, and current measurement locations.

2.3 Bathymetry

The model bathymetry for the sedimentation study was created based on the United States Geological Survey (USGS) Digital Elevation Model (DEM) for nearshore regions and survey data. Details of applied bathymetry data are given below:

- **USGS DEM** (2024): A digital bathymetric model with 1/3 arc-second (approximately 10 m) resolution, derived from diverse source data. The DEM covers the conterminous United States, Hawaii, Puerto Rico, other territorial islands, and limited areas of Alaska. See Figure 2-2 for extent of coverage available within the model domain used in this study.

- **Local Bathymetric Survey** (2024): Bathymetry data for the site (Figure 2-3) was produced by eTrac Engineering Inc, a Woolpert Company for COWI. Survey data was collected on July 24, 25, 27 and 29, 2024 and plotted on August 22, 2024.

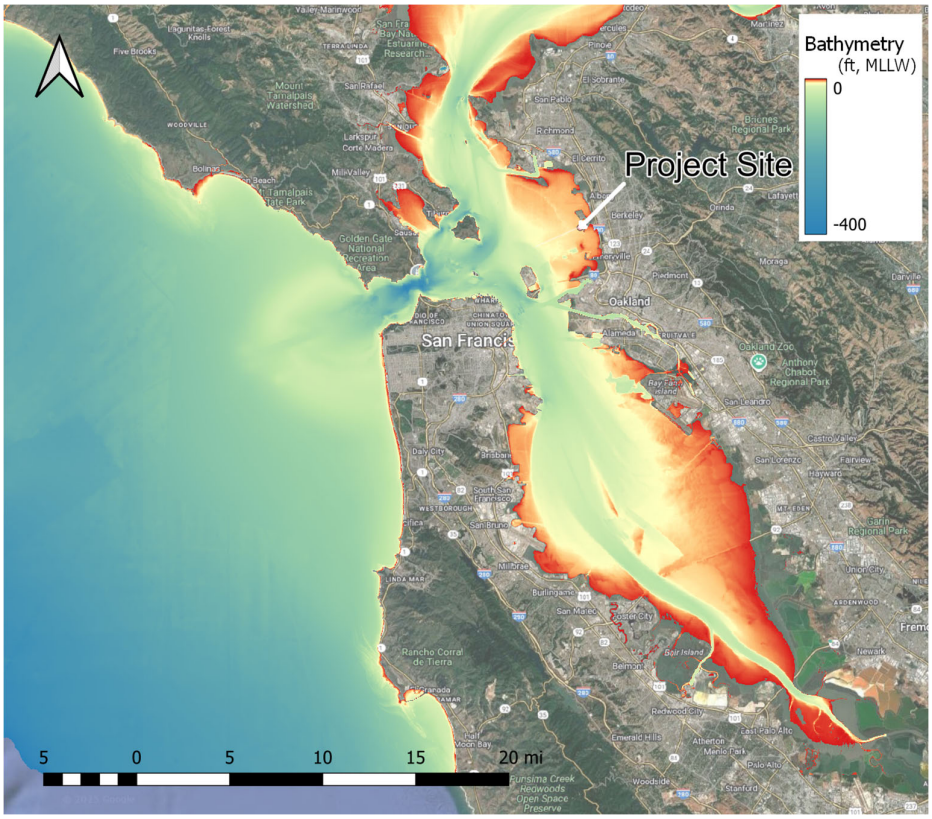


Figure 2-2 USGS DEM.

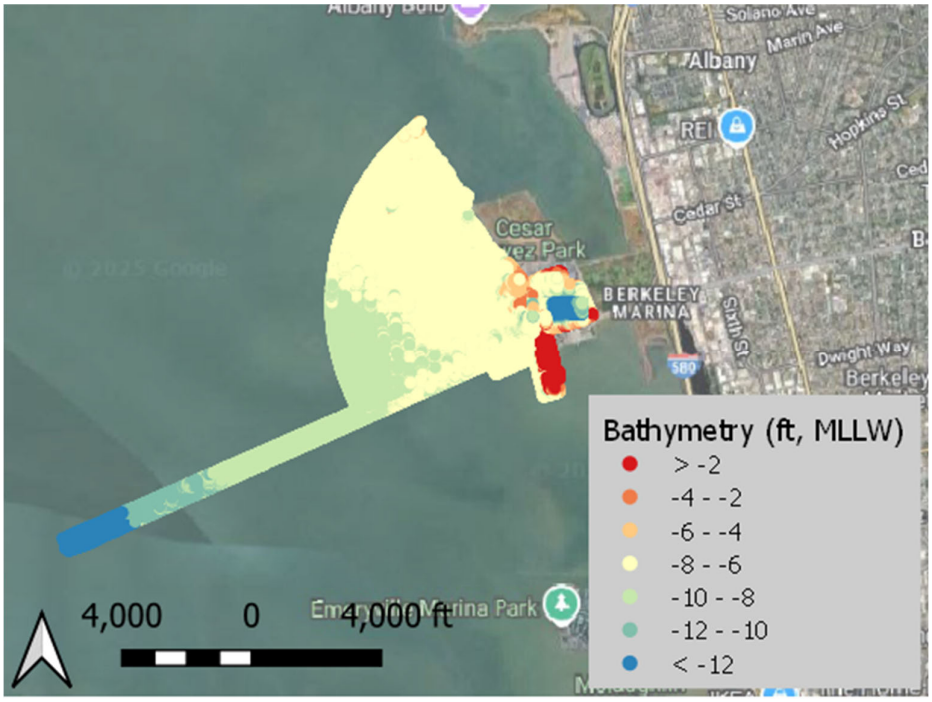


Figure 2-3 Bathymetric data from eTrac survey.

2.4 Seabed properties

The geotechnical report dated December 16, 2024, prepared by ENGEO Incorporated (Ref (ENGEO Incorporated, 2024)) contains soil boring samples at locations around the breakwater and berthing area. The seabed type in this area is generally of sandy silt type and the D50 is given to be 0.2 mm.

2.5 Suspended sediment concentration

A review of available literature such as USGS reports with suspended sediment concentration data in San Francisco Bay (Ref (Buchanan & Ganju, 2004) and Ref (Buchanan & Morgan, 2014)) show that the mean and median suspended sediment concentration in the Central San Francisco Bay region varies between 17-20 mg/L.

2.6 USACE hydrographic surveys for Berkely Marina Channel[1]

United States Army Corps of Engineers (USACE) hydrographic conditions surveys were retrieved to calculate the prevalent sedimentation rates in the project site. The latest survey data available is shown in Figure 2-4 to indicate the extent of data available. The sedimentation rates in the channel area indicated as black hatch in Figure 2-4 was calculated using survey data over four 3-year periods between 2011 and 2023 and are tabulated in

Table 2-4.

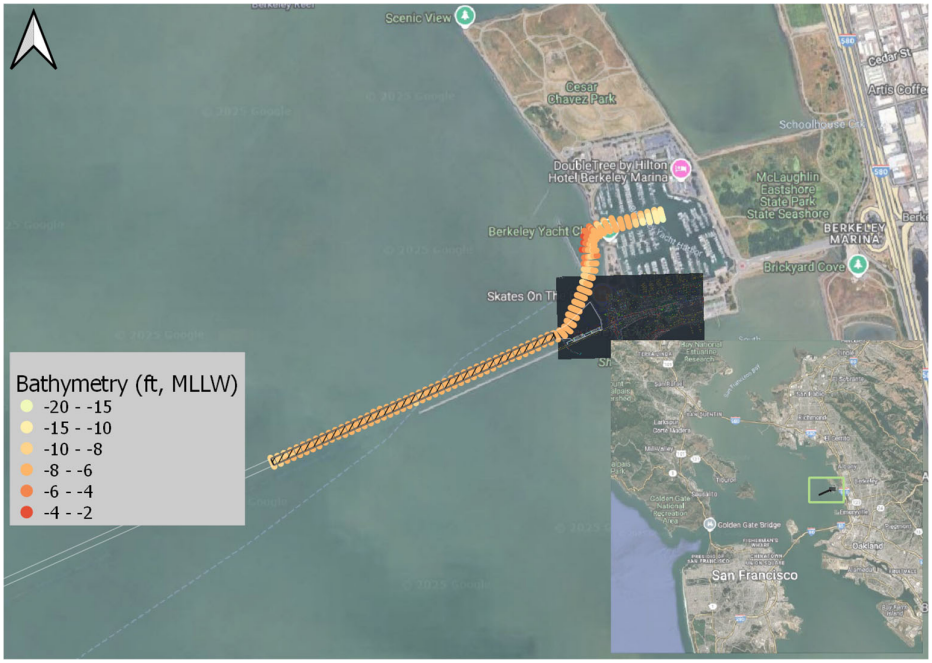


Figure 2-4 USACE hydrographic condition survey for Berkeley Marina Channel dated May 2024.

Table 2-4 Mean Sedimentation over different 3-year periods in Berkeley Marina Channel

From	To	Mean Sedimentation (ft)

[1] USACE Berkeley Marina Federal Navigation Channel (hereafter referred to as the Berkeley Marina Channel)

July 2011	Sep 2014	0.284
Sep 2014	Nov 2017	-1.267
Nov 2017	Apr 2020	0.147
Apr 2020	Dec 2023	0.044

Table 2-4 shows that the sedimentation was < 0.05 ft in the 2020-2023 period, indicating practically no sedimentation. The 2014-2017 period showed erosion to the channel, possibly related to dredging or issues with the survey/datum. These periods were excluded from the average used for calibration.

The two remaining periods with sedimentation have values of 0.147 ft. and 0.284 ft. Therefore, an average of 0.21 ft over 3 years, i.e., 0.07 ft/year is considered as the prevalent sedimentation rate for this channel segment.

3 Wind analysis

San Francisco Bay region is characterized by varied topography and water bodies which cause significant variations in the wind speed and directionality within and outside the Bay area. Figure 3-1 shows annual wind roses at the four measurement stations where data is available from 1997-2023 and indicates that the wind speed and directionality in the approaches to the project site indeed vary from the offshore region to the inside of San Francisco Bay.

While the offshore region winds are up to 30 knots and mainly from WNW-NW direction, wind speeds measured inside the San Francisco Bay are generally 10-15 knots with significant variation in the directionality.

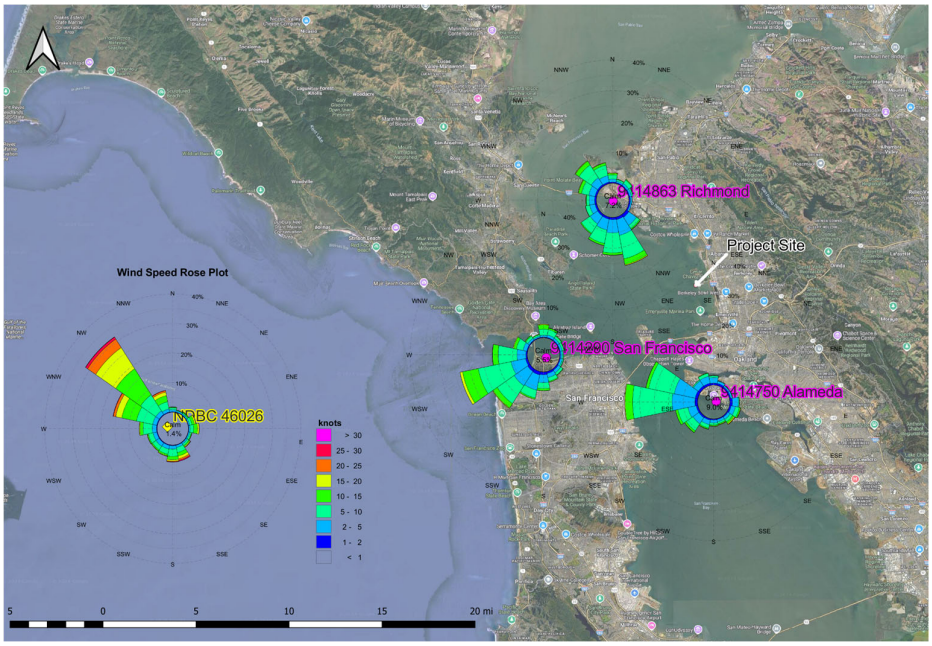


Figure 3-1 Wind roses from measured data in the vicinity of the project site.

Within San Francisco Bay, CO-OPS 9414290 is located to the east of the project site and CO-OPS 9414863 is located north. Winds at CO-OPS 9414290 are seen to be from SW-W directions (conforming to the geographical features of the area), whereas at CO-OPS 9414863, the winds are predominantly from S-SSE and from NW. At CO-OPS 9414750, which is located south of the project site, winds are from W-WNW directions.

Wind data from U.S. National Center for Environmental Information (NCEI) stations near the project site, shown in Figure 3-2, were also used to ascertain the variation in directionality near the project site.

Since wind conditions are one of the main inputs to a numerical sedimentation model, it is important to specify a spatially varying wind field in this scenario.

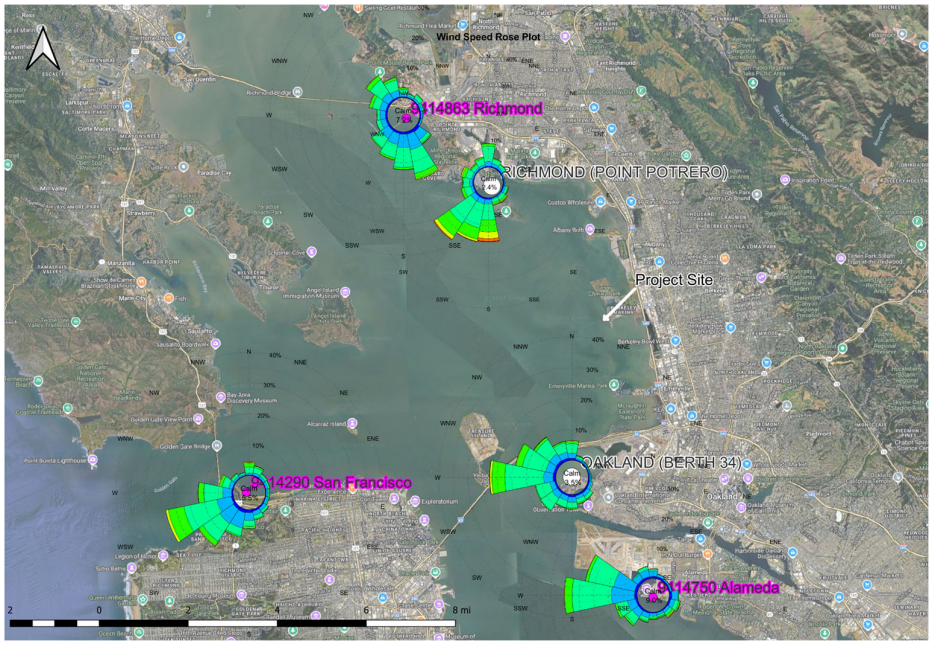


Figure 3-2 *Wind roses from measured data within the San Francisco Bay near the project site.*

4 Hydrodynamic modeling

A hydrodynamic modeling study was carried out by setting up a regional model to derive flow conditions at the site. The regional model was forced using astronomical tidal constituents for surface elevation and velocities from OSUTide database (Ref (Egbert & Erofeeva, 2002)) at its offshore boundaries.

4.1 Model description

The hydrodynamic model was developed using the MIKE 21 HDFM (Hydrodynamic Flexible Mesh) module of the comprehensive 2-dimensional MIKE 21 modeling package from DHI, Denmark. MIKE 21 HDFM is a modeling system for 2D free-surface flows (Ref (DHI, 2023)). It can be applied to a wide range of hydraulic and related phenomena. This includes modeling of tidal hydraulics, wind and wave generated currents, storm surges, and flood waves.

The HDFM module is the basic module of the system and is used in the simulation of hydraulics and related phenomena in lakes, estuaries, bays, coastal areas, and seas where flexibility inherited in the unstructured meshes can be utilized.

The applied MIKE 21 HDFM model requires the following main input for flow simulations:

- Bathymetry of the area.
- Hydrographic boundary conditions (water levels, flow velocity components or fluxes).
- Wind and air pressure of the area.
- Eddy viscosity and bed resistance (Manning number).

4.2 Modeling Methodology

The purpose of the hydrodynamic study is to derive flow conditions close to the project site for a 3-year period.

The following approach is adopted for the regional hydrodynamic study.

- A regional hydrodynamic model was established using the bathymetry data sets described in Section 2.3.
- Derivation of tidal boundary conditions from OSUTide database in the form of flow velocities and surface elevations.
- Wind speed and direction time series from NOAA NDBC Buoy 46026, CO-OPS Stations 9414290, 9414750, and 9414863 were used to create of a spatial wind field mosaic using the extents shown in Figure 4-1. The mosaic thus created was used as wind forcing. See Section 3 for wind data and directionality analysis.
- The regional hydrodynamic model was validated with water level and current speed measurements from NOAA CO-OPS at multiple locations within the model domain.

› The validated regional hydrodynamic model was used to generate boundary conditions for the local model.



Figure 4-1 Polygons used to create spatial wind field mosaic.

4.3 Model bathymetry

The model domain and mesh established for the regional hydrodynamic model is presented in Figure 4-2. The model domain includes the entire southern and central portions of San Francisco Bay and extends in the north up to San Pablo Bay (up to Benicia Bridge). The offshore boundaries are located near the NDBC Buoy 46026.

The domain is divided into several zones with different triangular mesh resolutions. The spatial resolution of the mesh varies from an average element size of ~500 m in the region outside San Francisco Bay, transitioning gradually to a minimum of 100 m in the Central San Francisco Bay where the site is located.

Figure 4-3 shows the model bathymetry.

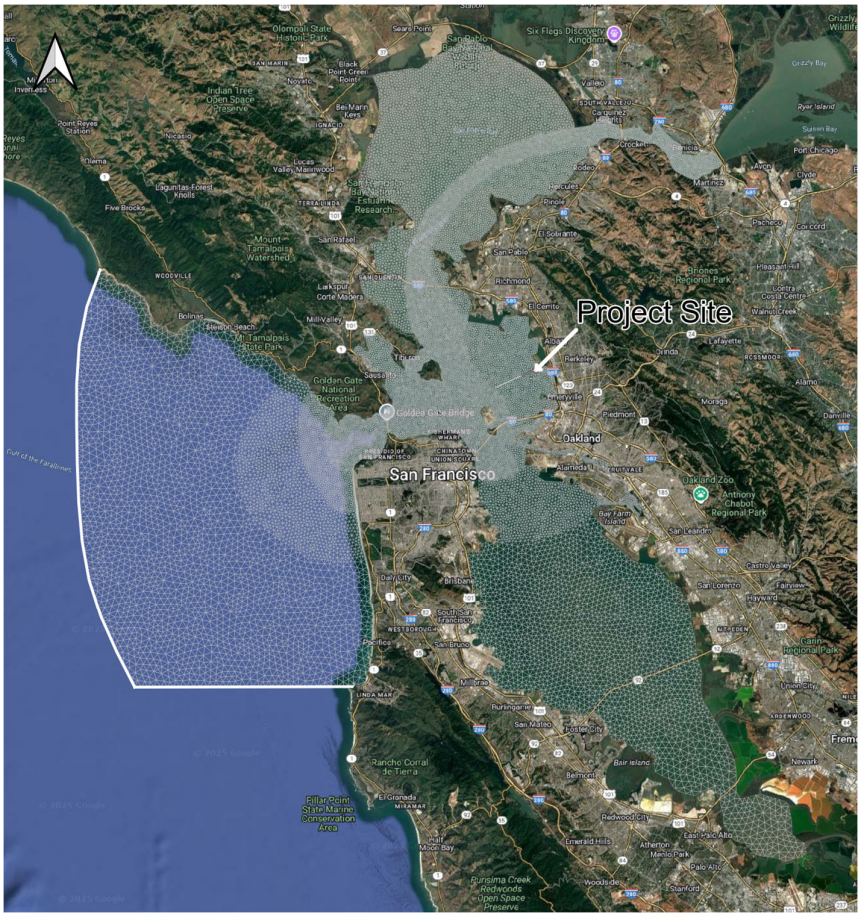


Figure 4-2 Model domain and mesh.

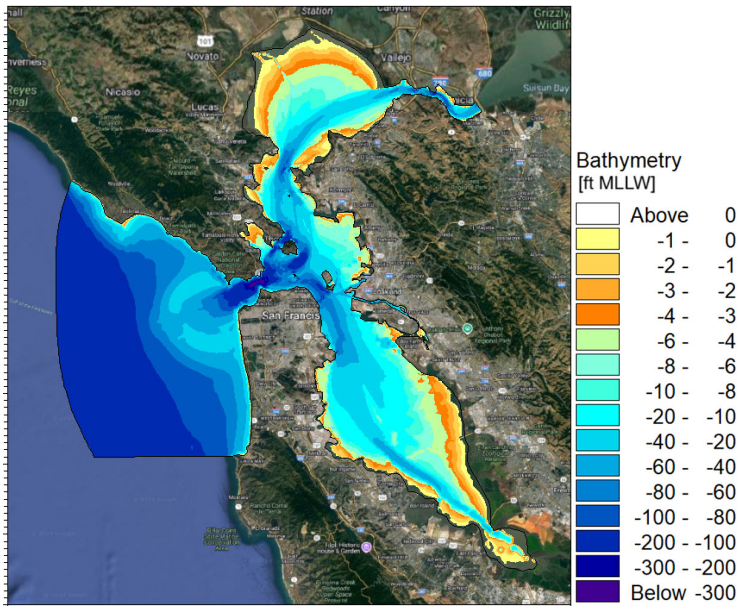


Figure 4-3 Model bathymetry.

4.4 Model validation

The regional hydrodynamic model was evaluated using measurements of water levels available at NOAA CO-OPS Stations 9414750 (Alameda), 9414290 (San Francisco), and 9414863 (Richmond). The model performance was also compared against measured current speeds at NOAA CO-OPS Stations s080010 and s090010. Figure 2-1 shows the locations of water levels and current measurements.

Figure 4-4 shows the comparison of water levels between model results and measurements during August 2023.

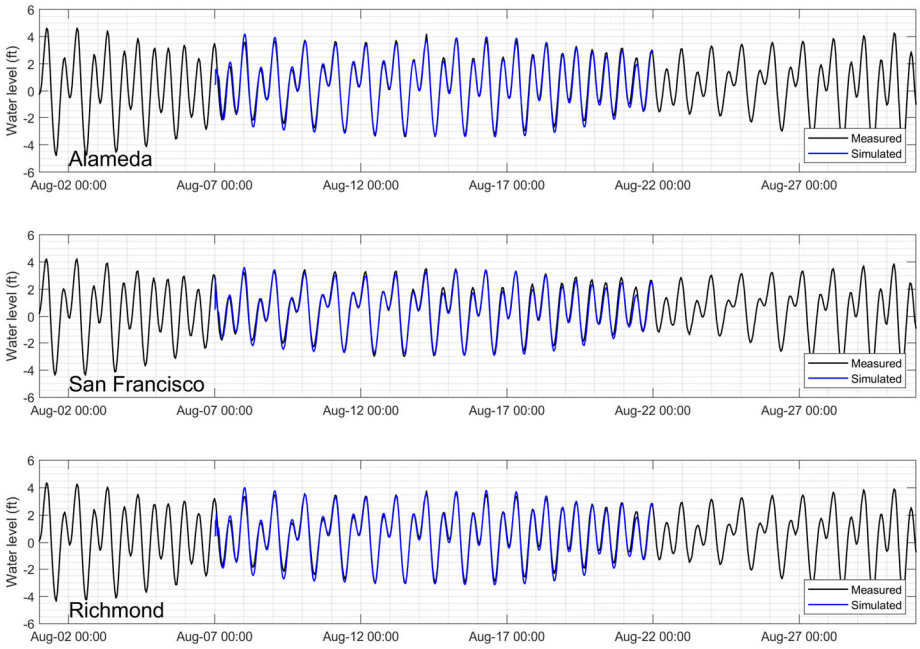


Figure 4-4 *Comparison of water levels between model results and measurements at NOAA CO-OPS stations during August 2023.*

A time series comparison of current speeds between model results and measurements are shown in Figure 4-5 during August 2023.

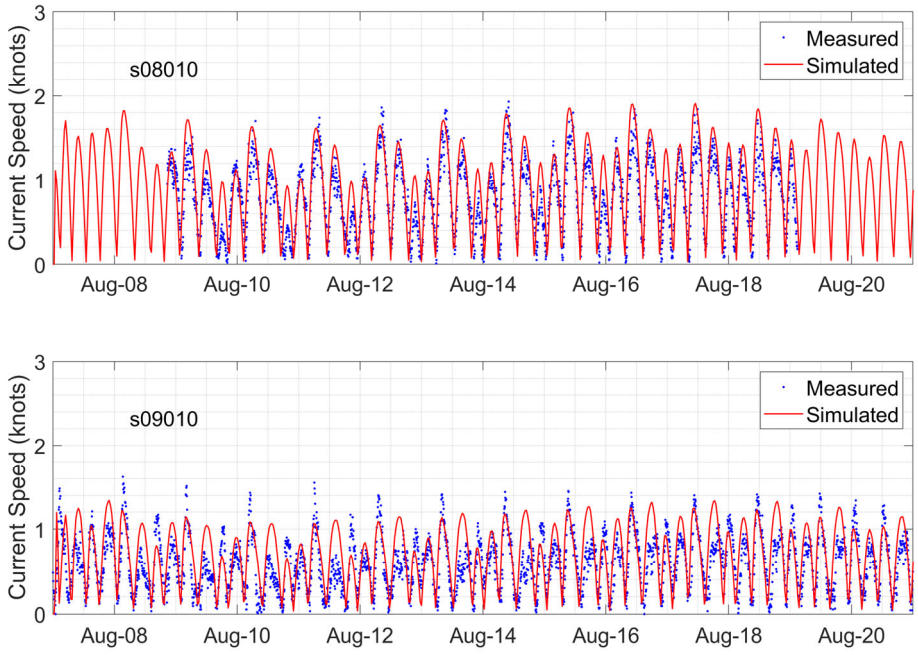


Figure 4-5 *Comparison of current speeds between model results and measurements at NOAA CO-OPS stations during August 2023.*

Figure 4-4 and Figure 4-5 indicate that the regional hydrodynamic model is adequately representing the flow conditions within Central San Francisco Bay.

4.5 Model parameters

Table 4-1 contains a description of model parameters selected to obtain the comparison shown in the sections above.

Table 4-1 *Model Parameters*

Parameters		Value
Flood and Dry	Drying depth	0.005 m (0.20 in)
	Wetting depth	0.1 m (4 in)
Eddy Viscosity	Smagorinsky formulation	0.28
Bed Resistance	Manning number	55
Wind Friction	Constant	0.001255

4.6 Model results

The regional hydrodynamic model was simulated for a continuous period of 3-years, i.e., 2020-2023 and was used to extract flow parameters at local model boundaries during sedimentation model setup and development.

Figure 4-6 and Figure 4-7 show flow pattern in the project site during typical spring flood and ebb phase of the tide. The current speeds at the project site are between 0.2 – 0.4 knots during peak flood and ebb tides and are generally less than 1 knot in the extended vicinity.

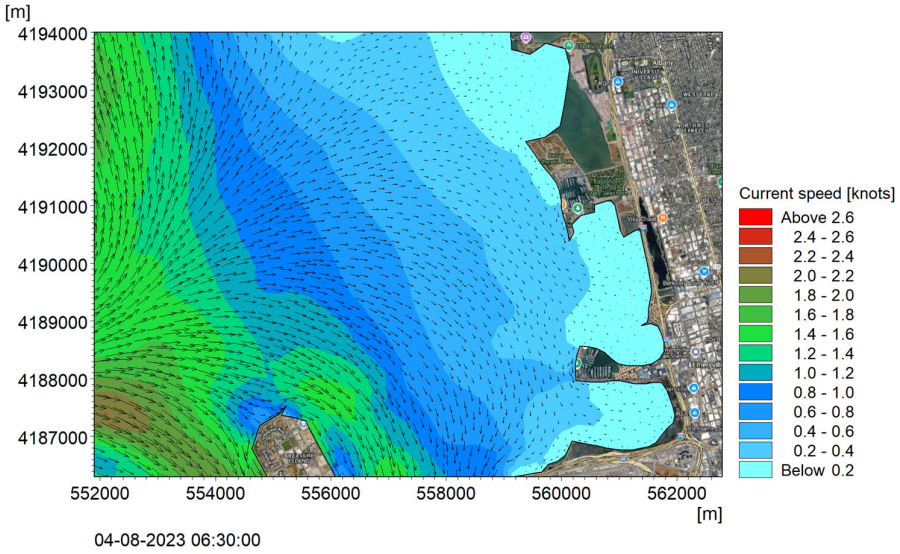


Figure 4-6 Flow field near the project site during a typical peak flood phase of a spring tide.

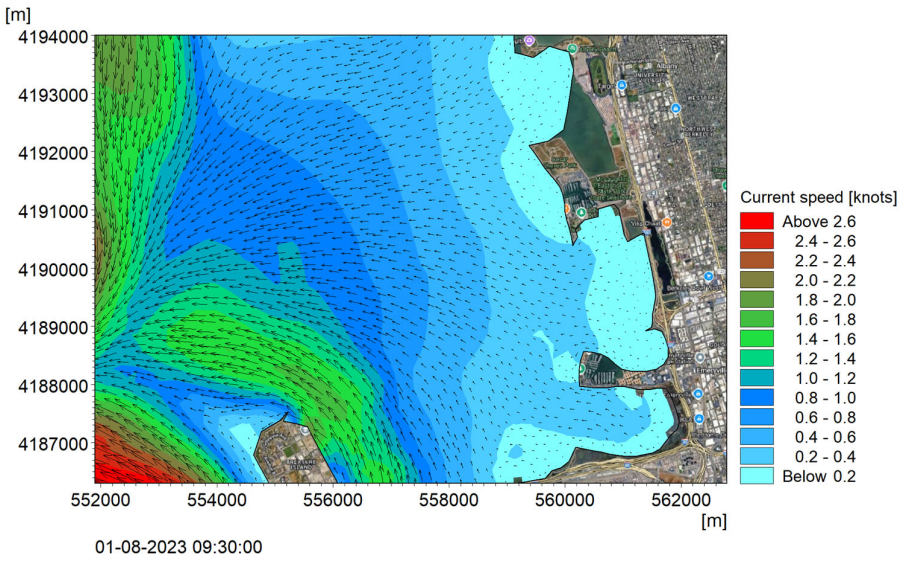


Figure 4-7 Flow field near the project site during a typical peak ebb phase of a spring tide.

5 Sedimentation model

The sedimentation assessment was carried out using the MIKE 21 coupled model that combines HD (hydrodynamics), SW (spectral wave), and MT (mud transport) models and describes the bottom evolution of the seabed for combinations of currents, water levels, and waves. The simulation was carried out for a period of 6 years to assess the sediment deposited at the project site with deepened bathymetry incorporated.

5.1 Modeling methodology

The following approach was adopted for the sedimentation study.

- A coupled model was established for the assessment of flow, wave, and sediment transport condition.
- Sediment grain size information for the sedimentation study was established based on the analysis of seabed sampling provided in Ref (ENGEO Incorporated, 2024). An average grain diameter of d_{50}=0.2 mm was assumed.
- Analysis of USACE hydrographic surveys available for Berkeley Marina Channel (ref Section 2.6) showed that the average sedimentation for a 3-year period was 0.21 ft. Hence, a 3-year calibration period was selected as the model duration to determine model parameters.
- Wave boundary conditions for the Coupled model was established from the Wave Transformation Study, ref. (COWI, 2023).
- Flow boundary conditions for the Coupled model were established using the validated regional model (Section 4).
- The coupled model was simulated for a 6-year period to determine sedimentation rates within the project site.

5.2 Modeled scenarios

Two scenarios were modeled:

Scenario 1: Existing seabed

Scenario 2: 400-ft breakwater with approach channel and berth pocket dredged to -12 ft MLLW.

5.3 Model bathymetry

A local model was established for the sedimentation study. The local model domain and mesh is presented in Figure 5-1 and is defined for extent of ~5 kms west and north-south of the project site. The model mesh is comprised of triangular elements, whose side, i.e., mesh resolution, varies from 100 m at the offshore boundaries to 10 m in the approach channel and close to the project site.



Figure 5-1 Model domain and mesh used for sedimentation study.

The model bathymetry used for the existing seabed scenario is given in Figure 5-2 and the model bathymetry for 400-ft breakwater scenario is given in Figure 5-3.

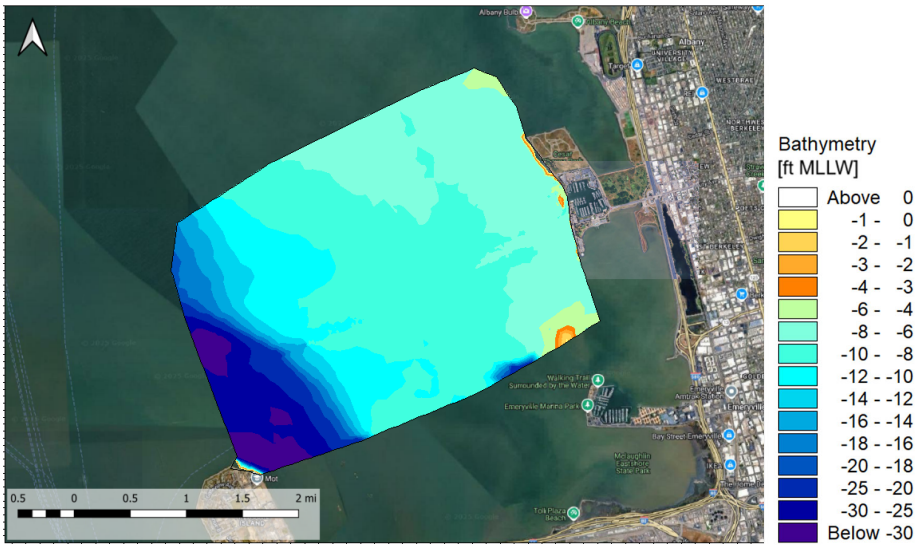


Figure 5-2 Model bathymetry used for existing seabed scenario.

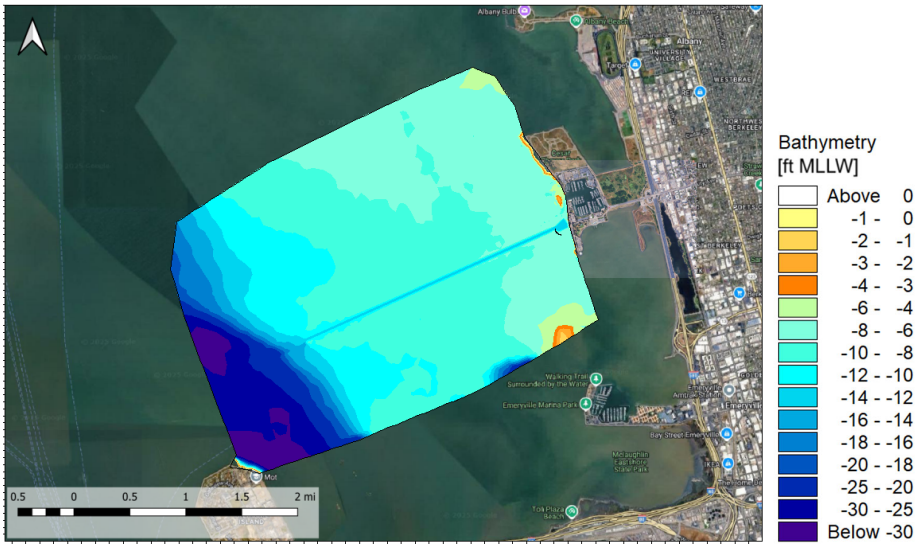


Figure 5-3 *Model bathymetry used for 400-ft breakwater scenario with approach channel and berth pocket dredged to -12 ft MLLW.*

5.4 Model results

The coupled model was simulated for the two scenarios described in Section 5.2. The model parameters were adjusted such that the sedimentation in the existing Berkely Marina Channel was 0.2 ft for a 3-year period. Subsequently, the same parameters were applied to simulate the scenario with 400-ft breakwater and deepened approach channel and berth pockets.

Both scenarios were simulated for a 6-year period to obtain net deposition patterns and sedimentation assessment within different areas.

Figure 5-4 and Figure 5-5 illustrate the net deposition patterns for the two scenarios simulated after 6 years of simulation. Figure 5-4 presents the results for the existing scenario. Similarly, Figure 5-5 shows the estimated sedimentation pattern for the scenario with 400-ft breakwater and deepened approach channel and berth pockets.

Table 5-1 provides the net sediment deposition estimated within the different areas for the two scenarios simulated.

Figure 5-6 shows the extent of the areas provided in Table 5-1.



Figure 5-4 *Net deposition map for existing seabed scenario using results from 6 years of simulation. Berths and dredge envelope shown for illustration purpose only.*



Figure 5-5 *Net deposition map for 400-ft breakwater scenario using results from 6 years of simulation with approach channel and berth pocket dredged to -12 ft MLLW.*

Table 5-1 Summary of mean sedimentation rates using results from 6 years of simulation.

Area	Mean sedimentation in existing seabed scenario (ft/yr)	Mean sedimentation with 400-ft breakwater, approach channel and berths dredged to -12 ft MLLW scenario (ft/yr)
Berkeley Marina Channel	0.05	0.10
Berth 1	0.07	0.04
Berth 2	0.04	0.08
Berth Pocket	0.05	0.09
Channel to -12 ft MLLW	0.1	0.20

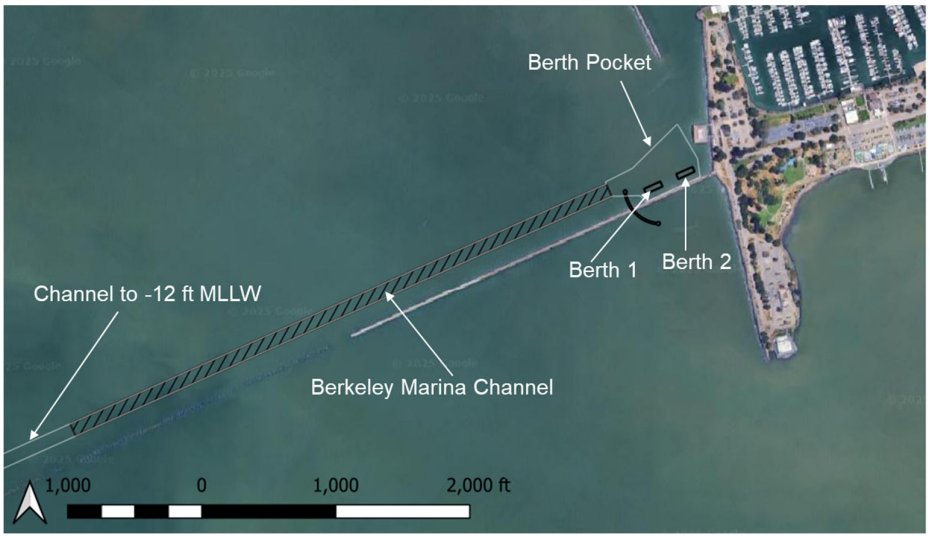


Figure 5-6 Areas for net deposition rates calculation.

Table 5-2 presents the simulated sedimentation rates in each year of the BWTPF project over the six-year modeled period. The table shows results for the scenario after dredging within the overall dredged areas (berth and channel) and separated berth pocket and channel. The results show that overall sedimentation after dredging across the full dredged area averages 0.15 ft/yr, with the highest accumulation occurring in the first year (0.32 ft). Within the berth pockets, sedimentation is relatively low and stable, averaging 0.09 ft, while the channel areas (excluding berths) experience higher and more variable deposition, with an average of 0.20 ft and a peak of 0.48 ft in the first year.

These results indicate that the channel is more prone to sediment accumulation than the berth pockets, and that most of the infilling occurs during the early years following dredging, after which deposition rates decline and stabilize. The relatively low deposition within the berth pockets reflects the sheltering effect of the breakwater, which reduces wave-driven sediment transport into these more protected areas and helps limit infilling compared to the more exposed channel.

Table 5-2 Yearly net sediment deposition per dredged area

Year	Overall sedimentation (ft) – full dredged area	Sedimentation in berth pocket (ft)	Sedimentation in channel (ft) Not including berth
1	0.32	0.10	0.48
2	0.18	0.10	0.24
3	0.12	0.09	0.14
4	0.11	0.09	0.12
5	0.10	0.08	0.11
6	0.09	0.08	0.11
Annual Average	0.15	0.09	0.20

5.5 Maintenance dredging

Table 5-3 provides an estimate of the maintenance dredging frequency required for the BWTPF project, using the average annual sedimentation rates obtained from the six-year simulations. The results are based on an average of 0.20 ft/year of infilling in the channel and 0.09 ft/year in the berth pockets. Two potential operational depths (–10 ft and –9 ft) were assessed to determine the approximate time until dredging would be needed, recognizing that the current dredged depth is –12 ft.

The analysis indicates that the channel would require dredging approximately every 10 years if maintained at –10 ft, extending to 15 years at –9 ft. In comparison, the berth pockets accumulate sediment at a slower rate due to the sheltering provided by the breakwater, leading to longer dredging intervals of 22 and 33 years for the same operational depths.

Overall, the findings suggest that dredging frequency is strongly dependent on the selected operational depth. A deeper target elevation would ensure improved navigational reliability but comes with shorter dredging intervals, particularly in the channel. Operating at a shallower depth reduces the dredging frequency significantly but may limit navigational access for higher ferry drafts. Balancing these trade-offs is critical for optimizing long-term maintenance costs while maintaining the required depth for safe operations.

Table 5-3 Estimated dredging intervals (years) for channel and berth pockets

Operational Depth (ft)	Est. Years until Maintenance Dredging	
	Channel	Berth Pocket
-10	10	22
-9	15	33

5.6 Post-dredging sedimentation assessment at Berkeley Marina

Dredging operations at the Berkeley Marina were completed in late 2024. A post-dredge bathymetric survey was subsequently conducted in December 2024 to establish baseline conditions. A follow-up survey was carried out in June 2025, enabling a comparative analysis of the two datasets to evaluate sedimentation rates during the initial six-month period following dredging.

Although the target dredging depth was -8 ft MLLW, the spatial proximity of the dredged area to the BWTTP channel allows for a qualitative assessment of sediment dynamics in relation to the channel's influence. Figure 5-7 highlights, within the red rectangle, the specific region where the two consecutive surveys were analysed. This comparative analysis provides insight into early-stage infill patterns and sediment redistribution processes post-dredging.



Figure 5-7 Berkeley Marina dredged area (surveys 2024 and 2025)

Figure 5-8 shows a differential analysis of bathymetric data collected over a six-month interval, comparing the December 2024 post-dredge survey with the June 2025 follow-up. To increase interpretability, the colour scale was selectively muted for regions exhibiting minimal sedimentation or erosion, improving visual contrast in areas of significant morphological change. The results indicate a mean sediment accumulation of approximately 0.3 ft within the delineated area of interest. This observed change in seabed elevation provides a preliminary estimate of the sedimentation rate during the initial post-dredging period.

A comparative assessment of sedimentation rates between the Berkeley Marina and the BWTPF dredge channel reveals consistent patterns in post-dredging infill dynamics. The BWTPF channel, unaffected by breakwater influences and thus directly comparable to the dredged section at Berkeley Marina, exhibited an average modeled sedimentation rate of 0.48 ft during the first year following dredging. It is well established that sedimentation rates are typically higher in the initial months post-dredging, gradually decreasing over time as the system approaches equilibrium. In this context, the observed mean sediment accumulation of 0.3 ft within the Berkeley Marina’s area of interest during

the first six months aligns closely with model predictions for the BWTPF channel. This correspondence supports the validity of the sedimentation model and underscores the similar depositional processes governing both sites during the early post-dredging phase.

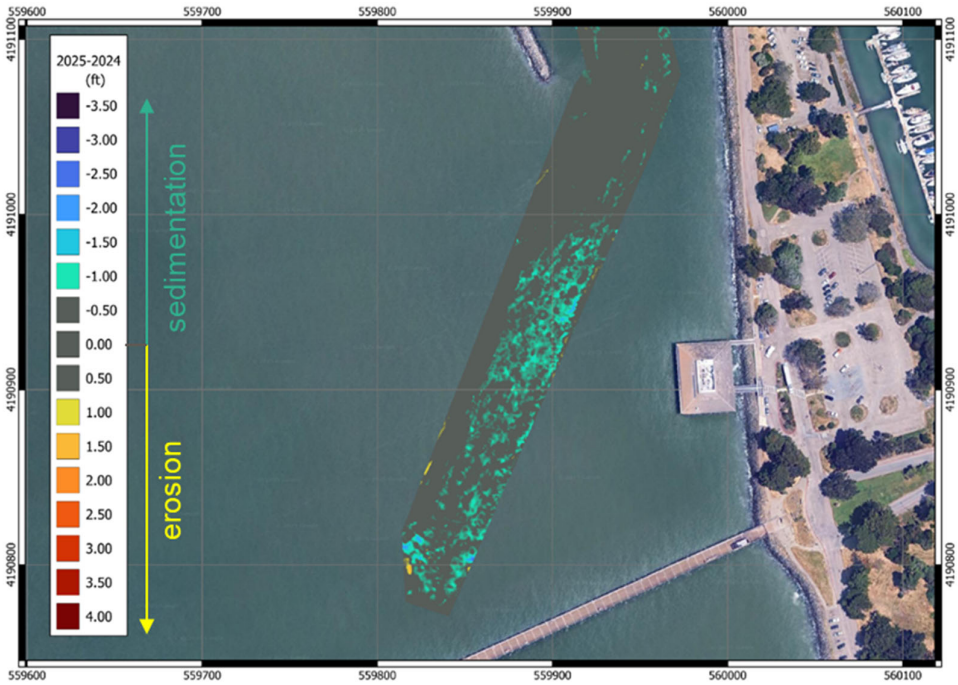


Figure 5-8 Bathymetric change over six months post dredge

5.7 Conclusion

The model results presented in this report are recommended to be used only within the project footprint, i.e., the areas where bathymetry changed or deepened, due to reasons as discussed in this section.

The survey data used to derive existing mean sedimentation rates near the project site are only available along the narrow segment called “Berkely Marina Channel” (see Figure 5-6). Hence any model sedimentation away from the channel has no survey data available for comparison.

Moreover, no site-specific suspended sediment concentration data, i.e., the sediment dispersed in water column, is available. This parameter is one of the main inputs, which determines the amount of sediment available or susceptible to settling in the seabed. Due to unavailability of site-specific data, a value derived from available literature, such as USGS reports with suspended sediment concentration data in San Francisco Bay (Ref (Buchanan & Ganju, 2004) and Ref (Buchanan & Morgan, 2014)), was used. However, suspended sediment concentration could be much lower in the project site due to very low sedimentation rates experienced in the existing Berkeley Marina Channel (0.07 ft / year average, see Section 2.6).

It should also be noted that the sedimentation experienced in the existing Berkeley Marina Channel, although generally low, is varying between 0.015 and 0.09 ft / year, as shown in Section 2.6. USGS report on sedimentation in San Francisco Bay (Ref (Fregoso, Foxgiver, & Jaffe, 2024)) also indicates that some segments along the channel deepening footprint have experienced sedimentation rates of

0.03-0.1 ft/year historically. Although this range encompasses the sedimentation rate selected to validate the model in this study, the USGS report also shows that there are segments where sedimentation is almost zero. In this study, a value slightly higher than median, i.e., 0.06-0.07 ft/year was considered mainly because very low or negligible sedimentation rates (as seen in USGS report) are hardly achievable in a numerical modeling exercise for such large area.

Comparison with the 2024/2025 Berkeley Marina Channel survey further supports this conclusion, as the measured sedimentation rates fall within the same order of magnitude as the modeled results for the first-year post-dredge. This alignment between recent survey data and the model increases confidence in the localized predictions within the project footprint, while also emphasizing the need for continued monitoring to refine site-specific assumptions. It is therefore recommended that the sedimentation study and methodology be revisited as more field data, particularly additional bathymetric surveys and suspended sediment concentration measurements, become available.

The sediment transport analysis was conducted in parallel with the breakwater length optimization study, based on a layout incorporating a 400 ft breakwater. A subsequent reduction of the breakwater length by approximately 100 ft is not anticipated to cause any significant variation in sediment transport rates within the navigation channel. In addition, the simulated sedimentation rates at the berth and turning basin are already minimal. Therefore, the overall results remain representative for the 300 ft breakwater configuration and provide a reasonable approximation of sediment transport patterns, considering the inherent limitations of the model and input data.

6 References

Buchanan, P., & Ganju, N. (2004). *Summary of Suspended-Sediment Concentration Data, San Francisco Bay, California, Water Year 2002: U.S. Geological Survey Open-File Report 2004-1219.*

Buchanan, P., & Morgan, T. (2014). *Summary of Suspended-Sediment Concentration Data, San Francisco Bay, California, Water Year 2010, U.S. Geological Survey Data Series 808.*

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DHI. (2023). *Hydrodynamic Module User Guide.*

Egbert, G. D., & Erofeeva, S. Y. (2002). Efficient inverse modeling of barotropic ocean tides. *Journal of Atmospheric and Oceanic Technology, 19.2*, 183-204.

ENGEO Incorporated. (2024). *Geotechnical Report.*

Fregoso, T. A., Foxgiver, A. C., & Jaffe, B. E. (2024). *Sediment Deposition, Erosion, and Bathymetric Change in San Francisco Bay, California, 1971-1990 and 1999-2020, U.S. Geological Survey.*

APPENDIX E: MISCELLANEOUS MEMORANDUMS

A number of side studies were developed to validate the design parameters included in this design document:

- Gangway Length Slope Study
- Shoreline Protection Review

DRAFT

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MEMO

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TITLE Berkeley Pier Determination of Gangway Length
DATE 06-01-2025
TO James Connolly, PM

COPY Jessica Rivas, DPM
Jacob Shaw, Structural Project Engineer

FROM Wen Huai Tsao
PROJECT NO A272767 Concept

1 Purpose

The goal of this memo is to provide enough information to help stakeholders make an informed decision regarding the Ferry Landing gangway length at the Berkeley Pier.

The gangway is to comply with both the ADA requirements[1] and the "Proposed Passenger Vessels Accessibility Guidelines"[2] in terms of maximum gangway slope. To this end, the slope of gangways 80, 90, 100, 110, and 120 feet long are studied to determine the number of hours per year where each gangway slope would exceed the maximum slope provided by the above guidelines. Moreover, the gangway length that would be required to reach the Lowest Observed Water Elevation without exceeding the maximum design slope is provided.

2 Accessible Route Requirements

According to Section 4.8. (Ramps) of the ADA Requirements, the maximum slope of a ramp in new construction shall be 1:12, see Appendix A.

According to Section 4.10 (V410 Gangways) of the Proposed Passenger Vessels Accessibility Guidelines, gangway runs shall have a running slope not steeper than 1:12, see Appendix B. The guidelines do not provide input on the range of tides to be considered.

[1] http://www.ada-compliance.com/ada-compliance/ada-ramp
[2] https://www.access-board.gov/guidelines-and-standards/transportation/passenger-vessels/proposed-accessibility-guidelines/1386-chapter-v4-onboard-accessible-routes-and-accessible-passenger-boarding-systems

However, codes note that if a gangway is at least 120 ft, gangways shall not be required to comply with V410.2

3 Tide Datum and Water Levels

The tide datums at the Berkeley Pier are as follows, for present day conditions:

Datum	Value [ft]	Value to NAVD88 [ft]	Description
HAT	7.65	7.08	Highest Astronomical Tide
MHHW	6.10	5.53	Mean Higher-High Water
MHW	5.49	4.92	Mean High Water
MSL	3.28	2.71	Mean Sea Level
NAVD88	0.57	0.00	North American Vertical Datum of 1988
MLLW	0.00	-0.57	Mean Lower-Low Water
LAT	-2.25	-2.82	Lowest Astronomical Tide

Table 3-1 Tidal reference levels at Berkeley Marina.

As there is no water level time history available at the Berkeley Pier, we relied on data from nearby NOAA stations: Alameda (9414750), Richmond (9414863), and San Francisco (9414290) as shown in Figure 3-1. The distances to the Alameda (9414750) and Richmond (9414863) stations are nearly identical. However, considering that the Alameda station is more sheltered, we chose to use water level data from Richmond (9414863), as it has similar exposure and tidal characteristics to the Berkeley Pier.

The observation period spans from January 2014 to December 2023, with the terminal assumed to operate 365 days a year from 6 AM to 10 PM. Accordingly, only data within this timeframe is analyzed. The dataset, recorded at 6-minute intervals, represents water levels in feet relative to the NAVD88 datum. The Highest Observed Water Level (HOWL) is 8.127 ft (recorded at 6:12 p.m. on January 5, 2023), and the Lowest Observed Water Level (LOWL) is -1.86 ft (recorded at 3:54 p.m. on May 18, 2022).

The analysis for this gangway calculation was conducted using the NAVD88 vertical datum, which is 0.57 ft lower than the Berkeley MLLW datum. This difference does not impact the gangway length design, as both the waterline and the pier elevations are referenced to the same vertical datum, ensuring consistency in the calculations.

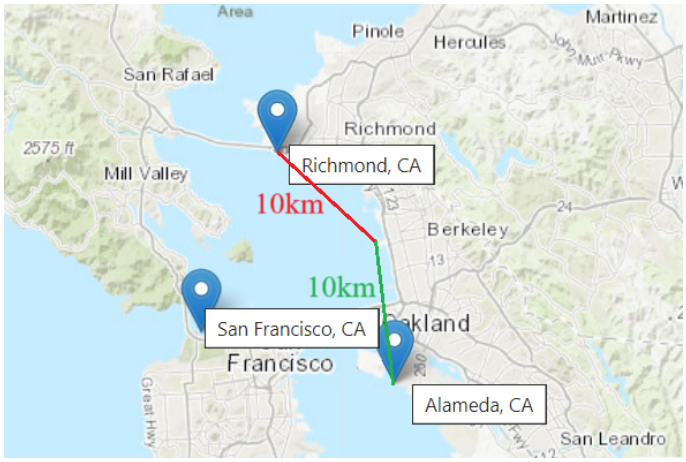


Figure 3-1 *Water level data available from NOAA stations near Berkeley Pier includes Alameda 9414750, Richmond 9414863, and San Francisco 9414290.*

4 Pier and Float Elevations

This section provides the slope of a 100-foot-long gangway at Berkeley Pier for various water elevations. The gangway connects a fixed pier (El. +17.0 feet-NAVD88) with a landing platform (+8.63 feet above the water line) on the float. Figure 4-1 illustrates the gangway configuration at the highest tide (El. +7.52 feet-NAVD88) and the lowest tide (El. -2.38 feet-NAVD88).

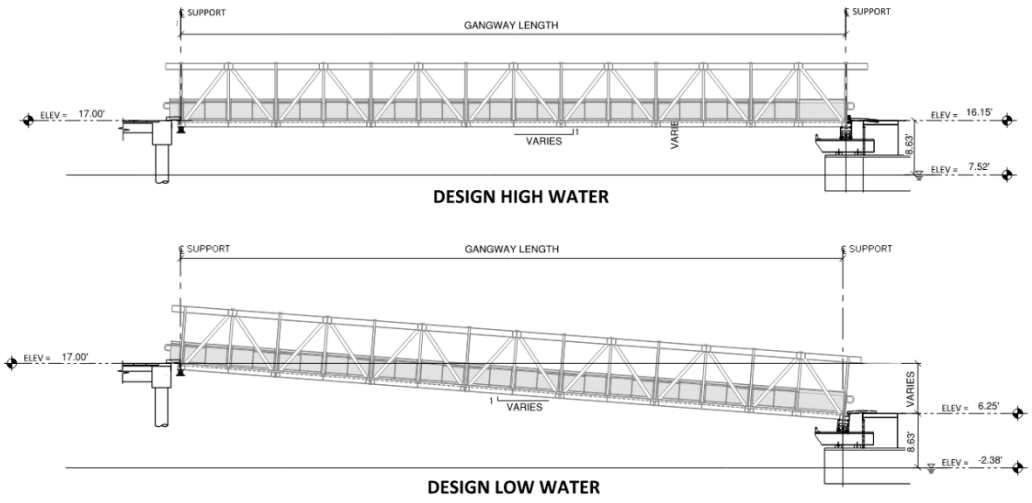


Figure 4-1 *The gangway connects a fixed pier (El. +17.0 feet-NAVD88) with a landing platform (+8.63 feet above the water line) on the float at the highest tide (El. +7.52 feet-NAVD88) and the lowest tide (El. -2.38 feet-NAVD88).*

5 Gangway 80-feet Long

5.1 Slope at Design Water Elevations

Typical design water elevations are LAT, MLLW, MHHW, and HAT.

Water Elevation	Gangway Slope	Maximum Design Slope	Design Requirement Compliant (ADA)	Accessible Board Guidelines Compliant
LAT	1:7.4, downwards	1:12	Non-Compliant	Non-Compliant
MLLW	1:9.4, downwards	1:12	Non-Compliant	Non-Compliant
MHHW	1:33.3, downwards	1:12	Compliant	Compliant
HAT	1:94.1	1:12	Compliant	Compliant

Table 5-1 *80-foot-long gangway slope for design water elevations.*

5.2 Slope at Extreme Water Elevations Based on Richmond Data

The 80-foot-long gangway slope has been checked for the HOWL and for the LOWL, see Table 5-2.

Water Elevation	Gangway Slope	Maximum Design Slope	Design Requirement Compliant (ADA)	Accessible Board Guidelines Compliant
LOWL	1:7.8, downwards	1:12	Non-Compliant	Non-Compliant
HOWL	1:329, downwards	1:12	Compliant	Compliant

Table 5-2 80-foot-long gangway slope for extreme water elevations.

5.3 Design Slope Exceedance Frequency based on Richmond Data

The 80-foot gangway will exceed the 1:12 requirement when the water elevation drops below the threshold +1.73 feet-NAVD88. Figure 5-1 shows the instances where the water level drops below the water elevation limit within the observation period. On average each year, there were 303 days when the 1:12 slope requirement was not met, resulting in a total failure time of 1,069 hours. The gangway will exceed a 1:12 slope for 18.3% of the terminal's operational time.

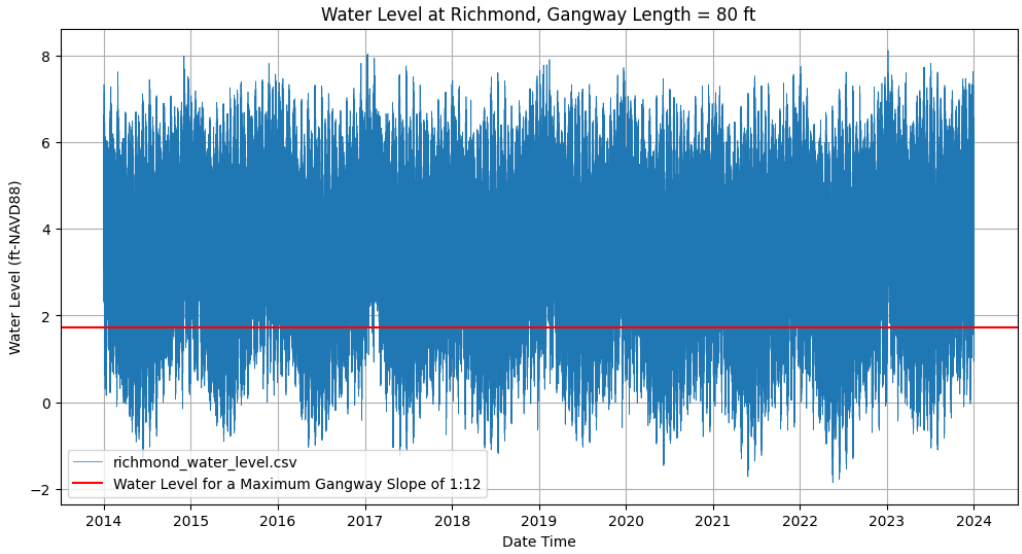


Figure 5-1 Water levels at Richmond 9414863 station and minimum water level required for an 80-foot-long gangway to not exceed the design slope of 1:12.

6 Gangway 90-feet Long

6.1 Slope at Design Water Elevations

Typical design water elevations are LAT, MLLW, MHHW, and HAT.

Water Elevation	Gangway Slope	Maximum Design Slope	Design Requirement Compliant (ADA)	Accessible Board Guidelines Compliant
LAT	1:8.3, downwards	1:12	Non-Compliant	Non-Compliant
MLLW	1:10.5, downwards	1:12	Non-Compliant	Non-Compliant
MHHW	1:37.5, downwards	1:12	Compliant	Compliant
HAT	1:105.9	1:12	Compliant	Compliant

Table 6-1 90-foot-long gangway slope for design water elevations.

6.2 Slope at Extreme Water Elevations Based on Richmond Data

The 90-foot-long gangway slope has been checked for the HOWL and for the LOWL, see Table 6-2.

Water Elevation	Gangway Slope	Maximum Design Slope	Design Requirement Compliant (ADA)	Accessible Board Guidelines Compliant
LOWL	1:8.7, downwards	1:12	Non-Compliant	Non-Compliant
HOWL	1:370, downwards	1:12	Compliant	Compliant

Table 6-2 90-foot-long gangway slope for extreme water elevations.

6.3 Design Slope Exceedance Frequency based on Richmond Data

The 90-foot gangway will exceed the 1:12 requirement when the water elevation drops below the threshold +0.90 feet-NAVD88. Figure 6-1 shows the instances where the water level drops below the water elevation limit within the observation period. On average each year, there were 203 days when the 1:12 slope requirement was not met, resulting in a total failure time of 538 hours. The gangway will exceed a 1:12 slope for 9.2% of the terminal's operational time.

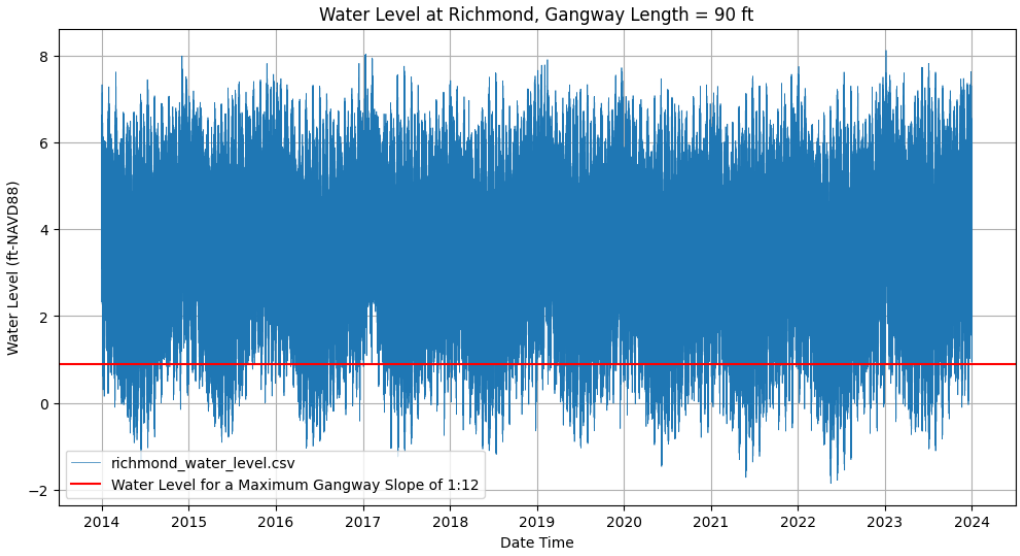


Figure 6-1 *Water levels at Richmond 9414863 station and minimum water level required for a 90-foot-long gangway to not exceed the design slope of 1:12*

7 Gangway 100-feet Long

7.1 Slope at Design Water Elevations

Typical design water elevations are LAT, MLLW, MHHW, and HAT.

Water Elevation	Gangway Slope	Maximum Design Slope	Design Requirement Compliant (ADA)	Accessible Board Guidelines Compliant
LAT	1:9.3, downwards	1:12	Non-Compliant	Non-Compliant
MLLW	1:11.7, downwards	1:12	Non-Compliant	Non-Compliant
MHHW	1:41.7, downwards	1:12	Compliant	Compliant
HAT	1:117.6	1:12	Compliant	Compliant

Table 7-1 *80-foot-long gangway slope for design water elevations.*

7.2 Slope at Extreme Water Elevations Based on Richmond Data

The 100-foot-long gangway slope has been checked for the HOWL and for the LOWL, see Table 7-2.

Water Elevation	Gangway Slope	Maximum Design Slope	Design Requirement Compliant (ADA)	Accessible Board Guidelines Compliant
LOWL	1:9.7, downwards	1:12	Non-Compliant	Non-Compliant
HOWL	1:411, downwards	1:12	Compliant	Compliant

Table 7-2 100-foot-long gangway slope for extreme water elevations.

7.3 Design Slope Exceedance Frequency based on Richmond Data

The 100-foot gangway will exceed the 1:12 requirement when the water elevation drops below the threshold +0.07 feet-NAVD88. Figure 7-1 shows the instances where the water level drops below the water elevation limit within the observation period. On average each year, there were 84 days when the 1:12 slope requirement was not met, resulting in a total failure time of 169 hours. The gangway will exceed a 1:12 slope for 2.9% of the terminal's operational time.

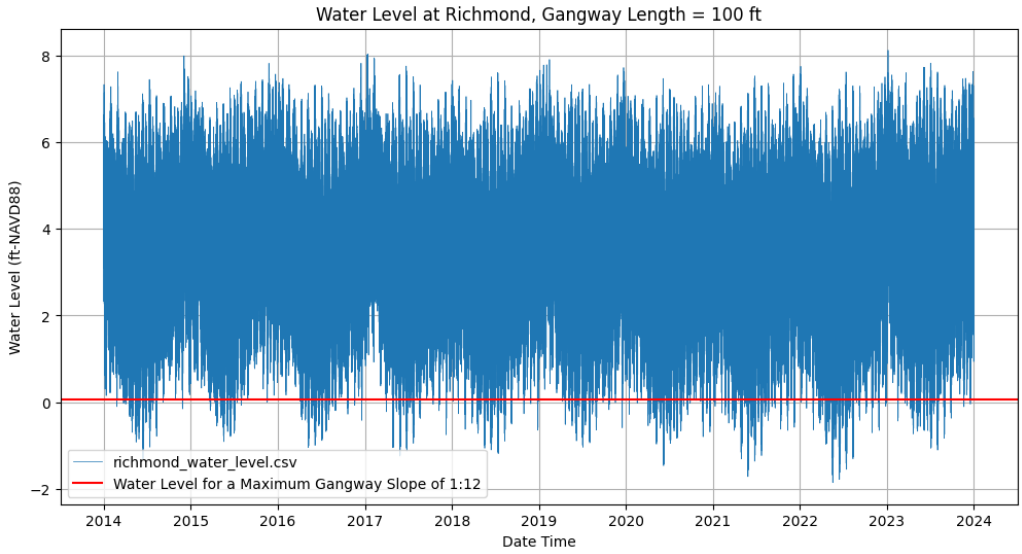


Figure 7-1 Water levels at Richmond 9414863 station and minimum water level required for a 100-foot-long gangway to not exceed the design slope of 1:12.

8 Gangway 110-feet Long

8.1 Slope at Design Water Elevations

Typical design water elevations are LAT, MLLW, MHHW, and HAT.

Water Elevation	Gangway Slope	Maximum Design Slope	Design Requirement Compliant (ADA)	Accessible Board Guidelines Compliant
LAT	1:10.2, downwards	1:12	Non-Compliant	Non-Compliant
MLLW	1:12.9, downwards	1:12	Compliant	Compliant
MHHW	1:45.8, downwards	1:12	Compliant	Compliant
HAT	1:129.4	1:12	Compliant	Compliant

Table 8-1 110-foot-long gangway slope for design water elevations.

8.2 Slope at Extreme Water Elevations Based on Richmond Data

The 110-foot-long gangway slope has been checked for the HOWL and for the LOWL, see Table 8-2.

Water Elevation	Gangway Slope	Maximum Design Slope	Design Requirement Compliant (ADA)	Accessible Board Guidelines Compliant
LOWL	1:10.7, downwards	1:12	Non-Compliant	Non-Compliant
HOWL	1:452, downwards	1:12	Compliant	Compliant

Table 8-2 110-foot-long gangway slope for extreme water elevations.

8.3 Design Slope Exceedance Frequency based on Richmond Data

The 110-foot gangway will exceed the 1:12 requirement when the water elevation drops below the threshold of -0.77 feet-NAVD88. Figure 8-1 shows the instances where the water level drops below the water elevation limit within the observation period. On average each year, there were 14 days when the 1:12 slope requirement was not met, resulting in a total failure time of 21 hours. The gangway will exceed a 1:12 slope for 0.37 % of the terminal's operational time.

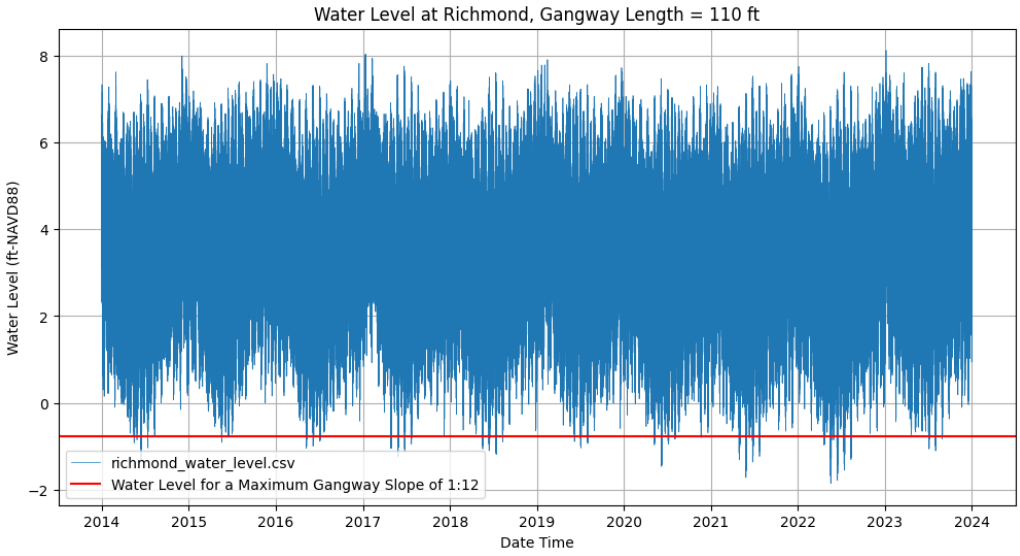


Figure 8-1 *Water levels at Richmond 9414863 station and minimum water level required for a 110-foot-long gangway to not exceed the design slope of 1:12.*

9 Gangway 120-feet Long

9.1 Slope at Design Water Elevations

Typical design water elevations are LAT, MLLW, MHHW, and HAT.

Water Elevation	Gangway Slope	Maximum Design Slope	Design Requirement Compliant (ADA)	Accessible Board Guidelines Compliant
LAT	1:11.1, downwards	1:12	Non-Compliant	Non-Compliant
MLLW	1:14.1, downwards	1:12	Compliant	Compliant
MHHW	1:50.0, downwards	1:12	Compliant	Compliant
HAT	1:141.2, downwards	1:12	Compliant	Compliant

Table 9-1 *120-foot-long gangway slope for design water elevations.*

9.2 Slope at Extreme Water Elevations Based on Richmond Data

The 120-foot-long gangway slope has been checked for the HOWL and for the LOWL, see Table 9-2.

Water Elevation	Gangway Slope	Maximum Design Slope	Design Requirement Compliant (ADA)	Accessible Board Guidelines Compliant
LOWL	1:11.7, downwards	1:12	Non-Compliant	Non-Compliant
HOWL	1:494, downwards	1:12	Compliant	Compliant

Table 9-2 120-foot-long gangway slope for extreme water elevations.

9.3 Design Slope Exceedance Frequency based on Richmond Data

The 120-foot gangway will exceed the 1:12 requirement when the water elevation drops below the threshold -1.60 feet-NAVD88. Figure 9-1 shows the instances where the water level drops below the water elevation limit within the observation period. On average each year, there were less than 1 day when the 1:12 slope requirement was not met, resulting in a total failure time of 0.7 hours. The gangway will exceed a 1:12 slope for 0.01% of the terminal's operational time.

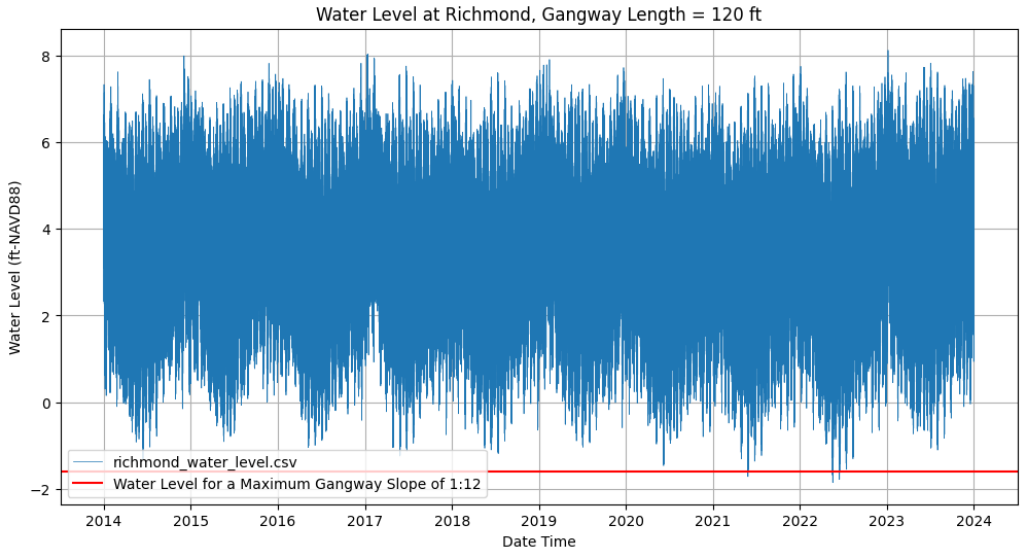


Figure 9-1 Water levels at Richmond 9414863 station and minimum water level required for a 120-foot-long gangway to not exceed the design slope of 1:12.

10 Summary

Table 10-1 summarizes the performance of gangways with varying lengths under different tidal conditions.

Gangway Length (ft)	Gangway Slope at LOWL (Richmond Station)	Days out of compliance (per year)	Hours* out of compliance (per year)	Exceedance frequency (%)	Gangway Slope at LAT
80	1:7.8	303	1,069	18.3	1:7.4
90	1:8.7	203	538	9.2	1:8.3
100	1:9.7	84	169	2.9	1:9.3
110	1:10.7	14	21	0.37	1:10.2
120	1:11.7	<1	0.7	0.01	1:11.1

Table 10-1 *Summary of gangway performance metrics, including slope, compliance duration, and exceedance frequency, for various gangway lengths. *The number of hours between 6AM and 10PM in a year.*

The minimum gangway length required to meet the 1:12 slope requirement at the LAT (-2.31 feet NAVD88) is 129.1 feet, which aligns with Section 3.2.3.3 of the GHD feasibility report [1].

Additional considerations are:

- As the sea level rises, the frequency of exceedance during extreme low tides will be less over time.
- The observation period spans from January 2014 to December 2023, with the terminal assumed to operate 365 days a year from 6 AM to 10 PM.
- At Alameda Sea Plane Lagoon signage is provided indicating that crew members are available to assist passengers when the ramp slope exceeds 1:12 slope. See Figure 10-1. The length of the current gangway is 91.5 ft, resulting in 1 hour out of compliance in a year (from 6AM to 10PM). The details can be referred to Appendix C.



Figure 10-1 Ramp slope notice sign.

11 Reference

[1] GHD. Feasibility Study: Ferry Facility at Berkeley Municipal Pier. June 2023

Appendix A

ADA Ramp Requirements

ADA RAMP



ADA Ramp

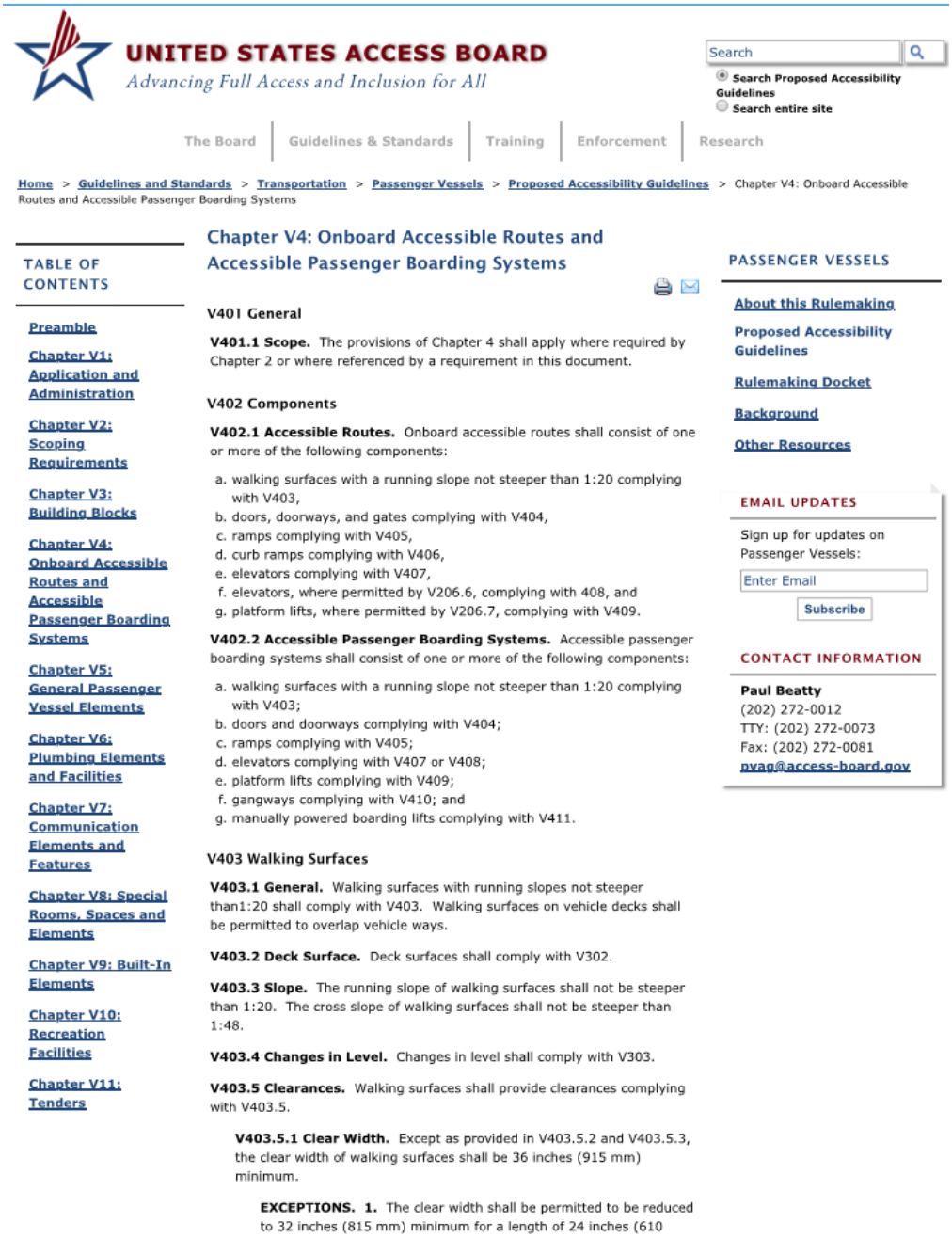
4.8 Ramps

4.8.1* General
Any part of an accessible route with a slope greater than 1:20 shall be considered a ramp and shall comply with 4.8.

4.8.2* Slope and Rise
The least possible slope shall be used for any ramp. The maximum slope of a ramp in new construction shall be 1:12. The maximum rise for any run shall be 30 in (760 mm). Curb ramps and ramps to be constructed on existing sites or in existing buildings or facilities may have slopes and rises as allowed in 4.1.6(3)(a) if space limitations prohibit the use of a 1:12 slope or less.

Appendix B

U.S. Access Board Recommendations on Accessible Routes



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- Chapter V9: Built-In Elements
- Chapter V10: Recreation Facilities
- Chapter V11: Tenders

Chapter V4: Onboard Accessible Routes and Accessible Passenger Boarding Systems

V401 General

V401.1 Scope. The provisions of Chapter 4 shall apply where required by Chapter 2 or where referenced by a requirement in this document.

V402 Components

V402.1 Accessible Routes. Onboard accessible routes shall consist of one or more of the following components:

a. walking surfaces with a running slope not steeper than 1:20 complying with V403,
b. doors, doorways, and gates complying with V404,
c. ramps complying with V405,
d. curb ramps complying with V406,
e. elevators complying with V407,
f. elevators, where permitted by V206.6, complying with 408, and
g. platform lifts, where permitted by V206.7, complying with V409.

V402.2 Accessible Passenger Boarding Systems. Accessible passenger boarding systems shall consist of one or more of the following components:

a. walking surfaces with a running slope not steeper than 1:20 complying with V403;
b. doors and doorways complying with V404;
c. ramps complying with V405;
d. elevators complying with V407 or V408;
e. platform lifts complying with V409;
f. gangways complying with V410; and
g. manually powered boarding lifts complying with V411.

V403 Walking Surfaces

V403.1 General. Walking surfaces with running slopes not steeper than1:20 shall comply with V403. Walking surfaces on vehicle decks shall be permitted to overlap vehicle ways.

V403.2 Deck Surface. Deck surfaces shall comply with V302.

V403.3 Slope. The running slope of walking surfaces shall not be steeper than 1:20. The cross slope of walking surfaces shall not be steeper than 1:48.

V403.4 Changes in Level. Changes in level shall comply with V303.

V403.5 Clearances. Walking surfaces shall provide clearances complying with V403.5.

V403.5.1 Clear Width. Except as provided in V403.5.2 and V403.5.3, the clear width of walking surfaces shall be 36 inches (915 mm) minimum.

EXCEPTIONS. 1. The clear width shall be permitted to be reduced to 32 inches (815 mm) minimum for a length of 24 inches (610

PASSENGER VESSELS

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pvag@access-board.gov

V410.1 General. Gangways that are part of accessible passenger boarding systems shall comply with V410.

V410.2 Slope. Gangway runs shall have a running slope not steeper than 1:12.

EXCEPTIONS. 1. Where gangways have only one run and the gangways do not exceed a rise of 6 inches (150 mm), the gangways shall be permitted to have running slopes in accordance with Table V410.2.

2. Where an existing gangway run or series of gangway runs is replaced or altered, an increase in the length of the gangway run shall not be required to comply with V410.2.

3. On vessel carried gangways, where the total length of the gangway run or series of runs is at least as long as the beam of the vessel, gangways shall not be required to comply with V410.2.

4. On pier provided gangways, where the total length of a gangway run or series of runs is at least 120 feet (37 m), gangways shall not be required to comply with V410.2.

5. On passenger vessels which carry vehicles, where the only way for pedestrian passengers to embark or disembark is by way of a gangway that also functions as a vehicle transfer bridge, gangways shall not be required to comply with V410.2.

Table V410.2 Alternate Running Slopes Permitted by Exception 1

Slope[1]	Maximum Rise
Steeper than 1:10 but not steeper than 1:8	3 inches (75 mm)
Steeper than 1:12 but not steeper than 1:10	6 inches (150 mm)
1. A slope steeper than 1:8 is prohibited.	

V410.3 Cross Slope. Cross slope of gangway runs shall not be steeper than 1:48.

Advisory V410.3 Cross Slope. Cross slope is the slope of the surface perpendicular to the direction of travel. It is measured the same way as slope is measured: the rise over the run.

V410.4 Surfaces. Surfaces of gangway runs shall comply with V302. Changes in level other than the running slope and cross slope are not permitted on gangway runs.

EXCEPTIONS. 1. Where gangways are deployed and conditions result in slopes greater than 1:4, changes in level complying with V303.2 or V303.3 shall be permitted on surfaces of gangway runs.

2. Changes in level shall be permitted on gangway run surfaces that extend beyond the minimum clear width specified in V410.5.

V410.5 Clear Width. The clear width of a gangway run and, where handrails are provided, the clear width between handrails shall be 36 inches (915 mm) minimum.

EXCEPTION. Where the largest deck is less than 3,000 square feet (279 m²), the width of gangway runs and the distance between handrails, if provided, shall be permitted to have clear widths of 32 inches (815 mm) minimum.

V410.6 Transition Plates. Transition plates provided at the ends of gangway runs shall contrast visually with adjacent passenger walkways either light-on-dark or dark-on-light.

V410.7 Landings. Gangways shall have landings at the top and the bottom of each gangway run. Landings shall comply with V410.7.

Appendix C

This analysis examines the conditions of the active Alameda Seaplane Ferry Terminal using the gangway's as-built dimensions, shown in Figure C-1, and tidal data from the Alameda station (9414750). Table C-1 summarizes the compliance parameters based on the observation period from January 2014 to December 2023, between 6 AM and 10 PM.

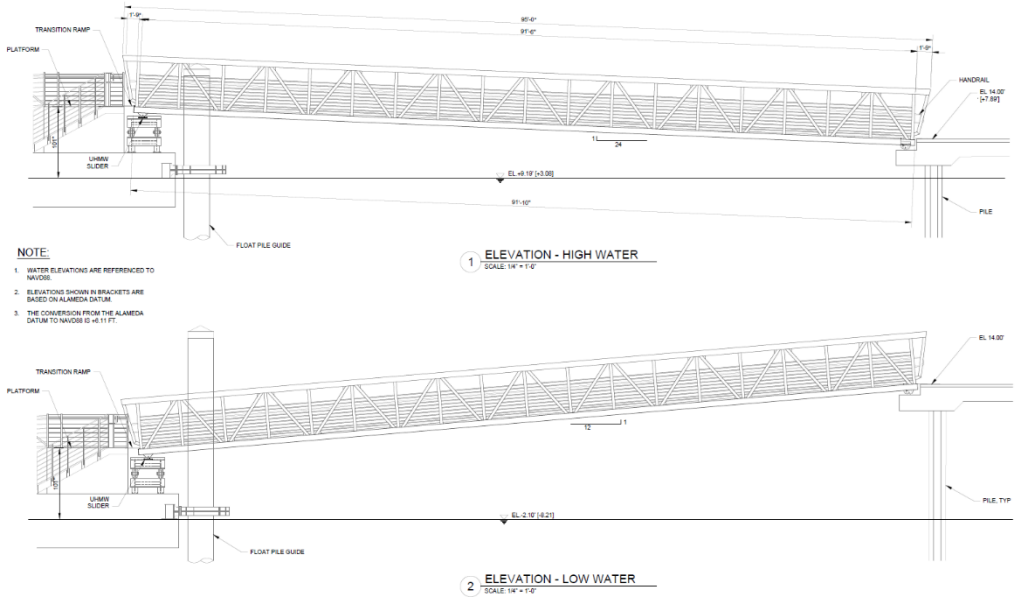


Figure C-1 *The Alameda gangway connects a fixed pier (El. +14.0 feet-NAVD88) with a landing platform (+8.42 feet above the water line) on the float at the highest tide (El. +9.19 feet-NAVD88) and the lowest tide (El. -2.10 feet-NAVD88).*

Gangway Length (ft)	Gangway Slope at LOWL (Alameda Station)	Days out of compliance (per year)	Hours* out of compliance (per year)	Exceedance frequency (%)	Gangway Slope at LAT
91.5	1:11.55	1	1	0.02	1:11.87

Tabel C-2 *Summary of the Alameda gangway performance metrics, including slope, compliance duration, and exceedance frequency, for various gangway lengths. *The number of hours between 6AM and 10PM in a year.*

DECEMBER 2025
CITY OF BERKELEY

BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT

SHORELINE PROTECTION REVIEW

COWI

ADDRESS COWI North America
555 12th Street
Suite 1700
Oakland, CA 94607

DECEMBER 2025
CITY OF BERKELEY

BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT

SHORELINE PROTECTION REVIEW

PROJECT NO.

A272767

VERSION	DATE OF ISSUE	DESCRIPTION	PREPARED	CHECKED	APPROVED
A	12 December 2025	Final	GAML / ARFG	BRLW	JRVS/JMC

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1 Introduction

The Berkeley Water Transportation Pier Ferry (BWTPF) Project is a major waterfront infrastructure initiative intended to enhance waterborne transit access along the San Francisco Bay. The project (BWTPF) aims to revitalize a significant piece of waterfront infrastructure in Berkeley, transforming the existing pier into a modern, dual-purpose ferry terminal and public pier.

This report presents the findings of the existing shoreline protection review adjacent to the Berkley Municipal Pier. The long-standing shoreline protection system of the Berkeley Marina has a history of maintenance and modification to support recreational, commercial, and transportation uses. The introduction of the BWTPF creates a need for a focused assessment of shoreline conditions to ensure that existing protection is compatible with new project demands, resilient under projected sea level rise and storm conditions, and sufficient to safeguard both public access and ferry operations.

The extent of the shoreline protection considered in this assessment spans from the Skates On the Bay restaurant, north of the pier to the south end of Seawall Drive. The shoreline under the pier is excluded from this study as it currently lacks sufficient protection and will be addressed as part of the BWTPF new shoreline plaza features.

The corresponding chainage of the existing shoreline protection as part of this study is outlined in Figure 1.

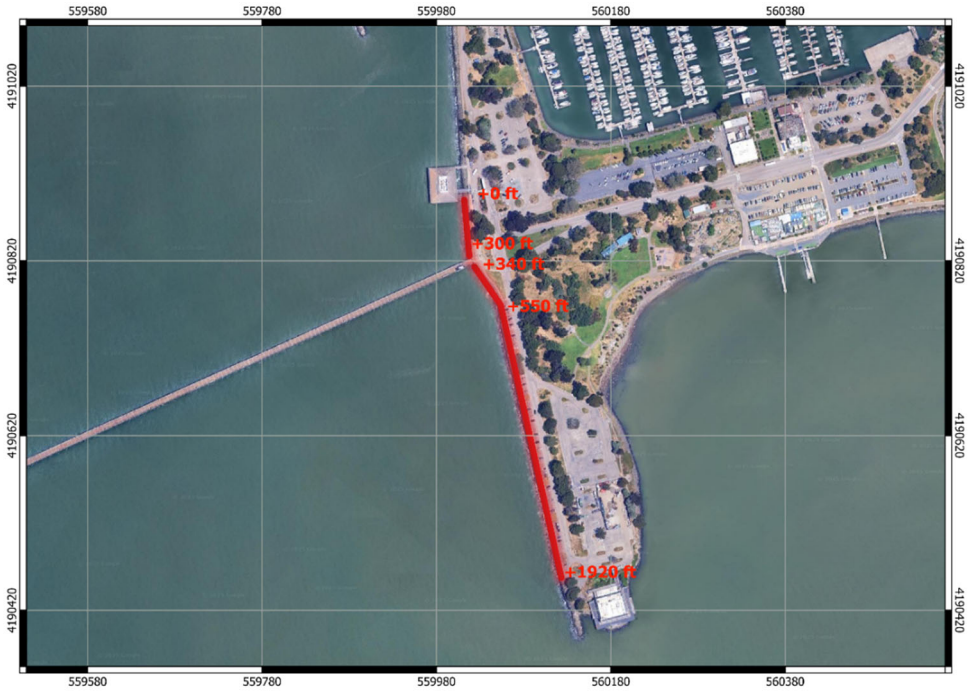


Figure 1 Shoreline chainage within scope.

The assessment consists of a combination of existing documentation, site-specific observations, previous site survey, and project design criteria. A visual field inspection was conducted in August 2025. It involved above water visual inspections, photographic documentation, sample measurements of rocks, and documentation of erosion or displacement. These findings are reviewed in the context of the BWTPF design requirements to provide a comprehensive characterization of the shoreline protection and to identify areas requiring maintenance, upgrade, or integration with the new ferry project infrastructure.

The assessment indicates that the existing rock revetment is generally stable and performing well, with localized issues such as minor crest erosion and pockets of undersized rock that should be addressed during future upgrades. Design checks confirm that the current material is adequate, aligning with Caltrans Class IV specifications and exceeding the requirements for both present and projected conditions, based on the limited measurements taken on site. Wave overtopping results show that most shoreline sections meet tolerable thresholds under current and future scenarios, but lower-crest areas (Sections 2 and 4) are projected to exceed acceptable limits with sea level rise. Targeted improvements in these sections, including raising crest elevations and correcting localized erosion, will help maintain long-term resilience and support the operational and safety needs of the BWTPF Project.

2 Existing Shoreline Protection

Sections of the shoreline protection adjacent to the Berkeley Municipal Pier were constructed as early as the 1930s with riprap revetment, primarily north of the pier. The southwest shoreline protection was likely constructed in the 1970s, along with the upgrades to the southeast corner of the Marina, using armor rock revetment. Since 1976, no major upgrades were made to the shoreline protection system in the area (Berkeley, 2019).

This section provides the baseline condition against which the recent field investigations and surveyed sections analyses are compared. Any information regarding the toe of the shoreline protection, or lack thereof, is based on bathymetric surveys only and could not be confirmed during recent field investigations from land.

2.1 Shoreline Sections

A topo-bathymetric survey from 2024 (eTrac, 2024) was used to extract sections 1 to 5, as shown in Figure 2.



Figure 2 Locations of sections from 2024 Survey.

2.1.1 Section 1

Section 1 is located north of the Berkeley Pier, close to the restaurant. Figure 3 shows the plotted section from the 2024 survey and a photo taken on August 2025. Key features include:

- Crest elevation: approximately +15 ft MLLW.
- Toe is not visible from section.
- Slope approximately 2.2H:1V.

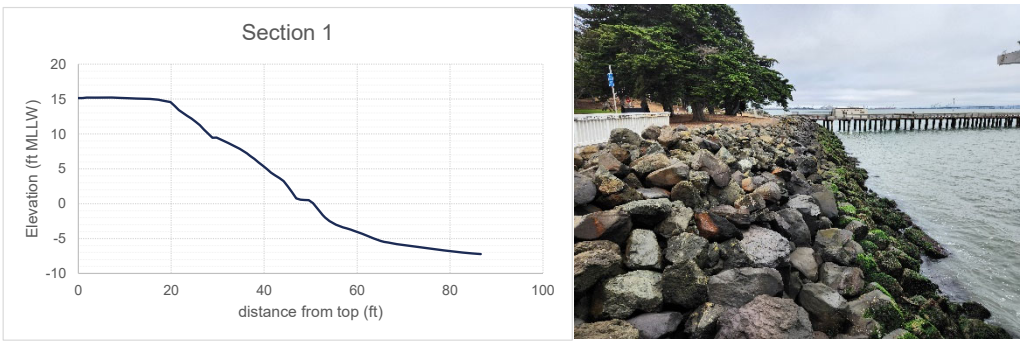


Figure 3 *Section 1 profile and site photo at approximate location*

2.1.2 Section 2

Section 2 is located immediately north of Berkeley Pier. Figure 4 shows the plotted section from the 2024 survey and a photo taken on August 2025. Key features include:

- Crest elevation: approximately +12.5 ft MLLW.
- Designed toe protection is not visible from section.
- Slope approximately 2.2H:1V.

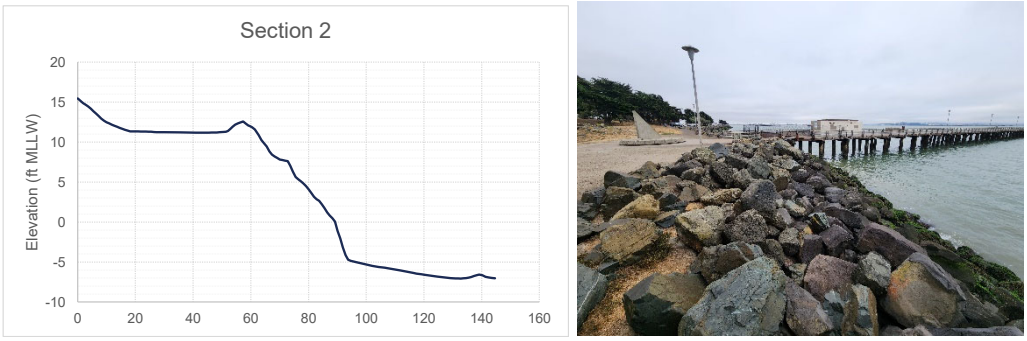


Figure 4 *Section 2 profile and site photo at approximate location.*

2.1.3 Section 3

Section 3 is located immediately south of Berkeley Pier. Figure 5 shows the plotted section from the 2024 survey and a photo taken on August, 2025. Key features include:

› Crest elevation: approximately +13.0 ft MLLW.

› Designed toe protection is not visible from section.

› Slope approximately 2.5H:1V.

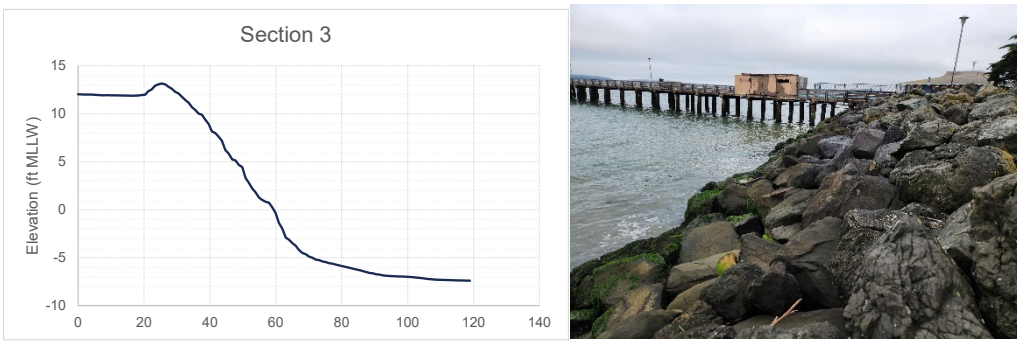


Figure 5 *Section 3 profile and site photo at approximate location.*

2.1.4 Section 4

Section 4 is located south of Berkeley Pier, slightly north of the parking lot entrance. Figure 6 shows the plotted section from the 2024 survey and a photo taken on August 2025. Key features include:

› Crest elevation: approximately +12.6 ft MLLW.

› Designed toe protection is not visible from section.

› Slope approximately 2.6H:1V.

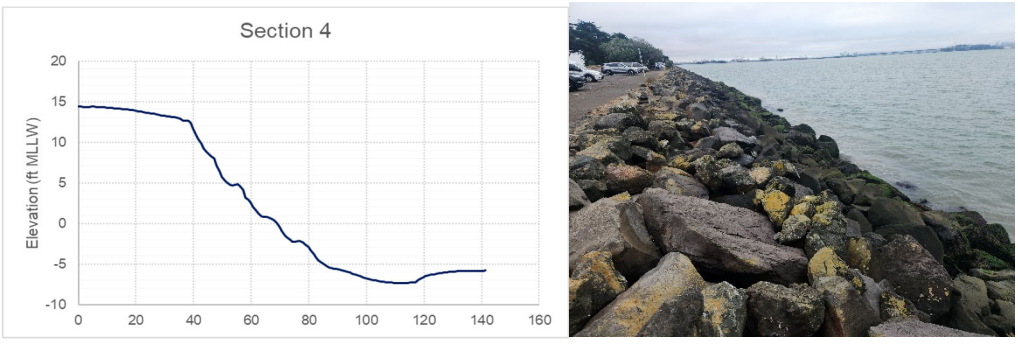


Figure 6 *Section 4 profile and site photo at approximate location.*

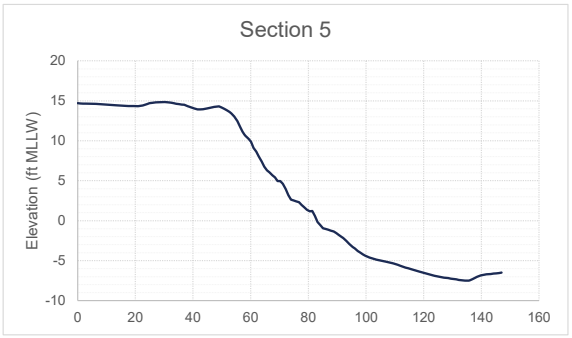
2.1.5 Section 5

Section 5 is located immediately south of Berkeley Pier, on the northern side of the parking lot. Figure 7 shows the plotted section from the 2024 survey and a photo taken on August 2025. Key features include:

› Crest elevation: approximately +12.6 ft MLLW.

› Designed toe protection is not visible from section.

› Slope approximately 2.6H:1V.



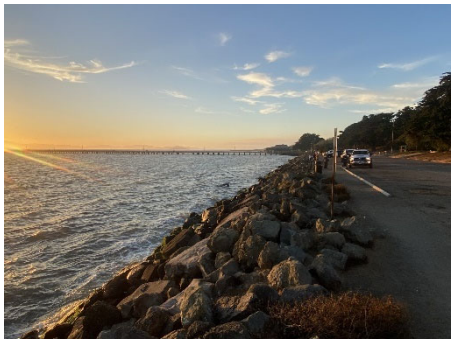


Figure 7 *Section 4 profile and site photo at approximate location.*

3 Conditions Assessment

This section presents a summary of the regional metocean conditions and site-specific field observations to evaluate the performance of existing shoreline protection at the Berkeley Marina. This combined data provides the framework for interpreting profile geometry and identifying vulnerabilities in the current system.

3.1 Metocean Conditions

The data and methodologies used to characterize water levels, currents, winds, and waves are presented in the (COWI, 2025) providing the criteria for assessment of the current shoreline conditions. A depth limited wave condition at the revetment toe has been considered for this assessment.

3.2 Field Inspection

Visual, above water, field inspections were conducted on August 22 and 25, 2025,Figure 8 to Figure 11 document the conditions of the revetment slope north and south of the pier. Overall, the revetment protection is in good condition with sufficient packing/interlock between rocks and adequate coverage along the shoreline.

A small number of rocks along the slope seem to deviate from standard dimensional requirements, showing irregular shapes and non-uniform three-dimensional proportions. While these out-of-spec rocks are noticeable, their limited quantity suggests they are not currently compromising the overall effectiveness of the slope protection.

Minor to moderate erosion of the revetment crest was observed mostly along the southern shoreline. Based on crest elevations (excluding the lower-elevation sections near the pier) and current metocean conditions, this erosion is likely caused by surface runoff rather than wave action. In the lower-elevation areas there is a need for targeted maintenance and reinforcement. However, erosion issues will be addressed as part of the project, since the planned plaza design incorporates higher elevations and will include additional protective measures.

Figure 13 shows a section of the revetment crest where erosion is evident. The surface in this area appears to be relatively impermeable, which may be contributing to concentrated surface runoff and, consequently, localized erosion. At this specific location, the gaps between amour rock indicate absence of geotextile or filter layers.

Overall, the visual inspection did not allow for a clear determination of the number of armor layers or the presence of underlayers such as geotextile. Based on current site conditions, however, the revetment appears to be performing adequately.

Figure 8 to Figure 14 are pictures illustrating conditions at different sections of the shoreline.



Figure 8 Slope protection north of the pier to the Skates On the Bay restaurant (north view).



Figure 9 Slope protection north of the pier (south view).



Figure 10 Slope protection south of the pier (north view).



Figure 11 Slope protection south of the pier (south view).

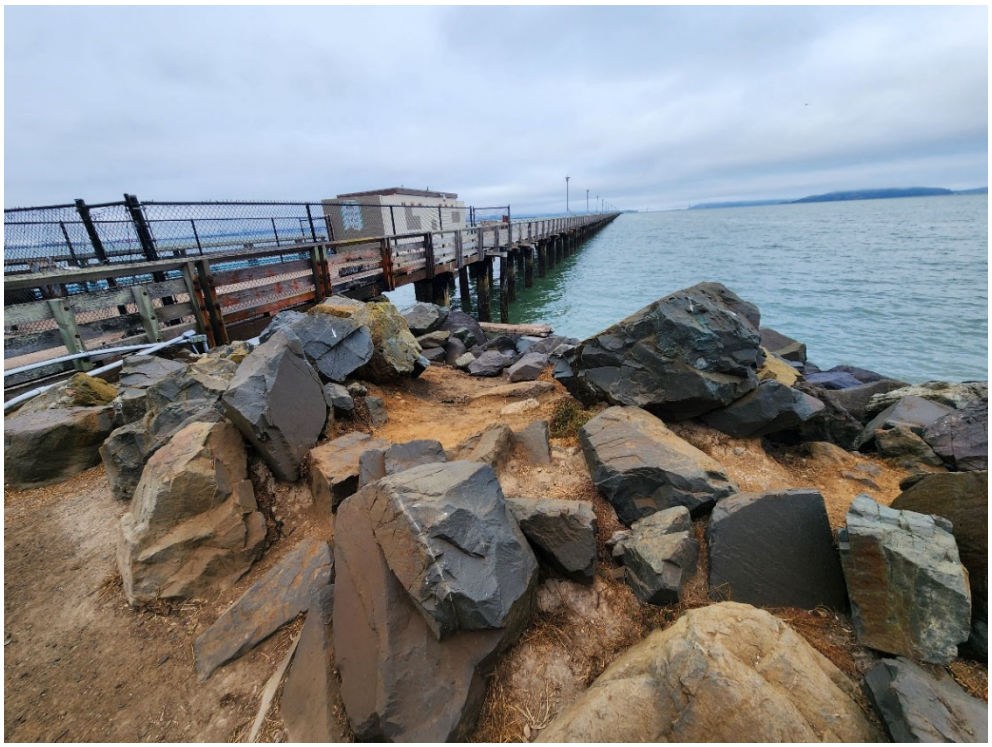


Figure 12 Chainage +300 ft – Minor erosion of slope crest adjacent to north end of the pier.



Figure 13 *Chainage +340 ft to +440 ft – Moderate erosion of slope crest adjacent to south end of the pier.*



Figure 14 *Chainage +525 ft – Moderate undermining of pavement at slope crest, south of the pier.*

4 Design Review

The design review assesses key parameters including crest elevation, rock gradation, and overtopping levels. These evaluations will be conducted in the context of both existing and projected metocean conditions, to ensure slope protection resilience and functionality for current and future environmental scenarios.

4.1 Rock Gradation

Sample measurements of the rocks were taken during the field visit, as shown in

Figure 15. Assuming a rock density of 2,630 kg/m3, Table 1 shows the approximate D_{50} of the measured samples.



Figure 15 Chainage +200 ft, measurement of rock sample north of pier (approximately 15").

Table 1 Section crest elevations (ft MLLW) and approximate slope.

Sample	X (in)	Y(in)	Z(in)	D_{50} (in)
1	33	26	10*	20.5
2	27	19	8	16
3	25	34	16	23.9
4	15	14	7	11.4

*assumed average

Based on the limited number of rock samples measured during the inspection, the existing rock material appears to correspond approximately to Caltrans Class IV, with an average median diameter (D_{50}) of approximately 15 inches. A comparison of the metocean conditions with the slope geometry and crest elevations obtained from the topo-bathymetric survey indicates that Caltrans Class III rock (D_{50} = 12 inches) would be sufficient to meet both current and projected design conditions. While a full gradation analysis was not possible due to the small sample size, the initial findings provide a positive indication of compliance with design expectations.

A screening-level assessment of the potential influence of the ferry azimuth thrusters on the shoreline protection was completed. Figure 16 shows the minimum separation between the dredge basin and the rock slope (approximately 100 ft). Using this offset, representative fast-ferry parameters, and assuming thruster power in the basin is less than 30 percent of installed capacity, the calculated bank-normal velocities at the shoreline from thruster wash are lower than those associated with the design wave condition. Therefore, the existing rock sizing remains appropriate as wave loading governs, and thruster effects do not govern design parameters.

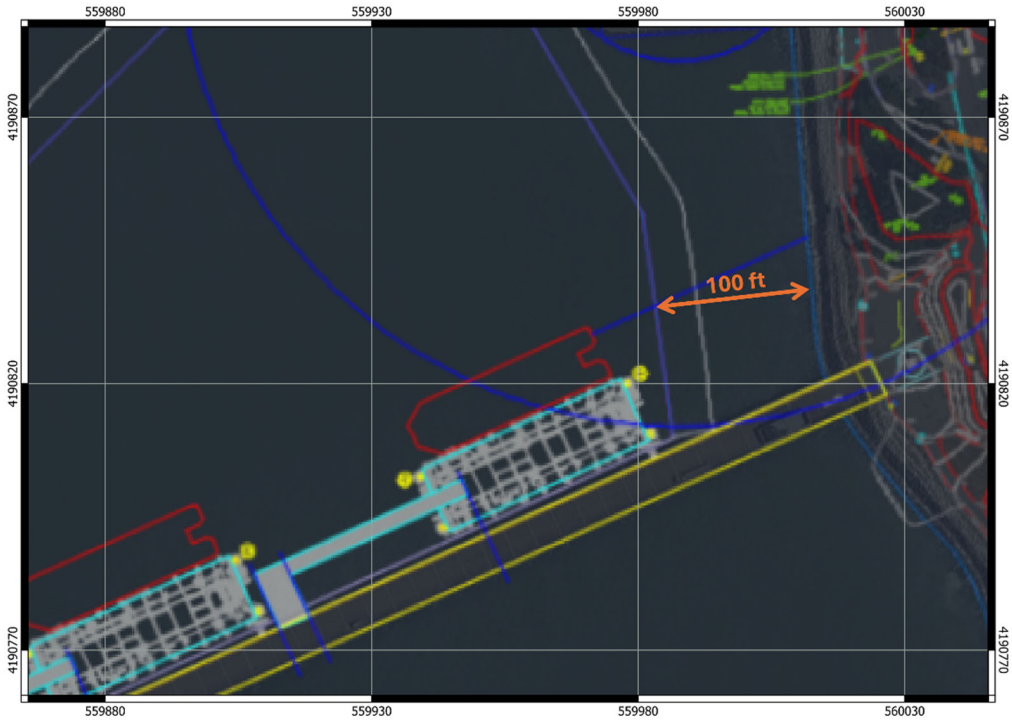


Figure 16 Approximate distance from ferry to rock slope toe.

4.2 Crest Elevation

Wave overtopping volumes were assessed across five shoreline sections, using the input parameters detailed in Table 2. The crest elevation along the shoreline is non-

uniform, with measured values ranging from 12.5 to 14.9 ft MLLW. Similarly, slope gradients varied between 2.2H:1V and 2.6H:1V across the analyzed segments.

Overtopping estimates were calculated for both current and projected design water levels, corresponding to a 100-year return period. These calculations incorporated depth-limited wind wave conditions at the toe of the revetment. For conservative design purposes, the revetment was assumed to be impermeable, consisting of a single layer of armor rock.

Table 2 identifies whether each section meets the tolerable wave overtopping criteria under current and future conditions. It indicates compliance with a threshold of 10 liters per second per meter (l/s/m), which reflects the lower tolerable limit for pedestrian safety near the waterfront.

Sections 1, 3, and 5 show sufficient elevation and slope characteristics to meet tolerable overtopping thresholds both now and under future sea level rise scenarios. Section 3, while compliant, still experiences some overtopping volume. The mean volume remains below the threshold, considered safe for pedestrian exposure.

Sections 2 and 4, with the lowest crest elevations among the sections analyzed, are only compliant under current conditions. Their inability to meet future tolerable overtopping limits suggests a vulnerability to increased water levels, highlighting the need for potential mitigation measures such as crest elevation adjustments or additional revetment reinforcement.

Table 2 Sections crest elevations, approximate slope, and tolerance to overtopping in current and future conditions.

Section	Crest Elevation (ft MLLW)	Slope H:1V	Tolerance to Overtopping Current Conditions	Tolerance to Overtopping Future Conditions
1	14.9	2.2	Yes	Yes
2	12.5	2.2	Yes	No
3	13	2.5	Yes	Yes
4	12.6	2.6	Yes	No
5	14.2	2.6	Yes	Yes

5 Conclusions

The assessment of the shoreline protection adjacent to the Berkeley Municipal Pier indicates that the existing revetment generally performs well under current environmental conditions. Field inspections confirmed that the rock revetment is mostly intact, with adequate interlocking and coverage. However, localized issues such as minor to moderate crest erosion, particularly along the southern shoreline, and the presence of undersized or irregular rock elements, were observed. These conditions, while not currently compromising the overall stability of the revetment, warrant attention in the context of future infrastructure upgrades.

Design evaluations based on crest elevation, slope geometry, and rock gradation suggest that the existing revetment material is broadly consistent with Caltrans Class IV specifications. Analysis indicates that Class III rock would be sufficient to meet both current and projected design conditions, indicating that the existing rock size is conservatively adequate.

Wave overtopping assessment across five shoreline sections revealed that Sections 1, 3, and 5 meet tolerable overtopping thresholds under both current and future sea level rise scenarios. Section 3, while compliant, does experience some overtopping with the mean volume remaining below the threshold considered safe for pedestrian use. In contrast, Sections 2 and 4, which have lower crest elevations, are only compliant under current conditions and are projected to exceed tolerable overtopping limits in the future. These findings highlight the need for targeted upgrades in these specific areas to ensure long-term resilience.

In summary, the current shoreline protection system is functionally adequate for present-day conditions. It would require selective reinforcement in specific sections with current lower crest to meet future performance criteria. It is recommended that:

- Crest elevations in Sections 2 and 4 (and similar sections) be raised or otherwise reinforced to mitigate future overtopping risks during the project design life.
- Areas exhibiting erosion or irregular rock placement be monitored and maintained to preserve structural integrity.

These measures will help ensure that the shoreline protection system remains robust and compatible with the operational and safety requirements of the BWTPF Project.

6 References

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