Geotechnical Engineering Services Report
Revision 3

Port of Ilwaco, Marina Structures Replacement and Dredging, Engineering, and Permitting
1170 Howerton Avenue East
Ilwaco, Washington

for
Moffatt & Nichol Engineers

October 4, 2023

GeoEngineers
Earth Science + Technology

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File No. 21551-003-00

October 4, 2023

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1.0 INTRODUCTION AND PROJECT UNDERSTANDING

GeoEngineers, Inc. (GeoEngineers) is pleased to submit this report presenting the results of our geotechnical engineering services for the proposed Port of Ilwaco, Marina Structures Replacement and Dredging, Engineering, and Permitting Upgrades project. This report summarizes our understanding of subsurface conditions in the project area and provides geotechnical recommendations and design criteria for the project. The project site is located at 1170 Howerton Avenue East, Ilwaco, Washington 98624, as shown on the Vicinity Map, Figure 1.

The project includes designing repairs and improvements to the existing wharf east bulkhead. The existing bulkhead consists of creosote treated timber piles, lagging, and walers. Wire strand tiebacks connected to the timber waler are presumed to connect to buried deadman anchors in the upland area. Three steel pipe piles are located along the face of the existing bulkhead and are assumed to be used for mooring of vessels. We understand that a replacement bulkhead consisting of a sheet pile wall embedded into the underlying siltstone will be constructed in front of the existing wharf east bulkhead. We understand tiebacks will be used to secure the top of the wall.

Improvements to shoreline areas surrounding the wharf east bulkhead are also planned. The majority of the improvements consist of slope armoring using rip rap. We understand that within the shoreline area northeast of the proposed bulkhead a relic timber wall on the shoreline slope will be removed, rip rap slope protection will be installed, and a small berm will be constructed at the top of the slope. The berm will be on the order of 1 foot tall and is being included to mitigate the effects of future sea-level rise. At the south end of the bulkhead we understand that existing concrete rubble slope armoring will be removed and replaced in-kind with riprap on the order of 18 inches thick.

2.0 PURPOSE AND SCOPE OF SERVICES

The purpose of our services is to provide design recommendations to support replacement of the Port of Ilwaco (POI) wharf east bulkhead and installation of slope protection on the shoreline slope to the northeast of the new bulkhead. Design recommendations included in this report are based on available existing subsurface information, our site explorations conducted on March 14 and March 19, 2022, and our experience in the project vicinity.

Our specific scope of services is presented in our Scope and Fee Estimate dated December 13, 2019 and Service Agreement with Moffatt & Nichol dated January 25, 2022.

3.0 SITE CONDITIONS

3.1. Surface Conditions

The site is located at the west end of the Ilwaco marina on a wharf currently occupied by multiple buildings associated with a fish processing facility. The existing bulkhead, which will be repaired as part of this project, delineates the eastern edge of the wharf. The buildings are generally located along the western edge of the wharf. The retained area between the bulkhead and buildings is approximately 27 feet wide. The shoreline at the north end of the bulkhead consists of gravel and grasses at the surface sloping down at approximately 2H:1V (horizontal to vertical) to the shoreline. At the south end of the bulkhead, the shoreline is sloped at approximately 1H:1V and consists of fill and riprap.

3.2. Site Geology

We reviewed the Geologic Map of Washington-Southwest Quadrant (Walsh, et al. 1987) to develop an understanding of the site geology. The surface geology of the project site is mapped as "Beach Deposits," and potentially underlain by bedrock mapped as "Oligocene to upper Eocene marine sedimentary rocks." The Beach Deposits are described as fine to coarse sand. The marine sedimentary bedrock is described as siltstone, and/or fine sandstone. Based on the site history and human modification, we also anticipate that fill material is present in the project vicinity.

3.3. Subsurface Exploration

We explored site subsurface conditions by completing two borings (B-1D and B-2A) at the approximate locations shown on the Site Plan Figure 2. The borings were advanced to depths of 65 and 70 feet below ground surface (bgs) using subcontracted track-mounted drilling equipment and vacuum trucks operated by drillers subcontracted to GeoEngineers. During our initial site exploration effort, six attempts were made to use a hollow stem auger drilling method to drill within the wharf footprint, but each attempt met practical refusal at depths of less than 5 feet. Attempted borings B-1, B-1A, B-1B, B-1C are also shown on the attached Site Plan Figure 2. We were able to complete boring B-2A just upland of the wharf footprint during this initial visit. We returned to site at a second time and were able to successfully complete boring B-1D in the same location as the original B1-D attempt using a sonic drill rig. Additional details of the exploration program and summary logs of the explorations are included in Appendix A, Field Explorations.

Soil samples obtained from the borings were taken to our geotechnical laboratory for further evaluation. Testing included moisture content determinations, percent fines determination and gradation analyses. A description of the laboratory test procedures and test results are presented in Appendix A and/or on the boring logs.

TABLE 1. UNSUCCESSFUL BORING ATTEMPTS

Boring	Depth of Termination (ft)	Reason for Termination	Observed Soils	Comment
B-1	3	Refusal on pipe	GP-GM	Corrugated Steel pipe at 3 feet bgs
B-1A	4	Refusal In cobbles	GP-GM	Yellow Plastic pipe (approx. 2-inch-diameter) Patch of clean sand fill approximately 6 inches around pipe
B-1B	3.5	Refusal In cobbles	GP-GM	
B-1C	3.6	Refusal In cobbles	GM	Layer of sandy silt with gravel and cobbles around 2 to 2½ feet
B-1D	3.6	Refusal In cobbles	GP-GM	
B-2	4.3	Refusal In cobbles	GP-GM	

3.4. Soil Conditions

Alluvial deposits in the site vicinity generally consist of soils with high silt content. The predominant soil types are sandy silt and silt, but these are often closely interbedded and may include lenses of variable thickness and/or inclined layers as well as regions of cleaner sands. Soils observed in our explorations generally consist of fill overlying native alluvial deposits overlying the regional bedrock, as described in the following paragraphs.

3.4.1. Fill

All borings and attempts except B2-A were advanced through asphalt pavement. Thickness of asphalt observed ranged from 3 to 6 inches. Boring B-2A encountered about 2 inches of silty sand topsoil. Starting below the asphalt (or below the topsoil in B2-A) to approximately 5 feet bgs, we observed brown fine to course gravel with silt and cobbles in a loose and moist condition. Occasional lenses of higher silt and sand content were observed as well.

3.4.2. Submerged Fill

Underlying the fill unit we generally observed brown silty fine to medium sand in a loose and wet condition, which we interpret to be a separate fill unit. For differentiation purposes, we have identified this fill unit as submerged fill. The top of the unit was observed at 5 feet bgs and the base varied from 12 feet bgs in B-1D and 15 feet bgs in B-2A.

3.4.3. Alluvial Deposits

Beneath the submerged fill unit, we interpret soils to consist of native alluvial deposits. Alluvial deposits generally consisted of interbedded layers of clay, silt with varying sand content, and silty sand. During drilling of boring B-2A we observed a transition in stiffness/density and based upon this observation, we divided the alluvial deposits into an upper and lower unit.

3.4.3.1. Upper Alluvial Deposit

The upper alluvial deposits were observed directly below the submerged fill unit and extending to about 30 feet in B-1D and 40 feet in B-2A. Soils observed in this unit were typically silts and clays with varying sand content. We also observed occasional interbeds of silty sand, typically 5 feet thick or less. The unit is generally soft/loose and wet. In addition, wood debris was consistently observed throughout the unit.

3.4.3.2. Lower Alluvial Deposit

Below the upper alluvial deposits, we observed lower alluvial deposits in boring B-2A, which extend to approximately 60 feet bgs. Soils observed generally consist of soft to medium stiff silt and brown fat clay. The unit is soft at the top and ranges to medium stiff at its base. Wood organic debris was observed in the upper 5 feet of the unit. Note that the lower alluvial deposits unit was not observed in boring B-1D.

3.4.4. Weathered Siltstone

Below the alluvial deposits, both borings encountered what we interpret to be weathered siltstone bedrock, extending to depths of 55 feet in B-1D and 65 feet in B-2A. The samples retrieved typically consisted of wet medium stiff to very stiff silt, but the material was observed to break into a blocky texture when cut with a soil knife.

The upper and lower boundaries of this unit were somewhat indistinct because the general soil type was very similar in the alluvial deposit and the more intact siltstone (described below). The extent of the weathered siltstone unit was delineated through changes in standard penetration test (SPT) blow counts and observed texture of the samples retrieved. The interpreted degree of weathering is relatively high, based on the consistency and the ability to drill through the material using hollow stem auger drilling and collect samples using standard penetration testing.

3.4.5. Siltstone

Below the weathered siltstone, we observed what we interpret to be more intact, less weathered siltstone, extending to the full depths explored. The samples retrieved typically consisted of hard, moist silt. As with the weathered siltstone described above, the material was observed to break into a blocky texture when cut with a soil knife, but also exhibited significantly higher resistance to the soil knife and drilling and sampling efforts.

3.5. Groundwater

At the time of our explorations, groundwater was encountered at approximately 5 feet bgs. Given the site's proximity to the tidal-influenced water, the water table should be expected to vary with tide level—but given the silt content of the upper most soils—saturated soils should be expected up to the high tide elevation.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1. Seismic Design Considerations

4.1.1. Seismic Design Approach

Based on our explorations and analysis, the project site is underlain by liquefiable soils. Liquefaction could result in surface settlements, soil strength loss and movement of the waterway slope (lateral spreading). The following sections provide additional information regarding liquefaction and associated effects. Based on our discussion with the design team, we understand that, in order to resist seismic loading and limit liquefaction risk, the bulkhead sheet pile wall will be driven into the underlying siltstone and tiebacks will be anchored in the siltstone as well.

4.1.2. Seismic Design Parameters

We understand that seismic consideration for this project fall under the *International Building Code* 2018 (IBC 2018) which references the 2016 *Minimum Design Loads for Buildings and Other Structures* (American Society of Civil Engineers [ASCE] 7-16).

As addressed in the sections below, our review of the existing data at the site indicates potentially liquefiable soils are present from the surface to the existing mudline (approximately Elevation -14 feet). In accordance with the design documents referenced above, sites with liquefiable soils shall be classified as Site Class F and a site-specific response analysis shall be performed. An exception is provided in Section 20.3.1 of ASCE 7-16, which states that for structures with a fundamental period of vibration less than or equal to 0.5 seconds, a site-specific seismic evaluation is not required. Our scope of services does not include site-specific response analysis.

As a basis for a simplified design and analysis we recommend using a response spectrum for Site Class D. Recommended Site Class D seismic design parameters are presented in Table 2 below.

TABLE 2. SEISMIC DESIGN CRITERIA

ASCE 7-16 Seismic Design Parameters[1]	
Site Class	F
Spectral Response Acceleration at Short Periods (S_S)	1.427g
Spectral Response Acceleration at 1-Second Periods (S_1)	0.738g
Short-Period Site Coefficient (F_a)	1.20
Long-Period Site Coefficient (F_v)	1.7
Design Spectral Response Acceleration at Short Periods ($S_{DS} = 2/3 * F_aS_s$)	1.142g
Design Spectral Response Acceleration at 1-Second Periods ($S_{D1} = 2/3 * F_vS_1$)	1.255g[2]
Design Peak Ground Acceleration (PGA_M)	0.798g

Notes:

[1] Parameters developed based on Latitude 46.3048196° and Longitude -124.0410238° using the ATC Hazards online tool.

[2] Per ASCE 7-16 Supplement 3 Section 11.4.8 item 1, parameter has been increased by 50 percent or has increased as a result of adjusted S_{m1} Value.

4.1.3. Liquefaction Potential

Liquefaction refers to the condition by which vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils and the subsequent loss of strength in the affected soil deposit. In general, soils that are susceptible to liquefaction include very loose to medium dense clean to silty sands and some silts below the water table. Liquefaction effects on foundations can include a temporary loss of bearing capacity, settlement of the ground surface and downdrag loads on pile and shaft foundations.

We reviewed the "Liquefaction Susceptibility Map of Pacific County, Washington" (Palmer et al. 2004). According to the map, the potential for liquefaction at this site is high.

We evaluated the liquefaction potential of the site soil using simplified methods that utilize Atterberg limits to evaluate liquefaction potential (Idriss and Boulanger 2008 and Bray and Sancio 2006). These methods apply limits to liquefaction potential based on the plastic index and moisture content of the soil. Based on the results of our Atterberg limit testing and using the above methodology, the majority of native soils at the site are not expected to be liquefiable. There is, however, some potential for soil strength reduction due to seismic shaking. We have considered this reduction in development of our post-seismic design recommendations presented below.

The upper 15 feet (approximate Elevations 11 to -4 feet) consists of primarily fill and the upper portion of the alluvium shows interbedded silty sands and we consider this region to have some susceptibility to liquefaction.

Based on our review and analysis, it is our opinion potentially liquefiable soils are present at the site from the surface to 15 feet bgs (approximate Elevation -4 feet).

4.1.4. Liquefaction-Induced Settlement

Based on our explorations, lab data, and liquefaction susceptibility evaluation, we estimated liquefaction-induced settlement at the ground surface considering liquefaction to a depth of 15 feet. We estimate liquefaction-induced settlement could range from about 1 to 2 inches at the ground surface as a result of the design level earthquake (Magnitude 9.08, PGA_M = 0.798g). Areas of liquefaction can be relatively discontinuous and separated by layers of non-liquefied soil. Due to the variability of soils in the upper 15 feet and the inherent unpredictability of seismic soil liquefaction, differential settlements could be as much as the total settlement.

4.1.5. Lateral Spreading Potential

Liquefaction-induced soil strength loss can also result in slope instability and lateral spreading. Lateral spreading related to seismic activity typically involves lateral displacement of large, surficial blocks of non-liquefied soil when an underlying soil layer loses strength during seismic shaking. Alternatively, when the majority of the soil profile loses strength a flow-type failure may occur. Lateral spreading usually develops in areas where sloping ground or large grade changes are present. Lateral spreading can induce significant lateral loads on embedded structures (kinematic loading).

Based on our understanding of the subsurface conditions, liquefaction risk and current site topography, it is our opinion there is a risk of lateral spreading during the design earthquake in regions not confined by the bulkhead.

4.1.6. Surface Rupture Potential

According to the Washington State Department of Natural Resources “Interactive Natural Hazards Map” (accessed online July 14, 2022), the nearest known major seismic feature is the Cascadia Subduction Zone (CSZ). The eastern most extent of this region is mapped approximately 11 miles west of the project site. In addition to the CSZ, there are two additional mapped faults approximately 8 miles from the site. The Willapa Bay Oblique-slip fault is located 8 miles north of the site and a strike-slip fault associated with the CSZ is located 8 miles southwest of the site. Based on this information it is our opinion the risk for seismic surface rupture at the site is low.

4.2. Soil Parameters

Based on our explorations and testing, we developed a generalized soil profile with associated parameters for use in engineering analysis completed as part of the project. Tables 3 and 4 below summarize our recommended design soil properties for static conditions and post-earthquake (liquefied) conditions. Elevation ranges for each soil unit are provided based on the explorations reviewed and are referenced to the elevation at the top of the existing pavement (approximate elevation 11 feet).

TABLE 3. RECOMMENDED STATIC SOIL PARAMETERS

Depth[1,2] (feet)	Soil Unit	USCS Soil Type	Total Unit Weight (pcf)[3]	Effective Unit Weight (pcf)[3]	Friction Angle (degrees)	Cohesion (pcf)[3]	Ka[4]	Kp[5]	Active Equivalent Fluid Density[6] (pcf)	Allowable Equivalent Fluid Density[7] (pcf)	Allowable Passive Pressure[8] (psf)
0 to 5	Fill	GP-GM	120	--	30	--	0.33	3.0	40.0	240	--
5 to 12	Submerged Fill	SM	120	58	28	--	0.36	2.75	21	107	--
12 to 35	Upper Alluvium	ML/CH	105	43	--	250	--	--	--	--	335
35 to 55	Lower Alluvium and weathered Siltstone	ML/CH	110	48	--	800	--	--	--	--	1,067
55 and below	Siltstone	Rx	120	58	42	--	--	5.04	--	194	--

Notes:

1 Depths are referenced to the top of pavement behind existing bulkhead.

2 Mudline in front of bulkhead assumed to be at 15 feet.

3 Groundwater is assumed to be at 5 feet below ground surface.

4 Ka = Active earth pressure coefficient.

5 Kp=Passive earth pressure coefficient (ultimate, does not include a factor of safety).

6 Active equivalent fluid density provided for soils retained by the bulkhead and do not include hydrostatic pressures.

7 Allowable passive equivalent fluid densities include a FOS of 1.5. These values do not include hydrostatic pressures.

8 Allowable passive pressures (rectangular distribution) provided for cohesive soils and include a FOS of 1.5.

TABLE 4. RECOMMENDED POST-SEISMIC CONDITIONS

Depth[1,2] (feet)	Soil Unit	USCS Soil Type	Total Unit Weight (pcf)[3]	Effective Unit Weight (pcf)[3]	Friction Angle (degrees)	Cohesion (pcf)[3]	Ka[4]	Kp[5]	Active Equivalent Fluid Density[6] (pcf)	Allowable Equivalent Fluid Density[7] (pcf)	Allowable Passive Pressure[8] (psf)
0 to 5	Fill	GP-GM	120	–	30	–	0.33	3.0	40.0	240	–
5 to 12	Liquified Fill	SM	120	58	22	–	0.45	2.2	26	107	–
12 to 35	Upper Alluvium (strain Softened)	ML/CH	105	43	–	200	–	–	–	–	335
35 to 55	Lower Alluvium and weathered siltstone (strain Softened)	ML/CH	110	48	–	640	–	–	–	–	1,067
55 and below	Siltstone	Rx	120	58	42	–	–	5.04	–	244	–

Notes:

1 Depths are referenced to the top of pavement behind existing bulkhead.

2 Mudline in front of bulkhead assumed to be at 15 feet.

3 Groundwater is assumed to be at 5 feet below ground surface.

4 Ka = Active earth pressure coefficient.

5 Kp=Passive earth pressure coefficient (ultimate, does not include a factor of safety).

6 Active equivalent fluid density provided for soils retained by the bulkhead and do not include hydrostatic pressures.

7 Allowable passive equivalent fluid densities include a FOS of 1.2. These values do not include hydrostatic pressures.

8 Allowable passive pressures (rectangular distribution) provided for cohesive soils and include a FOS of 1.2.

4.3. Geotechnical Pile Design Recommendations

4.3.1. Axial Pile Resistance

Based on our experience with driven piles in near shore environments, end bearing resistance can be highly variable, depending on the specific soil conditions at the tip of each pile. Therefore, we typically assume low end bearing resistance values for design if not driven into bedrock. However, it is our understanding that piles for this project will be driven into the underlying siltstone providing considerably more tip capacity than in alluvium sediment deposits. If it becomes desirable to drive piles to depths above the underlying siltstone we can provide further recommendations.

Based on our understanding of site conditions and planned development, we estimated axial resistance available for piles driven at the site, for static and post seismic conditions. Because pile sizes may need to vary, we provided estimated unit resistances for each soil layer. Estimated resistances are presented in Tables 5 and 6.

Mudline at the outboard edge of the existing bulkhead is currently at approximately Elevation -4 feet. We understand that the new mudline will be at approximate elevation -16 feet to account for future dredging activities. Skin friction above the planned future mudline should be disregarded when computing total pile capacities.

Because of the complex stratigraphy and variability of soils in the site vicinity, we anticipate that actual ultimate axial resistances may vary by as much as 20 to 25 percent. Allowable resistances should be used for designing the piles. Allowable static axial pile resistances presented in the table below include a factor of safety (FS) equal to 2 for end bearing, 3 for skin friction and 2.5 for uplift resistance. Allowable seismic axial pile resistances include a FS equal to 1.5 for end bearing, 3 for skin friction and 1.5 for uplift resistance.

TABLE 5. AXIAL PILE RESISTANCES (STATIC CONDITIONS)

Depth[1,2] (feet)	Soil Unit	USCS Soil Type	Allowable Unit Skin Resistance[3,4] (ksf)	Allowable Unit End Bearing Resistance[3,5] (ksf)	Allowable Unit Uplift Resistance[3,6] (ksf)
0 to 5	Fill	GP-GM	-	-	-
5 to 12	Submerged Fill	SM	-	-	-
12 to 35	Upper Alluvium	ML/CH	0.075	0.9	0.0625
35 to 55	Lower Alluvium and Siltstone	ML/CH	0.24	2.9	0.2
55 and below	Siltstone	RX	0.75	17	0.63

Notes:

1 Depths are referenced to the top of pavement behind existing bulkhead.

2 Mudline in front of bulkhead assumed to be at relative depth of 27 feet.

3 Resistances for fill not provided. Pile Resistance should be accounted for starting where pile becomes fully embedded (portion of pile below future mudline).

4 Includes a factor of safety of 2.5.

5 Includes a factor of safety of 2.5.

6 Includes a factor of safety of 3.0.

7 To calculate allowable skin and uplift resistance, multiply allowable skin/uplift resistance by the pile perimeter (ft) and the length of the pile embedded into the given layer.

8 To calculate allowable end bearing resistance, multiply unit end bearing resistance by pile tip area (sf) for the soil unit at the pile tip depth.

TABLE 6. AXIAL PILE RESISTANCES (POST SEISMIC CONDITIONS)

Depth[1,2] (feet)	Soil Unit	USCS Soil Type	Allowable Unit Skin Resistance[3,4] (ksf)	Allowable Unit End Bearing Resistance[3,5] (ksf)	Allowable Unit Uplift Resistance[3,6] (ksf)
0 to 5	Fill	GP-GM	-	-	-
5 to 12	Submerged Fill	SM	-	-	-
12 to 35	Upper Alluvium	ML/CH	0.075	0.9	0.0625
35 to 55	Lower Alluvium and Siltstone	ML/CH	0.24	2.9	0.19
55 and below	Siltstone	RX	0.75	17	0.63

Notes:

1 Depths are referenced to the top of pavement behind existing bulkhead.

2 Mudline in front of bulkhead assumed to be at relative depth of 27 feet.

3 Resistances for fill not provided. Pile Resistance should be accounted for starting where pile becomes fully embedded (portion of pile below future mudline).

4 Includes a factor of safety of 2.0.

5 Includes a factor of safety of 2.0.

6 Includes a factor of safety of 2.5.

7 To calculate allowable skin and uplift resistance, multiply allowable skin/uplift resistance by the pile perimeter (ft) and the length of the pile embedded into the given layer.

8 To calculate allowable end bearing resistance, multiply unit end bearing resistance by pile tip area (sf) for the soil unit at the pile tip depth.

4.3.2. Settlement

Based on our understanding of the project, soil profile and properties, and assuming the piles are embedded into the underlying siltstone unit, we anticipate settlement of piles should be on the order of 1 inch or less with differential settlement of ½ inch or less.

4.3.3. LPILE Soil Parameters

We understand that lateral load performance of the proposed piles will be evaluated using the computer software program LPILE produced by Ensoft, Inc. Our recommended LPILE soil parameters are presented in the tables below.

For the purpose of this report, we assume piles are spaced at least 5 diameters (5D) center to center in the direction of loading. If spacing is less than 5D, P multipliers will be required for the LPILE analysis and are available upon request.

TABLE 7. RECOMMENDED STATIC LPILE SOIL PARAMETERS (STATIC CONDITIONS)

Depth[1,2] (feet)	Soil Unit	USCS Soil Type	Friction Angle (degrees)	Undrained shear Strength/ Cohesion (psf)	Lateral Analysis Parameters – Static Conditions				
					P.Y Curve Model	Total Unit Weight3 (pcf)	Effective Unit Weight2 (pcf)	Soil Modulus K (pci)	Strain Factor e50
0 to 12	Fill[4]	GP-GM	-	-	-	-	-	-	-
12 to 35	Upper Alluvium	ML/CH	-	250	Soft Clay	105	43	-	0.02
35 to 55	Lower Alluvium and W. Siltstone	ML/CH	-	800	Soft Clay	110	48	-	0.02
55 and below	Siltstone	RX	42	-	Sand (Reese)	120	58	150	-

Notes:

1 Depths are referenced to the top of the pavement behind existing bulkhead.

2 Mudline in front of bulkhead assumed to be at relative depth of 27 feet.

3 Assume static groundwater levels at 5 feet below surface for design. Effective unit weights should be used for soil layers below the groundwater table.

4 Resistances for fill not provided. Pile Resistances should be accounted for starting where pile becomes fully embedded (portion of pile below mudline).

TABLE 8. RECOMMENDED STATIC LPILE SOIL PARAMETERS (POST SEISMIC CONDITIONS)

Depth[1,2] (feet)	Soil Unit	USCS Soil Type	Friction Angle (degrees)	Undrained shear Strength/ Cohesion (psf)	Lateral Analysis Parameters – Static Conditions				
					P.Y Curve Model	Total Unit Weight3 (pcf)	Effective Unit Weight2 (pcf)	Soil Modulus K (pci)	Strain Factor e50
0 to 12	Fill[4]	GP-GM	-	-	-	-	-	-	-
12 to 35	Upper Alluvium (strain softened)	ML/CH	-	200	Soft Clay	105	43	-	0.02
35 to 55	Lower Alluvium and W. Siltstone (strain softened)	ML/CH	-	640	Soft Clay	110	48	-	0.02
55 and below	Siltstone	RX	42	-	Sand (Reese)	120	58	150	-

Notes:

1 Depths are referenced to the top of pavement behind existing bulkhead.

2 Mudline in front of bulkhead assumed to be at relative depth of 27 feet.

3 Assume Static groundwater levels at 5 feet below ground surface for design. Effective unit weights should be used for soil layers below the groundwater table.

4 Resistances for fill not provided. Pile Resistance should be accounted for starting where pile becomes fully embedded (portion of pile below mudline).

4.3.4. Pile Installation Considerations

Provided subsurface conditions are as assumed, we anticipate conventional vibratory driving methods can be used to advance open-tip steel pipe piles through the overlying fill (if present at the mudline) and native alluvial deposits at the site. The reviewed explorations do not indicate the presence of gravel or other potential impediments to vibratory pile driving within the alluvial soils; however, very dense zones or other obstructions such as logs could be present. Vibratory pile driving equipment will need to be selected based on the pile size. If significant penetration into the siltstone unit is planned, impact driving is likely to be required. We recommend that project plans and specifications include selecting and providing an impact hammer of sufficient capacity to continue driving the pile if vibratory installation methods reach refusal before the design tip elevation.

We recommend that a GeoEngineers representative be present on site during pile installation, particularly if impact driving is used. Our representative can observe whether piles are installed in accordance with the project plans and specifications, check for consistency in pile resistance during vibratory installation and evaluate pile resistance during impact driving. We can also provide recommendations for sizing vibratory and impact hammers for installation, if requested.

4.4. Lateral Earth Pressures

We developed lateral earth pressure recommendations for use in design of the replacement sheet pile bulkhead. Recommended lateral earth pressures under static and post-seismic conditions are presented on Figures 3 to 8, respectively. Lateral earth pressures were developed for the purpose of the lateral loading analysis for the proposed sheet pile wall and are presented relative the proposed structures and their relationship with the site stratigraphy.

4.5. Tieback Anchors

Tieback anchors should extend far enough behind the wall to develop anchorage beyond the “no-load” zone (See Figures 3 through 8 for definition of the no-load zone) and within a stable soil mass. We recommend that spacing between tiebacks be at least five times the diameter of the anchor hole to minimize group interaction.

We understand that tieback anchors will be installed into the intact siltstone, which was encountered around 55 to 65 feet below ground surface. For tiebacks installed into siltstone we recommend using an ultimate bond strength of 50 psi for design. We recommend that tiebacks be designed using a factor of safety of at least 2.0 for static conditions, which can be reduced to 1.5 for seismic conditions. We recommend that tieback anchors have a minimum bond length of 10 feet.

4.6. Shoreline Slope Stability

4.6.1. General

We completed slope stability analyses to evaluate the proposed modifications to the shoreline slope to the northeast and south of the new bulkhead. Proposed slope modifications to the northeast of the bulkhead include removal of a relic timber wall, installation of rip rap and construction of a new berm at the top of the slope. We understand that the thickness of the rip rap armoring will be on the order of 18 inches. The proposed berm will be set back about 2 feet from the crest of the slope, will have a crest elevation of around 14 feet (about 1 foot above existing grade) and will be about 30 feet wide. The approximate location of the proposed berm and the area of slope armoring is shown on Figure 9.

No significant modifications to the existing slope geometry are proposed in the area to the south of the proposed bulkhead. We understand that concrete rubble on the slope will be removed, and new riprap slope armoring will be added. The riprap thickness is expected to be on the order of 18 inches. The approximate location of the proposed slope armoring area south of the bulkhead is shown in Figure 10.

Slope stability analyses were completed using the computer program SLOPE/W (GEO-SLOPE International, Ltd. 2020). SLOPE/W evaluates the stability of numerous trial shear surfaces using a vertical slice limit-equilibrium method. This method compares the ratio of forces and moments driving slope movement versus forces and moments resisting slope movement for each trial shear surface and presents the result as the factor of safety (FOS). The program then sorts the trial shear surfaces and identifies the surface with the lowest factor of safety, or the “critical” shear surface. We assumed a circular arc slip surface and used the Morgenstern-Price method to calculate the forces.

We did not consider pseudo-static (seismic) or post seismic (residual strength) conditions in our slope stability analyses because the considered slopes do not directly support the proposed bulkhead. Additionally, evaluating surrounding slopes for these conditions is beyond the scope of this project. Pseudo-static and post seismic slope stability will primarily be controlled by the magnitude of seismic inertial forces and the residual soil strength properties of the underlying soils. The proposed slope improvements will not impact either of these analysis inputs. In our opinion the existing slopes likely do not meet minimum seismic slope stability factor of safety values however, the proposed slope modifications are unlikely to significantly change the stability of the existing slope considering pseudo-static and post seismic conditions.

4.6.2. Slope Stability Results – Shoreline Slope Northeast of Bulkhead

The approximate location of the slope cross section considered in our stability analysis along with the analysis results are shown in Figure 9. Our slope stability analysis indicates that the proposed shoreline slope configuration meets target static factor of safety requirements presented in the Washington State Department of Transportation Geotechnical Design Manual (1.5 for static conditions). In our opinion the proposed slope modifications can be completed without destabilizing the shoreline slope.

4.6.3. Slope Stability Results – Shoreline Slope South of Bulkhead

The approximate location of the slope cross section considered in our stability analysis is shown in Figure 10. For our analysis of this slope, we considered static slope stability both before and after removal of the existing concrete rubble armoring and the installation of the riprap armoring. We limited our analysis to evaluating the impact that placing the riprap will have shallow surficial slope stability.

Slope stability analysis results for the existing and proposed shoreline slope configuration south of the bulkhead are shown on Figure 10. Our analysis results indicate that replacement of the slope protection with riprap armoring will not significantly change the existing slope factor of safety (FOS=1.2) with respect to shallow surficial slope stability. The calculated FOS is less than the typical target FOS for new construction. Based on our assessment, a FOS of 1.2 does not imply that the slope is inherently unstable or at immediate risk of shallow surficial movement. In our opinion the proposed slope armoring can be completed without destabilizing the shoreline slope or impacting the proposed bulkhead and upland structures.

We did not evaluate global stability of the shoreline slope, as improving global slope stability is beyond the intent of the repairs and, in our opinion, replacement of the existing armoring with riprap will not significantly affect global slope stability. We also did not evaluate stability of the slope for the temporary condition after concrete rubble removal but prior to new riprap placement as this condition is not expected to present a risk to upland structures. Maintaining excavation stability during construction is the responsibility of the contractor performing the work. The contractor should follow best practices during construction and applicable guidelines for temporary excavations to maintain a stable excavation.

4.7. Pavement Design

4.7.1. General

We understand that existing asphalt pavements behind the bulkhead and along the wharf will be replaced as part of this project. The replacement pavement areas are primarily used by standard duty vehicles, 1.5-ton pneumatic tire forklifts, delivery trucks and occasional semi-trucks with trailers. Specific vehicle loading and frequency of use was not provided to us.

4.7.2. Design Parameters

We completed our pavement design following the methodology presented in the American Association of State Highway Transportation Officials (AASHTO) 1991 Flexible Pavement Design Standards and the 1993 AASHTO Guide for Design of Pavement Structures.

The recommended pavement section is based on a 20-year design life assuming an annual growth percentage of 0.1 percent. A 20-year design life for a pavement means that it is expected to be worn to the point of requiring a full replacement after 20 years. Some crack sealing and minor patching could be required before that time. Typically, full crack sealing (chip seal or resurfacing) is required after about 10 years of use, to prevent water instruction and accelerated deterioration.

The average daily traffic repetitions assumed in our analysis are summarized in Table 9 below. Other design input parameters necessary to complete the analysis such as reliability and serviceability index were selected based on our experience. We should be notified if specific traffic volumes or vehicle types should be considered as part of the pavement design.

TABLE 9. VEHICLE LOADING FREQUENCY

Vehicle Type	Assumed Daily Repetitions
Standard Duty Vehicle	30
1.5 Ton Pneumatic Tire Forklift	50
Delivery Truck *Single tandem axle box truck*	5
Semi-truck and trailer *100-ton gross vehicle weight, HS20-44 wheel configuration*	2

4.7.3. Recommended Pavement Section

Our recommended asphalt concrete pavement section is provided below. The recommended section is suitable for support of around 5,000,000 equivalent single axel loads (ESALs) over the assumed design life. In our opinion this is appropriate for a light industrial area. The provided pavement section may not be adequate for heavy construction traffic loads such as those imposed by concrete transit mixers, dump trucks or cranes. Additional pavement thickness may be necessary to prevent pavement damage during construction if other loading types are planned.

Recommended Pavement Section

- 5 inches of hot mix asphalt, class ½ inch, PG 58-22
- 12 inches of compacted crushed surfacing base course (CSBC)
- Subgrade prepared as recommended in Section 4.7.4 below.

The top approximate 2 inches of the CSBC section may consist of crushed surfacing top course (CSTC) as a leveling layer and for more precise grade development. CSBC and CSTC should conform to applicable sections of 4-04 and 9-03.9(3) of the WSDOT Standard Specifications. Crushed surfacing materials should be moisture conditioned to near optimum moisture content and compacted to at least 95 percent of the theoretical MDD per ASTM D 1557.

Hot mix asphalt should conform to applicable sections of 5-04, 9-02 and 9-03 of the WSDOT Standard Specifications.

4.7.4. Subgrade Preparation

Subgrades for pavements should be thoroughly compacted to a uniformly firm and unyielding condition on completion of demolition/excavation and before placing structural fill. We recommend that subgrades be evaluated, as appropriate, to identify areas of yielding or soft soil. Probing with a steel probe rod or proof-rolling with a heavy piece of wheeled construction equipment are appropriate methods of evaluation.

If soft or otherwise unsuitable subgrade areas are revealed during evaluation that cannot be compacted to a stable and uniformly firm condition, we recommend that: (1) the unsuitable soils be scarified (e.g., with a ripper or farmer's disc), aerated and recompacted, if practical; or (2) the unsuitable soils be removed and replaced with compacted structural fill, as needed.

Based on the current condition of the wharf pavements, we expect that the majority of the existing subgrade areas will not be suitable for pavement support in their current condition. We recommend that the project budget and schedule include contingencies for subgrade remediation. For preliminary estimating purposes we recommend assuming that 40 percent of the existing subgrade area will require up to 12 inches of overexcavation and replacement during remediation, 40 percent of the existing subgrade area will require up to 6 inches of overexcavation and replacement during remediation and 20 percent of the existing subgrade can be prepared to a suitable condition without overexcavation.

Based on our conversations with the project team and our observations while onsite, it appears likely that relic timber piles will be exposed within the subgrade area. We recommend that relic piles (or other remnant structural elements) be cut off at least 12 inches below the bottom of the design pavement section during subgrade preparation. Voids caused by removal of the timber piles should be backfilled with compacted structural fill.

4.7.5. Additional Considerations

Pavement design life and durability can be impacted by factors outside of vehicle repetitions including impact loading and use by special vehicles. These factors were not considered as part of developing the recommended pavement section.

Impact loading can cause surface damage and full depth pavement cracking. Cracks provide a pathway for moisture to enter the pavement section which can saturate the base course and subgrade materials, reducing the pavement design life. If cracks form in the pavement section, they should be sealed, or the damaged area should be replaced as soon as possible.

We anticipate that the pavement areas may occasionally be used by unusual or special use vehicles. An example of this would be a “warehouse” forklift with small hard rubber tires. While these types of vehicles are typically not heavy, they can produce high concentrated loads. Additionally, certain tire types can shove and rut pavements. If the pavement area is expected to be regularly used by solid tire forklifts or other special use vehicles a different pavement type or a thicker pavement section may need to be considered.

5.0 LIMITATIONS

We have prepared this report for the exclusive use of Moffatt & Nichol, the Port of Ilwaco, and their authorized agents. Moffatt & Nichol and the Port of Ilwaco may distribute copies of this report authorized agents and regulatory agencies as may be required for the project.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. The conclusions, recommendations, and opinions presented in this report are based on our professional knowledge, judgment, and experience. No warranty or other conditions, express or implied, should be understood.

Please refer to Appendix B titled “Report Limitations and Guidelines for Use” for additional information pertaining to use of this report.

6.0 REFERENCES

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FIGURES

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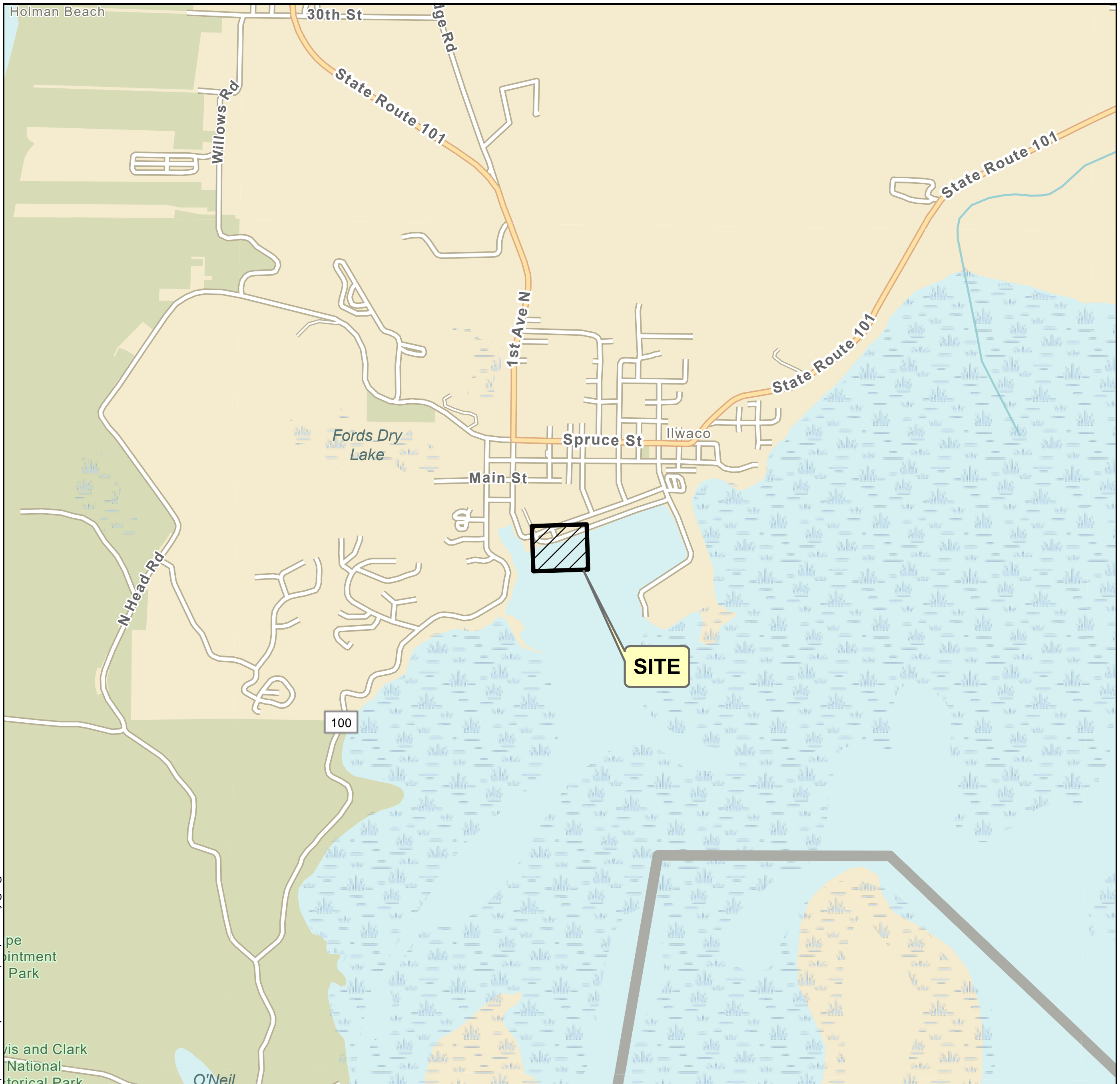

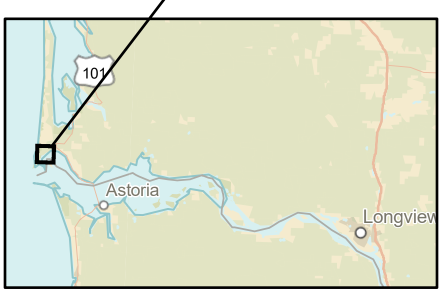

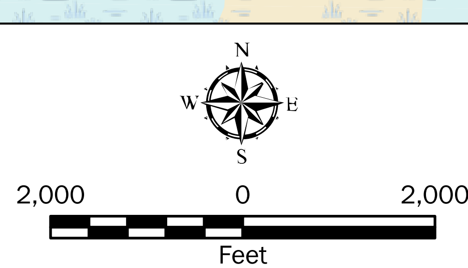

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: ESRI

Projection: NAD 1983 UTM Zone 10N

Vicinity Map
Port Ilwaco Bulkhead Replacement Ilwaco Washington Pacific County

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Figure 1

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Legend

B-1 Boring by GeoEngineers, Inc., 2022

Notes:
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Data Source: Aerial from Google Earth Pro dated 10/12/2018.

Projection: Washington State Plane, South Zone, NAD83, US Foot

N
W E
S

60 0 60
Feet

Site Plan

Port Ilwaco Bulkhead Replacement
Ilwaco, Washington
Pacific County

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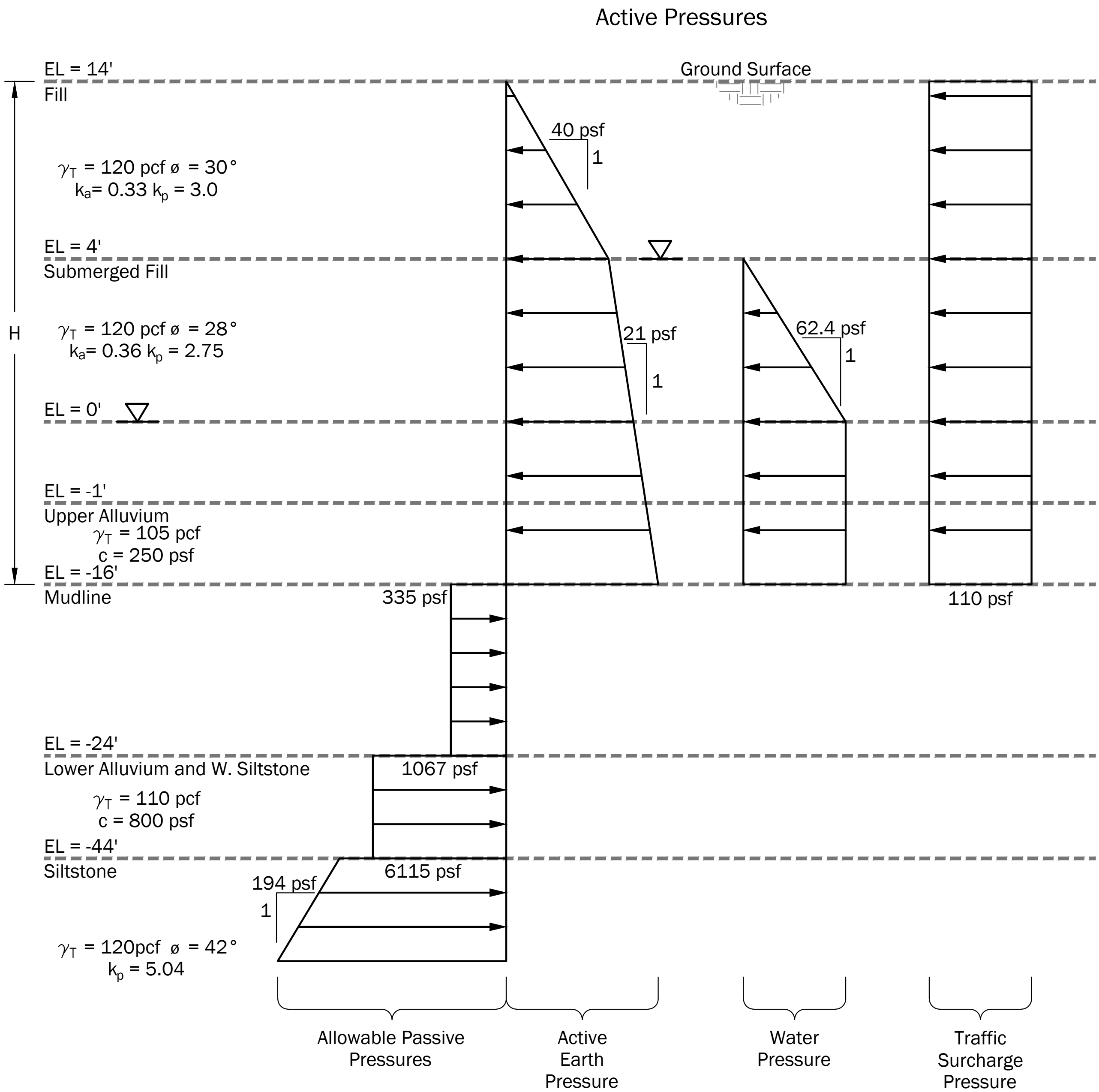
Figure 2

P:\21\21551003\CAD\00\Geotech Report\2155100300_F02_Site Plan.dwg F02 Date Exported:6/24/2022 1:52 PM - by Gabby Register

Static Conditions Cantilever Wall



Legend

H = Exposed Wall Height

Not To Scale

Earth Pressure Diagram
Static Conditions Cantilever Wall

Port Ilwaco Bulkhead Replacement
Ilwaco, Washington
Pacific County

GeoEngineers

Figure 3

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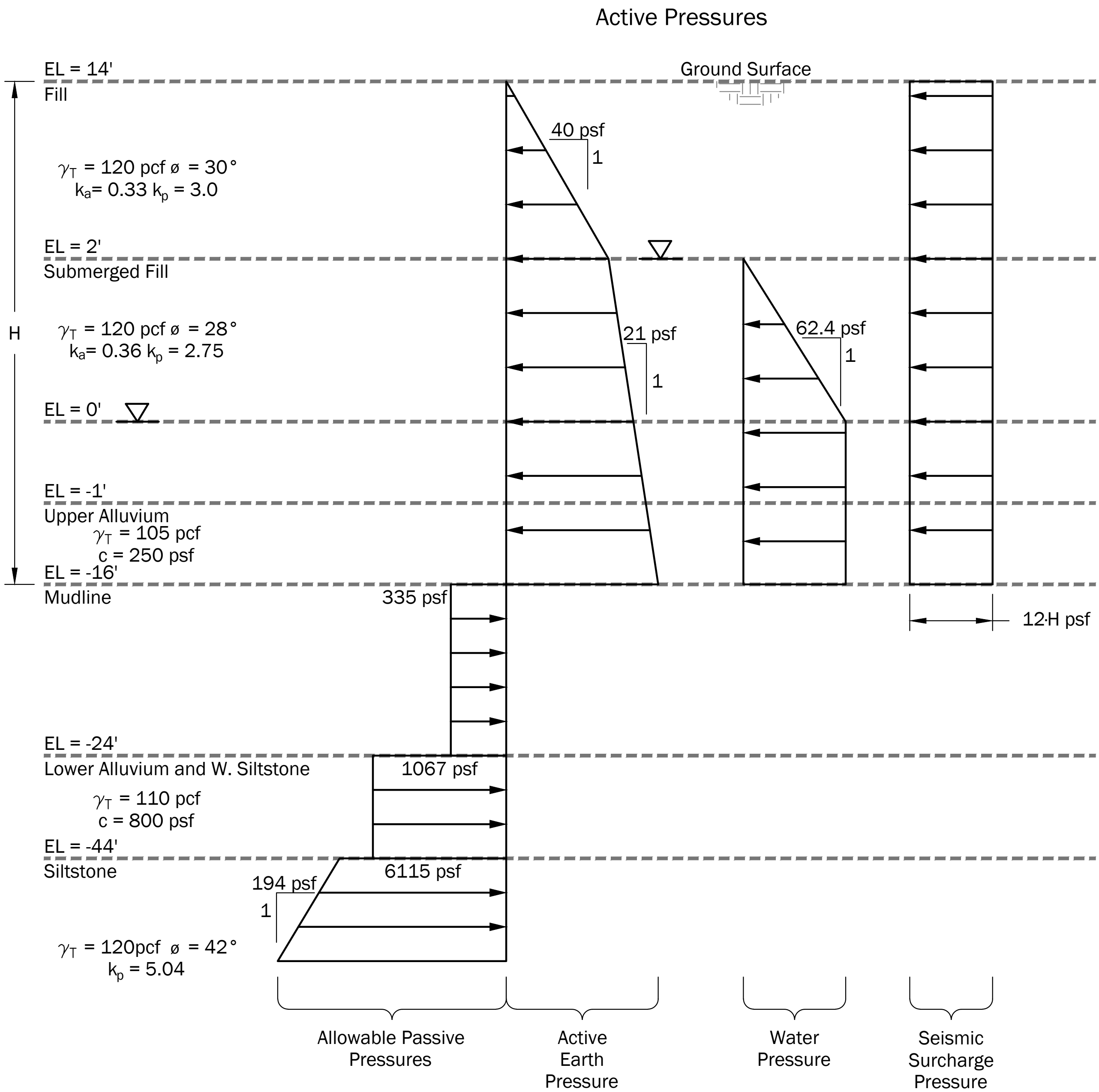
Seismic Conditions Cantilever Wall



Legend

H = Exposed Wall Height

Not To Scale

Notes:

1. Apply uniform 110 psf lateral traffic surcharge over distance H.

Earth Pressure Diagram
Seismic Conditions Cantilever Wall

Port Ilwaco Bulkhead Replacement
Ilwaco, Washington
Pacific County

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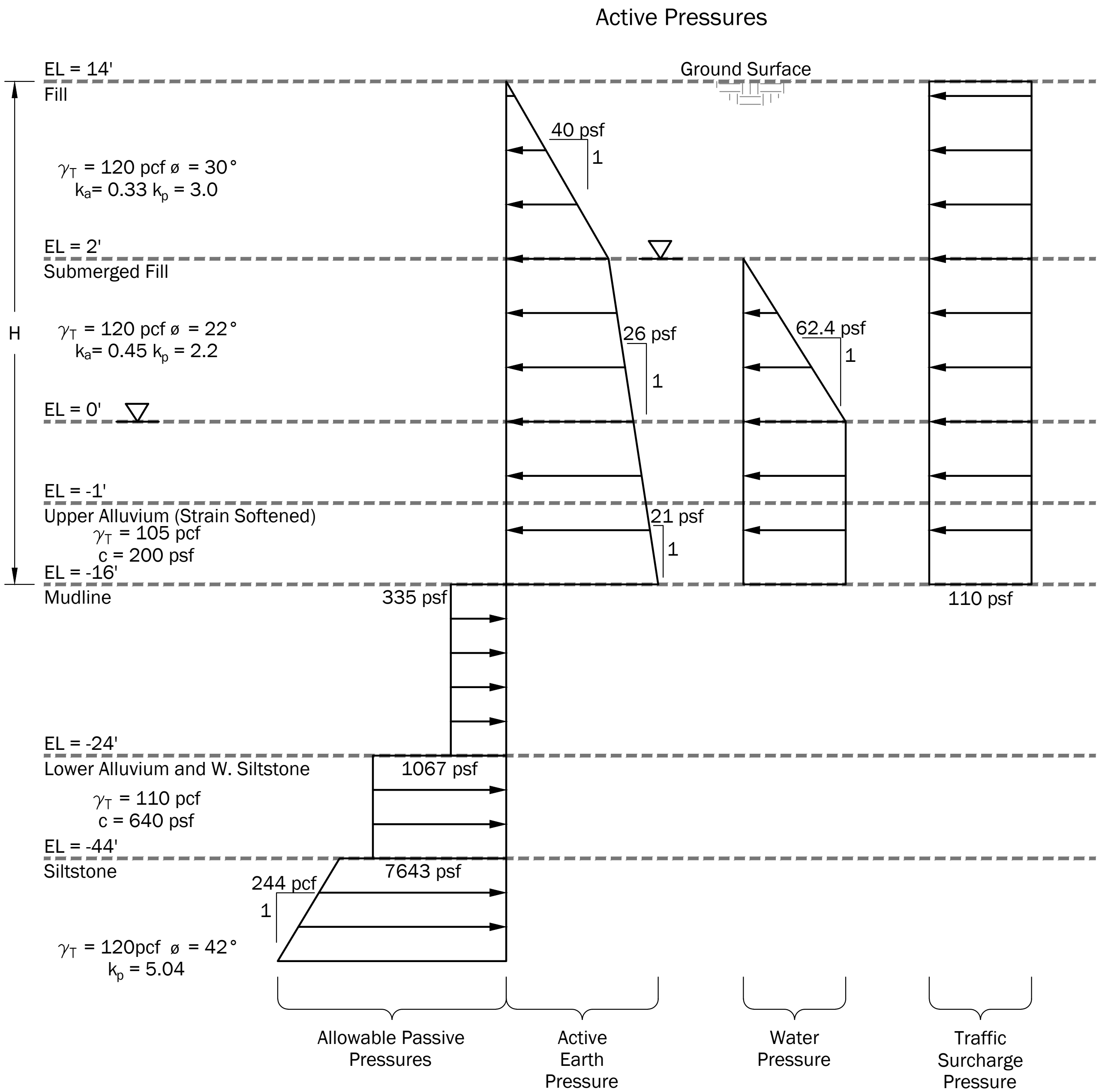
Figure 4

Post-Seismic Conditions Cantilever Wall



Legend

H = Exposed Wall Height

Not To Scale

Earth Pressure Diagram
Post-Seismic Conditions Cantilever Wall

Port Ilwaco Bulkhead Replacement
Ilwaco, Washington
Pacific County

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Figure 5

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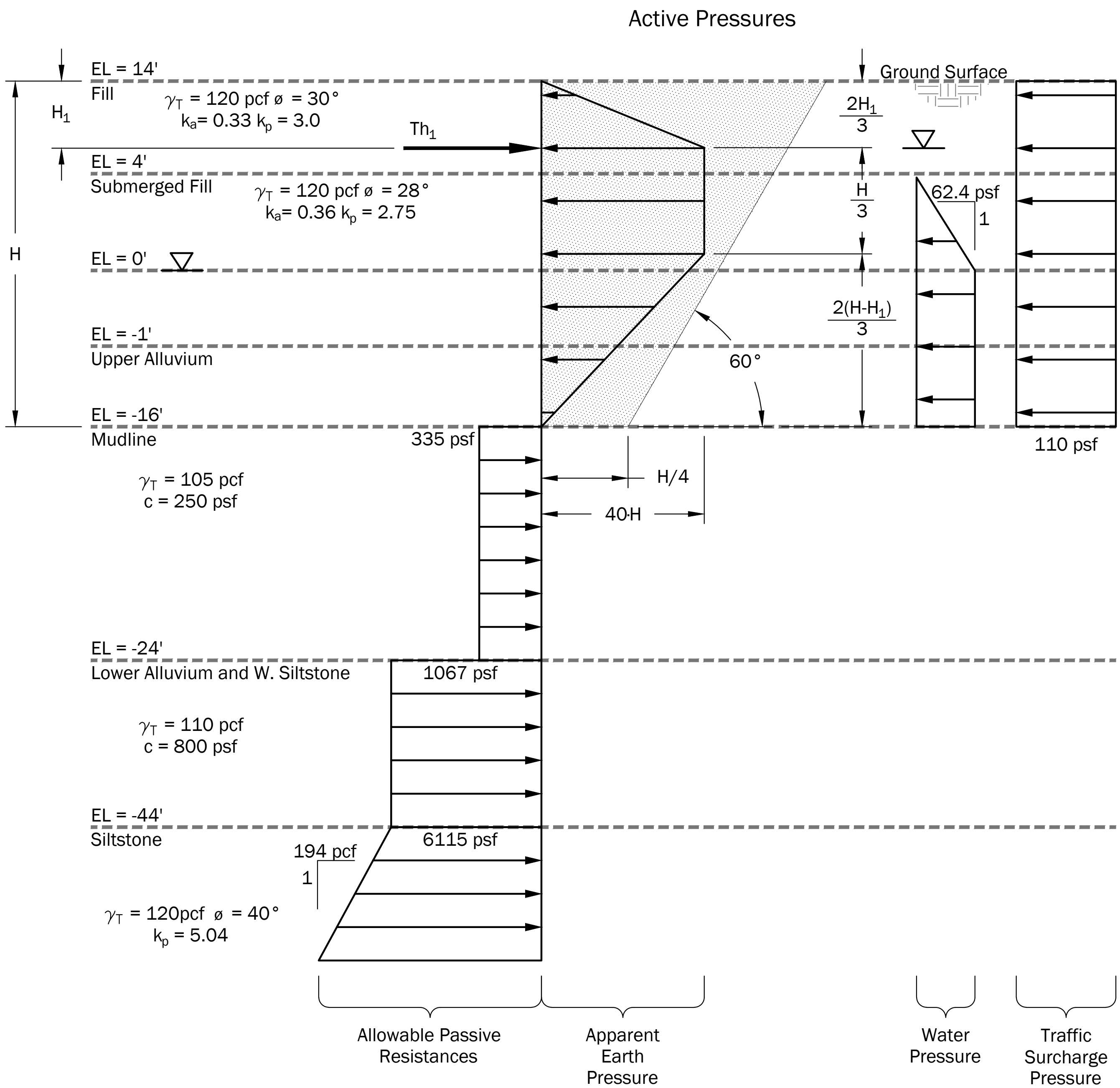
Static Conditions Wall with Tieback



Not To Scale

Legend

No Load Zone

H = Exposed Wall Height

H_1 = Distance from Ground Surface to Upper most Tieback

Th_1 = Horizontal Load in Uppermost Ground Anchor

Earth Pressure Diagram
Static Conditions Wall with Tieback

Port Ilwaco Bulkhead Replacement
Ilwaco, Washington
Pacific County

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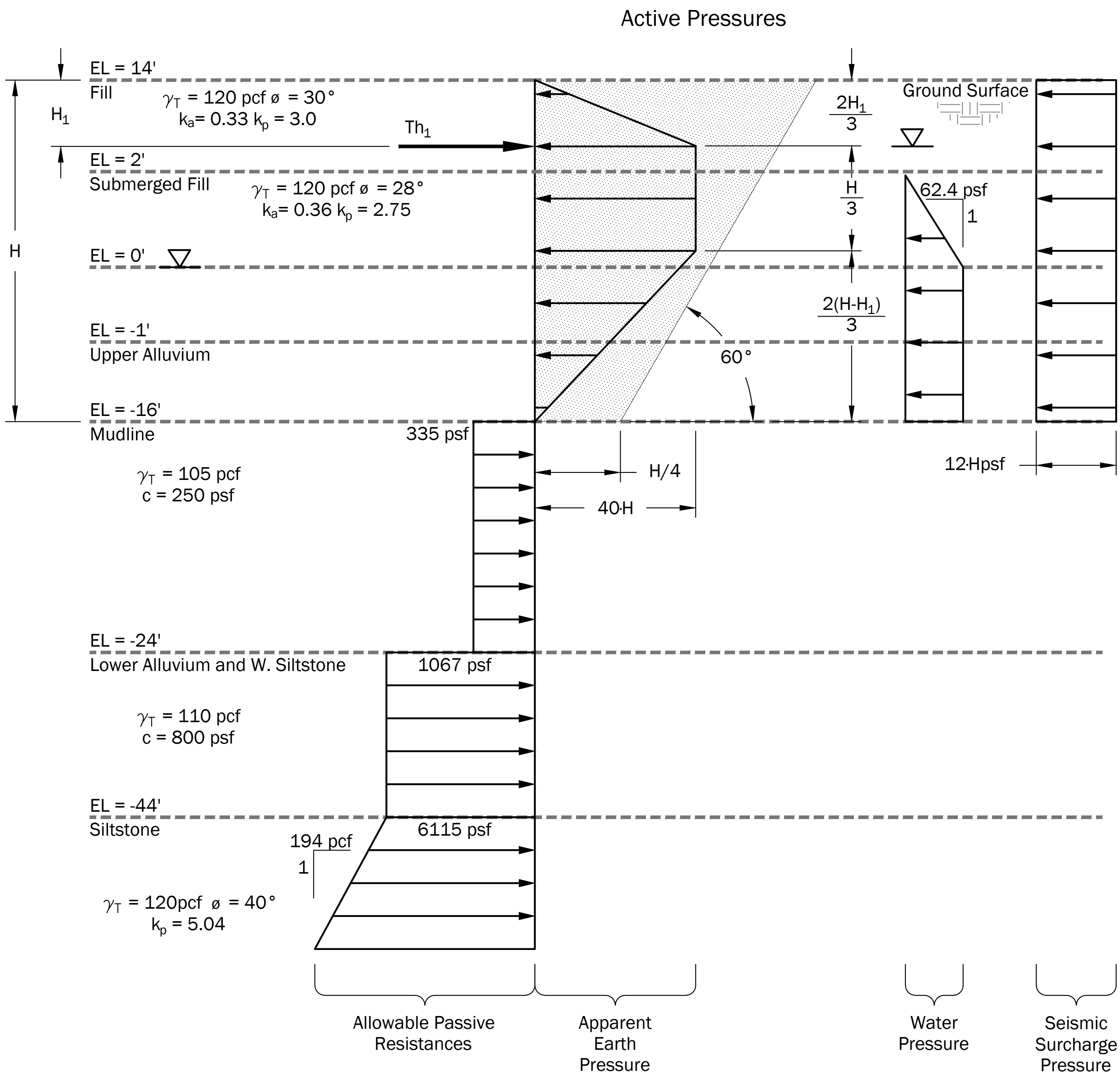
Figure 6

Seismic Conditions Wall with Tieback



Legend

No Load Zone

H = Exposed Wall Height

H_1 = Distance from Ground Surface to Upper most Tieback

Th_1 = Horizontal Load in Uppermost Ground Anchor

Not To Scale

Notes:

1. Apply uniform 110 psf lateral traffic surcharge over distance H.

Earth Pressure Diagram
Seismic Conditions Wall with Tieback

Port Ilwaco Bulkhead Replacement
Ilwaco, Washington
Pacific County

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Figure 7

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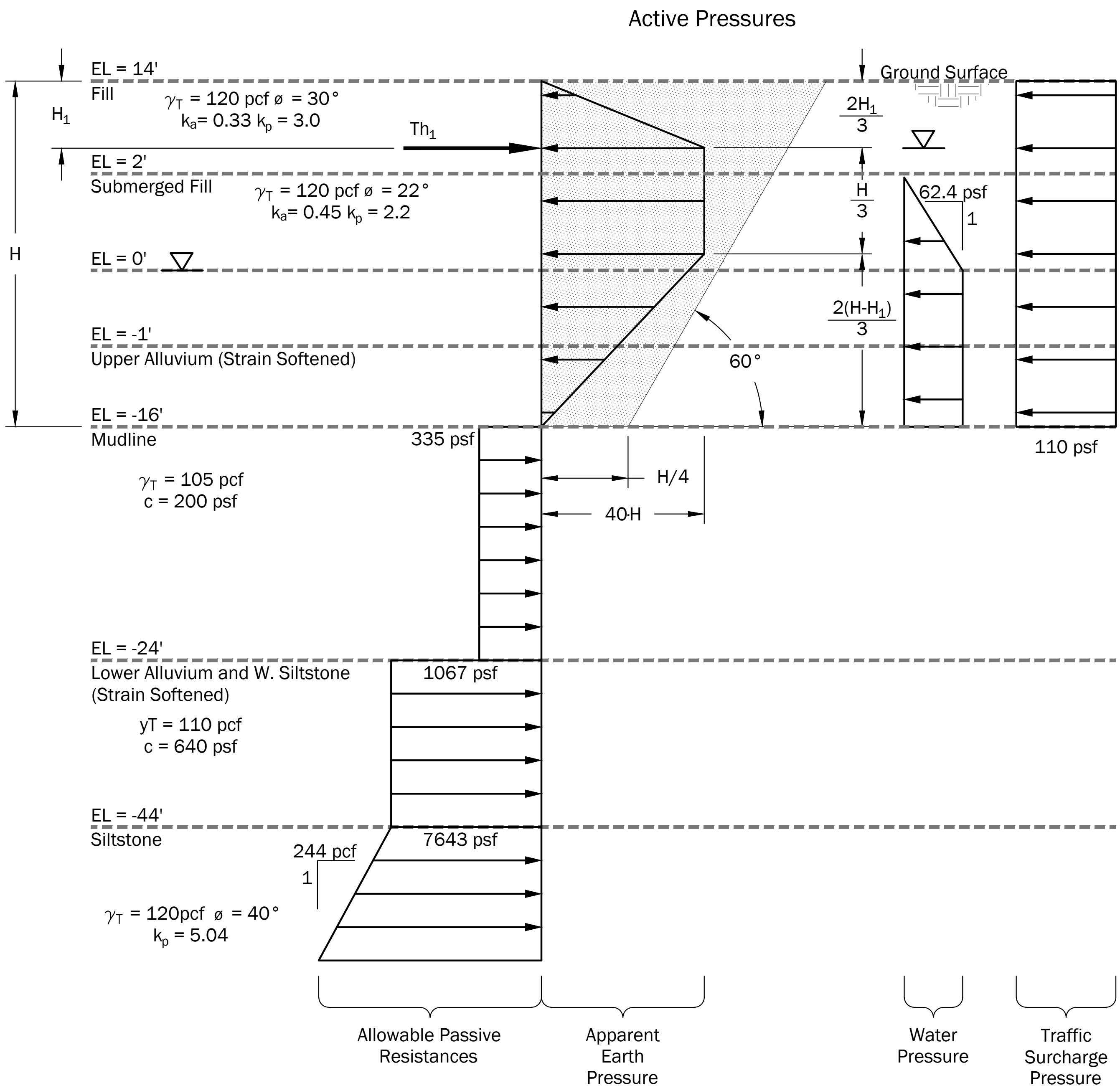
Post-Seismic Conditions Wall with Tieback



Not To Scale

Legend

No Load Zone

H = Exposed Wall Height

H_1 = Distance from Ground Surface to Upper most Tieback

Th_1 = Horizontal Load in Uppermost Ground Anchor

Earth Pressure Diagram Post-Seismic Conditions Wall with Tieback	
Port Ilwaco Bulkhead Replacement Ilwaco, Washington Pacific County	
GeoEngineers	Figure 8

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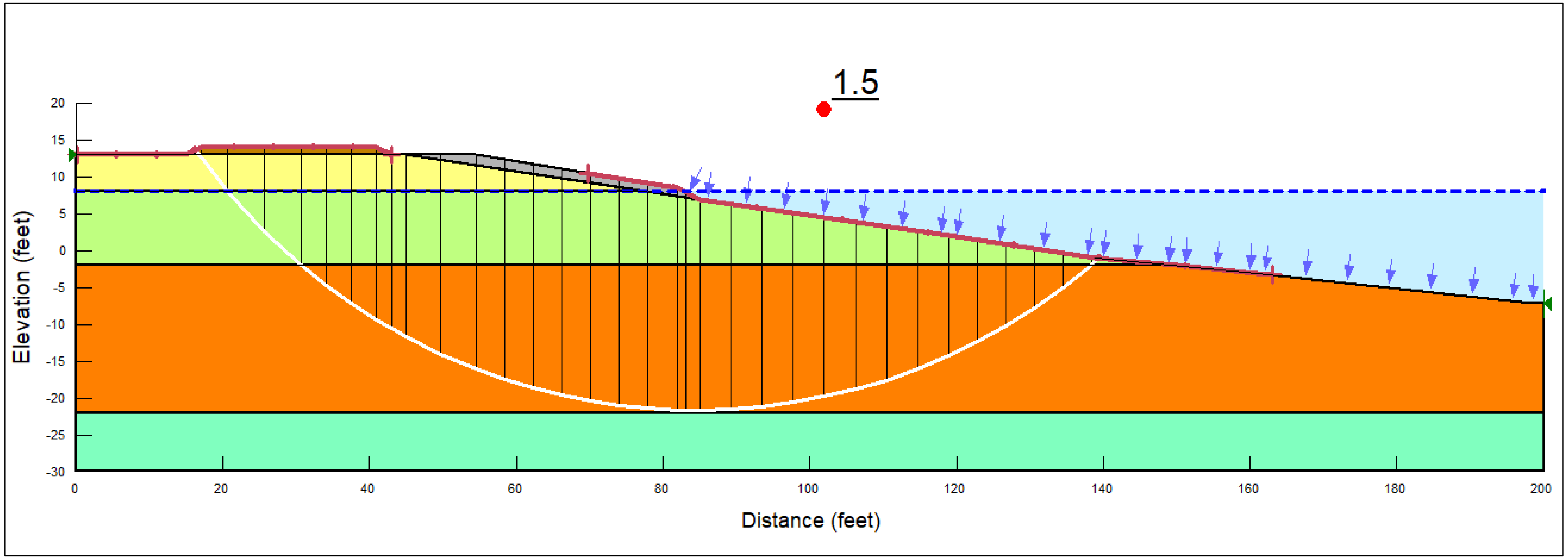
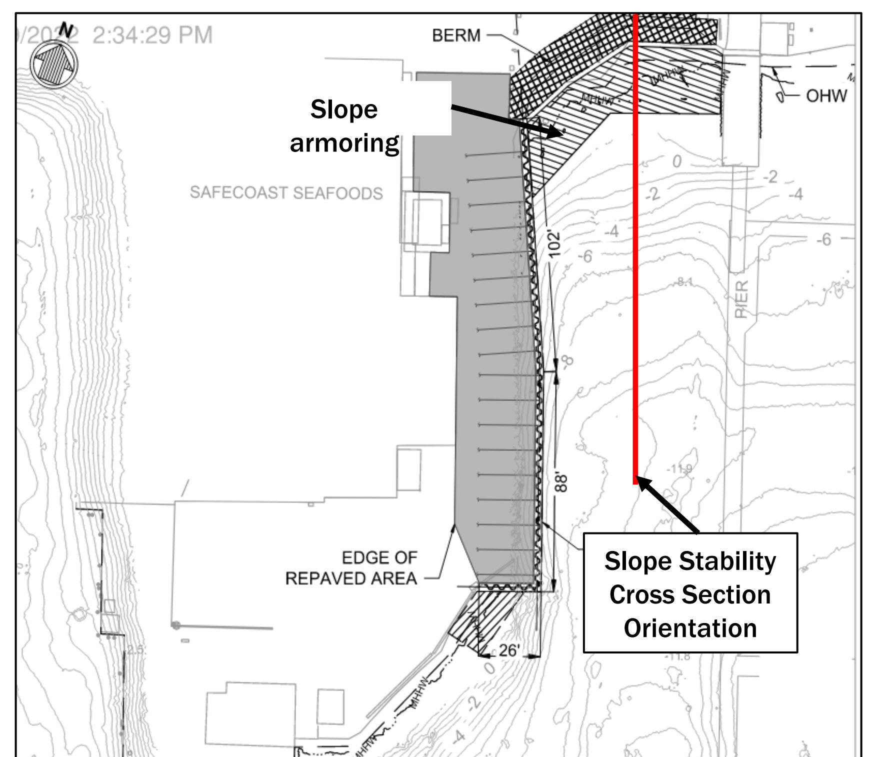


Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Berm	120	0	38
	Fill	120	0	30
	Lower Alluvium	110	800	0
	Lower Fill	120	0	28
	Rip Rap	105	0	48
	Upper Aluvium	105	250	0

Soil Properties Used in Slope Stability Analyses



Not to Scale

Slope Stability Analysis Results
Slope Northeast of Bulkhead

Port of Ilwaco Bulkhead Replacement
Ilwaco, Washington

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Figure 9

Notes:
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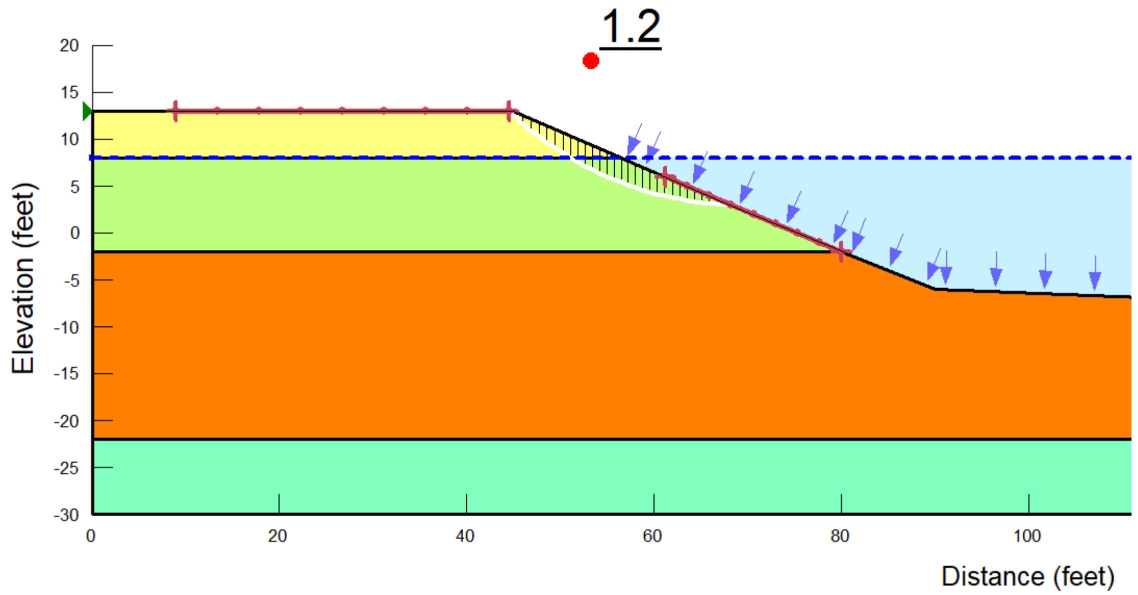
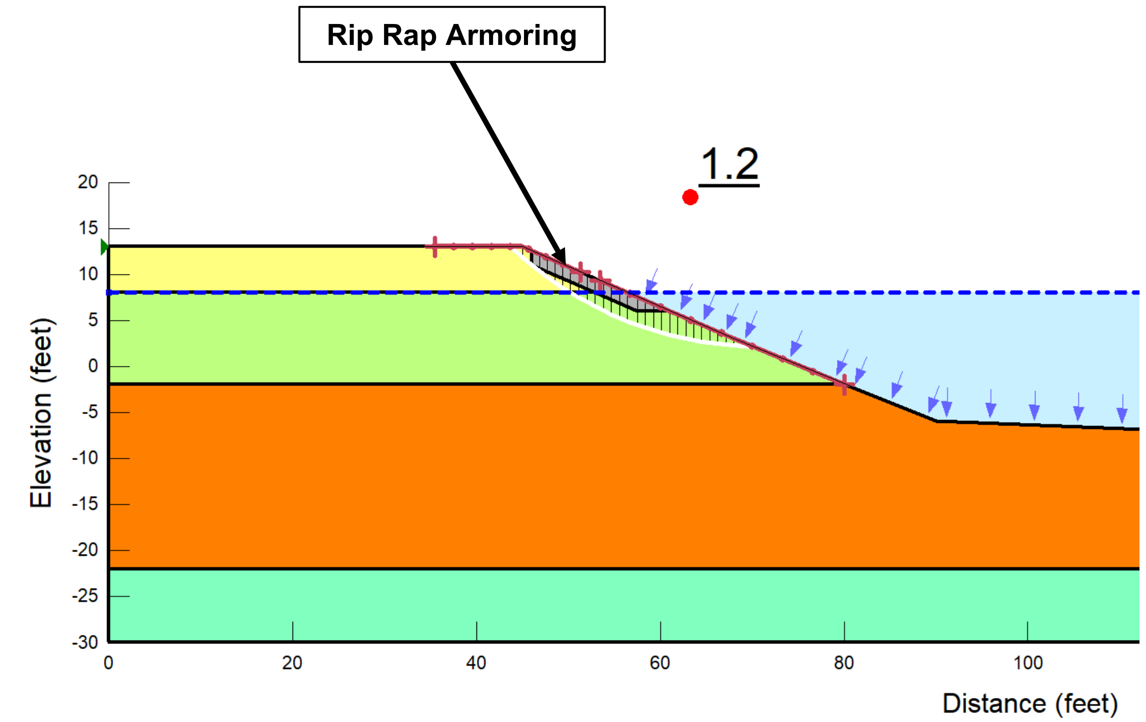


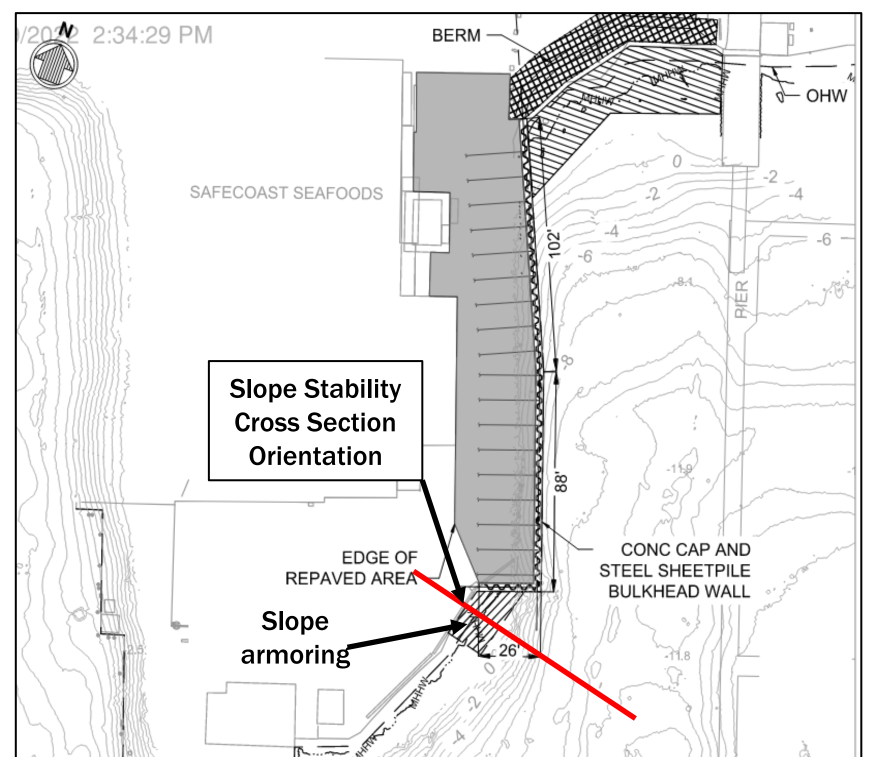
Existing Conditions



Proposed Conditions

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Berm	120	0	38
	Fill	120	0	30
	Lower Alluvium	110	800	0
	Lower Fill	120	0	28
	Rip Rap	105	0	48
	Upper Aluvium	105	250	0

Soil Properties Used in Slope Stability Analyses



Not to Scale

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Slope Stability Analysis Results
Slope South of Bulkhead

Port of Ilwaco Bulkhead Replacement
Ilwaco, Washington

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Figure 10

APPENDICES

APPENDIX A
Subsurface Explorations and Laboratory Testing

APPENDIX A
SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

Subsurface Explorations

Soil and groundwater conditions at the site were explored by completing two borings on March 14, 2022 (B-2A) and May 19, 2022. Locations of the explorations are shown on Figure 2, Site Plan. Locations of the explorations were determined in the field using an electronic tablet with global positioning system (GPS) software. The locations and elevations of the explorations should be considered approximate.

During our site explorations on March 14, 2022 and our time on site during March 15, 2022, we used a vacuum truck to attempt an additional 6 boring locations on the wharf. Each boring met refusal, at depths varying from 3 to 4.3 feet, due to undocumented and abandoned utility lines, or large cobbles. Based on the presence of cobbles, the project team decided that continuing to attempt hollow-stem auger drilling within the wharf footprint was not effective. We therefore returned to the site on May 19, 2022 with a sonic drill rig capable of easily advancing through cobbles.

Boring B-1D was performed using a Terrasonic CC150 sonic track drill rig provided and operated by Holt Drilling, Inc. under subcontract to GeoEngineers. Boring B-2A was performed using a Diedrich D70 Turbo Track drill rig provided and operated by Holocene Drilling, Inc. under subcontract to GeoEngineers. Borings were advanced using hollow-stem auger and Sonic drilling methods to nominal depths of approximately 70 (B-2) and 65 (B-1) feet below surrounding grade. Standard Penetration Tests (SPT) were completed using a 1.475-inch inner-diameter split-barrel sampler driven into the soil using a 140-pound hammer free-falling a distance of 30 inches. The number of blows required to drive the sampler the last 12 inches or other indicated distance is recorded on the logs as the blow count. SPTs were advanced at 5-foot intervals. Continuous sonic sampling was also conducted between SPT Samples for B-1).

During the exploration program our field representative obtained soil samples, classified the soils, maintained a detailed log of each exploration, and observed groundwater conditions. Soils were classified visually in general accordance with ASTM International (ASTM) D 2488. Figure A-1 includes a Key to Exploration Logs. Summary logs of the explorations are included as Figures A-2 through A-3, Logs of Borings. The densities noted on the boring exploration logs are based on the blow counts produced in the SPT and our experience and judgment.

Borings were backfilled by the driller in accordance with Washington State Department of Ecology requirements.

Laboratory Test Results

Soil samples obtained from the explorations were retained in sealed plastic bags and transported to the GeoEngineers' laboratory. Representative soil samples were selected for laboratory tests to evaluate pertinent geotechnical engineering characteristics of the soils and refine our field classification, as necessary. The following paragraphs provide a description of the tests performed.

Atterberg Limits Testing

Atterberg Limits were performed on selected samples in general accordance with ASTM Test Method D4318. This test method determines the liquid limit, plastic limit, and plasticity index of soil particles passing the U.S. No. 40 sieve. Results for plastic soils are presented in Figure A-4, Atterberg Limits Test Results. The liquid limit and plasticity index are also presented on the exploration logs at the respective sample depths.

Moisture Content (MC)

The moisture content of selected samples was determined in general accordance with ASTM Test Method D 2216. Test results are presented on the exploration logs at the respective sample depths.

Percent Fines (%F)

Selected samples were "washed" through the U.S. No. 200 sieve to estimate the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve (fines). Tests were conducted in general accordance with ASTM D 1140. Test results are presented on the exploration logs at the respective sample depths.

Particle Size Gradation - Sieve Analysis (SA)

Sieve analyses were performed on selected samples in general accordance with ASTM Test Method D 6913. This test method covers the quantitative determination of the distribution of particle sizes in soils. Typically, the distribution of particle sizes larger than 75 micrometers (μm) is determined by sieving. The results of the tests were used to verify field soil classifications. Figures A-23 and A-24 present the results of our sieve analyses.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

Sampler Symbol Descriptions

- 2.4-inch I.D. split barrel / Dames & Moore (D&M)
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Direct-Push
- Bulk or grab
- Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point lead test
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
UU	Unconsolidated undrained triaxial compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

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Figure A-1

Drilled	Start 5/19/2022	End 5/19/2022	Total Depth (ft)	65.5	Logged By LSP, Checked By BEL	Driller Holt Drilling, Inc.	Drilling Method Sonic

Surface Elevation (ft) 11, Vertical Datum NAVD88	Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Terrasonic CC150
Easting (X) 745863, Northing (Y) 374360	System Datum	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
10	0						AC	Approximately 3 inches of asphalt concrete			
							GP-GM	Brown fine to coarse gravel with silt, sand and cobbles (angular ballast rock) (loose, moist) (fill)			
5	5		6	6		1A		Becomes wet			Groundwater observed at 5 feet during drilling
			6			1B / SA	SM	Brown silty fine to medium sand (loose, wet) (submerged fill)	24	24	Driller noted smoother drilling at 6 feet
							ML	Dark gray sandy silt (soft, wet)			
0	10		16	2		2A / %F			59	69	
						2B / %F	SM	Dark gray silty fine sand (very loose, wet)	39	46	
			12			3A / AL	CH	Dark gray high plasticity clay with sand and occasional wood debris (very soft, wet) (upper alluvium)	55		AL (LL = 55; PI = 27)
							SM	Dark gray silty sand with occasional wood debris (very loose, wet)			
-5	15		13	0		3B / %F			39	39	
							MH	Gray elastic silt (very soft, wet)			
-10	20		18	1		4 / MC			50		
							MH	Gray sandy silt with occasional wood debris and shells (very soft, wet)			
-15	25		18	1		5 / MC	MH	Dark gray elastic silt (very soft, wet)	49		
							ML	Gray silt, wood debris (soft, wet) (lower alluvium)			
-20	30		18	2		6 / MC			69		
	35										

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on commercial grade GPS. Vertical approximated based on Google Earth.

Log of Boring B-1D

GEOENGINEERS

Project: Port of Ilwaco Marina Structure Replacement
Project Location: Ilwaco, Washington
Project Number: 21551-003-00

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
-25	35		18	3		7						
-30	40		18	5		8A MC / 8B MC		ML	Gray silt (medium stiff, moist) (weathered siltstone)	63 / 59		
-35	45		18	7		9						
									Becomes very stiff			
-40	50		18	26		10						Sample observed to crumble into semi-intact pieces of siltstone
-45	55		17	84		11		ML	Gray silt (hard, moist) (siltsone)			Siltstone pieces appeared more intact and distinct increase in density/consistency noted
-50	60		17	86		12						
	65		6	50/6"		13						

Log of Boring B-1D (continued)

GEOENGINEERS

Project: Port of Ilwaco Marina Structure Replacement

Project Location: Ilwaco, Washington

Project Number: 21551-003-00

Drilled	Start 3/14/2022	End 3/14/2022	Total Depth (ft) 70.5	Logged By LSP / Checked By BEL	Driller Holocene Drilling, Inc.	Drilling Method Mud Rotary

Surface Elevation (ft) 11 Vertical Datum NAVD88	Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment: Track-mounted Diedrich D70 Turbo
Easting (X) 745847 Northing (Y) 374488	System Datum	See "Remarks" section for groundwater observed

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
10	0							SM	Approximately 2 inches of dark brown silty fine sand (loose, moist)			Vac truck used to 5 feet bgs
								GP-GM	Brown fine to coarse gravel with silt and sand (medium dense, moist) (fill)			
5	5		9	5		1		SM	Gray silty fine to medium sand with gravel (loose, wet) (submerged fill)			Groundwater observed at 5 feet during drilling
			18	0		2 / MC		ML	Gray silt with occasional sand (very soft, wet)	81		
0	10		10	5		3 / %F		SM	Gray silty fine sand with occasional gravel (loose, wet)	39	38	
-5	15		10	0		4 / AL		MH	Gray elastic silt with sand (very soft, wet) (upper alluvium)	58		AL (LL = 53; PI = 24)
-10	20		11	0		5 / %F		SM	Gray silty fine sand (very loose, wet)	37	39	
								CL	Gray fat clay (very soft, wet)			
-15	25		18	1		6 / AL				47		AL (LL = 51; PI = 23)
-20	30		18	0		7						
	35											

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on commercial grade GPS. Vertical approximated based on Google Earth.

Log of Boring B-2A

GeoEngineers

Project: Port of Ilwaco Marina Structure Replacement
Project Location: Ilwaco, Washington
Project Number: 21551-003-00

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
-25	35	18	1	8 / MC			66		
-30	40	18	5	9 / MC	CH	Grayish brown fat clay with occasional fibrous organic matter (wood fragments) (medium stiff, wet) (lower alluvium)	81		
-35	45	18	6	10 / AL			54		AL (LL = 80; PI = 50)
-40	50	18	5	11A	SM	Brownish gray silty fine to medium sand with organic matter (loose, wet)			
				11B	MH	Gray elastic silt (medium stiff, wet)			
-45	55	18	2	12 / MC		Becomes soft	48		
-50	60	14	25	13	ML	Gray silt (medium dense, moist) (weathered siltstone)			Driller noted harder drilling at 59 feet bgs Sample observed to crumble into semi-intact pieces of siltstone
-55	65	9	50/3"	14	ML	Gray silt (hard, moist) (siltsone)			Siltstone pieces appeared more intact and distinct increase in density/consistency noted
	70	4	50/4"	15					

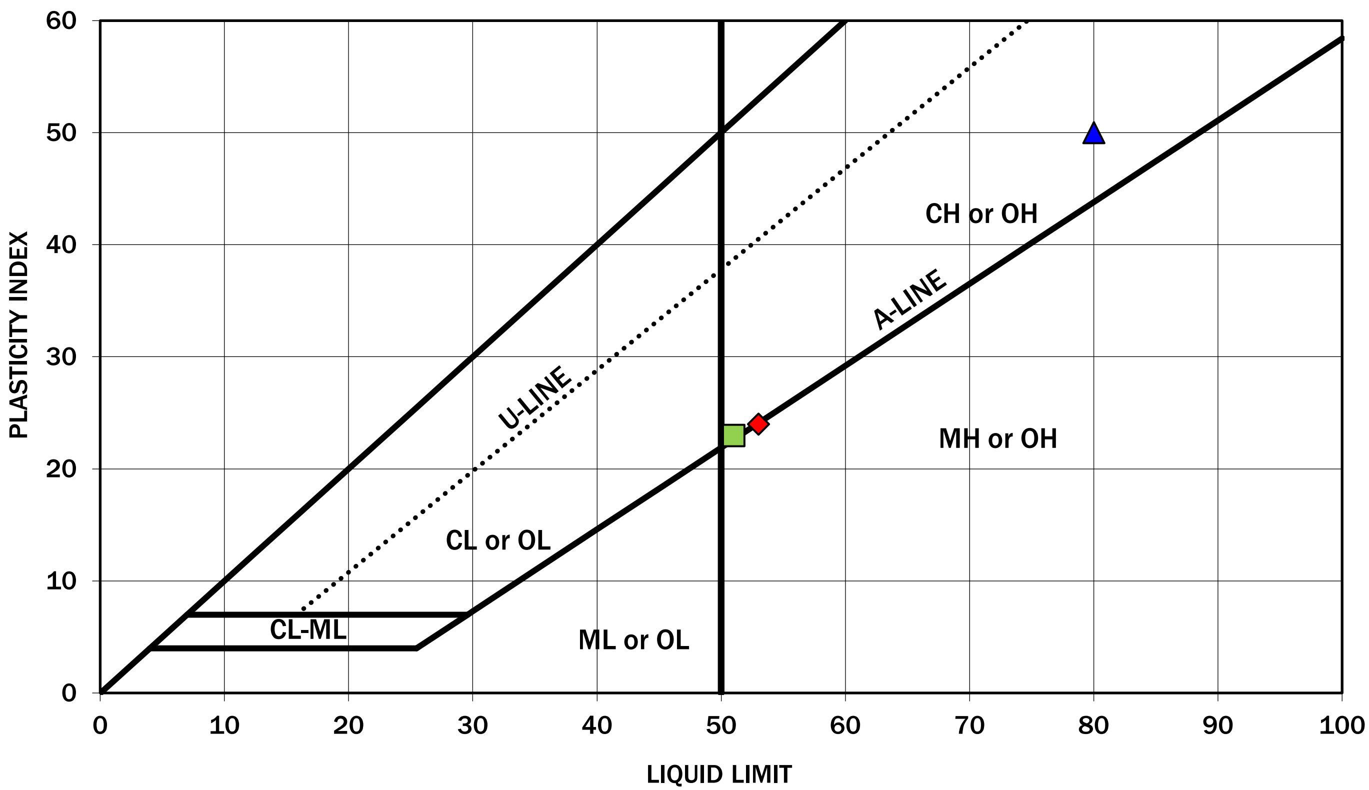
Log of Boring B-2A (continued)

GeoEngineers

Project: Port of Ilwaco Marina Structure Replacement
Project Location: Ilwaco, Washington
Project Number: 21551-003-00

Figure A-3
Sheet 2 of 2

PLASTICITY CHART



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆	B-2A	15	58	53	24	Elastic Silt (MH)
■	B-2A	25	47	51	23	Fat Clay (CH)
▲	B-2A	45	54	80	50	Fat clay (CH)

 The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

Atterberg Limits Test Results

Port of Ilwaco
Ilwaco, Washington

GEOENGINEERS

Figure A-4

APPENDIX B
Report Limitations and Guidelines for Use

APPENDIX B
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology, and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for Moffatt & Nichol and for the Project specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our Agreement with Moffatt & Nichol dated January 25, 2022 and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A Geotechnical Engineering or Geologic Report is based on a Unique Set of Project-Specific Factors

This report has been prepared for the Port of Ilwaco, Marina Structures Replacement and Dredging, Engineering, and Permitting located in Ilwaco, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- Not prepared for you,
- Not prepared for your project,
- Not prepared for the specific site explored, or
- Completed before important project changes were made.

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[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

For example, changes that can affect the applicability of this report include those that affect:

- The function of the proposed structure;
- Elevation, configuration, location, orientation, or weight of the proposed structure;
- Composition of the design team; or
- Project ownership.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

Information Provided by Others

GeoEngineers has relied upon certain data or information provided or compiled by others in the performance of our services. Although we use sources that we reasonably believe to be trustworthy, GeoEngineers cannot warrant or guarantee the accuracy or completeness of information provided or compiled by others.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical and Geologic Findings are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations are Not Final

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- Advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- Encourages contractors to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

Contractors are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.

GeoEngineers