Ilwaco Marina Conceptual Engineering Report

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1. Introduction and Background

Moffatt & Nichol (M&N) was retained by the Port of Ilwaco (the Port) for conceptual engineering design services for bulkhead repair, shoreline protection, and repair of various structures in the northwest portion of the Ilwaco Marina. The goal of this work was to complete an onsite condition assessment and conceptual engineering for repair and/or replacement of structures and slope protection in the three areas referred to as the “TraveLift Pier”, “Safe Coast Seafoods Wharf East Bulkhead” (formerly known as “Jessie’s Wharf East Bulkhead”), and the “West Gangway Access Pier”.

The Port of Ilwaco is located in Ilwaco, Washington on the north bank of Baker Bay, immediately east of the mouth of the Columbia River as shown on Figure 1.1. The Port area generally consists of a marina used for year-round moorage of recreational and commercial fishing vessels, upland commercial buildings, and a boatyard. The boatyard is located at the northwest end of the marina and includes the TraveLift Pier. The TraveLift Pier was constructed in the late 1970’s and is used to haul out large vessels.

As shown in Figure 1.2, several commercial buildings, including the Safe Coast Seafoods building, are located along the marina’s northern shoreline. The buildings associated with Safe Coast Seafoods are located east of the boatyard and TraveLift Pier. The Safe Coast Seafoods buildings are located on a timber pile supported wharf on the west side of the property and an earth-filled wharf on the east side. The earth-filled structure is protected by riprap armored slopes on its west and south sides and a timber bulkhead along the eastern limits of the wharf. The timber pile supported wharf structure along the west side of the Safe Coast Seafoods property is not included in the scope of this project.

The marina moorage is accessed from the shoreline by four timber access piers that support gangways to the marina floats. The West Access Pier is one of the main entry points for the marina for both pedestrians and utilities. The general arrangement of the Port and the facilities inspected is shown in Figure 1.2.

FIGURE 1.1: VICINITY MAP

FIGURE 1.2: PORT OF ILWACO

1.1. Scope of Work

Our scope of work was performed in general accordance with our proposal dated 28 August 2020. The project activities included condition assessment and conceptual design for repair and/or replacements of various structures and adjacent shoreline slopes in the three areas shown in Figure 1.2. The project areas and anticipated project components are summarized in Table 1.1.

TABLE 1.1. PROJECT AREAS AND ANTICIPATED WORK ACTIVITIES

Project Area	Location	Anticipated Project Activity
TraveLift Pier	Adjacent shoreline and abutment	Riprap repair or replacement
	TraveLift Pier structure – upgrade capacity from 50 metric tons (MT)to 75 MT	Condition assessment of existing pier (replacement is required based on conceptual review).
Safe Coast Seafoods Wharf	East Bulkhead (timber)	Bulkhead replacement (ground improvement and steel sheet piles to be considered)
	Wharf south slope protection	Riprap repair, or replacement with a bulkhead
	Miscellaneous piles (SE corner)	Replace, repair, or remove piles
West Gangway Access Pier	Slope protection	Riprap repair or replacement
	West Gangway Access Pier	Repair or replacement

Task 1 – Project Management

M&N provided scheduling, coordination meetings, invoicing, and administrative support to manage the project.

Task 2 – Conceptual Design

Task 2.1 – Site Visit / Condition Assessment

M&N completed a site visit to document existing site conditions. The condition assessments were conducted by an engineering team and a biologist. It was initially anticipated that a dive team would be mobilized for the underwater inspection of the TraveLift Pier and the Safe Coast Seafoods Wharf East Bulkhead. However, preliminary concept design efforts determined that the retrofit of the existing TraveLift Pier was considered impractical (to achieve increased load capacity and a raised grade), and due to the current condition of the East Bulkhead, repair was not considered feasible. Therefore, it was determined that the facility condition assessment could be conducted as an above-water inspection only. The condition assessment included the following:

- Observed and documented the above-water conditions of the shoreline and structures in the three project areas.
- Collected preliminary mudline depth data in the vicinity of the TraveLift structure, the Safe Coast Seafoods Wharf East Bulkhead, and the West Gangway project areas and located the Ordinary High-Water Line.
- Took photos and recorded narrative descriptions of shoreline conditions, vegetation, erosion areas, and riprap slope protection.

Deliverables:

- Prepared a memorandum summarizing the conditions observed and providing general recommendations for design of conceptual alternatives.

Task 2.2 – Conceptual Geotechnical Consultation

M&N coordinated with GeoEngineers (contracted separately by the Port of Ilwaco) for conceptual geotechnical consultation needed to identify realistic and feasible options to address the vulnerability of the site soils to seismic-induced liquefaction and lateral spreading.

Deliverables:

- GeoEngineers provided a memorandum with a summary of anticipated soil conditions and conceptual geotechnical engineering recommendations for TraveLift Pier pile capacity, and soil properties to be used for the East Bulkhead replacement, the West Access Pier, and repair of the shoreline protection in the vicinity of those structures.

Task 2.3 – Slope Protection

Slope protection engineering included evaluating slope stability, existing materials and conditions, and recommendations for repairs to reduce potential erosion and improve slope stability. Increasing the height of the protection relative to historic flood levels and anticipated sea level rise was considered for each project area.

Deliverables:

- Conceptual plan view and typical cross section depicting recommended slope protection repairs/improvements.
- Narrative describing the anticipated materials, quantities, construction methods, and appropriate BMPs to be used.
- Concept-level (-30% to +50% accuracy) opinion of probable construction cost for each alternative.

Task 2.4 – Safe Coast Seafoods Wharf East Bulkhead

Conceptual design for replacement of the Safe Coast Seafoods Wharf East Bulkhead included conceptual evaluation of slope stability, loading conditions, and the bulkhead height increase required for flooding reduction and future sea level rise for two alternatives.

The work also included identifying the use, need, and condition of a group of piles ("miscellaneous piles") located near the southeast corner of the Safe Coast Seafoods Wharf and determining if removal, repair, or replacement of these piles is needed.

Deliverables:

- Conceptual plan views and typical cross sections depicting two alternatives to be considered showing the existing conditions and proposed improvements.
- Narrative describing the anticipated materials, quantities, construction methods, and appropriate BMPs to be used for bulkhead replacement.
- Concept-level (-30% to +50% accuracy) opinion of probable construction cost for each alternative.

Task 2.5 – TraveLift Pier

Our conceptual review of the existing TraveLift pier indicates that repair and upgrade of the structure will not be practical to achieve the 75 metric ton capacity of the Port's recently purchased new TraveLift hoist. Upgrade of the pier structure was not considered further. We investigated two concepts for replacement of the structure.

Deliverables:

- Conceptual plan views and typical cross sections depicting the adjacent slope protection to be considered and two alternatives for replacement of the structure.
- Narrative describing the anticipated materials, quantities, construction methods, and appropriate BMPs to be used.
- Concept-level (-30% to +50% accuracy) opinion of probable construction cost for each alternative.

Task 2.6 – West Gangway Access Pier

Based on the results of the site visit and the evaluation performed in Task 2.1, the West Gangway Access Pier will be repaired or replaced. Observations of water levels at the site during high tides and/or storm events revealed the vulnerability of the pier deck and utilities to those high-water levels.

Two conceptual alternatives were developed: repair; or replace. Options such as raising the deck elevation and relocating the electrical service mounting locations were also considered to increase the resiliency of the access pier.

Deliverables:

- Conceptual plan views and typical cross sections depicting two alternatives to be considered showing the existing conditions and proposed improvements.
- Narrative describing anticipated materials, quantities, construction methods, and appropriate BMPs to be used.
- Concept-level (-30% to +50% accuracy) opinion of probable construction cost for each alternative.

2. Condition Assessment of Existing Facilities

Moffatt & Nichol (M&N) conducted a condition assessment of the TraveLift Pier, Safe Coast Seafoods Wharf East Bulkhead, Marina West Access Pier, and the shoreline protection at each structure. The scope of work included above-water inspection of facilities and adjacent shorelines, perimeter mapping, and preliminary bathymetric measurements. The observations noted in the field were analyzed to ascertain a condition assessment rating for the structures and determine repair or replacement recommendations.

An overall Condition Assessment Rating (CAR) was assigned to these three facilities. The CARs were based on the findings of the visual observations. The condition assessment scale includes the following six categories: Good, Satisfactory, Fair, Poor, Serious, and Critical. The six CARs and their descriptions are provided as an attachment to this report.

2.1. Existing Topography and Bathymetry

The existing topography and bathymetry were determined using handheld GPS measurements, Google Earth, and water depth soundings. These approximate elevations were used to provide a basis for the recommendations regarding upland grade changes, and water depths for the conceptual design efforts discussed later in this report. Figures 2.1 and 2.2 report the elevations recorded and used for this project. As can be seen, the upland elevations range from EL +9.0 ft MLLW to EL +13.0 ft MLLW with the lowest elevations observed around the Safe Coast Seafoods Wharf. Higher elevations were observed at the east end of the marina basin.

FIGURE 2.1: EXISTING APPROXIMATE TOPOGRAPHY

FIGURE 2.2: EXISTING APPROXIMATE BATHYMETRY

2.2. TraveLift Pier

The TraveLift Pier condition is rated as "*Fair*". All primary structural elements are sound; but minor to moderate defects and deterioration were observed. Localized areas of moderate deterioration are present on the concrete apron structure, but do not significantly reduce its structural capacity. The localized deterioration on the apron structure does not currently affect the structural capacity of the TraveLift Pier itself.

The following repairs are recommended to maintain the facility:

- Install shoreline protection along the shoreline.
- Restore fill supporting the concrete vault located between the two runways, beneath the deck.

Replacement of the existing TraveLift Pier is discussed in the design section of this report as an alternative that would increase the capacity of the structure to accommodate the larger lift equipment recently purchased by the Port.

FIGURE 2.3: TRAVELIFT PIER

FIGURE 2.4: TRAVELIFT PIER ABUTMENT AND ADJACENT SHORLINE

2.3. Safe Coast Seafoods Wharf East Bulkhead

The East Bulkhead is rated as “Serious”. Advanced deterioration and breakage have affected the load-bearing capacity of the bulkhead. The bulkhead experiences overtopping during extreme storm and tidal events. Due to the extent and nature of the deterioration, as well as overtopping of the bulkhead, it is assumed that repairing the structure in-kind is cost prohibitive, therefore, alternatives for replacement of the bulkhead were evaluated.

FIGURE 2.5: EAST BULKHEAD

FIGURE 2.6: EAST BULKHEAD AND ADJACENT SHORLINE

2.4. West Access Pier

The West Access Pier is rated as "*Serious*". Advanced deterioration of the timber piling has affected the load-bearing capacity of the pier. Due to the extent and nature of the deterioration, it is assumed that repairing the structure in-kind is cost prohibitive, therefore, alternatives for replacement of the pier should be evaluated. If full replacement is not the preferred alternative, the following repairs are recommended:

- Install structural pile jackets on all piles (nine total).
- Restore Piles 4:B and Pile 3:C to full bearing, and install steel straps to secure the piles to the pile cap.
- Repair broken electrical fittings.
- Replace cracked water flex hose.

FIGURE 2.7: WEST ACCESS PIER

FIGURE 2.8: WEST ACCESS PIER UNDER DECK AND UTILITIES

3. Conceptual Design

The conceptual design effort of the marina repairs uses the previously described condition assessment to identify repair and or replacement options that will address the observed deficiencies, increase the useable life of the marina facilities, satisfy sea level rise (SLR) concerns, and meet the operational requirements desired by the Port. The concept design for each structure is described in the following sections. Detailed sketches for each structure are provided in Appendix A. The associated cost estimates for each repair or replacement option are provided in Appendix B and discussed further in Section 4 of this report. The geotechnical recommendations used as the basis for the conceptual design were generated by GeoEngineers. Their geotechnical report is replicated in full in Appendix C. The detailed Condition Assessment Report is provided in Appendix D.

3.1. Sea Level Rise

Elevations along the top of bank for much of the marina basin are typically between EL +12.0 to EL +13.0 ft MLLW, with low point near the East Bulkhead of EL +11.0 ft MLLW. During king tides combined with extreme storm events, high tides and storm surge in the marina basin result in water levels that are within 1 to 3 feet of the top of bank. The existing top of bank is less than EL +12.0 ft MLLW between the Safe Coast Seafoods East Bulkhead and the West Access Pier and in the vicinity of the TraveLift Pier. Choppy conditions caused by high winds will increase runup in exposed areas of the shore.

The median projected sea level rise (50% exceedance) for Ilwaco for the year 2060 is 0.4 feet, with a 1% exceedance estimate of 1.3 feet. Table 3.1 summarizes average annual flood levels that would likely exceed EL +12.0 ft MLLW by 2060, with the 100-year event likely exceeding EL +14 ft MLLW.

TABLE 3.1. HIGH WATER LEVELS FOR VARIOUS 2060 SLR PROJECTIONS

Predicted High Water Levels	Sea Level Rise Scenarios			
	Current ft-MLLW	0.4ft in 2060 (50% exceedance likelihood)	0.8ft in 2060 (10% exceedance likelihood)	1.3 ft in 2060 (1% exceedance likelihood)
MHHW	8.07	8.47	8.87	9.37
Annual	11.7	12.1	12.5	13
2-yr	12.0	12.4	12.8	13.3
10-yr	12.4	12.8	13.2	13.7
25-yr	12.7	13.1	13.5	14
50-yr	13.3	13.7	14.1	14.6
100-yr	13.6	14	14.4	14.9

3.2. Shoreline Protection

3.2.1. Repair Alternatives

An adaptive strategy is proposed for the shoreline to reduce the vulnerability of the shoreline area from wave runup and overtopping, and to reduce flooding from sea level rise. The initial improvements would include raising the top of shoreline to a minimum elevation of +14.0 ft MLLW. Improvements to the existing rock slope protection (RSP) would include raising the crest (top) elevation and repairing the RSP to a stable configuration. In areas such as at the TraveLift Pier abutment, the RSP may be combined with a retaining wall in order to minimize the extent that the RSP slope extends out from shore.

Conceptual review of slope stability for the RSP (see Appendix C) indicates that structure slopes of 1.5H:1.0V to 2.0H:1.0V are only stable for a static loading case, and slope displacements and possible lateral spreading could occur due to seismic events.

For the conceptual design, the RSP structure slope is shown to be 2.0H:1.0V - stable static slope and marginally stable for a post seismic loading condition. The RSP typical section would consist of rock bedding layer and armor stone.

Two reaches of shoreline were identified for improvements to raise the elevation of the top of the shoreline embankment.

- Reach 1: In the vicinity of TraveLift Pier and the Boatyard (approximately 800 linear feet of shoreline)
- Reach 2: From the east side of the Safe Coast Seafood Wharf to the Marina West Access Pier (approximately 120 linear feet of shoreline)

3.2.2. Construction Considerations

Construction can be accomplished using land-based equipment. Equipment such as excavators or truck-mounted cranes can be used to place rock delivered to the site. If excavation is required in order to securely anchor the rock toe of the RSP, temporary measures such as turbidity curtains can be employed to minimize impacts to marina water quality.

3.3. TraveLift Pier

The TraveLift Pier is located on the west side of the Port of Ilwaco complex. An aerial photo of the area is shown in Figure 3.1. We understand the Port has replaced the original lift equipment with a new TraveLift boat hoist. The new hoist equipment has a greater lift capacity (75 metric tons) than the original TraveLift (capacity of 50 metric tons) and can be modified to be 4 feet wider. See Figure 3.2.

FIGURE 3.1: TRAVELIFT PIER CURRENT MUDLINE ELEVATIONS

As part of the Port's long-term plan for resiliency and sea level rise, the design elevation for the new pier would be +14.0 ft MLLW. Retrofit of the existing pier to accommodate the heavier loads was not considered practical and not considered further in the engineering review due to the 50% increase in equipment lift capacity and the increased pier deck elevation required. Conceptual pile capacities for tension and compression were provided by the geotechnical engineer for use in the development of conceptual-level design of the foundation for the new pier.

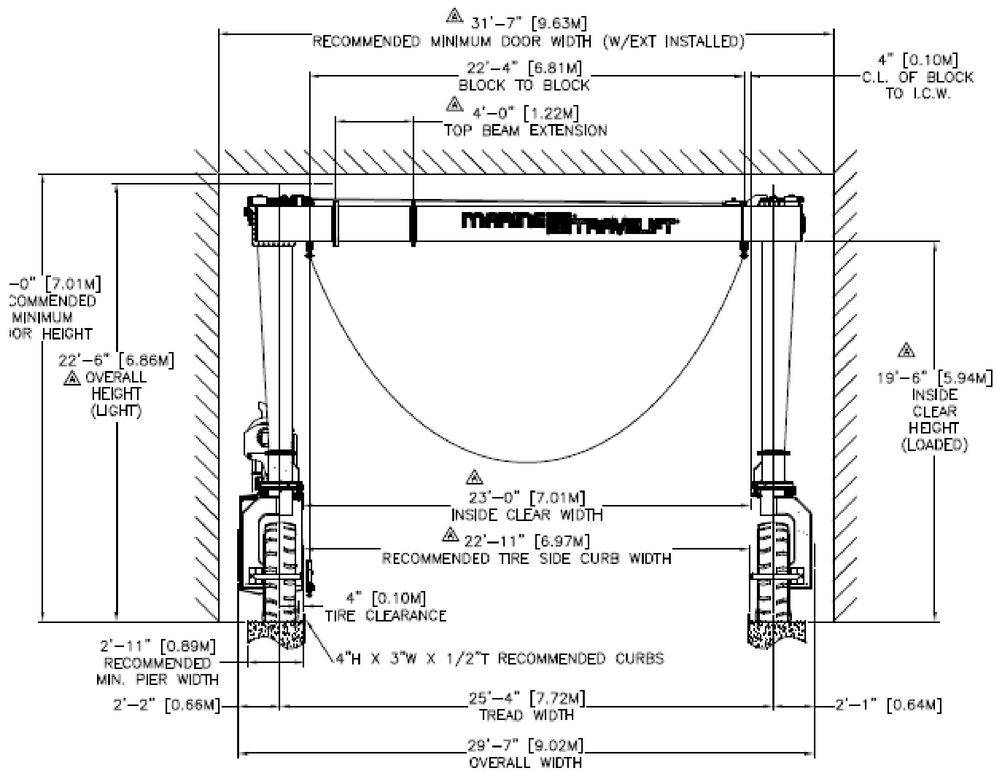


FIGURE 3.2: TRAVELIFT EQUIPMENT

Two construction alternatives were considered for the new pier. Both alternatives would have two three-foot-wide piers spaced 25 feet 10 inches (center to center) supported on pile caps at 22 feet on center. Each pile cap would consist of a plumb pile and a pile battered perpendicular to the pier, and two pile caps on each pier would have two piles battered longitudinal to the pier in addition to the perpendicular pile, see Figures A.7 and A.8. The two structure alternatives considered were:

- 18-inch-octagonal precast, prestressed concrete piles, with precast, prestressed haunched deck panels (Figure 3.4) supported on cast-in-place concrete two-stage pile caps.
- Open ended steel pipe piles, 18 inches in diameter with precast, prestressed haunched panel decks supported on cast-in-place concrete two-stage pile caps.

Concept-level analysis was performed and lateral loads from the equipment operation and earthquake loads were checked against the available pile capacities. The seismic loads are quite high and because battered piles cannot be directed into the shipway, some of the loads must be resisted through tension on the batter piles. At this point the concrete pile option was discarded as there is no way to drive the concrete piles into the siltstone to achieve the required tension capacity or install tension anchors with them. The geotechnical engineer has advised that open-ended steel pipe piles can be driven 2 to 3 feet into the siltstone and then tension anchors can be drilled and grouted into the siltstone. This alternative is presented in Appendix A Figures A.7 and A.8.

The adjacent upland area will require regrading to meet the new, higher, pier elevation. Regrading should be considered as part of an overall resiliency plan to raise the shoreline elevation. As discussed in the Condition Assessment Report, see Appendix D, the slope protection in this area will need remediation to prevent further slope erosion. The existing abutment and its integral vault will need to be repaired and the fill under the vault replaced regardless of whether the TraveLift Pier is replaced, as the vault at the top of the bank is currently being undermined and the adjacent slope is failing. There is a possibility that the increased vessel capacity will require dredging to provide for deeper draft vessels and/or to restore the basin to design depth. (The basin for the TraveLift was originally dredged to -10 ft MLLW. The current elevation ranges from approximately -4 to -6 ft MLLW). This consideration is outside the scope of this report but should be considered in further planning.

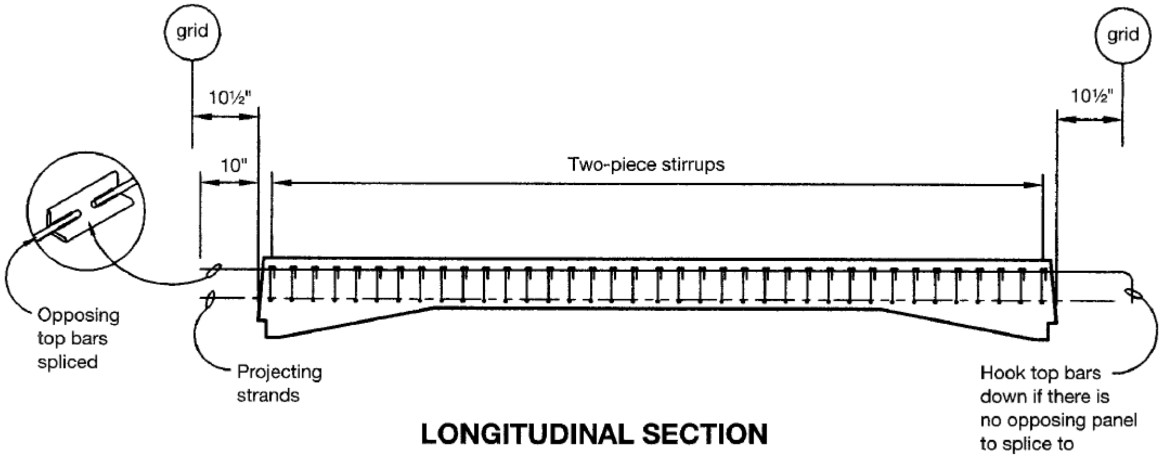


FIGURE 3.3: PRECAST DECK PANEL CONCEPT (SOURCE: CONCRETE TECHNOLOGY CORPORATION)

The geotechnical information is based on borings made onshore for another project. It may not represent the actual elevation of the siltstone horizon at the TraveLift Pier. Final design of the TraveLift Pier will require offshore borings to determine the location and character of the bearing strata.

3.4. Safe Coast Seafood Wharf East Bulkhead

The failing East Bulkhead is beyond repair and it is proposed to be replaced with a new steel sheet pile bulkhead positioned to the waterside of the existing timber bulkhead. Two bulkhead options were investigated for this concept-level design study. Both options utilize vertical steel plumb piles, a reinforced concrete pile cap, and grouted tie back anchors that will extend down to underlying bedrock at a slope of 1H:1V. The tie back anchors will be anchored within the concrete pile cap to protect the anchors from corrosion and provide a smooth waterside face on the bulkhead that will not damage vessels. A fendering system could be included along the face of the bulkhead to allow vessels to berth at the East Bulkhead if desired by the Port. This has not been included in the concept design at this time.

Sea level rise concerns are handled by raising the grade behind the wall to an elevation of +14.0 ft MLLW from the existing grade of approximately +11.5 ft MLLW. The existing timber bulkhead will be partially demolished with most of the structure buried in place. The gap between the new and existing bulkheads will be filled with drainage rock.

Bulkhead Option 1 will consist of using the bulkhead only to resist lateral earth pressure loads that include seismically induced liquefaction. Because the loads on a bulkhead wall from soil liquefaction are high, Option 2 will utilize ground improvement behind the wall to prevent liquefaction and reduce the loads on the bulkhead wall, thereby reducing the cost of steel required in the bulkhead and tieback anchors. Figures of the bulkhead design options are found in Appendix A, Figures A.5 and A.6.

Both bulkhead options are designed to support 250 pounds per square foot (psf) of uniform live load applied to the upland area behind the wall. Loading from AASHTO HS20-44 trucks are also checked. The following load cases in Table 3.2 were evaluated to encompass all likely live load cases, tidal fluctuations, and seismic loading conditions. Further details of the bulkhead analyses can be found in the Geotechnical Report included in Appendix C of this report.

TABLE 3.2: BULKHEAD LOADING CONDITIONS

Load Case	Dominant Load	Water Elevations (MLLW)		Notes
		Land Side	Water Side	
1	250 psf Uniform Live Load	+2.0 ft	+0.0 ft (MLLW)	Live loading with low tide conditions, 2 ft of tidal lag
2	HS20-44 Vehicle	+2.0 ft	+0.0 ft (MLLW)	
3	250 psf Uniform Live Load	+8.1 ft (MHHW)	+6.1 ft	Live loading with high tide conditions, 2 ft of tidal lag
4	HS20-44 Vehicle	+8.1 ft (MHHW)	+6.1 ft	
5	Hydrostatic, no Live Load	+4.0 ft	+0.0 ft (MLLW)	Tidal drawdown with 4 ft of tidal lag and no live load
6	Seismic Inertial, no Live Load	+0.0 ft (MLLW)	+0.0 ft (MLLW)	Inertial loading under the strong ground shaking of the design earthquake
7	Post-Seismic Liquefaction, no Live Load	+0.0 ft (MLLW)	+0.0 ft (MLLW)	Maximum liquefaction loads occurring after strong ground shaking. Only applicable to Bulkhead Option 1

The bulkhead sheet pile design assumes that the sheet piles will be coated with a marine paint coating system. It is expected this coating will last for 10 years before corrosion of the steel begins. Sacrificial steel thickness was included to allow for 40 years of corrosion to achieve the design life of 50 years.

Due to the height of the bulkhead and the elevation of the mudline, the maximum wall moments are located in the splash zone, which experiences high corrosion rates. And because cathodic protection (CP) does not provide a benefit in the splash zone, a CP system (impressed current or passive anodes) was not considered.

Several design challenges were identified during the bulkhead concept design. The first design challenge is the uncertainty for existing conditions buried behind the bulkhead wall. Record drawings were not available for the existing timber bulkhead, the existing buildings, or the timber wharf on the west side of the peninsula. The tie backs for the new bulkhead were oriented at a 1H:1V slope in an effort to minimize conflict with buried building foundations, utilities, and the existing bulkhead tie back anchors. Potential conflicts with buried existing conditions should be further investigated during final design.

The next design challenge is how the raised grade in the truck lane along the east side of the wharf will be tied into the existing buildings and the elevated west side of the peninsula. The transition from the East Bulkhead into the southern shoreline also needs to be addressed. The solution to this will be dependent on the development plans of the new owner of the facility, Safe Coast Seafoods. This includes coordination with tying into the riprap armored slope at the southeast corner of the peninsula (or extending a vertical bulkhead along the southern shoreline), and the loading dock area where the truck lane reaches the West Wharf. Coordination between the Port, Safe Coast Seafoods, and the design team should continue to develop a strategy for integration of the raised truck lane and the surrounding structures that will mitigate sea level rise and flooding concerns.

The final design challenge identified is the seismic performance of the entire peninsula. The new bulkhead is designed to comply with the current seismic design and performance requirements enforced by the local Building Code. Due to the soil conditions at the site (discussed in more detail in Appendix C), a design-level seismic event will likely induce widespread liquefaction and lateral spreading on the peninsula. The new bulkhead will protect against seismically-induced failure of the peninsula to the east, but does not protect against failure to the south and the west. To reduce the seismic and lateral spreading risk for the entire peninsula, seismic upgrades would also be required on the south and west sides. These structures were outside of the scope of this project, however the condition assessment, repair and/or replacement, and seismic upgrade of these facilities should be coordinated with the Port, Safe Coast Seafoods, and the design team.

3.5. West Marina Access Pier

The West Marina Access Pier provides access to a gangway that serves the west end of the marina and supports water and power utilities serving the marina's floating docks. The existing construction is timber piles and deck, and the existing utilities are supported under the deck and gangway. The Condition Assessment Report (provided in Appendix D) found that the Access Pier condition was rated "serious" based on the condition of the timber piles and the utilities. To address SLR the elevation of the abutment and deck will be raised to EL +14.0 ft MLLW. The upland area will require regrading to meet the new, higher, pier deck elevation. Regrading should be considered as part of an overall resiliency plan to raise the shoreline elevation and is not included in this discussion. Two options were investigated: one for repair and one for replacement. See Appendix A, Figures A.1 through A.4.

The repair option should be considered a short-term solution, with an additional design life of ten to fifteen years or less. This option takes advantage of the fact that the stringers, deck, and abutment are in good condition. For the repair option the nine timber piles would be jacketed and grouted. For this repair, one of several proprietary jacket and grout systems would be used. The timber piles are wrapped with fiberglass jackets and structural grout is pumped in to fill the annular void between the jacket and the pile. The system is installed from approximately two feet below the marina basin's design mudline to the top of the pile. The non-bearing piles would be repaired by adding shims or grout between the pile and pile cap. The deck elevation would be raised by adding timber sleepers and fiberglass grating on top of the existing deck boards. The guardrails would be replaced. The utilities would be rerouted to lie between the sleepers on top of the deck, and the compromised utility connections would be replaced. Vehicle access to the pier would be prohibited by the addition of traffic bollards. The existing abutment would be used with additional concrete added to match the new elevation of the pier deck. The existing gangway would be reused; reattached at the higher elevation. This scenario is based on an assumption that the cost of this option is low enough that it would not trigger an upgrade of the facilities to conform with the ADA, which would require replacement of the gangway. Due to the high cost of the pile wrap repairs, this option was found to be very comparable in cost to replacement, but it will have a much shorter service life.

The replacement option assumes a new ADA-compliant gangway would be provided. Compliant gangways in marinas are limited to a maximum length of 80 feet regardless of slope. In order to maintain the current location of the marina floats, and accommodate an 80-foot-long gangway, the Access Pier length would be reduced to approximately 10 feet. No vehicle access is also assumed for this option. The existing abutment would be used with additional concrete added to match the new elevation of the pier deck. Steel pipe piles 12 inches in diameter with a steel beam pile cap would be used for a single bent on the outboard end. Stringers, guardrails, and the deck would be timber construction. New utility runs would be placed under or alongside the new pier and gangway.

This option might require additional floatation to be added to the float supporting the end of the new, longer (heavier) gangway.

The replacement option is strongly recommended over the repair option as the high cost of repairing the West Access Pier for no more than a 15-year service life extension does not appear to be cost effective.

4. Opinion of Probable Construction Costs

Opinion of probable construction costs (OPCC) were developed based on available cost estimating resources and recent results of bid tabulations for similar work. In providing opinions of probable construction costs, it is recognized that neither the Client nor the Consultant has control over the costs of labor, equipment, materials, or over the Contractors' methods of determining prices and bids. This OPCC is based on the Consultant's reasonable professional judgment and experience. This estimate does not constitute a warranty, express or implied, that the Contractors' bids or negotiated prices of work will correspond with the Owner's budget or the OPCC prepared by the Consultant. Costs are presented in 2023 dollars, escalated from 2021 dollars with an assumed escalation rate of 3% per year. Costs include a contingency of 30% due to the conceptual nature of the design currently available.

This OPCC is based on the conceptual concepts presented in this report. It does not include costs of project management, permitting, or engineering. It does not include any upgrades to utilities. It is noted that these conceptual designs are based on approximate existing conditions based on rough measurements taken at the site. The final design will require a bathymetric and topographic survey. The material contained within the estimates reflects Moffatt & Nichol's best judgment considering the information available to it at the time of preparation. Any use which a third party makes of this OPCC, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Moffatt & Nichol accepts no responsibility for damages, if any, suffered by any third party because of decisions made or actions based on this OPCC.

Costs presented here do not include the cost of regrading the upland areas beyond these improvements to the higher elevation. Details of the cost estimates are included in Appendix B.

TABLE 4.1. OPINION OF PROBABLE CONSTRUCTION COSTS

Project Element	Cost	Remarks
Shoreline Slope Protection		
Reach 1: TraveLift Pier and Boatyard	$1,370,000	Combination of RSP and a new bulkhead structure at the TraveLift Pier is not included. Alternative configuration should be evaluated with updated survey data.
Reach 2: Between the Safe Coast Seafood Wharf (East Bulkhead) and the Marina West Access Pier	$203,000	
TraveLift Pier		
Replacement Option	$2,059,400	
Safe Coast Seafood Wharf East Bulkhead		
Replacement Option 1	$1,796,200	Tied back steel sheet pile bulkhead
Replacement Option 2	$3,166,100	Tied back steel sheet pile bulkhead with ground improvements
West Access Pier		
Repair Option	$228,500	All piles receive pile wraps
Replacement Option	$266,200	Dock size reduced; gangway lengthened

5. Next Steps

This report presents the results of condition assessment and conceptual design for repair and/or replacement of the Port of Ilwaco's TraveLift Pier, the Safe Coast Foods Wharf East Bulkhead, the West Access Pier for the marina, and the adjacent shoreline slopes in those areas. The next steps include the following:

- Port review and decision-making regarding the moving forward with the concepts.
- Additional site-specific studies including detailed topographic and bathymetric surveys.
- Detailed geotechnical site investigation.
- Hydraulic analysis for detailed flooding and SLR impact mitigation.
- 30% engineering design development.
- Preparation of project permit applications.
- 60%, 90%, and final engineering design.
- Contract plans and specifications package.
- Bidding support
- Construction support

M&N appreciates the opportunity to work with you regarding proposed improvements to the Port of Ilwaco Marina. Please contact us if you have questions about this report or regarding moving forward with the next steps for your projects.

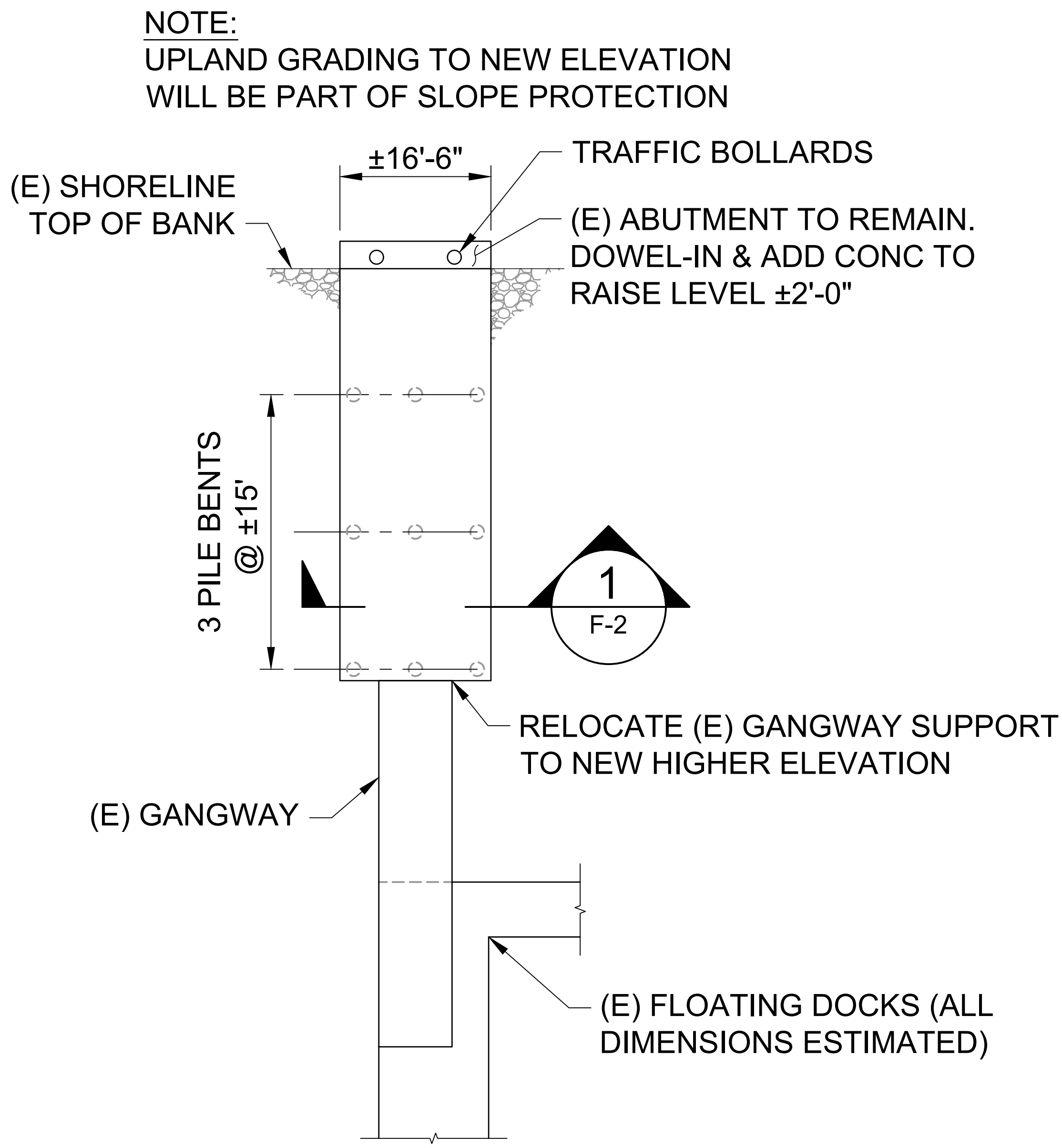
Sincerely,
MOFFATT & NICHOL, INC.

SJS signature

Sally Fisher
Senior Project Manager

Stuart Stringer, P.E., S.E.
Structural Engineer

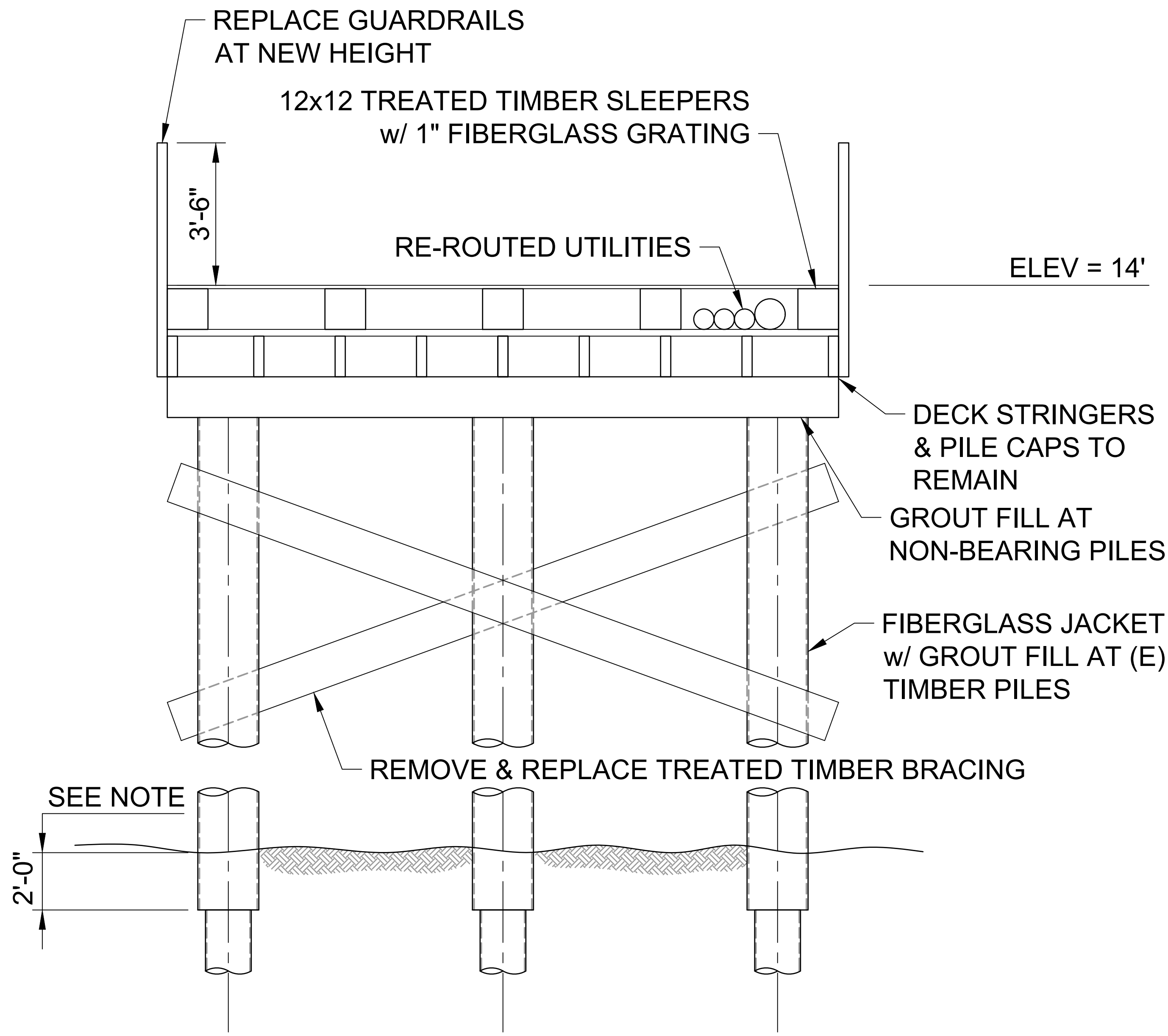
Appendix A – Drawings



PLAN - WEST ACCESS PIER - REPAIR OPTION

SCALE: 1"=20'

moffatt & nichol	PORT OF ILWACO	2021-03-19
	Figure A.1 - PLAN - WEST ACCESS PIER - REPAIR OPTION	

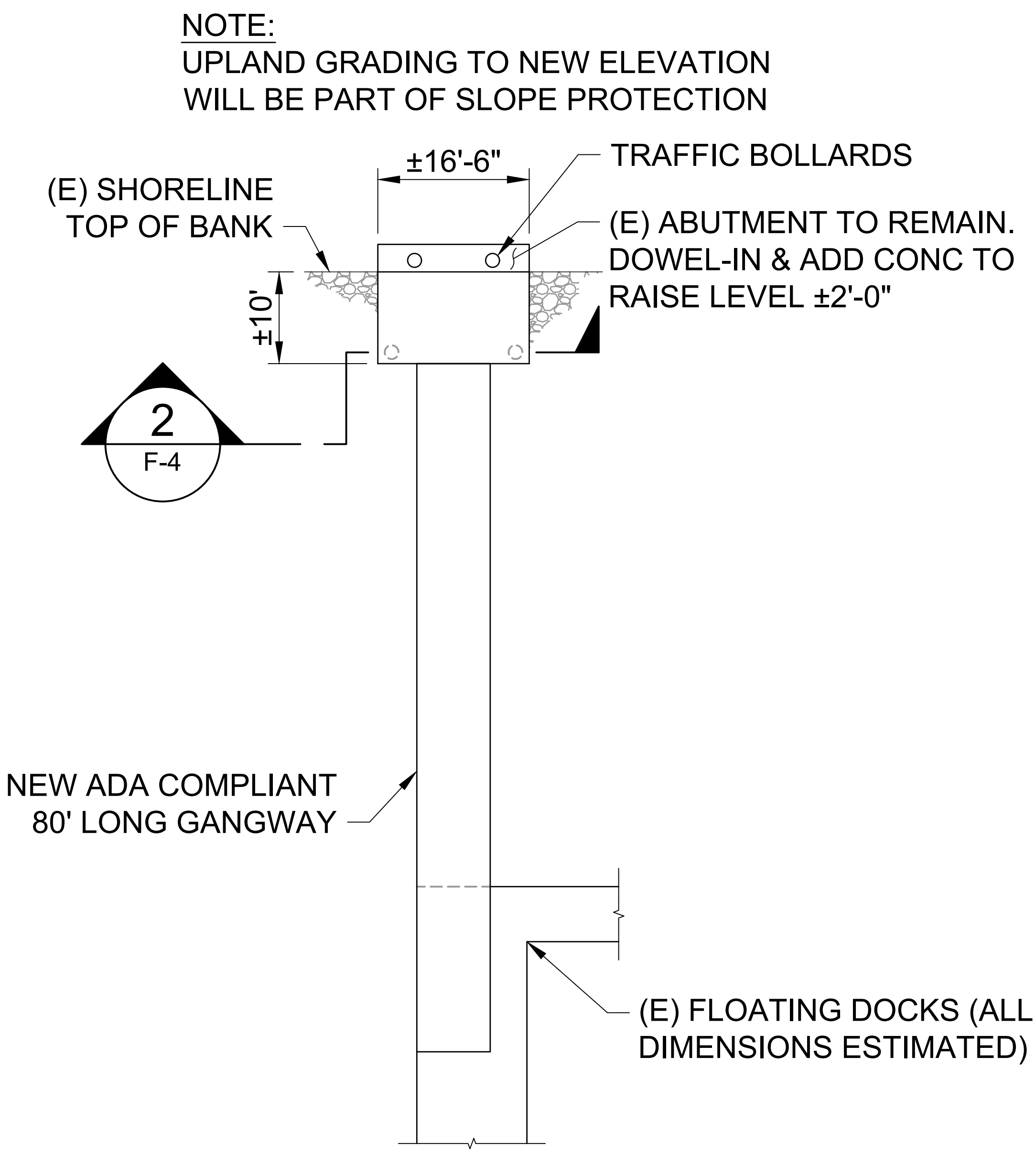


NOTE: BOTTOM OF JACKET TO BE 2-FT BELOW THE EXISTING MUDLINE OR THE BASIN'S DESIGN DREDGE ELEVATION, WHICHEVER IS DEEPER.

SECTION - WEST ACCESS PIER - REPAIR OPTION

SCALE: 1/4"=1'-0"

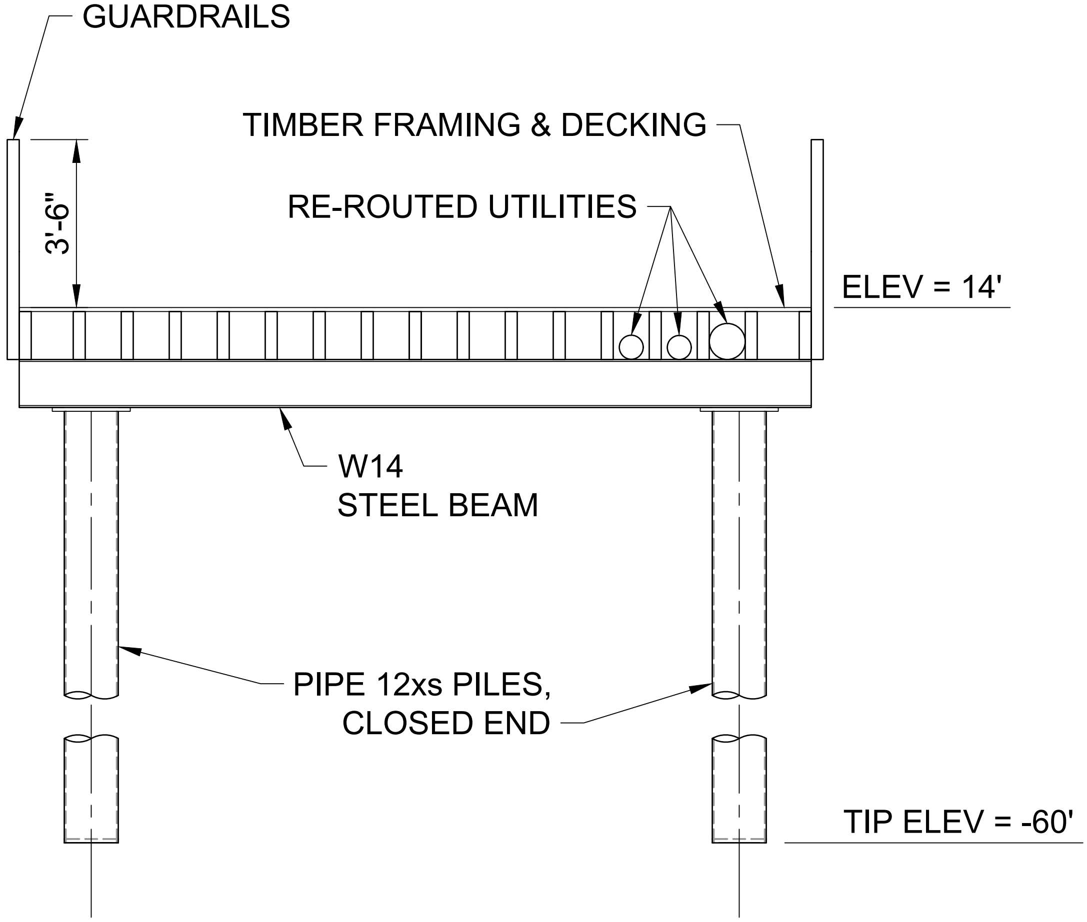
moffatt & nichol	PORT OF ILWACO	2021-03-19
	Figure A.2 - SECTION - WEST ACCESS PIER - REPAIR OPTION	



PLAN - WEST ACCESS PIER - REPLACE OPTION

SCALE: 1"=20'

moffatt & nichol	PORT OF ILWACO	2021-03-19
	Figure A.3 - PLAN - WEST ACCESS PIER - REPLACE OPTION	



SECTION - WEST ACCESS PIER - REPLACE OPTION

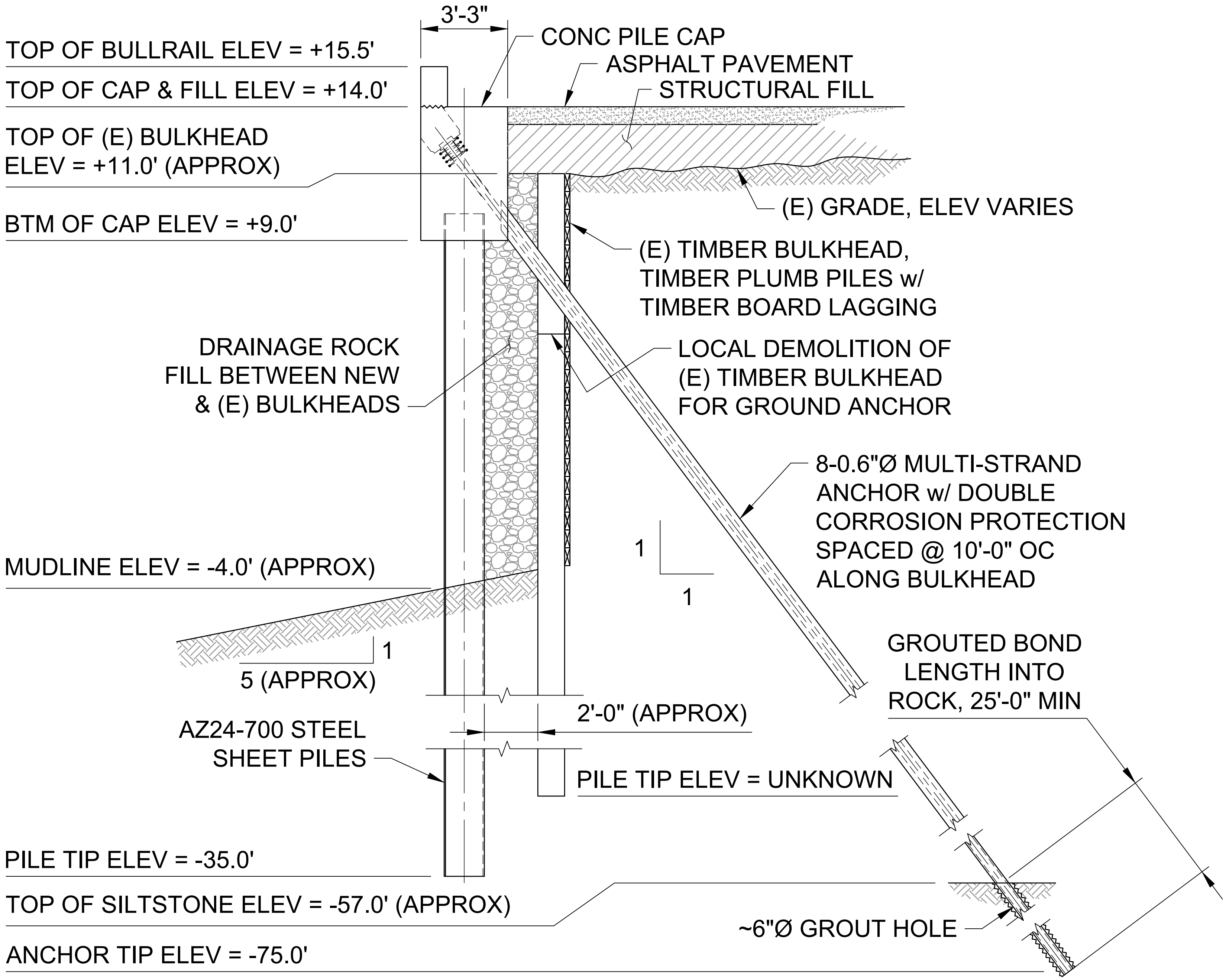
SCALE: 1/4"=1'-0"

moffatt & nichol

PORT OF ILWACO

Figure A.4 - SECTION - WEST ACCESS PIER - REPLACE OPTION

2021-03-19



SECTION - EAST BULKHEAD - NO GROUND IMPROVEMENT

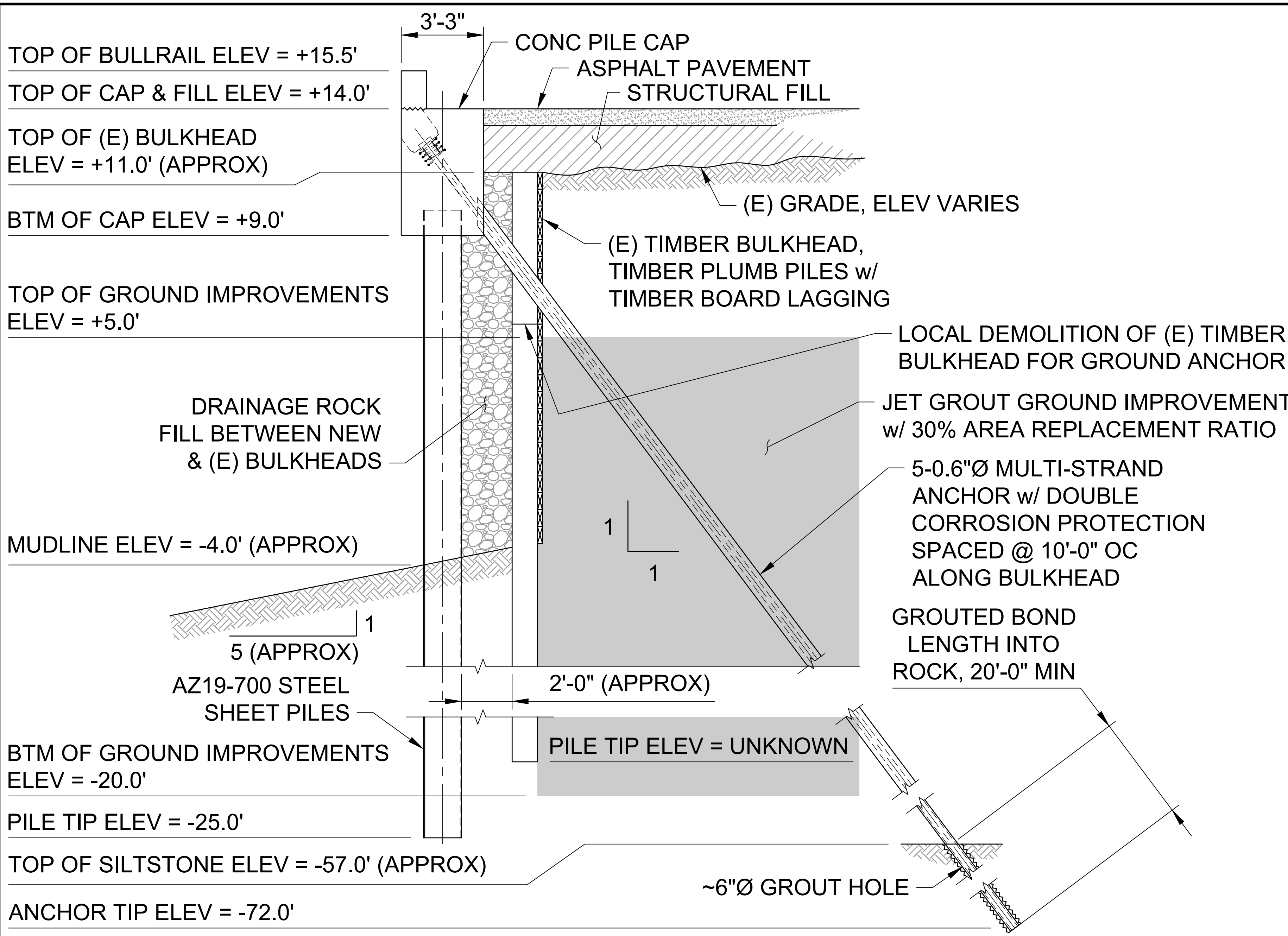
SCALE: 3/16"=1'-0"

moffatt & nichol

PORT OF ILWACO

Figure A.5 - SECTION - EAST BULKHEAD - NO GROUND IMPROVEMENT

2021-03-19



SECTION - EAST BULKHEAD - WITH GROUND IMPROVEMENTS

SCALE: 3/16"=1'-0"

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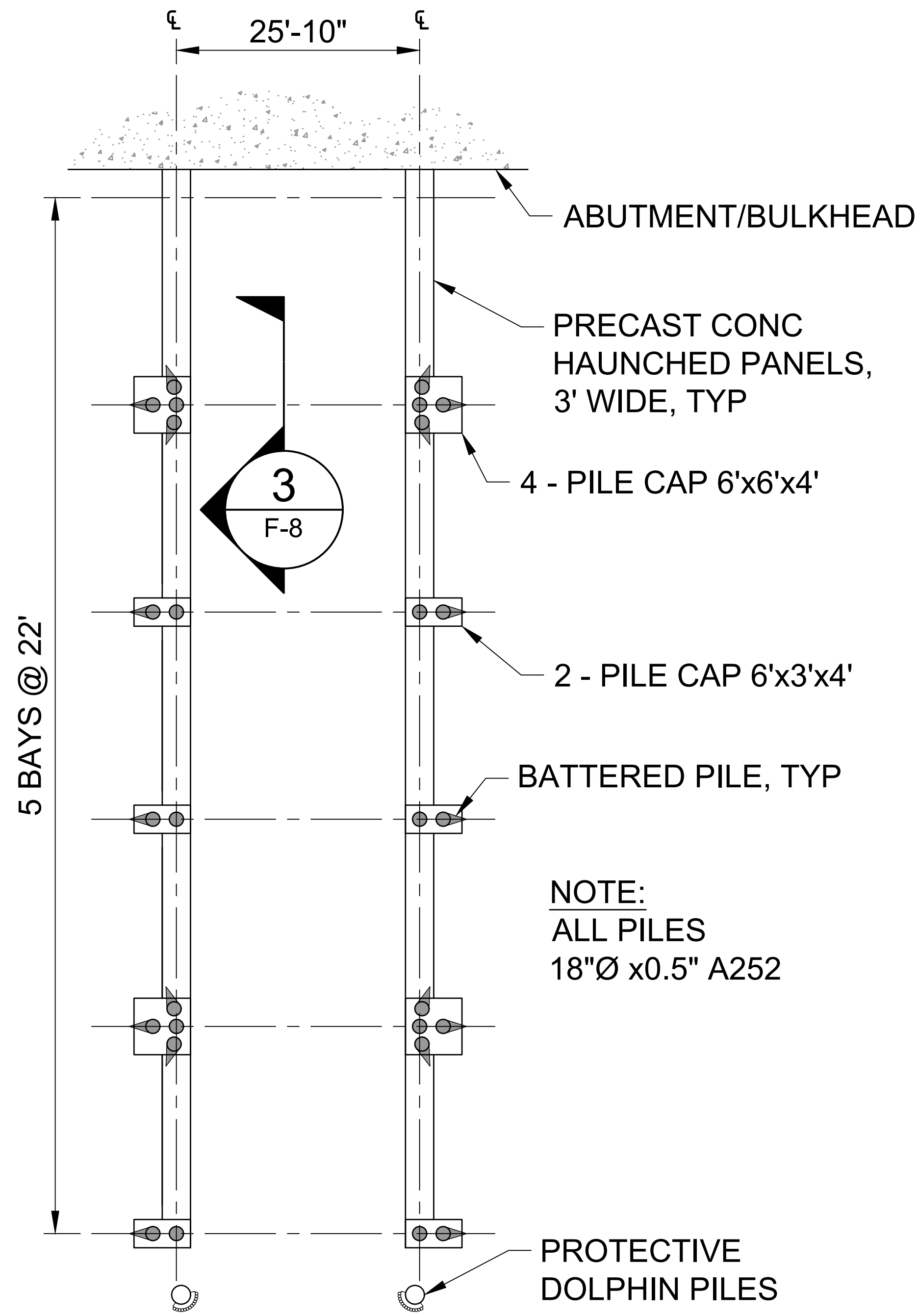
PORT OF ILWACO

Figure A.6 - SECTION - EAST BULKHEAD - WITH GROUND IMPROVEMENTS

2021-03-19



PLAN - TRAVEL LIFT PIER

SCALE: 1"=20'

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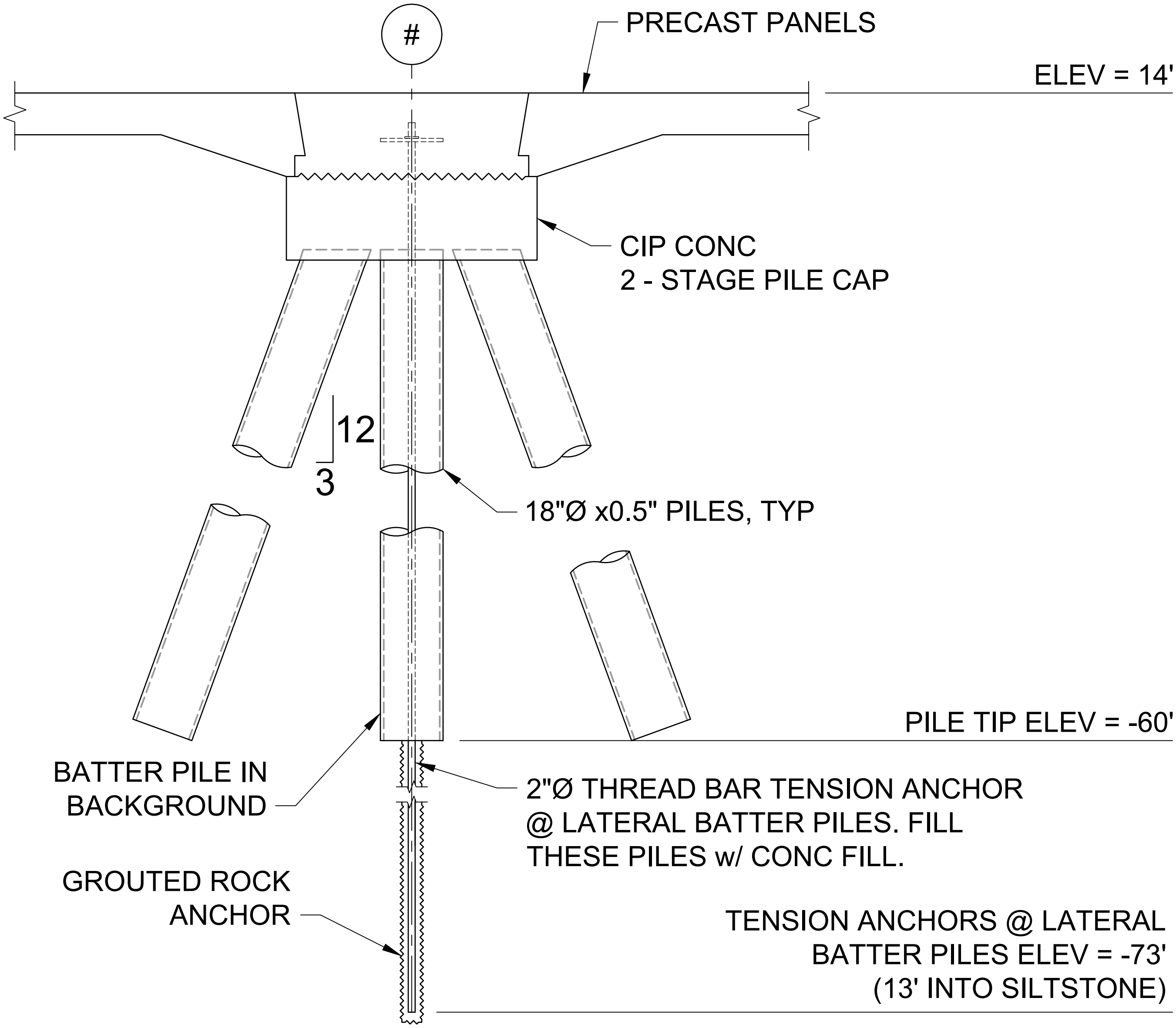
PORT OF ILWACO

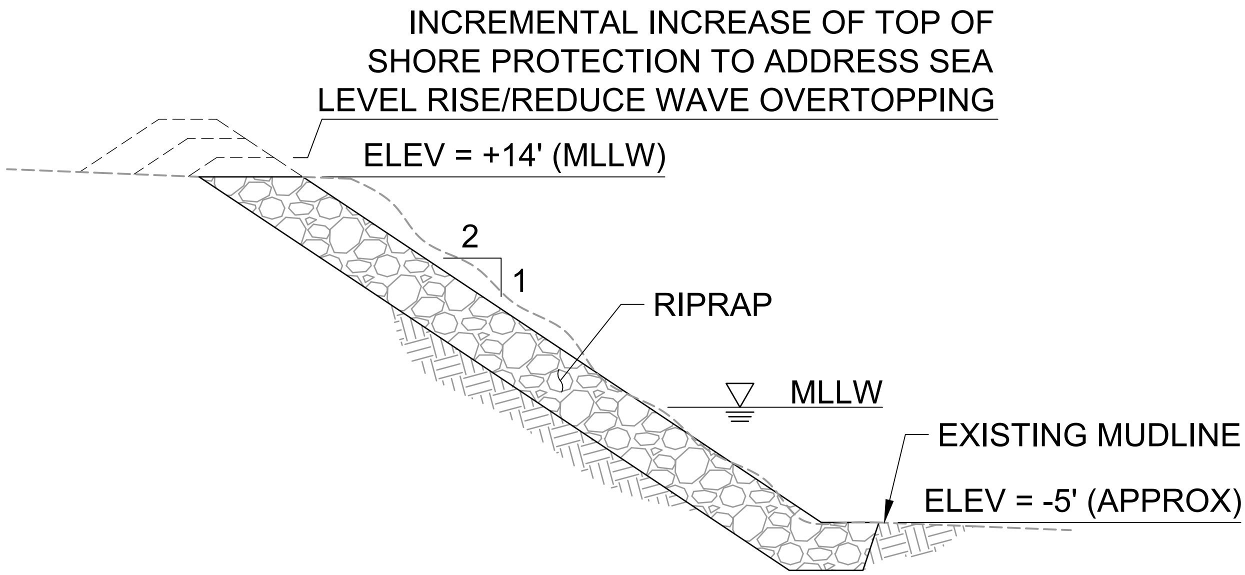
Figure A.7 - PLAN - TRAVEL LIFT PIER

2021-03-19



SECTION - TRAVEL LIFT PIER

SCALE: 1/4"=1'-0"



SECTION - MARINA SHORELINE - SLOPE PROTECTION

SCALE: NTS

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PORT OF ILWACO

Figure A.9 - SECTION - MARINA SHORELINE - SLOPE PROTECTION

2021-03-19

Appendix B – Detailed Cost Estimates

PORT OF ILWACO, MARINA REPAIRS CONCEPTUAL DESIGN

OPINION OF PROBABLE CONSTRUCTION COST

Updated 3-5-2021

Bid Item No.	Description	Unit	Quantity	Unit Cost 2023 $		Total Cost 2023 $	
	MOBILIZATION / DEMOBILIZATION	**LS**				**$**	**81,000**
	SLOPE PROTECTION (2.0H : 1.0V slope)					**$**	**1,006,800**
Reach 1	Furnish Filter Rock	TN	2,150	$	56	$	120,800
(800 LF)	Furnish Armor Stone	TN	6,440	$	72	$	464,200
	Grade/Prep Slope	LF	800	$	103	$	82,300
	Filter Fabric	SY	4,470	$	11	$	47,400
	Place Filter Rock	TN	2,150	$	11	$	22,800
	Place Armor Stone	TN	6,440	$	19	$	122,900
	Backfill	CY	150	$	92	$	13,800
Reach 2	Furnish Filter Rock	TN	330	$	56	$	18,500
(120LF)	Furnish Armor Stone	TN	970	$	72	$	69,900
	Grade/Prep Slope	LF	120	$	103	$	12,300
	Filter Fabric	SY	670	$	11	$	7,100
	Place Filter Rock	TN	330	$	11	$	3,500
	Place Armor Stone	TN	970	$	19	$	18,500
	Backfill	CY	30	$	92	$	2,800
				Construction Subtotal		**$**	**1,087,800**
			Design Contingency	30%		$	326,300
				CONSTRUCTION SUBTOTAL		$	1,414,100
			Sales Tax (Allow.)	8.1%		$	114,500
				PROJECT SUBTOTAL		**$**	**1,529,000**
			Construction Administration/Support (Allow.)	2%		$	30,600
				PROJECT TOTAL		**$**	**1,559,600**

ASSUMPTIONS/NOTES:

1. Estimates were developed using 2021 USD and escalated to Year 2023 using 3% per annum.
2. Shoreline separated into reaches:
   Reach 1: West of Boatyard, past TraveLift Pier to corner of parking lot
   Reach 2: East side of Jessie's Pier to Marina West Access Pier

PORT OF ILWACO, MARINA REPAIRS CONCEPTUAL DESIGN

OPINION OF PROBABLE CONSTRUCTION COST

Updated 3-3-21

Bid Item No.	Description	Unit	Quantity	Unit Cost 2023 $	Total Cost 2023 $
	MOBILIZATION / DEMOBILIZATION	**LS**	**1**		**$ 93,000**
	BULKHEAD REPLACEMENT OPTION 1				**$ 1,159,800**
1	Partial Demolition of Existing Bulkhead	LS	1	$ 20,000	$ 20,000
2	Furnish Sheet Piles, AZ 24-700, 45' Long	LS	1	$ 304,847	$ 304,800
	Install Sheet Piles	LS	1	$ 102,714	$ 102,700
3	Furnish Tie Back Anchors, 8 Strand	EA	18	$ 1,749	$ 31,500
	Install Tie Back Anchors	EA	18	$ 29,680	$ 534,200
4	Furnish and Install Concrete Pile Cap	CY	112	$ 1,060	$ 118,700
5	Drainage Fill Between Existing and New Bulkheads	TON	357	$ 28	$ 9,800
6	Structural Fill to Raise Grade	TON	893	$ 28	$ 24,600
7	Pavement	TON	98	$ 138	$ 13,500
				Construction Subtotal	**$ 1,252,800**
			Design Contingency	30%	$ 375,800
				CONSTRUCTION SUBTOTAL	$ 1,628,600
			Sales Tax (Allow.)	8.1%	$ 131,900
				PROJECT SUBTOTAL	**$ 1,761,000**
			Construction Administration/Support (Allow.)	2%	$ 35,200
				PROJECT TOTAL	**$ 1,796,200**

ASSUMPTIONS/NOTES:

1 All Estimates are in 2021 USD. 6% total assumed for escalation between 2021 and 2023.

2

PORT OF ILWACO, MARINA REPAIRS CONCEPTUAL DESIGN

OPINION OF PROBABLE CONSTRUCTION COST

Updated 3-3-21

Bid Item No.	Description	Unit	Quantity	Unit Cost 2023 $	Total Cost 2023 $
	MOBILIZATION / DEMOBILIZATION	**LS**	**1**		**$ 164,000**
	BULKHEAD REPLACEMENT OPTION 2				**$ 2,045,100**
1	Partial Demolition of Existing Bulkhead	LS	1	$ 20,000	$ 20,000
2	Furnish Sheet Piles, AZ 19-700, 35' Long	LS	1	$ 227,570	$ 227,600
	Install Sheet Piles	LS	1	$ 70,278	$ 70,300
3	Furnish Tie Back Anchors, 5 Strand	EA	18	$ 1,102	$ 19,800
	Install Tie Back Anchors	EA	18	$ 26,712	$ 480,800
4	Jet Grout Ground Improvement	CY	2,000	$ 530	$ 1,060,000
5	Furnish and Install Concrete Pile Cap	CY	112	$ 1,060	$ 118,700
6	Drainage Fill Between Existing and New Bulkheads	TON	357	$ 28	$ 9,800
7	Structural Fill to Raise Grade	TON	893	$ 28	$ 24,600
8	Pavement	TON	98	$ 138	$ 13,500
				Construction Subtotal	**$ 2,209,100**
			Design Contingency	30%	$ 662,700
				CONSTRUCTION SUBTOTAL	$ 2,871,800
			Sales Tax (Allow.)	8.1%	$ 232,600
				PROJECT SUBTOTAL	**$ 3,104,000**
			Construction Administration/Support (Allow.)	2%	$ 62,100
				PROJECT TOTAL	**$ 3,166,100**

ASSUMPTIONS/NOTES:

1 All Estimates are in 2021 USD. 6% total assumed for escalation between 2021 and 2023.

2

PORT OF ILWACO, MARINA REPAIRS CONCEPTUAL DESIGN

OPINION OF PROBABLE CONSTRUCTION COST

Updated 1/14/2021

Bid Item No.	Description	Unit	Quantity	Unit Cost 2023 $	Total Cost 2023 $
	MOBILIZATION / DEMOBILIZATION	**LS**			**$ 107,000**
	TRAVEL LIFT PIER OPTION 1, STEEL PIPE PILES				**$ 1,329,900**
1	Plumb Piles 18" dia x 0.5", Furnish	LF	705	$ 129	$ 91,200
	Plumb Piles 18" dia x 0.5", Install	EA	10	$ 6,360	$ 63,600
2	Batter Piles 18" dia x 0.5", Furnish	LF	1,530	$ 129	$ 197,900
	Batter Piles 18" dia x 0.5", Install	EA	18	$ 6,360	$ 114,500
	Grouted Tension Anchors	EA	10	$ 47,700	$ 477,000
3	Haunched Panels, Furnish	EA	10	$ 10,314	$ 103,100
	Haunched Panels Install	LS	1	$ 46,301	$ 46,300
4	CIP Pile Caps	CY	41	$ 2,173	$ 89,000
5	CIP Abutment	CY	16	$ 1,198	$ 18,600
6	Railing	LF	110	$ 122	$ 13,400
7	Dolphin Piles with Pile Wraps, Furnish	EA	2	$ 17,384	$ 34,800
	Dolphin Piles with Pile Wraps, Install	EA	2	$ 6,360	$ 12,700
8	Demolition	LS	1	$ 67,840	$ 67,800
				Construction Subtotal	**$ 1,436,900**
			Design Contingency	30%	$ 431,100
				CONSTRUCTION SUBTOTAL	$ 1,868,000
			Sales Tax (Allow.)	8.1%	$ 151,300
				PROJECT SUBTOTAL	**$ 2,019,000**
			Construction Administration/Support (Allow.)	2%	$ 40,400
				PROJECT TOTAL	**$ 2,059,400**

ASSUMPTIONS/NOTES:

1 All Estimates are in 2021 USD. 6% total assumed for escalation between 2021 and 2023.

2

PORT OF ILWACO, MARINA REPAIRS CONCEPTUAL DESIGN

OPINION OF PROBABLE CONSTRUCTION COST

Updated 1/14/2021

Bid Item No.	Description	Unit	Quantity	Unit Cost 2023 $	Total Cost 2023 $
	MOBILIZATION / DEMOBILIZATION	**LS**			**$ 14,000**
	ACCESS PIER OPTION 2, REPLACEMENT				**$ 171,400**
1	Furnish Piles, 12" XS, Top 30' Coated	LF	150	$ 87	$ 13,100
	Install Piles, 12" , Land Based Equipment	EA	2	$ 2,120	$ 4,200
2	W14 Pile Bent	LS	1	$ 14,776	$ 14,800
3	Timber Deck	BD-FT	640	$ 5	$ 3,000
4	Timber Railing	LF	20	$ 122	$ 2,400
5	Raise Abutment	CY	9	$ 975	$ 9,100
6	Relocate Utilities	LS	1	$ 10,600	$ 10,600
7	80' Gangway, Furnish and Install	LS	1	$ 84,800	$ 84,800
8	Demolish Old Pier	SF	765	$ 38	$ 29,400
				Construction Subtotal	**$ 185,400**
			Design Contingency	30%	$ 55,600
				CONSTRUCTION SUBTOTAL	$ 241,000
			Sales Tax (Allow.)	8.1%	$ 19,500
				PROJECT SUBTOTAL	**$ 261,000**
			Construction Administration/Support (Allow.)	2%	$ 5,200
				PROJECT TOTAL	**$ 266,200**

ASSUMPTIONS/NOTES:

1 All Estimates are in 2021 USD. 6% total assumed for escalation between 2021 and 2023.

2

PORT OF ILWACO, MARINA REPAIRS CONCEPTUAL DESIGN
OPINION OF PROBABLE CONSTRUCTION COST
Updated 1/14/2021

Bid Item No.	Description	Unit	Quantity	Unit Cost 2023 $	Total Cost 2023 $
	MOBILIZATION / DEMOBILIZATION	**LS**			**$ 12,000**
	ACCESS PIER OPTION 1, REPAIR				**$ 147,300**
1	12 x 12 Timber Sleepers	BD-FT	2,700	$ 2	$ 5,700
2	Fiberglass Pultruded Grating	SF	720	$ 25	$ 18,300
3	Timber Railing Each Side	LF	90	$ 122	$ 11,000
4	Wrap and Grout Timber Piles, 17' x 9 Piles	LF	108	$ 795	$ 85,900
5	Relocate Utilities	LS	1	$ 10,600	$ 10,600
6	Remove and Reattach Gangway	LS	1	$ 4,240	$ 4,200
7	Raise Abutment	CY	9	$ 975	$ 9,100
8	Remove Timber Bracing	BD-FT	408	$ 2	$ 600
	Replace Timber Bracing	BD-FT	408	$ 5	$ 1,900
				Construction Subtotal	**$ 159,300**
			Design Contingency	30%	$ 47,800
				CONSTRUCTION SUBTOTAL	$ 207,100
			Sales Tax (Allow.)	8.1%	$ 16,800
				PROJECT SUBTOTAL	**$ 224,000**
			Construction Administration/Support (Allow.)	2%	$ 4,500
				PROJECT TOTAL	**$ 228,500**

ASSUMPTIONS/NOTES:

1 All Estimates are in 2021 USD. 6% total assumed for escalation between 2021 and 2023.

2

Appendix C – Geotechnical Memo (GeoEngineers)

Geotechnical Engineering Report

Port of Ilwaco Marina Repairs Conceptual Design
Ilwaco, Washington

for
Port of Ilwaco

February 26, 2021

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Report

Port of Ilwaco Marina Repairs Conceptual Design
Ilwaco, Washington

for
Port of Ilwaco

February 26, 2021

GeoEngineers

4000 Kruse Way Place
Building 3, Suite 200
Lake Oswego, Oregon 97035
503.624.9274

Geotechnical Engineering Report

Port of Ilwaco Marina Repairs Conceptual Design
Ilwaco, Washington

File No. 21551-002-00

February 26, 2021

Prepared for:

Port of Ilwaco
165 Howerton Avenue
Ilwaco, Washington 98624

Attention: Guy Glenn, Port Manager

Prepared by:

GeoEngineers, Inc.
4000 Kruse Way Place
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Benjamin J. Hoffman, PE
Senior Engineer

GREG ALAN LANDAU
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PROFESSIONAL ENGINEER
2/26/21

Greg A. Landau, PE
Associate

BJH:GAL:cje

GEOENGINEERS

Table of Contents

1.0 INTRODUCTION

GeoEngineers, Inc. (GeoEngineers), is pleased to submit this geotechnical engineering report for the proposed Port of Ilwaco Marina Repairs Conceptual Design Project located at the Port of Ilwaco at 165 Howerton Avenue in Ilwaco, Washington. The site is located on the north bank of Baker Bay, immediately east of the mouth of the Columbia River. Based on information provided to us, we understand that the project generally consists of conceptual engineering design for repair or replacement of the following project elements: (1) Jessie's Wharf East Bulkhead; (2) TraveLift Pier; (3) West Access Pier; and (4) shoreline and slope protection adjacent to the evaluated structures. The goal of the project is to provide engineering concepts for the repair and/or replacement of the evaluated structures as well as shoreline and slope protection in adjacent areas. The location of the site highlighting project elements is shown in the Site Plan, Figure 1.

2.0 SCOPE OF SERVICES

The purpose of our services was to evaluate soil and groundwater conditions within the project area in order to identify realistic and feasible options for conceptual design and to address the vulnerability of the site soils to seismic-induced liquefaction and lateral spreading.

Our proposed scope of services included the following:

1. Coordinated with Moffat & Nichol, structural engineers for the project, during the conceptual design process.
2. Reviewed information regarding subsurface soil and groundwater conditions at the site, including reports in our files, selected geologic maps, and other geotechnical engineering related information for the project area.
3. Reviewed information provided by the Port of Ilwaco and/or Moffat & Nichol, including site sketches, survey information, and field-developed geometry.
4. Reviewed and evaluated preliminary concepts for the proposed improvements (provided by Moffat & Nichol) with respect to feasibility and seismic issues. Evaluation of proposed improvements included the following:
    a. Provide an opinion about the suitability of the existing geotechnical information for application to the conceptual design and if additional subsurface information and exploration is recommended to carry the design forward.
    b. Development of subsurface conditions for use in conceptual design that included soil layering and engineering properties as well as groundwater conditions.
    c. Preliminary liquefaction analyses.
    d. Preliminary slope stability analyses that included both static and seismic slope stability evaluations.
    e. Preliminary axial and lateral pile capacity analyses for use in repair and/or replacement of the pile supported TraveLift and West Access Piers.
    f. Preliminary geotechnical capacity and stability analyses for bulkhead replacement.

5. Provided a summary of our preliminary evaluations and analyses as well as project-specific recommendations in this geotechnical report.

Our geotechnical work has been directly supervised by a professional engineer licensed in the state of Washington.

3.0 FACILITY DESCRIPTION

The Port of Ilwaco generally consists of a marina used for year-round moorage of recreational and commercial fishing vessels, upland commercial buildings, and a boatyard. The boatyard is located at the northwest end of the marina and includes the TraveLift Pier. Several commercial buildings, including Jessie's Ilwaco Fish Company building, are located on an earth-filled and timber pile supported wharf structure that includes a timber bulkhead along the eastern limits of the wharf. The marina is accessed from the shoreline by four timber access piers and the West Access Pier is one of the main entry points for the marina for both pedestrians and utilities. The general arrangement of the Port and these facilities is shown in Figure 1.

A general description of each facility evaluated as part of this study is provided below. More detailed information regarding the Port of Ilwaco and the existing condition of these facilities is provided in the *Draft Port of Ilwaco Marina Repairs Condition Assessment Letter Report* (Moffatt & Nichol 2020).

- The TraveLift Pier consists of prestressed-concrete piles, a steel H-pile, reinforced concrete runway beams, a reinforced-concrete abutment, and steel handrails along the outer perimeter of the runway beams. A timber catwalk is located at approximately mid-length of the runway beams and consists of timber stringers and decking. Timber-pile dolphins are located at the southern end of each runway. The adjacent shoreline consists of various sized rock armoring.
- Jessie's Wharf East 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 in the upland area. Three steel pipe piles are located along the face of the bulkhead and are assumed to be used for mooring of vessels. The adjacent northern shoreline consists of a vegetated slope and a timber pile bulkhead retaining structure.
- The West Access Pier consists of treated timber components, including piles, pile caps, stringers, cross bracing, and decking. A concrete abutment connects the pier to the upland area. Water and electrical utilities that service the marina are supported along the pier and transition onto an aluminum gangway. The adjacent shoreline consists of a vegetated slope.

4.0 SITE CONDITIONS

4.1. Site Geology

Geology of the site area is mapped in *Geologic Map of Washington-Southwest Quadrant* (Walsh, et al. 1987) and was reviewed in order to develop an understanding of the site geology and underlying bedrock/basement formational materials. 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.

4.2. Subsurface Conditions

Previous geotechnical engineering work performed by GeoEngineers at a site located near the Port of Ilwaco was reviewed in order to approximate the general subsurface conditions used for conceptual design. The previous work performed at the nearby site is documented in *Geotechnical Engineering Report: Ilwaco Fish Company Cold Storage and Seafood Processing Facility, Ilwaco, Washington* (GeoEngineers 2014) and included the advancement of four soil borings. Based on review of the soil borings, as well as other data presented in the 2014 report prepared by GeoEngineers, the subsurface was divided into four general soil/rock layers for use in conceptual design for the project. The four general soil/rock layers considered for conceptual design are as follows:

- **Interbedded Silt and Sand.** Cohesionless alluvial soil consisting of interbedded very soft to very stiff silt and loose to medium dense sand with varying amounts of silt is presumed to be present from mudline to an elevation of approximately -15 feet.
- **Fat Clay.** The upper cohesionless alluvial soils are likely underlain by a cohesive alluvial soil layer consisting of fat clay that is medium stiff to stiff. The fat clay is presumed to be present from approximately Elevation -15 feet to -23 feet.
- **Silt with Organics and Peat.** The fat clay layer is likely underlain by lower alluvial soil consisting of very soft to very stiff silt with varying amounts of sand, organic material, and peat. The organic material and peat likely consists of intact and decomposed wood, partially decomposed wood debris, and various partially decomposed organic material. The silt with organics and peat is presumed to be present from approximately Elevation -23 feet to -57 feet.
- **Siltstone.** The alluvial soil layers are likely underlain by bedrock that generally consists of slightly decomposed, extremely soft, closely fractured siltstone. The upper 5 feet (or more) is likely slightly decomposed and extremely soft, but the quality and hardness of the siltstone is presumed to increase with depth.

4.3. Groundwater

Regional groundwater is likely equal to the elevation of Baker Bay and the Columbia River. Because the project site is located on or adjacent to Baker Bay and the Columbia River it is likely that groundwater elevation in upland areas is subject to tidal fluctuations.

5.0 CONCEPTUAL DESIGN

The following subsections present a summary of geotechnical design parameters developed and analyses performed for conceptual design for the project.

5.1. Soil Properties

As discussed in Section 4.2, the subsurface was divided into four general soil/rock layers. Table 1 presents estimated drained soil strength properties for each of the soil layers presumed to be present at the project site that were used for conceptual design.

TABLE 1. ASSUMED DRAINED SOIL STRENGTH PROPERTIES FOR CONCEPTUAL DESIGN

Layer No.	Soil Description	Bottom of Layer Elevation (feet)	Total Unit Weight (pcf)	Internal Angle of Friction, ϕ (deg)	Cohesion, c (psf)
1	Interbedded Silt and Sand	-15	110	30	0
2	Fat Clay	-23	105	29	0
3	Silt with organics and peat	-57	110	30	0
4	Weathered Siltstone	Unknown	130	0	14,400[a]

Notes:

[a] Weathered Siltstone estimated to have unconfined compressive strength of 200 pounds per square inch (psi), equivalent to undrained shear strength or cohesion of 100 psi (14,400 psf).

pcf = pounds per cubic foot; deg = degrees; psf = pounds per square foot

5.2. Seismic Design

Based on review of available geologic resources and subsurface conditions encountered nearby, including the presence of potentially liquefiable soils, Site Class F was selected for preliminary seismic design for the project. However, it is assumed that the fundamental period of each of the structures to be repaired or replaced for the project will be less than 0.5 seconds. Therefore, exceptions documented in Section 20.3.1 of the 2016 *Minimum Design Loads for Buildings and Other Structures* (American Society of Civil Engineers [ASCE] 7-16) were used to approximate recommended seismic design parameters for the project. In determining seismic design parameters, Site Class D was selected for the project. Parameters provided in Table 2 are based on the procedure outlined in the 2018 IBC, which references the ASCE 7-16. Per ASCE 7-16 Section 11.4.8, a ground motion hazard analysis or site-specific response analysis is required to determine the ground motions for structures on Site Class D sites with S_1 greater than or equal to 0.2g. As stated previously, the site is assumed to be classified as Site Class D and has a recommended S_1 value of 0.738g; therefore, the provision of 11.4.8 applies. Alternatively, the parameters listed in Table 2 below may be used to determine the design ground motions if Exception 2 of Section 11.4.8 of ASCE 7-16 is used. Using this exception, the seismic response coefficient (C_s) is determined by Equation (Eq.) (12.8-2) for values of $T \leq 1.5T_S$, and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$, where T represents the fundamental period of the structure and T_S=0.732 sec. If requested, we can complete a site-specific seismic response analysis, which might provide somewhat reduced seismic demands from the parameters in Table 2 and the requirements for using Exception 2 of Section 11.4.8 in ASCE 7-16. The reduced values will likely not be significant enough to warrant the additional cost of further evaluation if designing to 2018 IBC. For conceptual design purposes, we recommend seismic design be performed using the values presented in Table 2.

TABLE 2. MAPPED 2018 IBC SEISMIC DESIGN PARAMETERS

Parameter	Recommended Value[1,2]
Site Class	F
Mapped Spectral Response Acceleration at Short Period (S_S)	1.427 g
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.738 g
Site Modified Peak Ground Acceleration (PGA_M)	0.871 g
Site Amplification Factor at 0.2 second period (F_a)	1.20

Parameter	Recommended Value[1,2]
Site Amplification Factor at 1.0 second period (F_v)	1.70
Design Spectral Acceleration at 0.2 second period (S_{DS})	1.142 g
Design Spectral Acceleration at 1.0 second period (S_{D1})	0.836 g

Notes:

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

[2] These values are only valid if the structural engineer utilizes Exception 2 of Section 11.4.8 (ASCE 7-16).

5.3. Liquefaction

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay contents is the most susceptible to liquefaction. Low plasticity, silty sand may be moderately susceptible to liquefaction under relatively higher levels of ground shaking.

Liquefaction analyses were performed for the project site located nearby, and a summary of analyses is presented in the 2014 GeoEngineers Report. Based on review of analyses results, it assumed that Soil Layer 1, the interbedded silt and sand, is susceptible to liquefaction. Using available liquefaction data and analyses results for the interbedded silt and sand layer, methods developed by Seed and Harder (1990) were used to estimate the post-liquefaction residual strength of the soil layer for use in seismic design. A post-liquefaction residual shear strength equal to 300 psf was estimated for the interbedded silt and sand layer for use in seismic design for the project.

In addition, it is possible that due to the relatively large design ground shaking values (Table 2), Soil Layers 2 and 3 (fat clay and silt with peat and organics) are susceptible to strength loss due to strain softening during ground shaking. However, due to uncertainty and lack of subsurface data at the project site, strength loss was not considered for seismic design for the fat clay and silt with peat and organics layers. Due to this uncertainty, it is recommended that a subsurface exploration program be performed prior to final design in order to define the liquefaction (and/or strain softening) hazard present at the project site. See further discussion regarding recommendations for a subsurface exploration program later in this report.

5.4. Slope Stability

Preliminary slope stability analyses were performed for the proposed general shoreline protection geometry. The proposed shoreline protection geometry generally consists of an incremental increase (2 feet or less) of elevation at the top of the shoreline slopes as well as placement of riprap protection on slope surfaces at 1.5H:1V (horizontal to vertical) or 2H:1V. The riprap protection will also include a toe key at the base of slopes.

Limit equilibrium methods were used to evaluate shoreline slope stability for static and seismic loading conditions using the computer program SLIDE, Version 9 (Rocscience 2020). The Spencer method was used to calculate the factors of safety for each of the stability analyses performed. Additionally, a pseudo-static approach was implemented for the seismic evaluation in order to determine the yield acceleration, k_y. The yield acceleration is defined as the acceleration applied to the pseudo-static slope stability model that results in a factor of safety equal to 1.0. Based on Newmark (1965), if the PGA value exceeds the k_y value, the potential for permanent displacement of the critical failure plane exists. Methods developed by Bray and Travasarou (2007) were used to estimate the magnitude of permanent displacement using the k_y from the pseudo-static slope stability models as well as other parameters. In addition, the following conceptual design assumptions were used in the stability analyses:

- Live load at top of slope equal to 250 psf for static loading conditions. The live load was not used to evaluate stability for seismic or post-seismic loading.
- Pseudo-static PGA value equal to 0.435g (one-half the PGA at the ground surface) for the design seismic event. If the factor of safety using a PGA value equal to 0.435g is less than 1.0, determine the k_y value as discussed above.
- Post-seismic stability accounting for liquefaction of the interbedded silt and sand layer using a residual soil strength equal to 300 psf.

A summary of results of the preliminary slope stability analyses are presented in Table 3 for both a riprap slope surface at 1.5H:1V and 2H:1V, respectively. The results of the slope stability analyses for static and seismic loading conditions are provided in Appendix A, Figures A-1 through A-8.

TABLE 3. SUMMARY OF PRELIMINARY SLOPE STABILITY RESULTS

Loading Case	Factor of Safety	
	1.5H:1V	2H:1V
Static	1.5	1.6
Pseudo-Static (PGA = 0.435g)	0.6	0.6
Post-Seismic with Liquefaction	0.9	1.0

The results show that for static loading conditions, the proposed shoreline protection geometry meets the recommended minimum factor of safety for stability equal to 1.5 for both a 1.5H:1V and 2H:1V slope. The results also show that the proposed slope geometry is likely unstable during and following a design seismic event with factors of safety equal to or less than 1.0 for both the pseudo-static (with PGA equal to 0.435g) and post-seismic with liquefaction load cases. Because the pseudo-static load case with a PGA value equal to 0.435g was determined to have a factor of safety less than 1.0, a yield acceleration (k_y) value was determined for both slope geometries. Ky values of 0.16g and 0.19g were estimated for the proposed shoreline protection with a geometry of 1.5H:1V and 2H:1V, respectively. Using these k_y values, permanent slope displacement using methods developed by Bray and Travasarou (2007) were estimated to be on the order of 3 to 4 feet for the 1.5:1V slope, and 2 to 3 feet for the 2H:1V slope. Furthermore, based on the post-seismic factor of safety estimated to be less than or equal to 1.0 due to liquefaction, additional slope displacement and lateral spread of shoreline slopes should be expected following a design seismic event. It is difficult to determine the magnitude of slope displacement and/or lateral spread due to liquefaction without specific subsurface information at the location of the proposed shoreline protection. Due to this uncertainty, it is recommended that a subsurface exploration program be performed prior to final design in order to better estimate the magnitude of slope displacement and/or lateral spread. See further discussion regarding recommendations for a subsurface exploration program later in this report.

5.5. Axial Capacity of Driven Piles

It is our understanding that Moffatt & Nichol is proposing driven 16- or 18-inch-diameter pipe piles for replacement of the existing TraveLift Pier, and proposing driven 12-inch-diameter pipe piles for replacement of the existing West Access Pier. Conventional static analyses for compression and uplift pile capacity was performed using the “Beta Method” (after Fellenius 2017) for the proposed TraveLift and West Access Pier piles. The analyses used the general soil profile summarized in Table 1 as well as load transfer coefficients developed for each soil layer using the “Beta Method.” Based on subsurface conditions present at the project site, the axial capacity analyses assumed that the pipe piles would either be driven closed end or open-end with a plugged toe condition when driven into the weathered siltstone layer. Also, based on subsurface conditions and the presence of potentially liquefiable soil and/or compressible soil, it is recommended that all pier piles be driven to (and into) the siltstone layer.

Recommendations presented in *Design of Pile Foundations* (U.S. Army Corps of Engineers [USACE] 1991) were used to determine factors of safety used for compression and uplift pile capacity for both static and seismic loading conditions. Based on USACE (1991) a factor of safety equal to 3.0 was used to determine allowable compression and uplift pile capacity for static loading conditions and a factor of safety equal to 1.7 was used to determine allowable compression and uplift pile capacity for seismic loading conditions. These recommended factors of safety assume no pile testing during driving. Graphs presenting estimated allowable compression and uplift pile capacity versus pile toe elevation for piles proposed for the TraveLift and West Access Piers are provided in Appendix B, Figures B-1 through B-4. It should be noted that based on assumed subsurface conditions, refusal pile driving will likely be encountered at a penetration of 5 to 10 feet (or less) into the weathered siltstone layer, therefore axial capacity values were provided to an elevation of -65 feet for conceptual design purposes.

5.6. Design Parameters for Lateral Pile Analyses

It is our understanding that Moffatt & Nichol will be using the computer program LPILE (Ensoft 2019) to perform lateral analyses for the proposed TraveLift and West Access Pier piles. The soil profile presented in Table 1 as well as the soil properties presented in Table 4 below should be used in the LPILE analyses. Due to liquefaction, it is recommended for seismic design that Soil Layer 1 in Table 4 below be ignored in the LPILE analyses.

TABLE 4. LPILE SOIL PROPERTIES FOR USE IN CONCEPTUAL DESIGN

Layer No.	Soil Description	Soil Model	Effective Unit Weight (pcf)	Internal Angle of Friction, ϕ (deg)	Subgrade Modulus, k (pci)	Cohesion, c (psf)	Strain Factor, e50
1	Interbedded Silt and Sand	Sand (Reese)	46	30	30	N/A	N/A
2	Fat Clay	Stiff Clay w/ Free Water	41	N/A	100	1,000	0.01
3	Silt with organics and peat	Sand (Reese)	46	30	30	N/A	N/A

Layer No.	Soil Description	Soil Model	Effective Unit Weight (pcf)	Internal Angle of Friction, ϕ (deg)	Subgrade Modulus, k (pci)	Cohesion, c (psf)	Strain Factor, e50
4	Weathered Siltstone	Stiff Clay w/o Free Water	66	N/A	500	14,400	0.005

Notes:

pcf = pounds per cubic foot; pci = pounds per cubic inch

5.7. Jessie's Wharf East Bulkhead

Moffatt & Nichol has proposed constructing an anchored sheet pile bulkhead in front of the existing timber bulkhead structure along the eastern limits of Jessie's Wharf. As part of bulkhead construction, the grade at Jessie's Wharf will be raised to approximately Elevation 14 feet. Therefore, it was assumed the top of the new bulkhead will be located at Elevation 14 feet and a single row of anchors will be located at Elevation 12.5 feet. In addition, it was assumed that mudline in front of the new bulkhead will be located at Elevation -4 feet and slope away from the new bulkhead at 5H:1V. Conceptual geotechnical design and analyses for the proposed anchored sheet pile bulkhead was performed using the computer program CWALSHT (CASE 2002). The following conceptual design criteria were used in the CWALSHT analyses:

- Factor of safety equal to 1.5 applied to passive soil pressures to determine geotechnical stability.
- Interface friction angle equal to one-half drained strength friction angle for cohesionless soil layers (Soil Layers 1 and 3), and equal to zero for cohesive soil layers (Soil Layer 2).
- Free Earth Support Method with a stability factor of safety equal to 1.5 to determine minimum required sheet pile toe elevation.
- Fixed Earth Support Method with a factor of safety equal to 1.0 to determine reduced maximum bending moment and anchor loads.

Multiple load cases were considered for the conceptual design of the proposed anchored sheet pile bulkhead. The CWALSHT analyses evaluated the geotechnical stability, loads, and bending moments for each of the load cases summarized below.

1. **Load Case 1** – Static loading conditions with a uniform 250 psf live load above the bulkhead. Water elevation of 2 feet on the land side (within bulkhead backfill) and 0 feet on the water side (in Baker Bay) of the bulkhead.
2. **Load Case 2** – Static loading conditions with a HS20-44 vehicular live load above the bulkhead. Water elevation of 2 feet on the land side and 0 feet on the water side of the bulkhead.
3. **Load Case 3** – Static loading conditions with a uniform 250 psf live load above the bulkhead. Water elevation of 8.1 feet on the land side and 6.1 feet on the water side of the bulkhead.
4. **Load Case 4** – Static loading conditions with a HS20-44 vehicular live load above the bulkhead. Water elevation of 8.1 feet on the land side and 6.1 feet on the water side of the bulkhead.
5. **Load Case 5** – Static loading conditions with tidal drawdown and no live load. Water elevation of 4 feet on the land side and 0 feet on the water side of the bulkhead.

6. **Load Case 6** – Seismic loading conditions assuming some wall movement and no live load. Water elevation of 0 feet on either side of the bulkhead.
7. **Load Case 7** – Post-seismic loading conditions considering liquefaction of Soil Layer 1 and no live load. Water elevation of 0 feet on either side of the bulkhead.

The general subsurface profile presented in Table 1 was used in the CWALSHT analyses. In addition to the soil properties presented in Table 1, the following general subsurface conditions were used in the CWALSHT analyses:

- The analyses assumed that fill used to construct the existing Jessie's Wharf has a total unit weight of 120 pcf and an internal angle of friction equal to 32 degrees.
- Analyses were performed for Load Cases 1 through 7 (listed above) assuming no ground improvement to soil behind the proposed bulkhead.
- Analyses were performed for Load Cases 1 through 6 (listed above) assuming ground improvement by jet grouting methods due to the potential liquefaction hazard present in Soil Layer 1. General assumptions regarding the jet grout ground improvement include an improvement depth to Elevation -20 feet, an area replacement ratio equal to 30 percent, and a soil-cement unconfined compressive strength equal to 200 psi.

Tables 5 and 6 present a summary of results for the CWALSHT analyses for the proposed bulkhead without and with ground improvement, respectively. In addition, graphical results from CWALSHT presenting bending moment and shear versus depth for each load case without and with ground improvement are provided in Appendix C, Figures C-1 through C-13.

TABLE 5. CWALSHT (CASE 2002) ANALYSES SUMMARY – NO GROUND IMPROVEMENT

Load Case and Description	Recommended Pile Toe Elevation (feet)	Max Bending Moment (kip*feet)	Elevation at Max Bending Moment (feet)	Anchor Load (kips)
Water Elevations: Land Side = 2 feet and Water Side = 0 feet				
1. Static Loading, Uniform 250 psf Live Load	-25.5	26.7	0.6	3.8
2. Static Loading, HS20-44 Vehicular Live Load	-24.0	21.4	0.8	3.1
Water Elevations: Land Side = 8.1 feet and Water Side = 6.1 feet				
3. Static Loading, Uniform 250 psf Live Load	-23.3	22.8	1.6	3.6
4. Static Loading, HS20-44 Vehicular Live Load	-21.0	18.4	1.9	3.0
Water Elevations: Land Side = 4 feet and Water Side = 0 feet				
5. Static Loading, Tidal Drawdown	-26.3	29.3	0.2	3.8
Water Elevation at 0 feet				
6. Seismic Loading	-23.8	49.6	6.8	9.1
7. Post-Seismic with Liquefaction	-29.7	64.8	-3.2	5.9

Notes:

Max bending moment and anchor load values provided in Table 5 are per foot of wall.

TABLE 6. CWALSHT (CASE 2002) ANALYSES SUMMARY – GROUND IMPROVEMENT

Load Case and Description	Recommended Pile Toe Elevation (feet)	Max Bending Moment (kip*feet)	Elevation at Max Bending Moment (feet)	Anchor Load (kips)
Water Elevations: Land Side = 2 feet and Water Side = 0 feet				
1. Static Loading, Uniform 250 psf Live Load	-9.0	3.3	7.2	1.2
2. Static Loading, HS20-44 Vehicular Live Load	-9.0	3.2	7.3	1.1
Water Elevations: Land Side = 8.1 feet and Water Side = 6.1 feet				
3. Static Loading, Uniform 250 psf Live Load	-10.0	6.5	6.1	1.6
4. Static Loading, HS20-44 Vehicular Live Load	-9.9	6.5	6.1	1.6
Water Elevations: Land Side = 4 feet and Water Side = 0 feet				
5. Static Loading, Tidal Drawdown	-11.6	5.1	2.0	1.3
Water Elevation at 0 feet				
6. Seismic Loading	-23.3	37.8	6.2	7.0

Note:

Max bending moment and anchor load values provided in Table 6 are per foot of wall.

Based on the CWALSHT analyses results, it is recommended that a single row of anchors spaced at 8 to 10 feet along the length of the bulkhead be designed. It is also recommended that the bond zone for each anchor be installed within the siltstone layer, which as a consequence, results in estimated anchor lengths on the order of 130 to 150 feet based on the assumed subsurface conditions. For conceptual anchor design, it is recommended that an allowable unit bond strength within the siltstone layer of 35 psi be used.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on review of subsurface conditions present in the vicinity of the project site, review of proposed concepts for repair or replacement of highlighted project elements, and results of preliminary geotechnical analyses, it is our opinion that the site is suitable for the proposed project from a geotechnical standpoint. However, no subsurface data was found to be available for the project site, therefore analyses results and recommendations presented in this geotechnical report are based on assumed subsurface conditions and are intended for conceptual design purposes only. Final design analyses and recommendations, as well as construction documents, should be prepared only after a subsurface exploration program is performed specifically for this project.

The subsurface exploration program should generally consist of soil borings or a combination of soil borings and cone penetrometer test (CPT) soundings. A total of three to four boring/CPT locations should be advanced, one at the TraveLift Pier, one at the Jessie's Wharf East Bulkhead, one at the West Access Pier, and one at a select location where shoreline protection will be constructed. The soil borings should be advanced to depths of 10 to 20 feet below the top of the siltstone layer and rock coring techniques be used to sample the siltstone. The subsurface exploration program would provide data that would resolve issues due to uncertainty currently impacting conceptual design. In addition to refining soil layering and soil engineering properties for use in design, general geotechnical related items currently impacting conceptual design that would be addressed with a subsurface exploration program include the following:

- Definition of the extent of the liquefaction hazard present at the project site. This especially impacts lateral spreading estimates for overall project design, seismic slope stability of any proposed shoreline protection, the seismic design of the proposed bulkhead, and any proposed ground improvement.
- Definition of the depth to rock (siltstone). This especially impacts the estimated length of anchors for the proposed bulkhead as well as estimates for allowable pile capacity and pile length. Currently, it is our understanding that the estimated allowable uplift capacity of the TraveLift Pier piles is not large enough to carry the anticipated uplift loads, and therefore, Moffatt & Nichol is currently proposing anchoring the pile caps to the siltstone layer in order to produce enough uplift capacity.

7.0 LIMITATIONS

We have prepared this report for the exclusive use of Port of Ilwaco, Moffatt & Nichol, and their authorized agents and/or regulatory agencies for the proposed Port of Ilwaco Marina Repairs Conceptual Design Project in Ilwaco, Washington.

As stated previously, no subsurface data was found to be available for the project site, therefore analyses results and recommendations presented in this draft geotechnical report are based on assumed subsurface conditions and are intended for conceptual design purposes only. Final design analyses and recommendations, as well as construction documents, should be prepared only after a subsurface exploration program is performed specifically for this project.

This report is not intended for use by others, and the information contained herein is not applicable to other sites. No other party may rely on the product of our services unless we agree in advance and in writing to such reliance.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in the area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Please refer to Appendix D titled "Report Limitations and Guidelines for Use" for additional information pertaining to use of this report.

8.0 REFERENCES

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FIGURES

GeoEngineers

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Feet

Data Source: Bing Imagery

Site Plan

Port of Ilwaco Marina Repairs Conceptual Design
Port of Ilwaco, Washington

GEOENGINEERS

Figure 1

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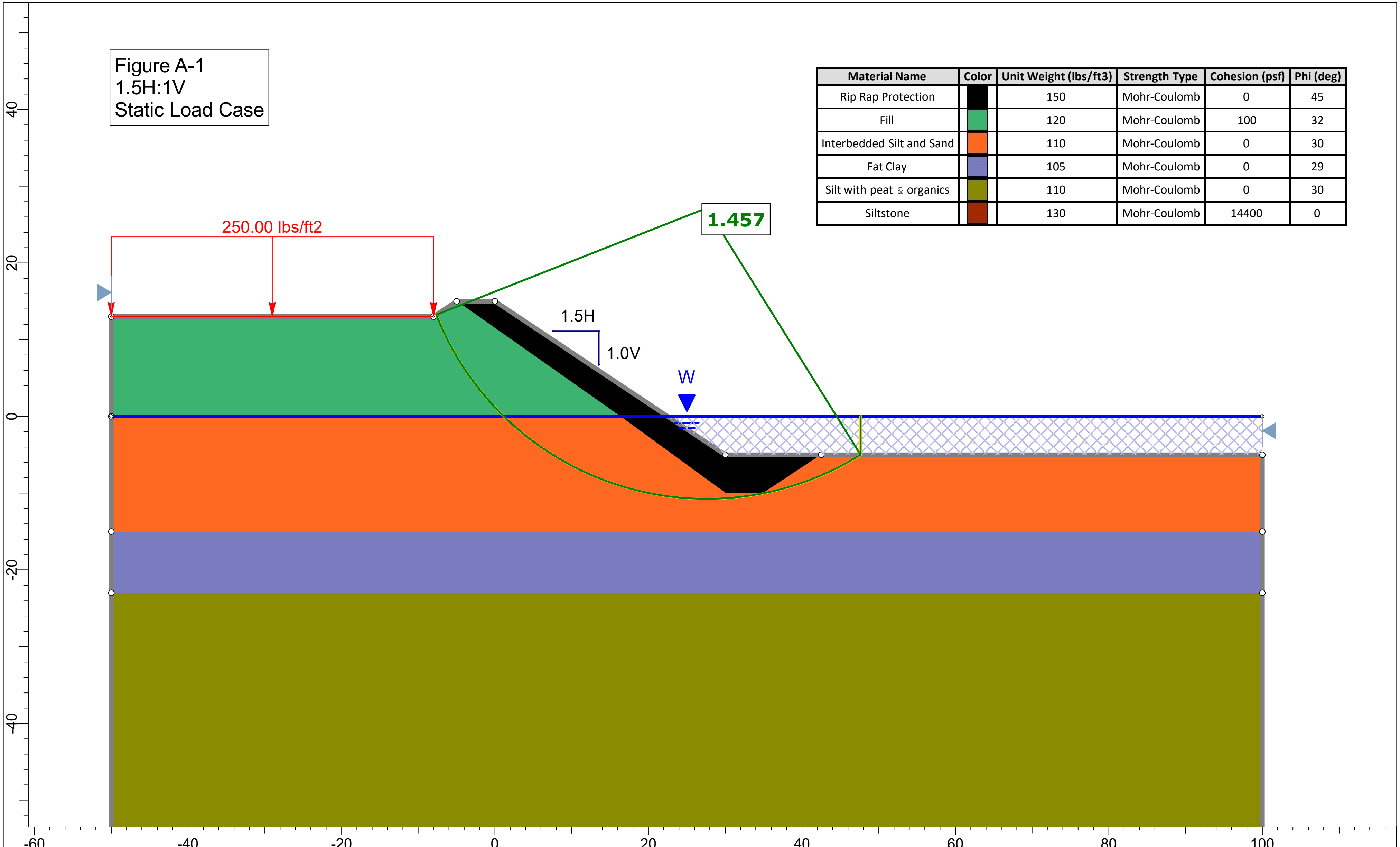
Notes:

APPENDICES

GeoEngineers

APPENDIX A
Slope Stability Results, SLIDE Version 9 (Rocscience 2020)



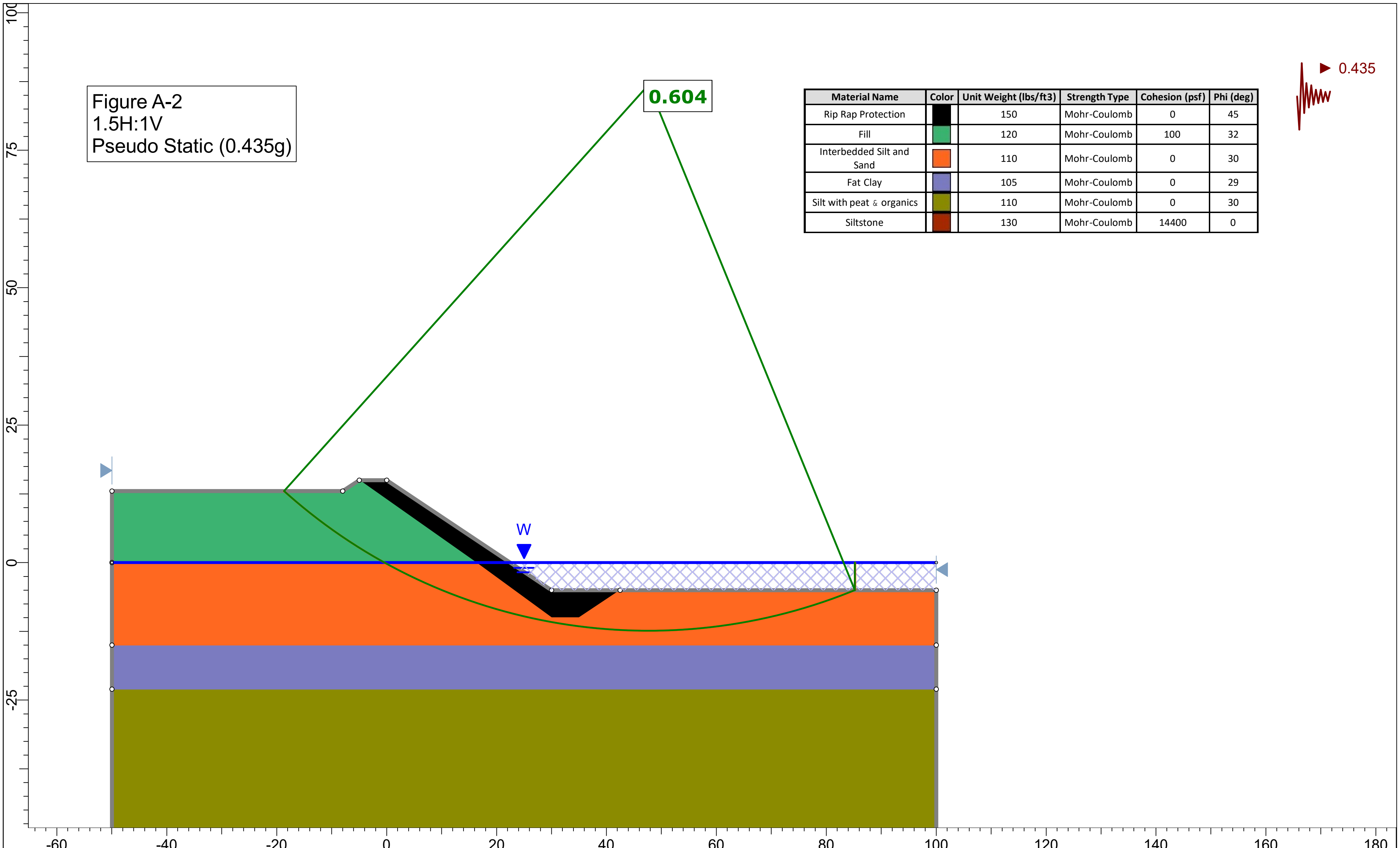
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Rip Rap Protection		150	Mohr-Coulomb	0	45
Fill		120	Mohr-Coulomb	100	32
Interbedded Silt and Sand		110	Mohr-Coulomb	0	30
Fat Clay		105	Mohr-Coulomb	0	29
Silt with peat & organics		110	Mohr-Coulomb	0	30
Siltstone		130	Mohr-Coulomb	14400	0

rocscience

SLIDEINTERPRET 9.012

Project	Port of Ilwaco		
Group	Static	*Scenario*	Master Scenario
Drawn By	B Hoffman	*Company*	GeoEngineers
Date	2/15/2021, 10:05:33 AM	*File Name*	Shoreline.slmd

Figure A-2
1.5H:1V
Pseudo Static (0.435g)



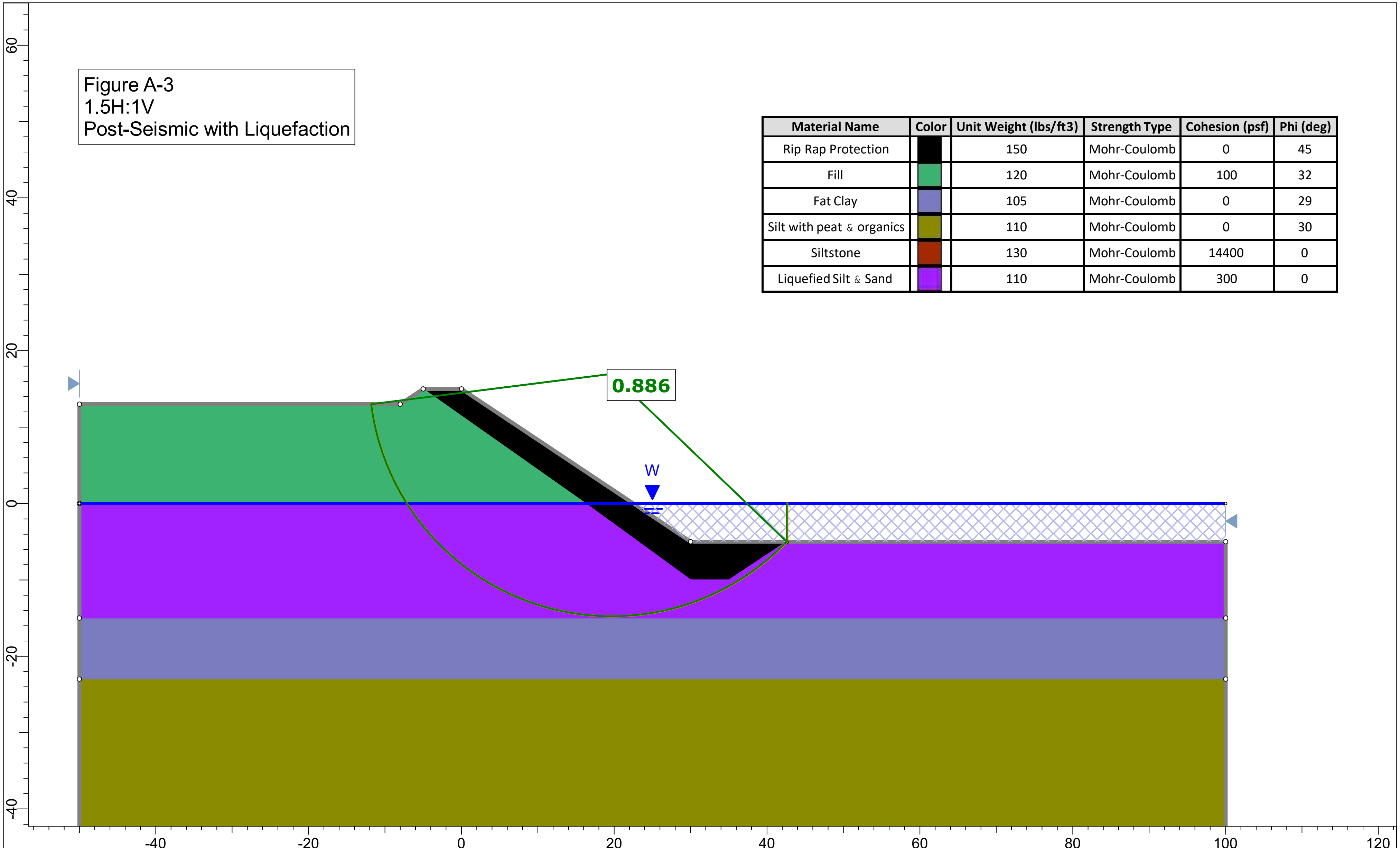
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Rip Rap Protection		150	Mohr-Coulomb	0	45
Fill		120	Mohr-Coulomb	100	32
Interbedded Silt and Sand		110	Mohr-Coulomb	0	30
Fat Clay		105	Mohr-Coulomb	0	29
Silt with peat & organics		110	Mohr-Coulomb	0	30
Siltstone		130	Mohr-Coulomb	14400	0

rocscience

SLIDEINTERPRET 9.012

Project	Port of Ilwaco		
Group	Seismic	*Scenario*	Pseudo-Static
Drawn By	B Hoffman	*Company*	GeoEngineers
Date	2/15/2021, 10:05:33 AM	*File Name*	Shoreline.slmd

Figure A-3
1.5H:1V
Post-Seismic with Liquefaction

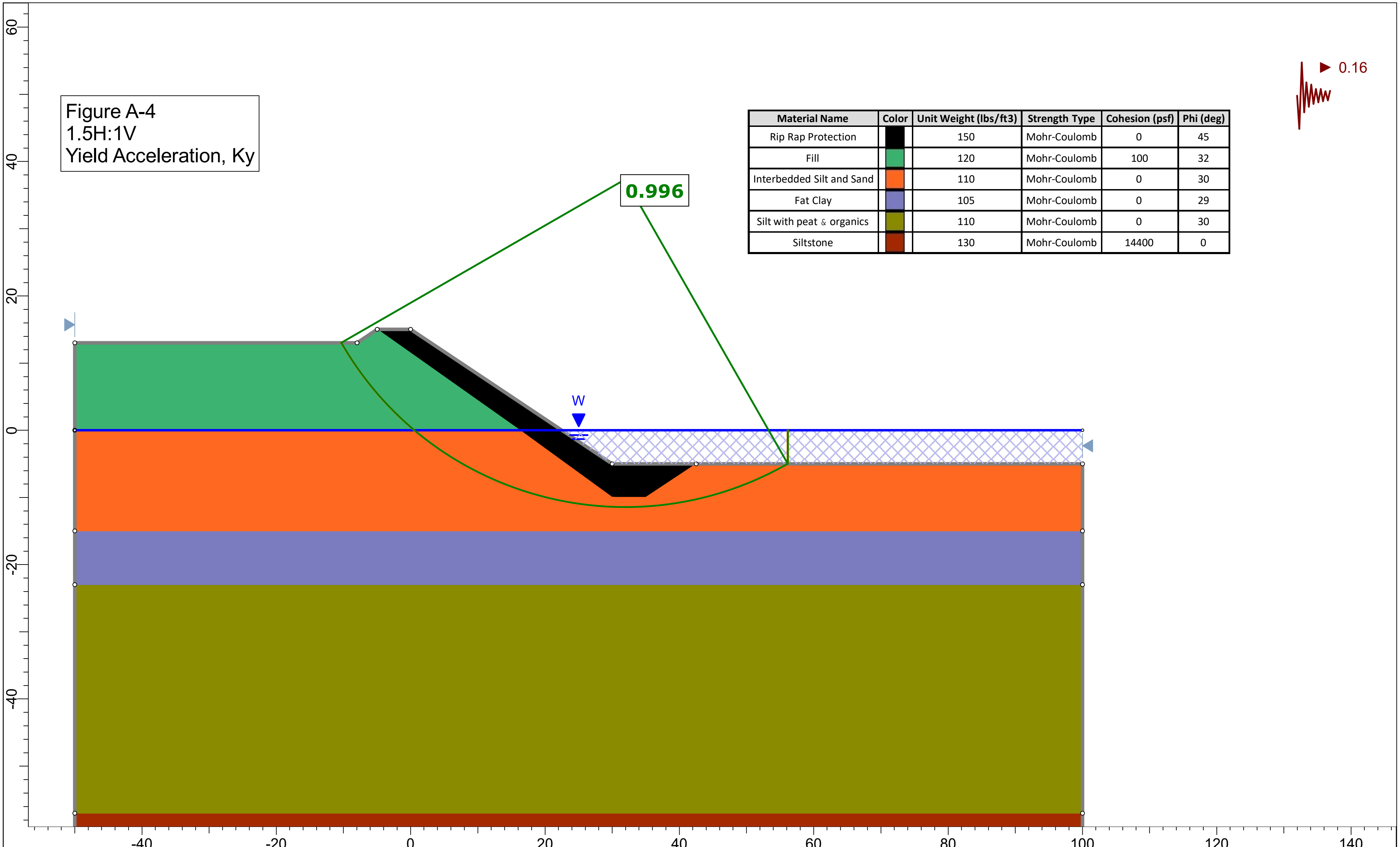


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Rip Rap Protection		150	Mohr-Coulomb	0	45
Fill		120	Mohr-Coulomb	100	32
Fat Clay		105	Mohr-Coulomb	0	29
Silt with peat & organics		110	Mohr-Coulomb	0	30
Siltstone		130	Mohr-Coulomb	14400	0
Liquefied Silt & Sand		110	Mohr-Coulomb	300	0

rocscience

Project	Port of Ilwaco		
Group	Seismic	*Scenario*	Post-Seismic
Drawn By	B Hoffman	*Company*	GeoEngineers
Date	2/15/2021, 10:05:33 AM	*File Name*	Shoreline.slmd

SLIDEINTERPRET 9.012



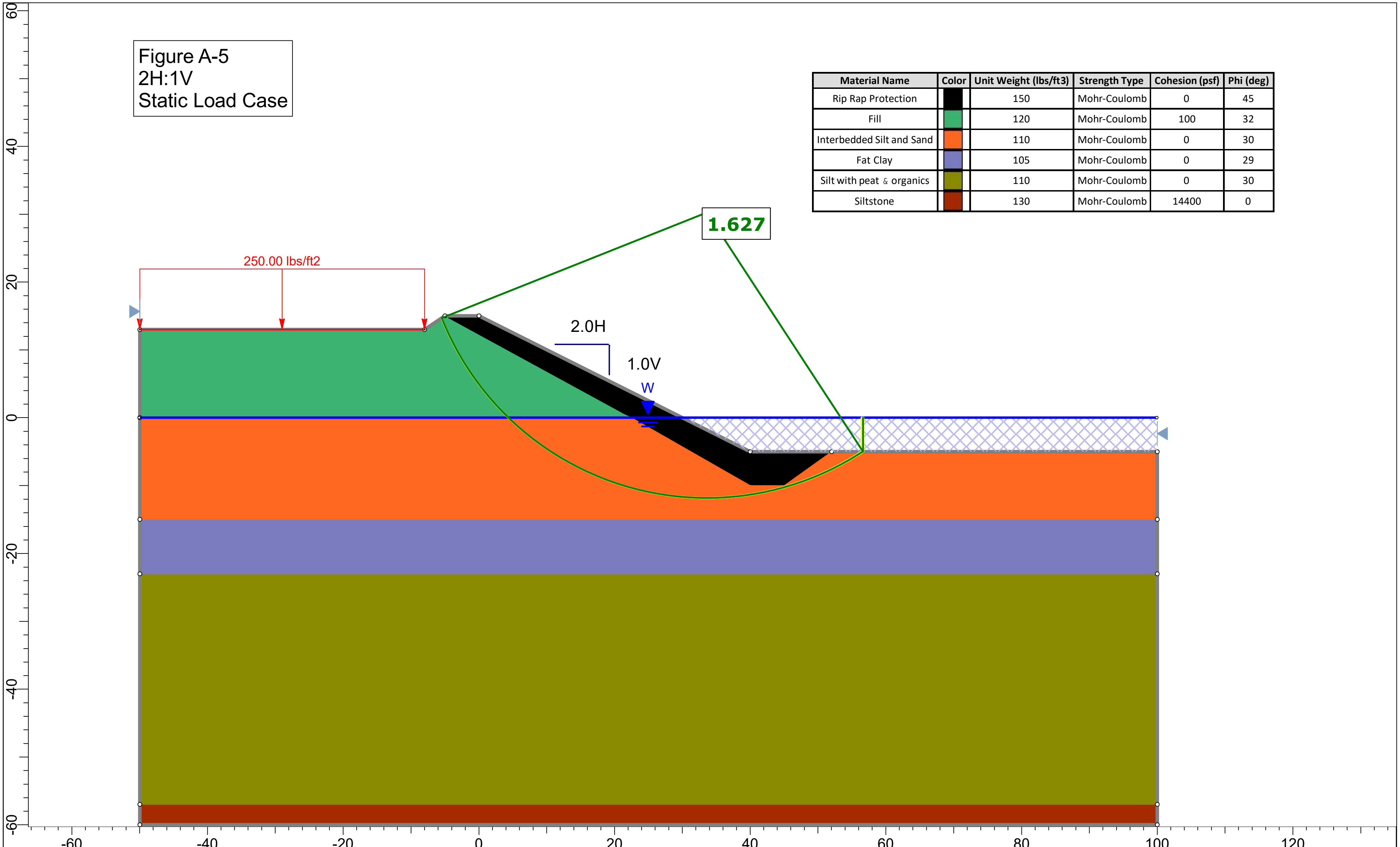
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Rip Rap Protection		150	Mohr-Coulomb	0	45
Fill		120	Mohr-Coulomb	100	32
Interbedded Silt and Sand		110	Mohr-Coulomb	0	30
Fat Clay		105	Mohr-Coulomb	0	29
Silt with peat & organics		110	Mohr-Coulomb	0	30
Siltstone		130	Mohr-Coulomb	14400	0

rocscience

SLIDEINTERPRET 9.012

Project	Port of Ilwaco		
Group	Seismic	*Scenario*	Yield Acceleration
Drawn By	B Hoffman	*Company*	GeoEngineers
Date	2/15/2021, 10:05:33 AM	*File Name*	Shoreline.slmd

Figure A-5
2H:1V
Static Load Case

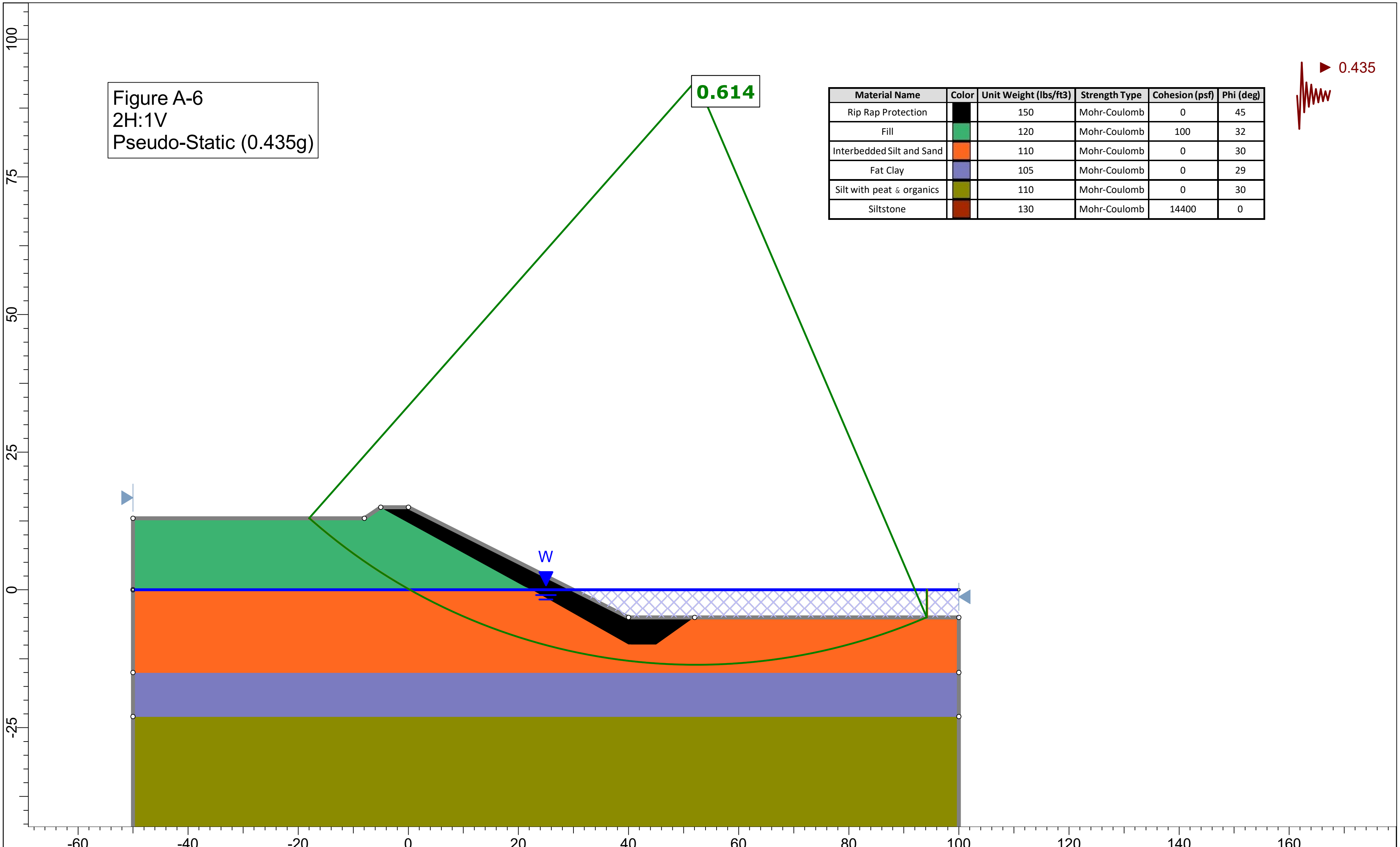


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Rip Rap Protection		150	Mohr-Coulomb	0	45
Fill		120	Mohr-Coulomb	100	32
Interbedded Silt and Sand		110	Mohr-Coulomb	0	30
Fat Clay		105	Mohr-Coulomb	0	29
Silt with peat & organics		110	Mohr-Coulomb	0	30
Siltstone		130	Mohr-Coulomb	14400	0

rocscience

SLIDEINTERPRET 9.012

Project	Port of Ilwaco		
Group	2:1 Static	*Scenario*	Master Scenario
Drawn By	B Hoffman	*Company*	GeoEngineers
Date	2/15/2021, 10:05:33 AM	*File Name*	Shoreline.slmd

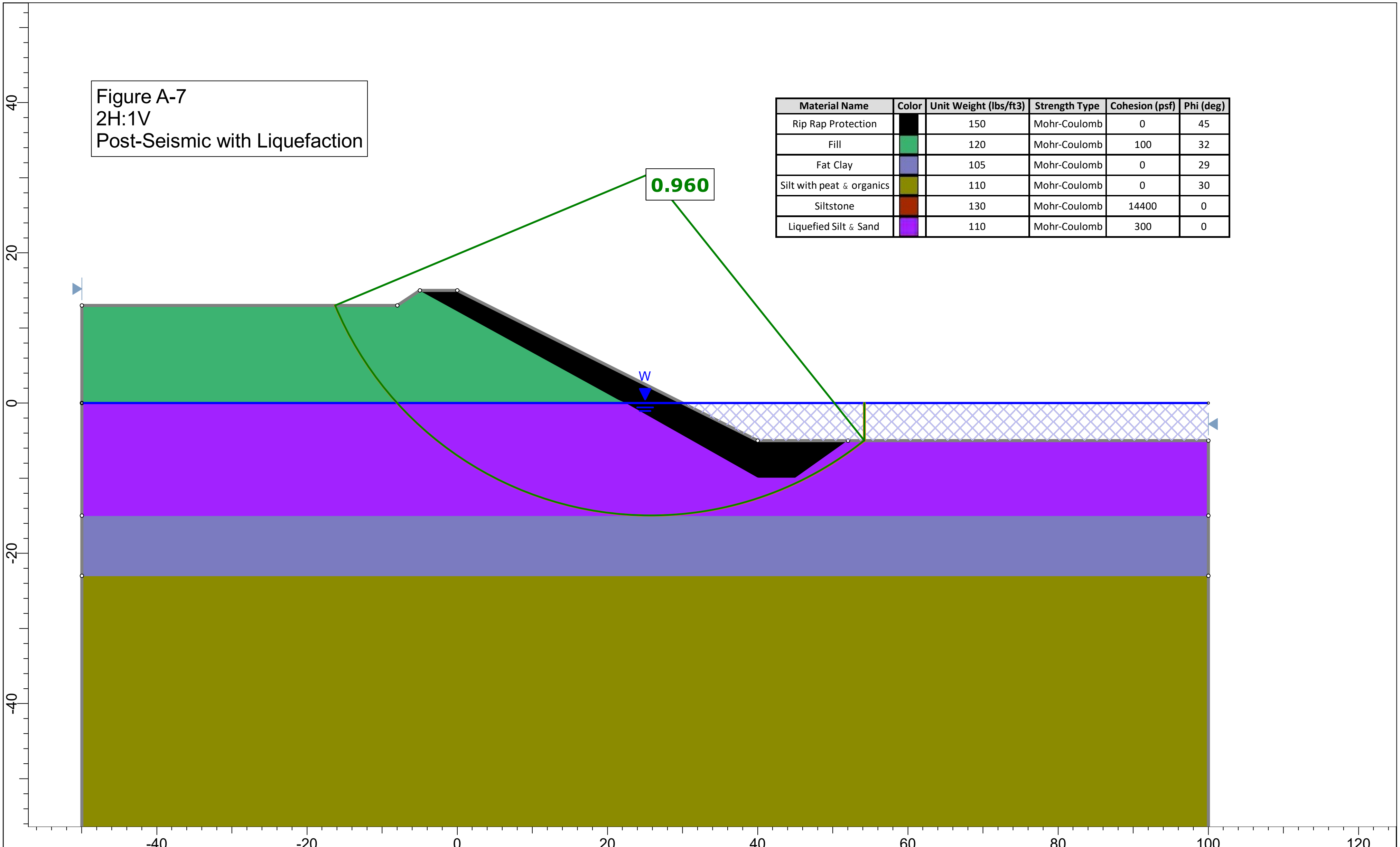


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Rip Rap Protection		150	Mohr-Coulomb	0	45
Fill		120	Mohr-Coulomb	100	32
Interbedded Silt and Sand		110	Mohr-Coulomb	0	30
Fat Clay		105	Mohr-Coulomb	0	29
Silt with peat & organics		110	Mohr-Coulomb	0	30
Siltstone		130	Mohr-Coulomb	14400	0

rocscience

SLIDEINTERPRET 9.012

Project	Port of Ilwaco		
Group	2:1 Seismic	*Scenario*	Pseudo-Static
Drawn By	B Hoffman	*Company*	GeoEngineers
Date	2/15/2021, 10:05:33 AM	*File Name*	Shoreline.slmd

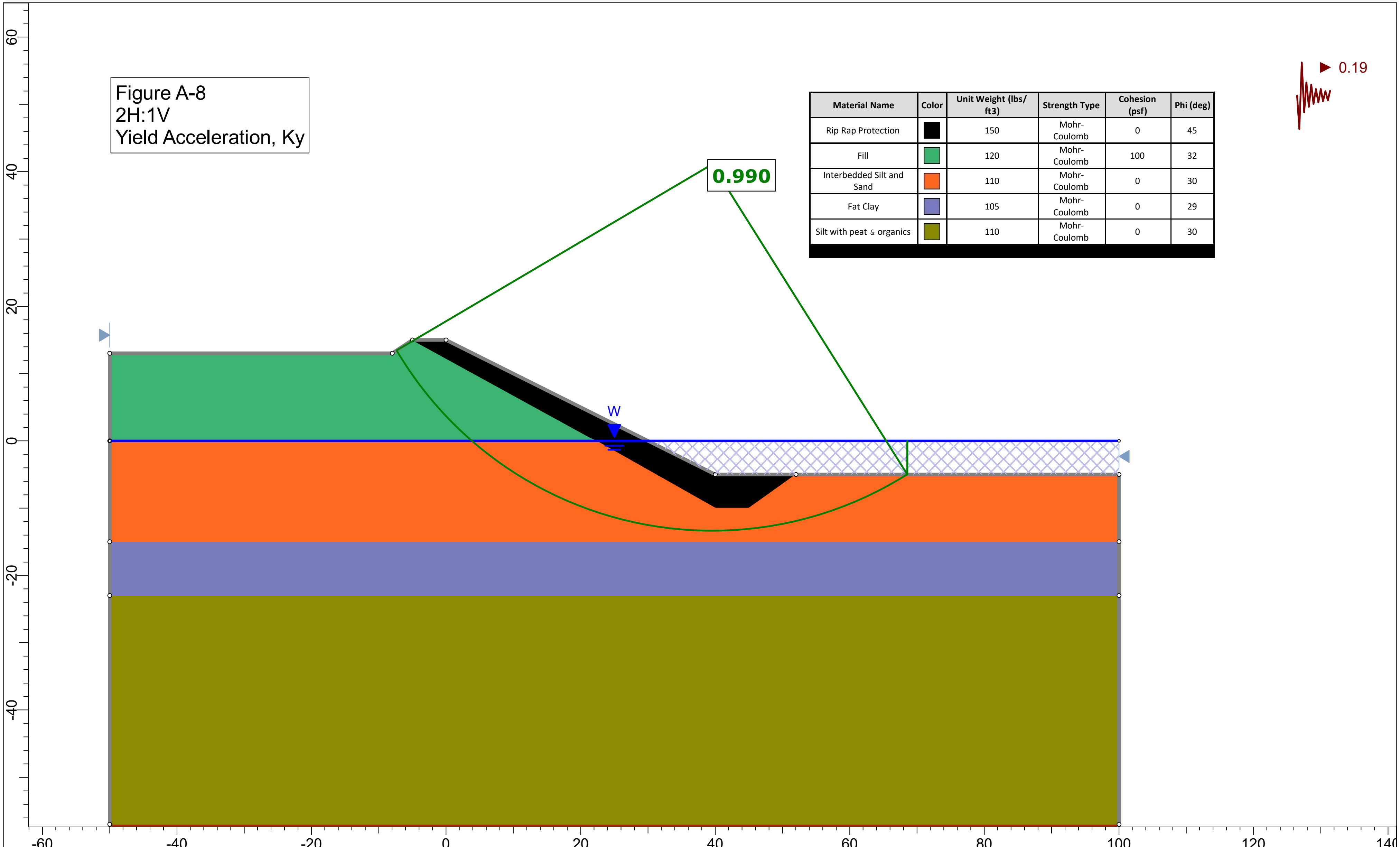


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Rip Rap Protection		150	Mohr-Coulomb	0	45
Fill		120	Mohr-Coulomb	100	32
Fat Clay		105	Mohr-Coulomb	0	29
Silt with peat & organics		110	Mohr-Coulomb	0	30
Siltstone		130	Mohr-Coulomb	14400	0
Liquefied Silt & Sand		110	Mohr-Coulomb	300	0

rocscience

SLIDEINTERPRET 9.012

Project	Port of Ilwaco		
Group	2:1 Seismic	*Scenario*	Post-Seismic
Drawn By	B Hoffman	*Company*	GeoEngineers
Date	2/15/2021, 10:05:33 AM	*File Name*	Shoreline.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Rip Rap Protection		150	Mohr-Coulomb	0	45
Fill		120	Mohr-Coulomb	100	32
Interbedded Silt and Sand		110	Mohr-Coulomb	0	30
Fat Clay		105	Mohr-Coulomb	0	29
Silt with peat & organics		110	Mohr-Coulomb	0	30

rocscience

SLIDEINTERPRET 9.012

Project	Port of Ilwaco		
Group	2:1 Seismic	*Scenario*	Yield Acceleration
Drawn By	B Hoffman	*Company*	GeoEngineers
Date	2/15/2021, 10:05:33 AM	*File Name*	Shoreline.slmd

APPENDIX B
Allowable Compression and Uplift Capacity Versus Elevation for Driven Piles

Figure B-1: TraveLift Pier

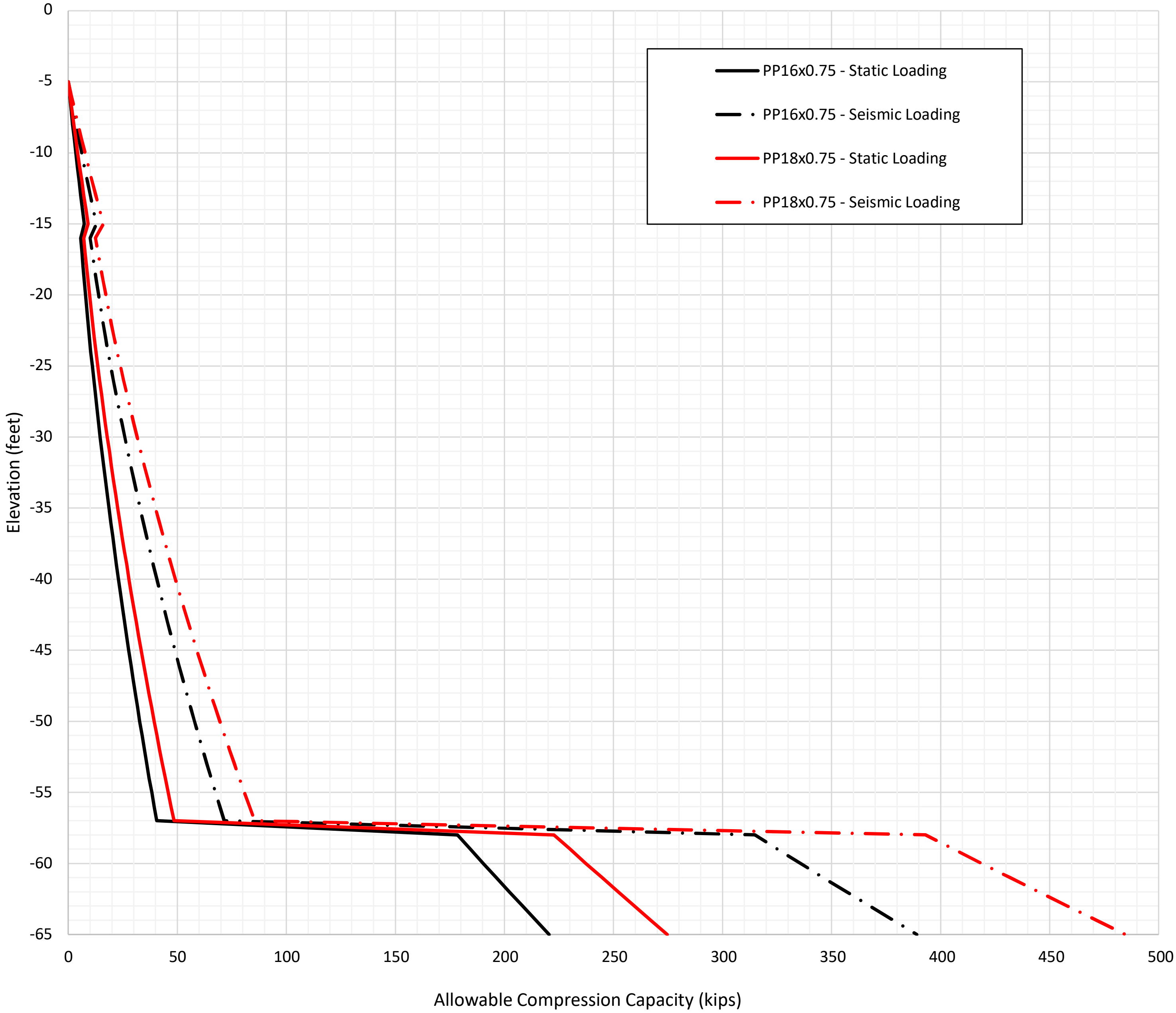


Figure B-2: TraveLift Pier

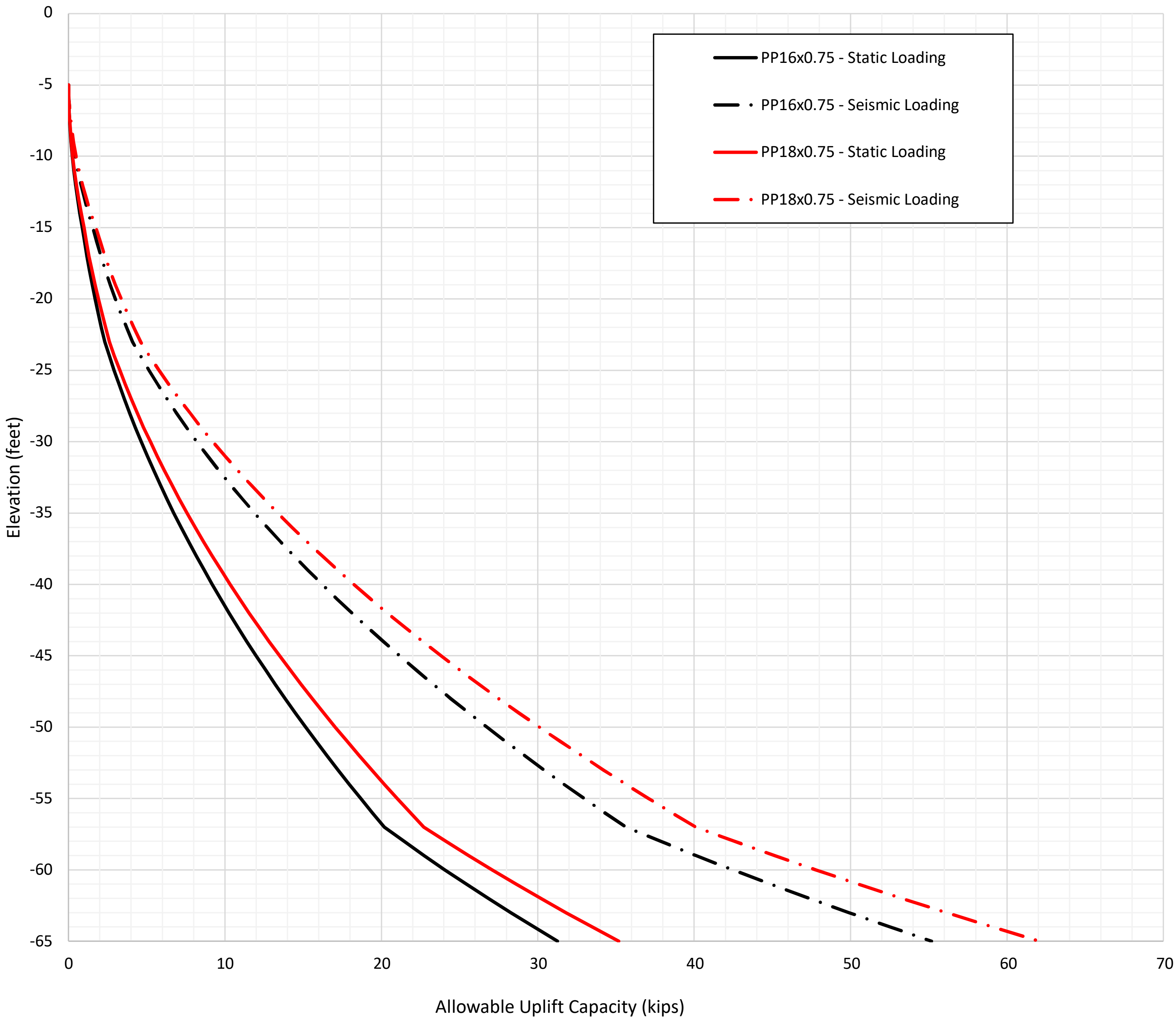


Figure B-3: West Access Pier PP12x0.75

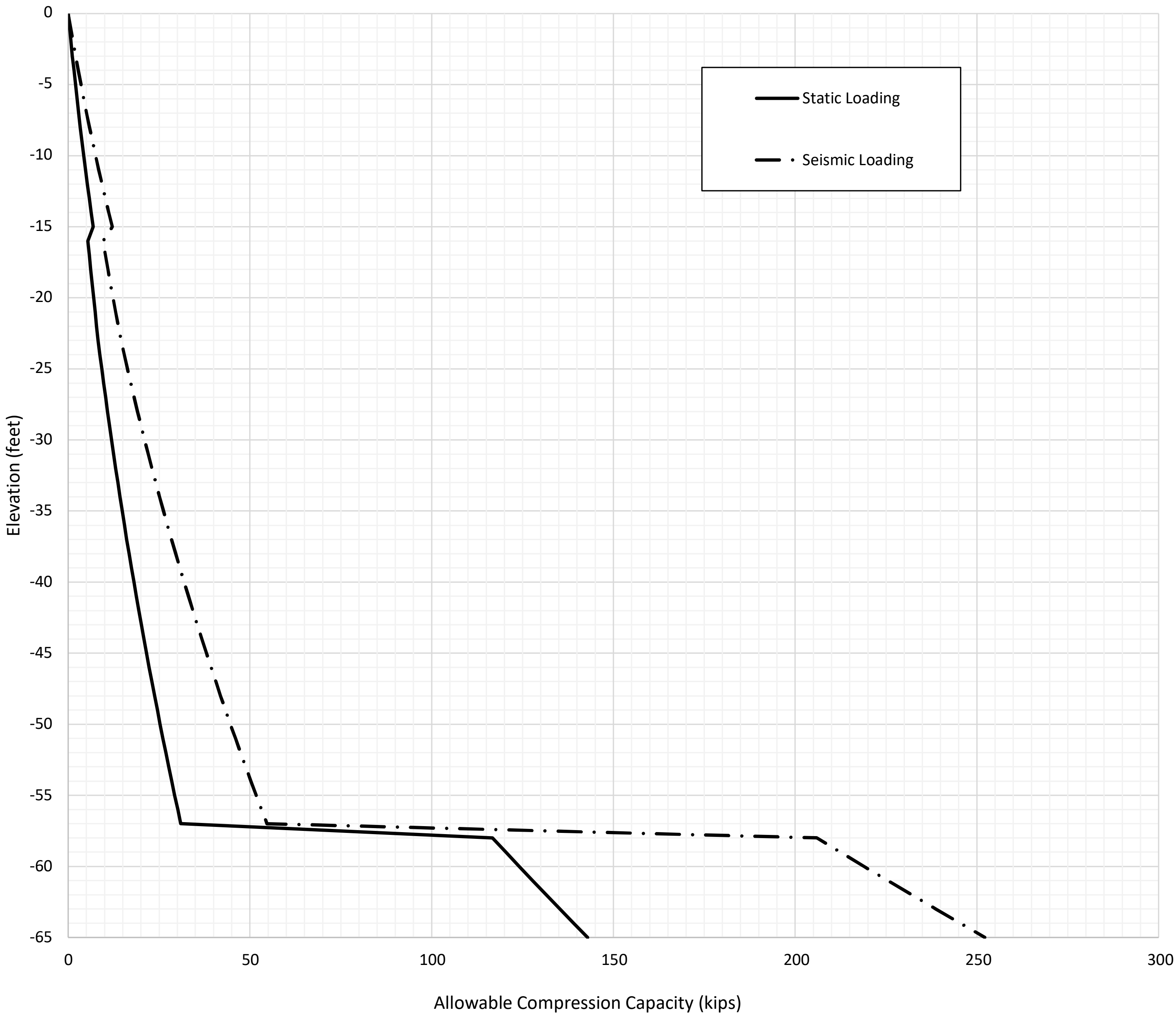
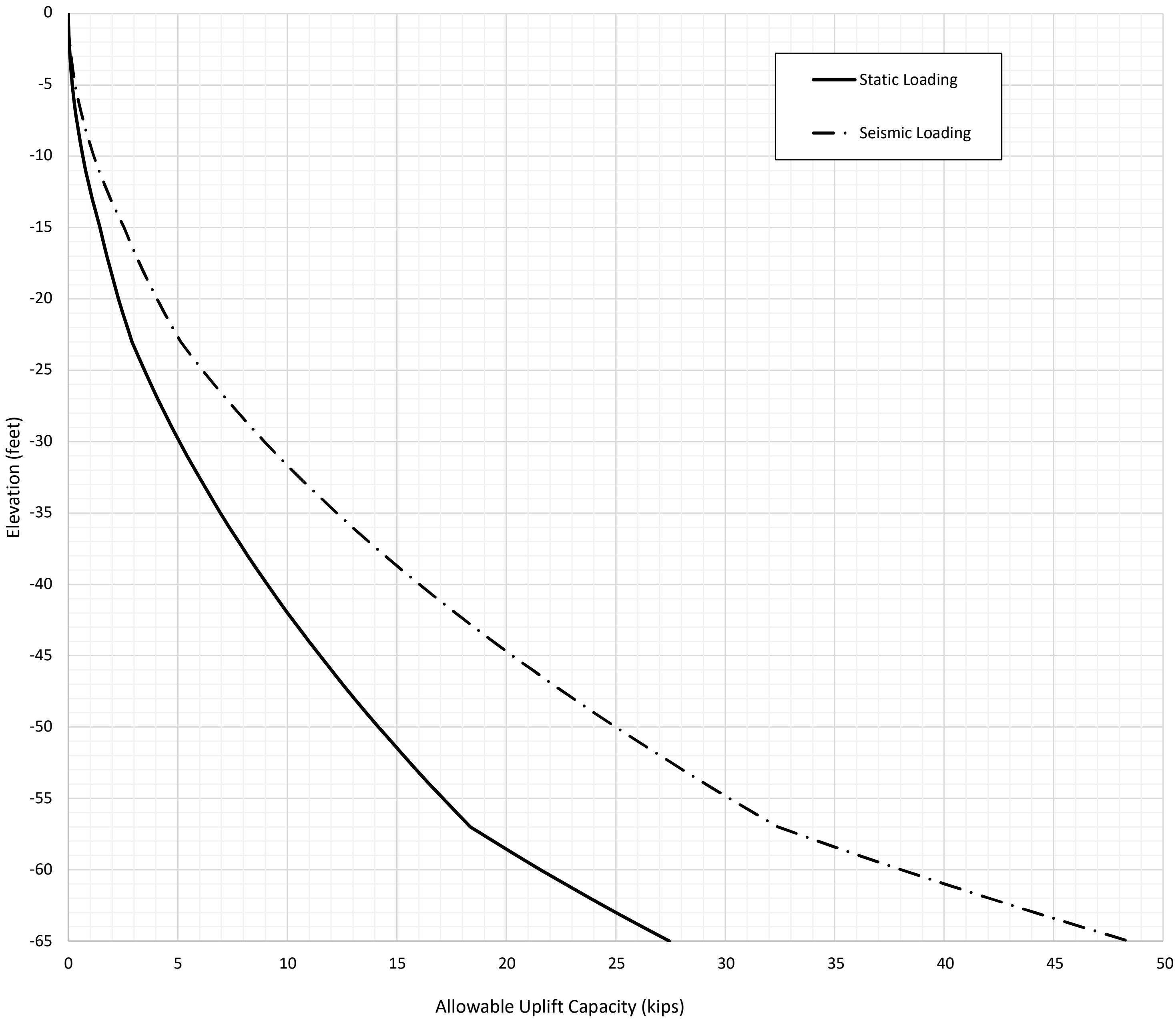


Figure B-4: West Access Pier PP12x0.75

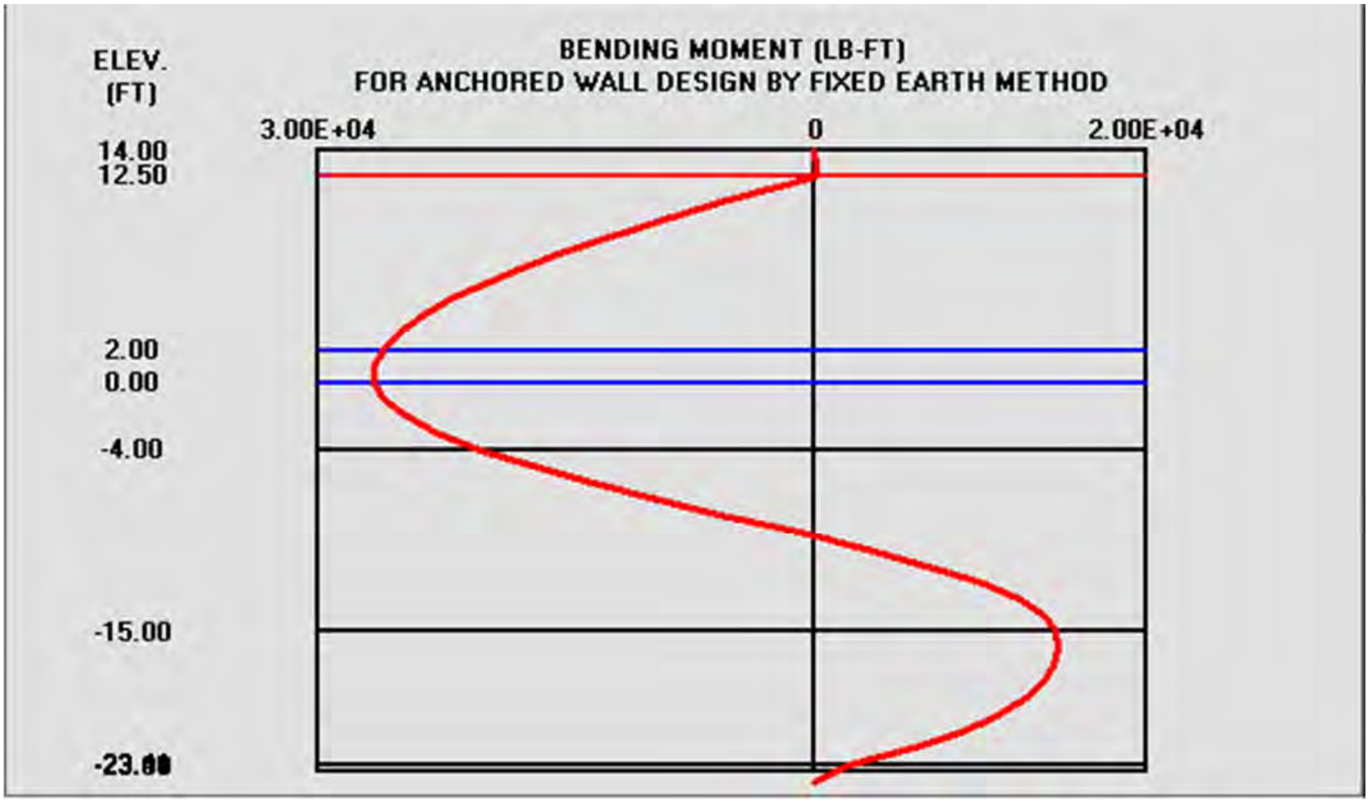


APPENDIX C
CWALSHT Results (CASE 2002)

<u>**Figure C-1 – No Ground Improvement**</u>

Load Case 1 – 250 psf Live Load

Water Elevations: Land Side = +2.0, Water Side = 0



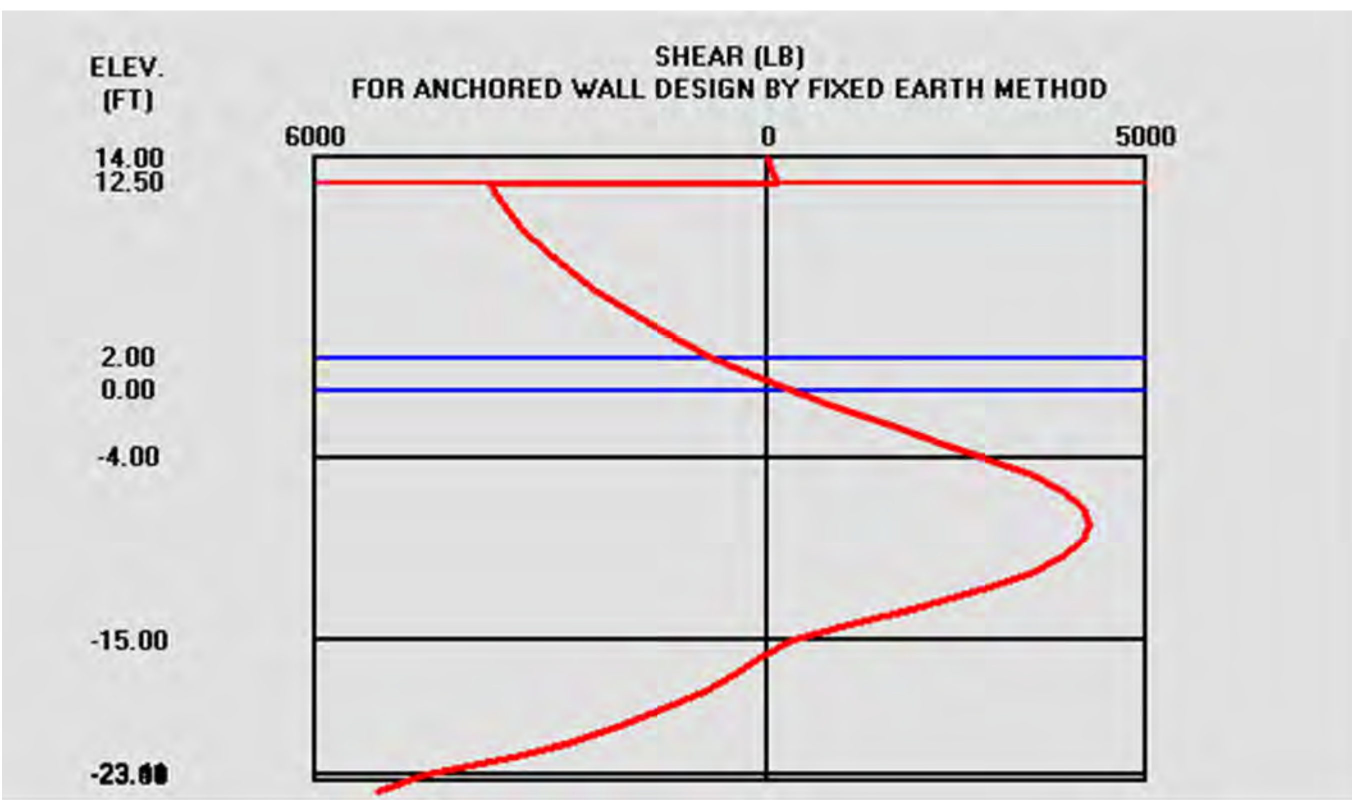
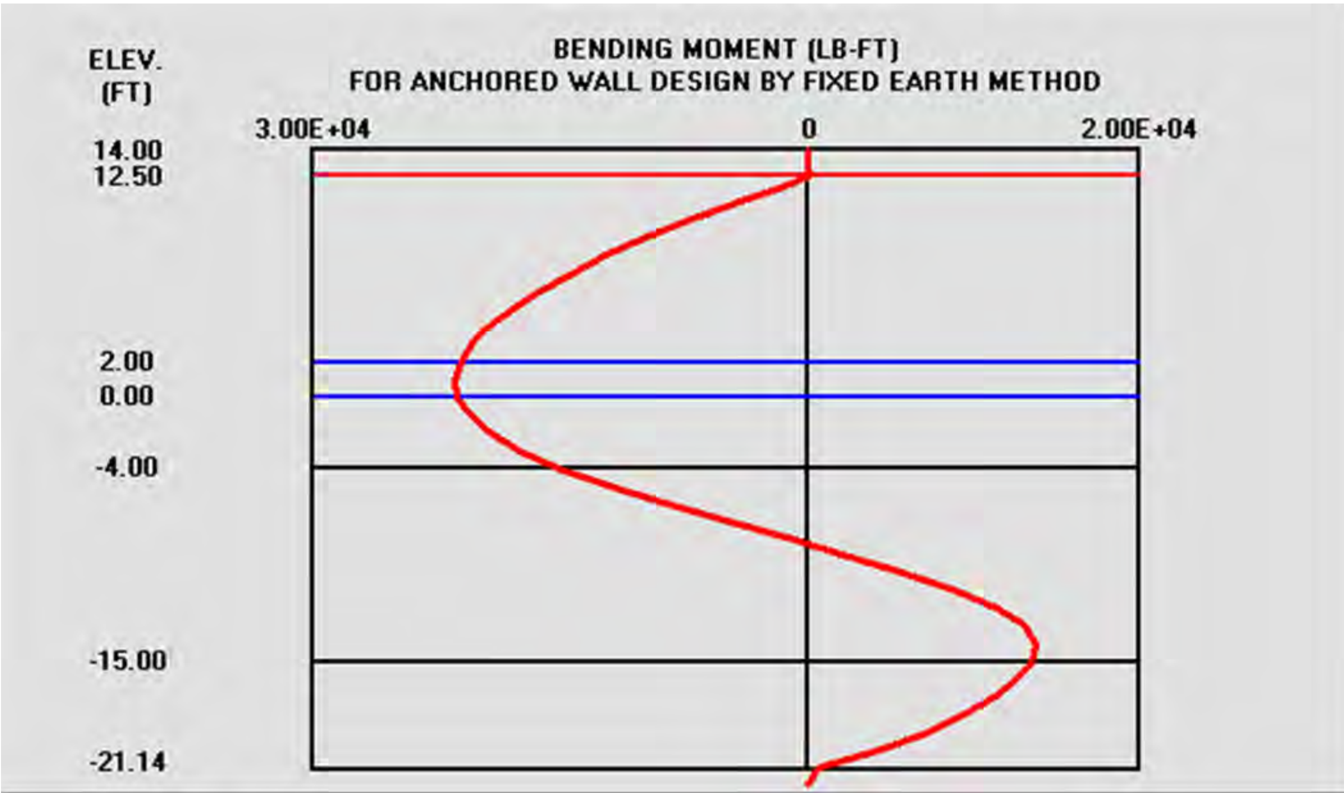


Figure C-2 – No Ground Improvement

Load Case 2 – HS20-44 Live Load

Water Elevations: Land Side = +2.0, Water Side = 0



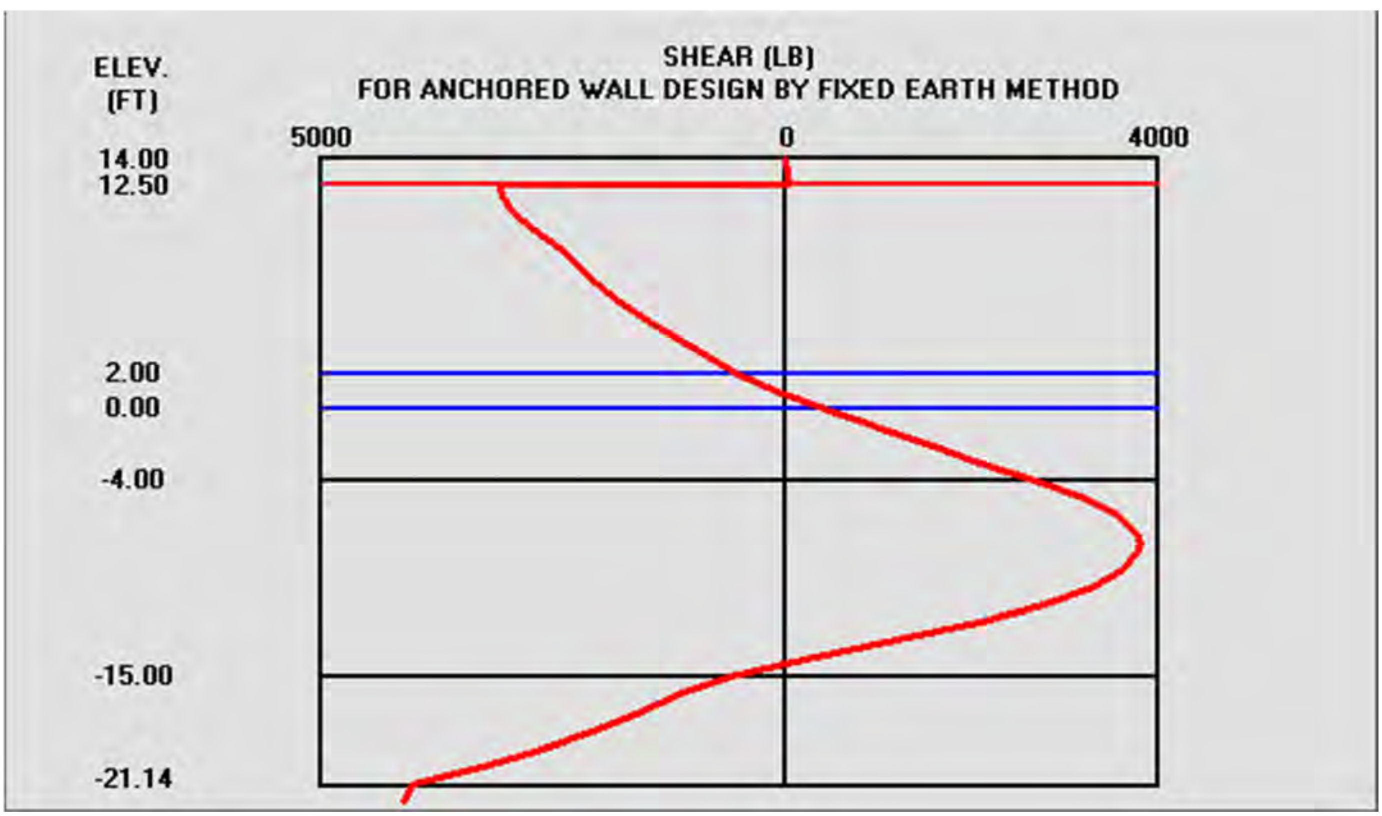
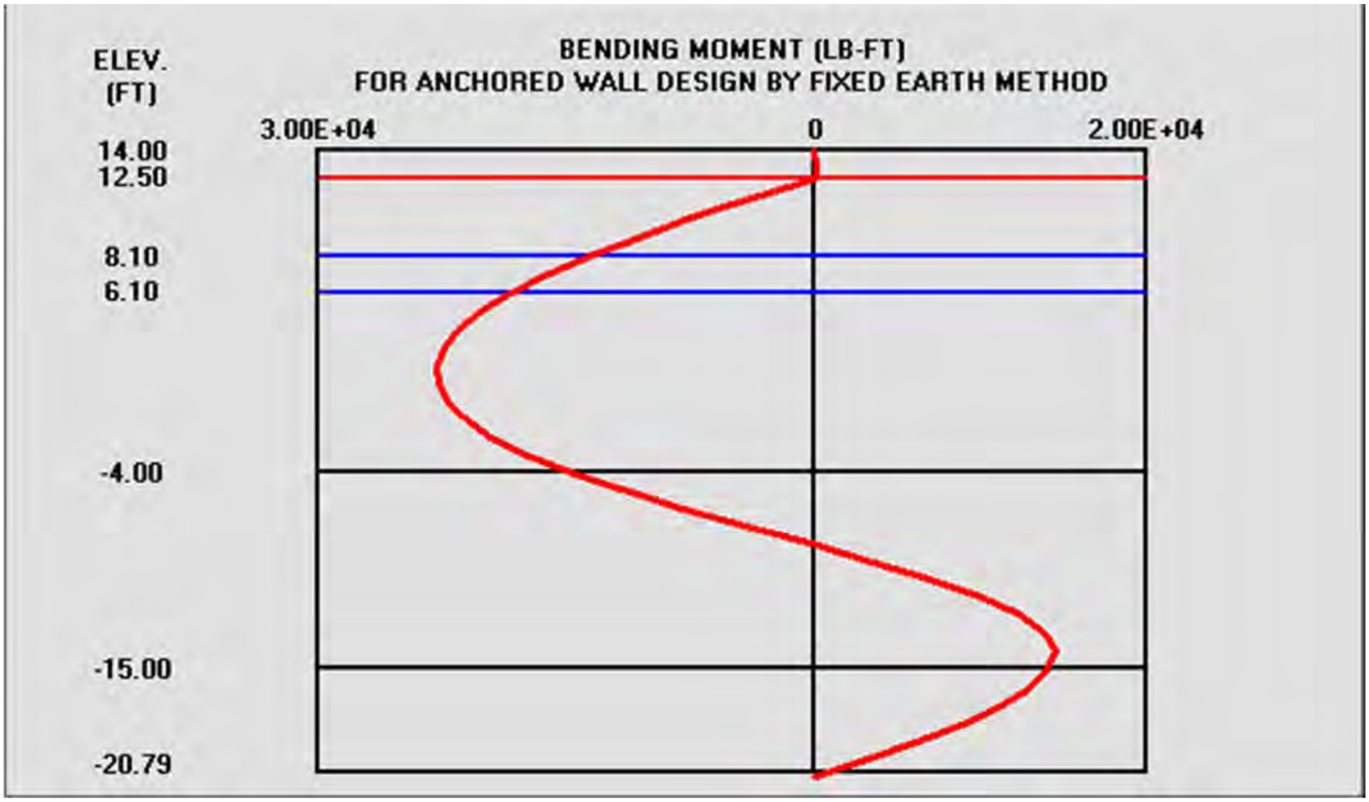


Figure C-3 – No Ground Improvement

Load Case 3 – 250 psf Live Load

Water Elevations: Land Side = +8.1, Water Side = +6.1



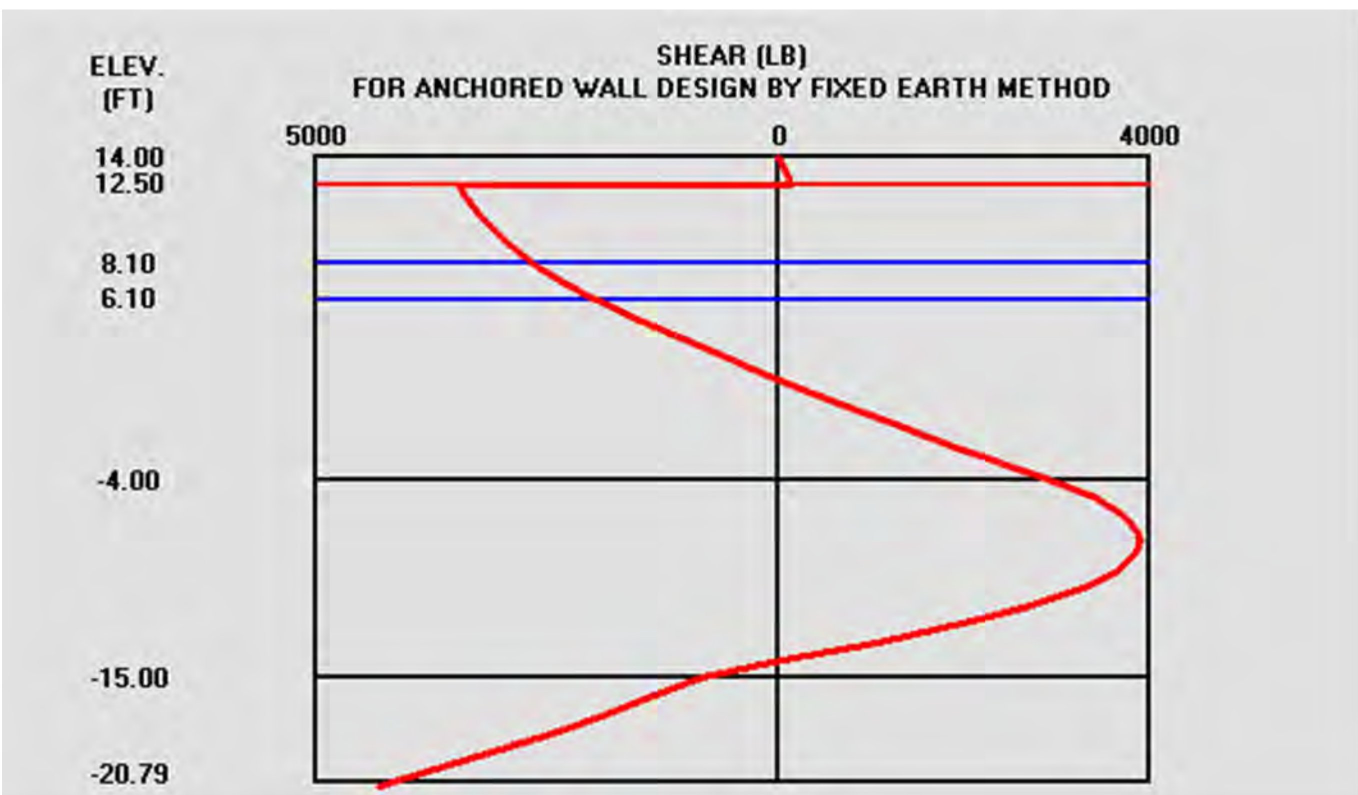
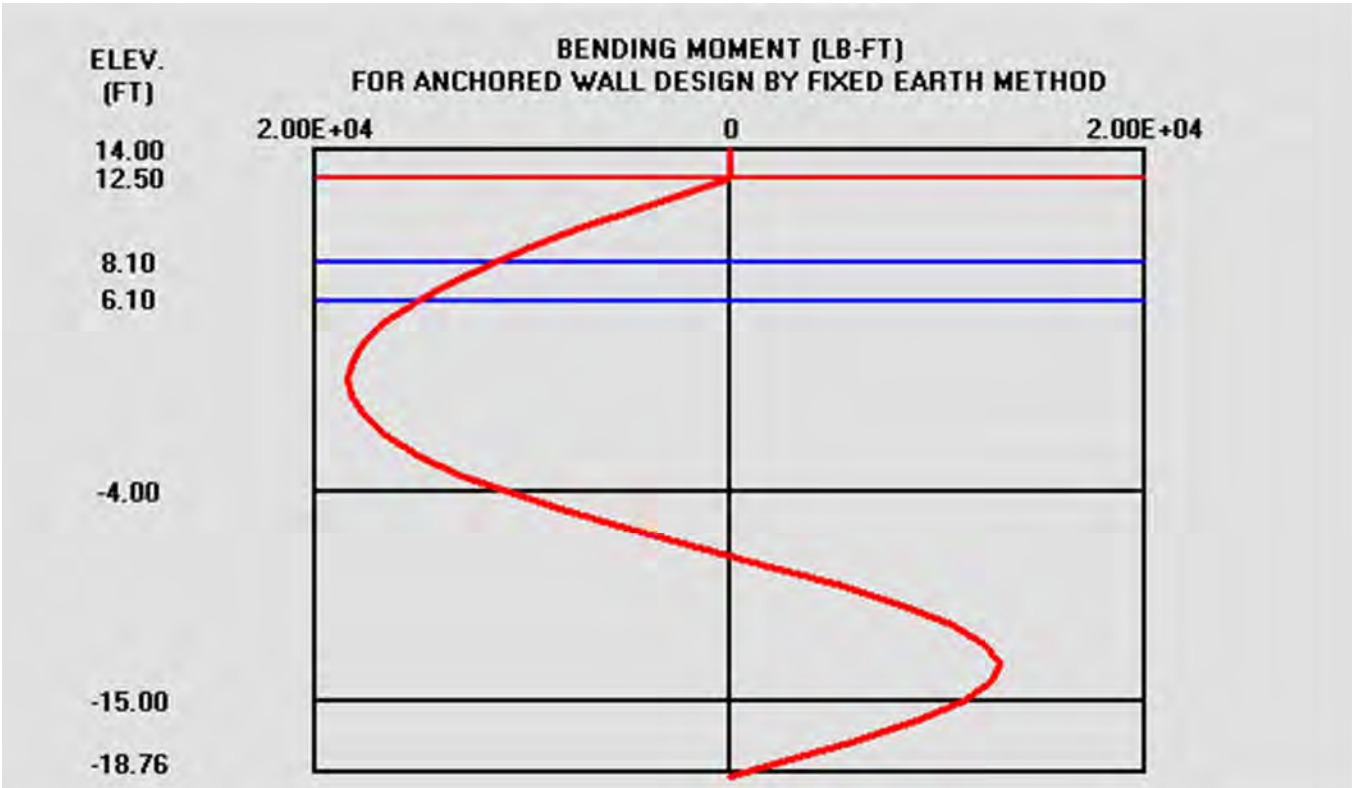


Figure C-4 – No Ground Improvement

Load Case 4 – HS20 Live Load

Water Elevations: Land Side = +8.1, Water Side = +6.1



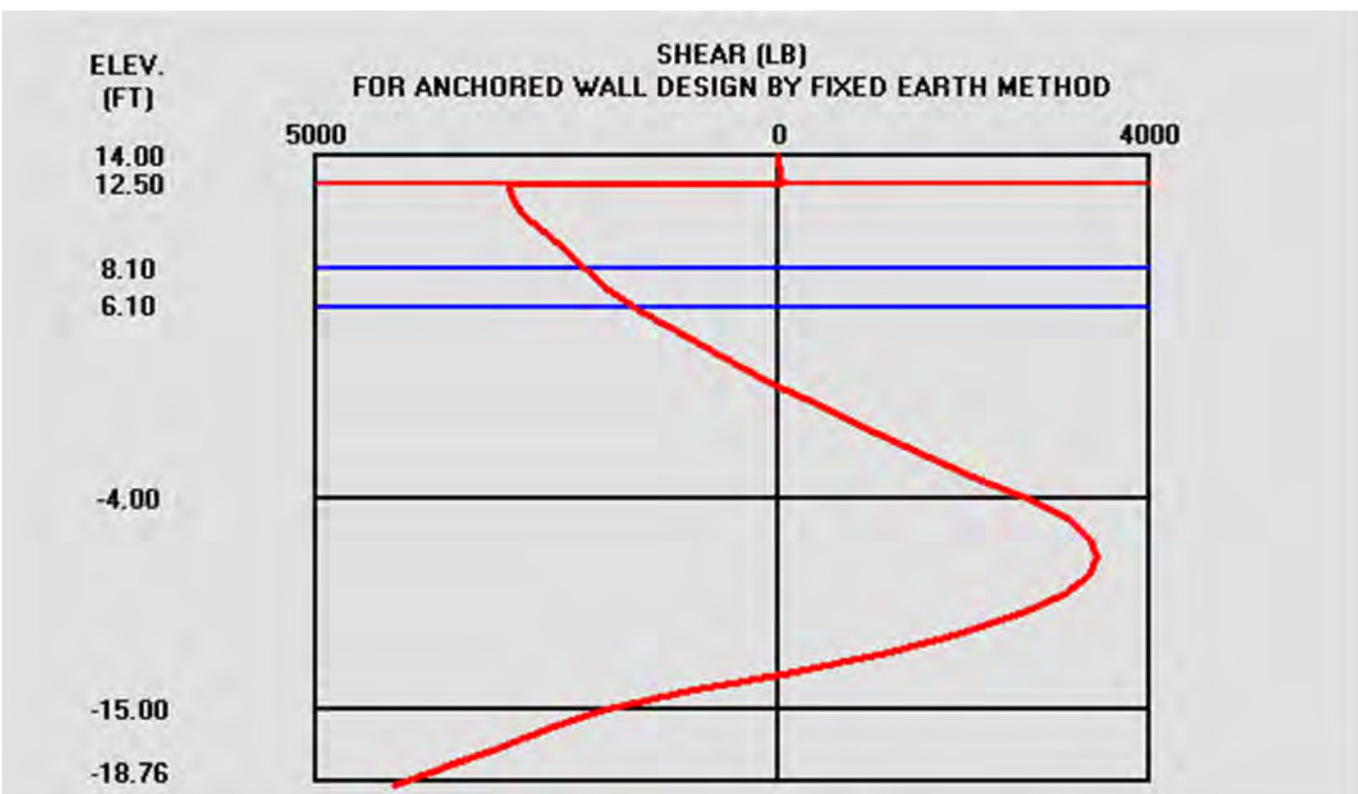
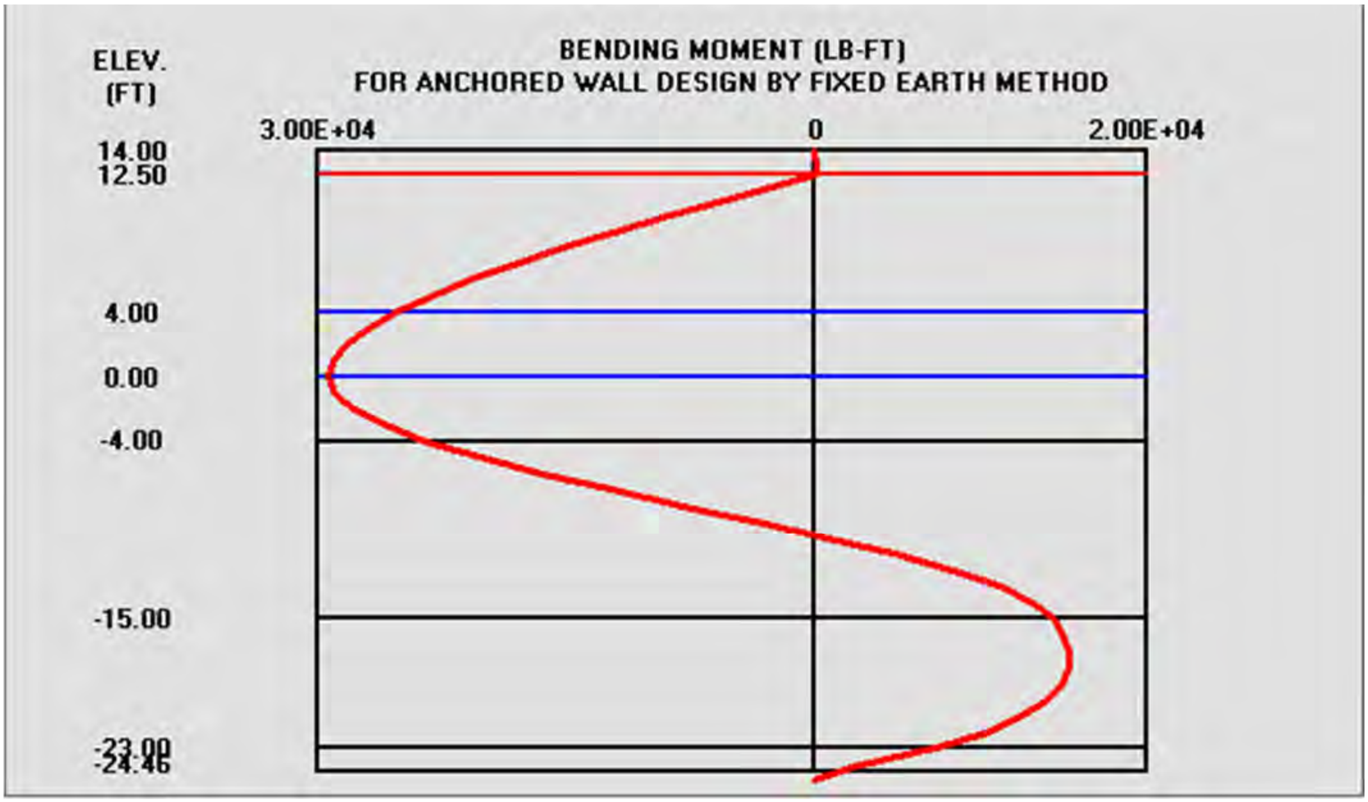


Figure C-5 – No Ground Improvement

Load Case 5 – Drawdown



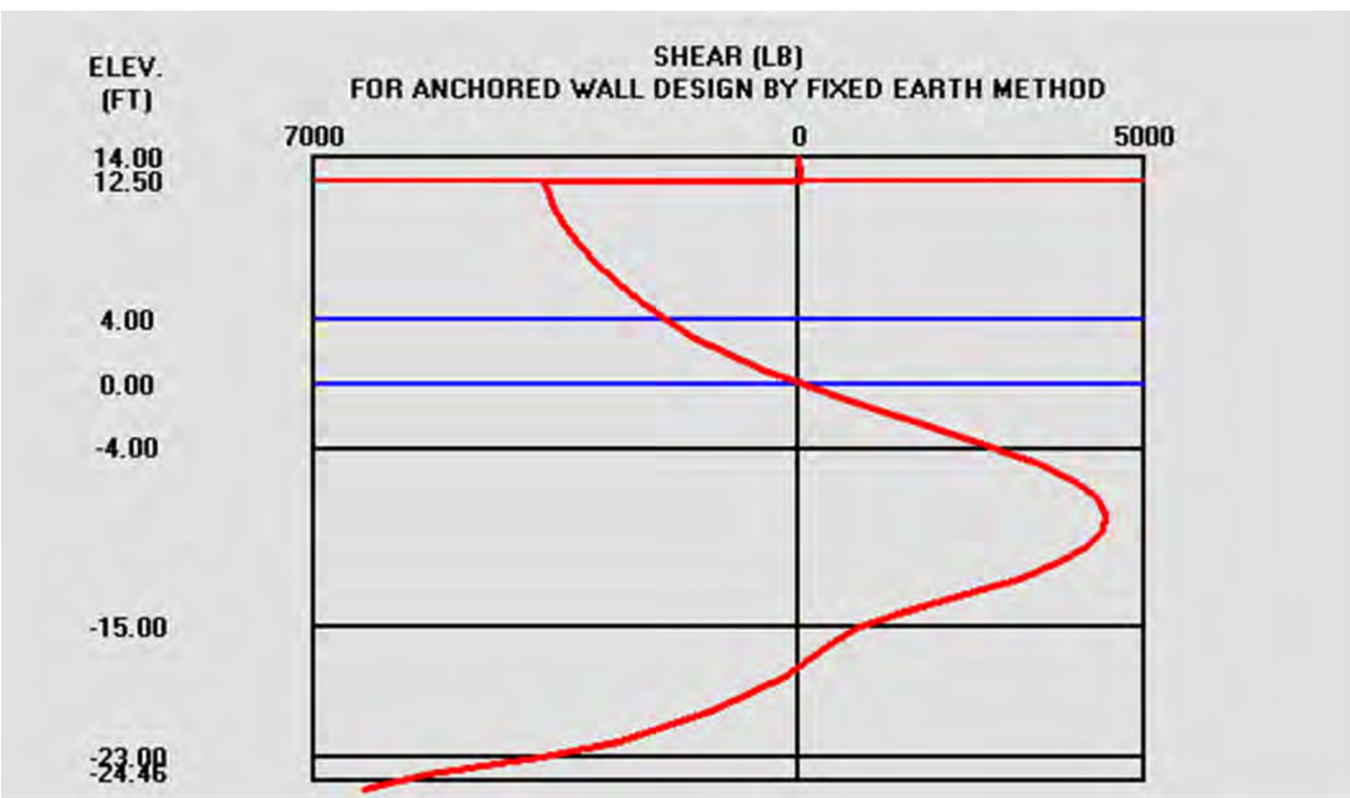
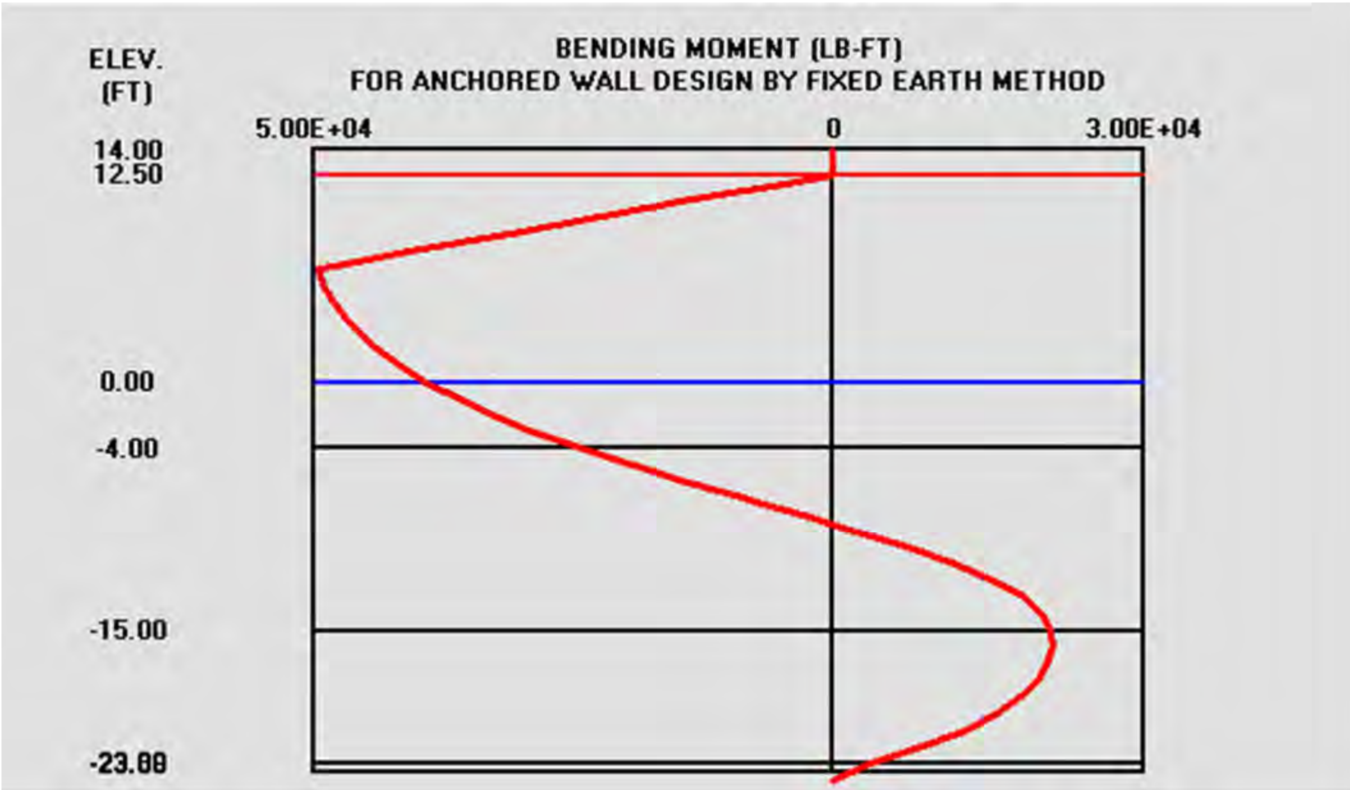


Figure C-6 – No Ground Improvement

Load Case 6 – Seismic



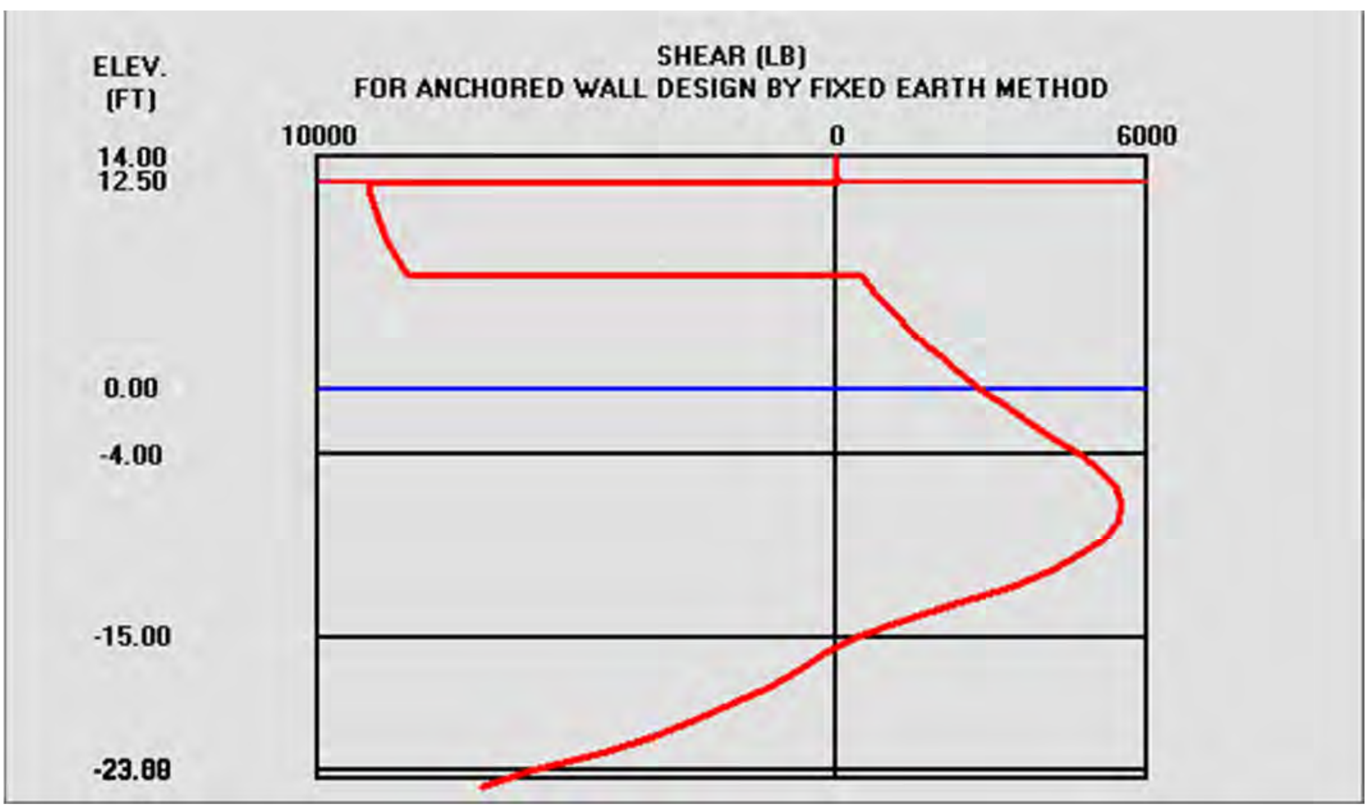
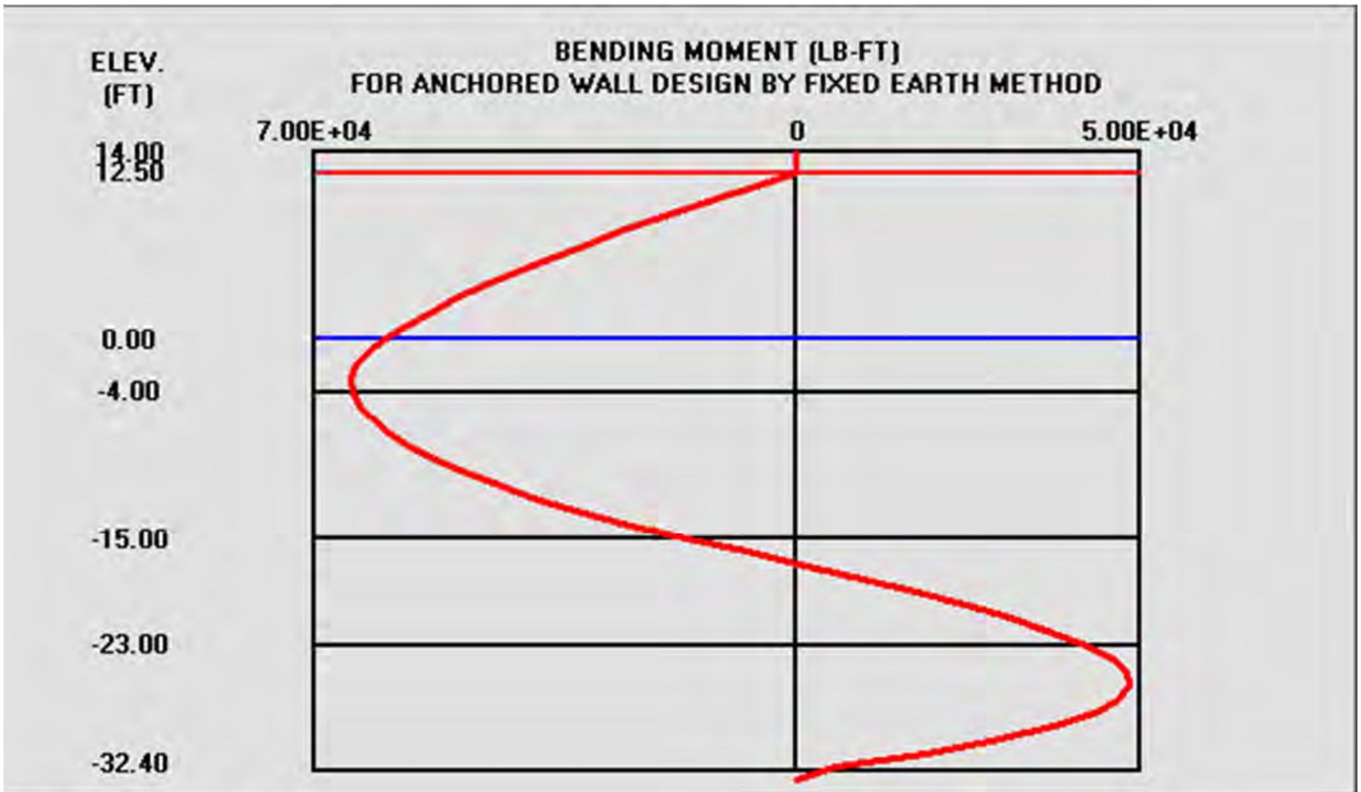


Figure C-7 – No Ground Improvement

Load Case 7 – Post-Seismic w/Liquefaction



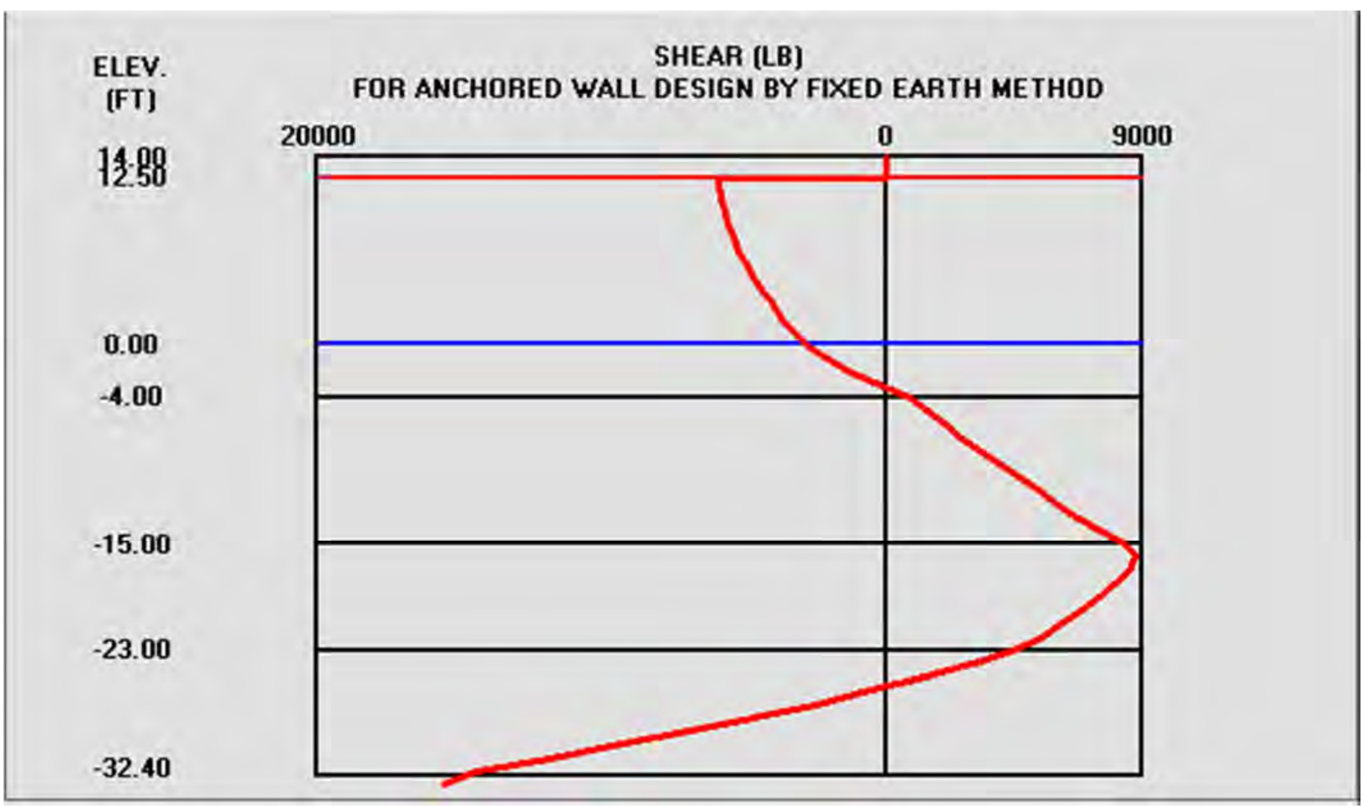
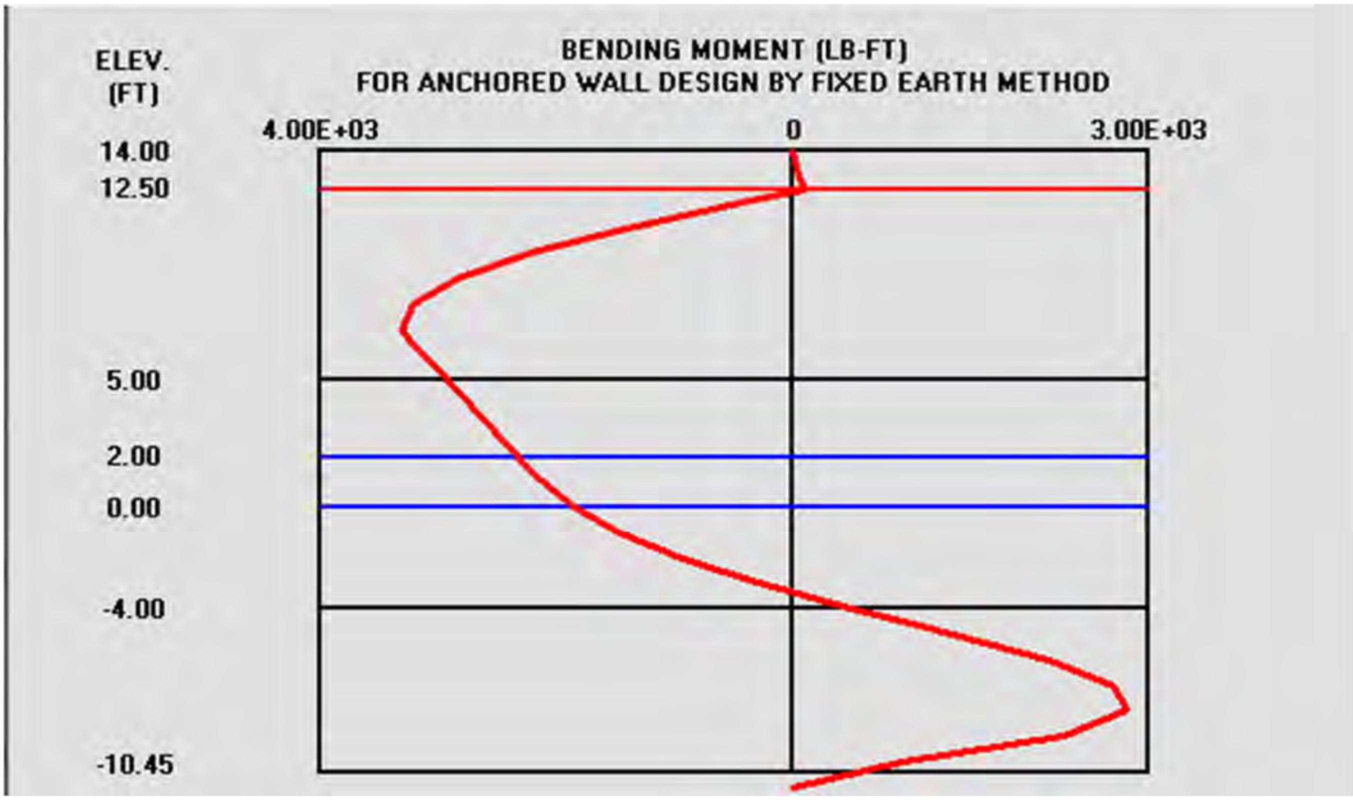


Figure C-8 – Ground Improvement

Load Case 1 – 250 psf Live Load

Water Elevations: Land Side = +2.0, Water Side = 0



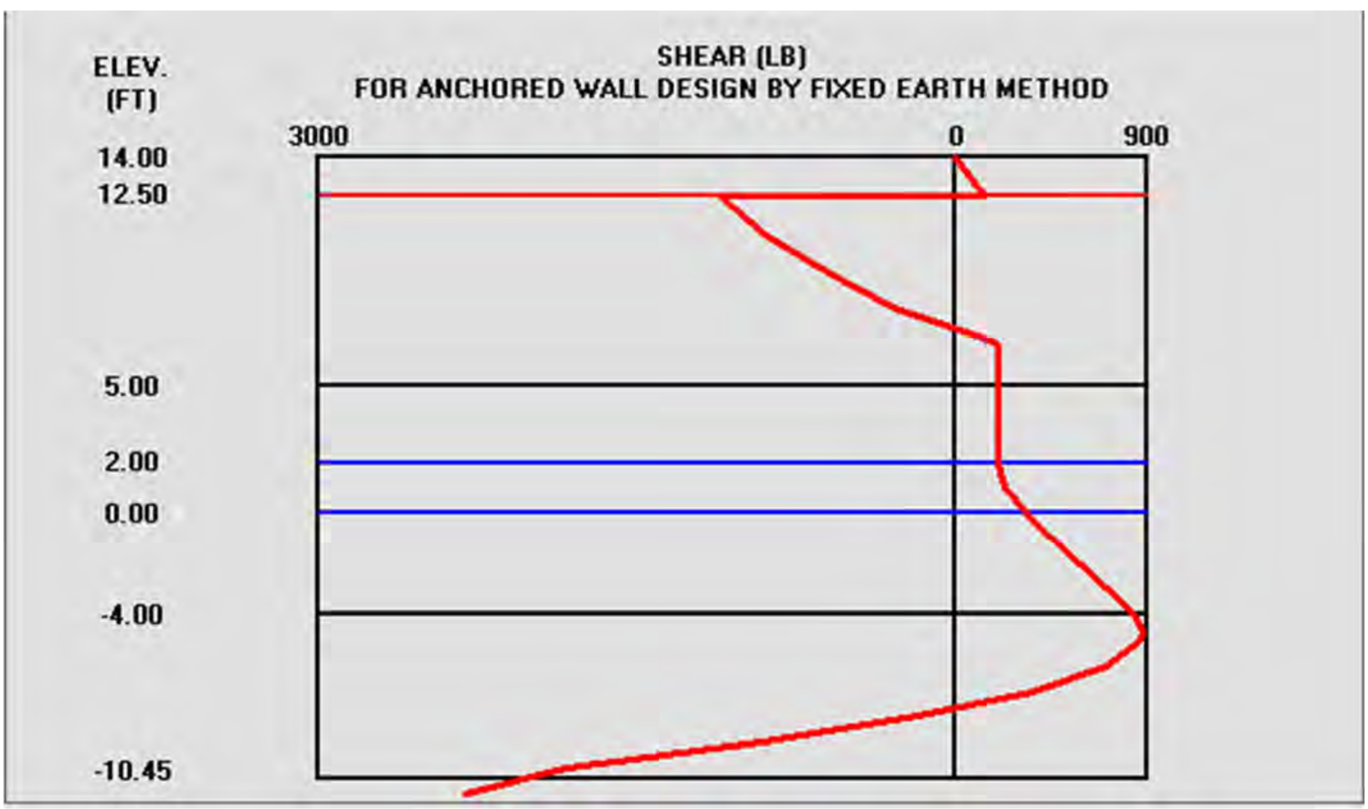
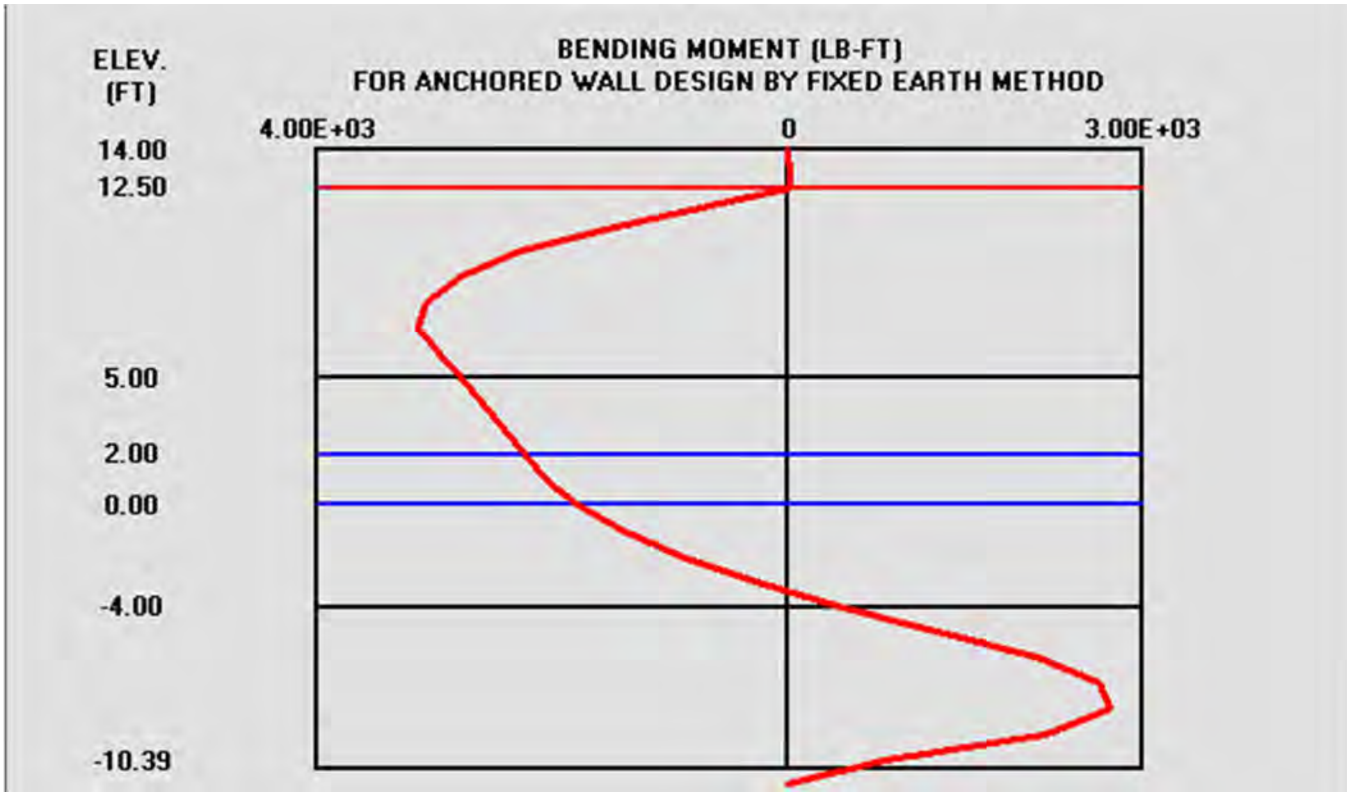


Figure C-9 – Ground Improvement

Load Case 2 – HS20-44 Live Load

Water Elevations: Land Side = +2.0, Water Side = 0



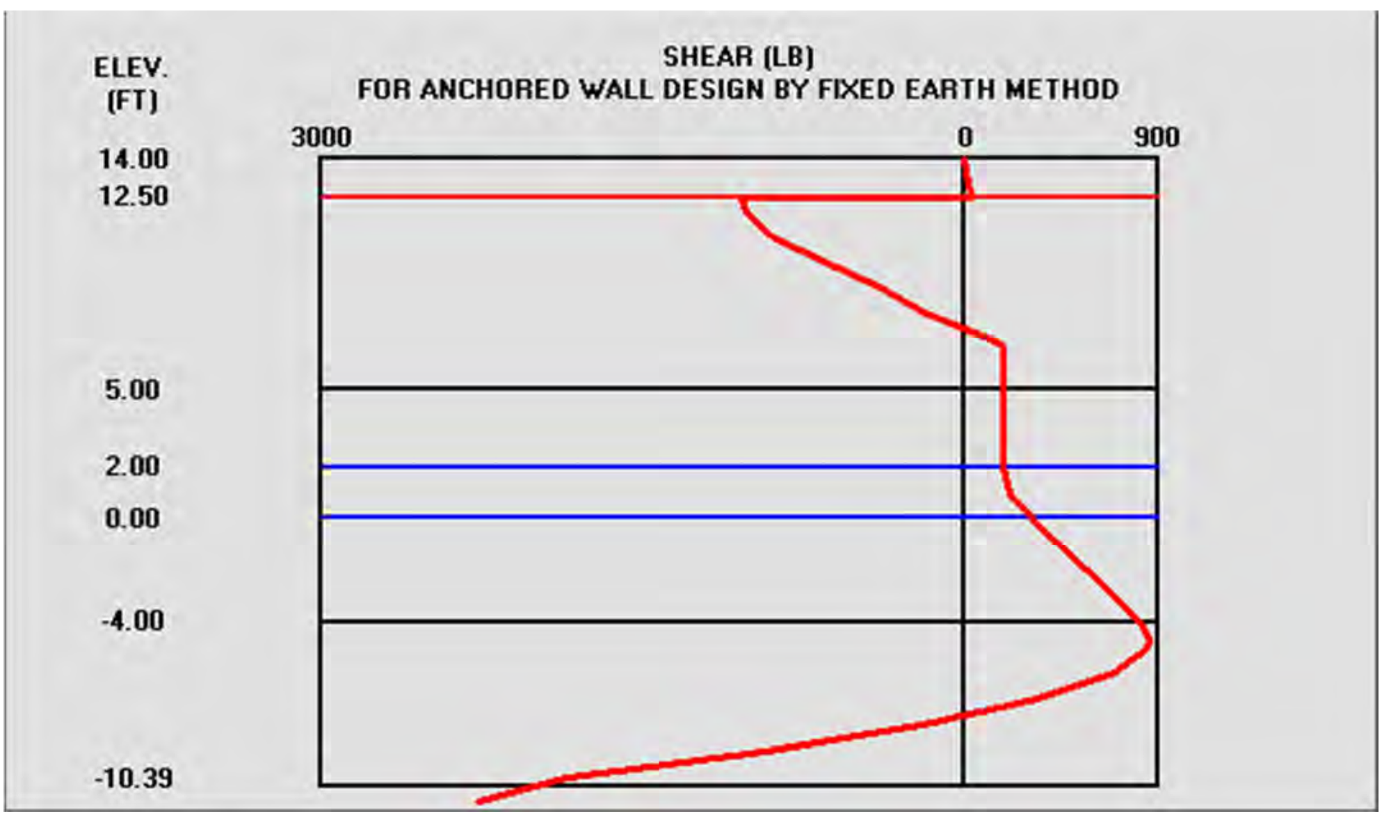
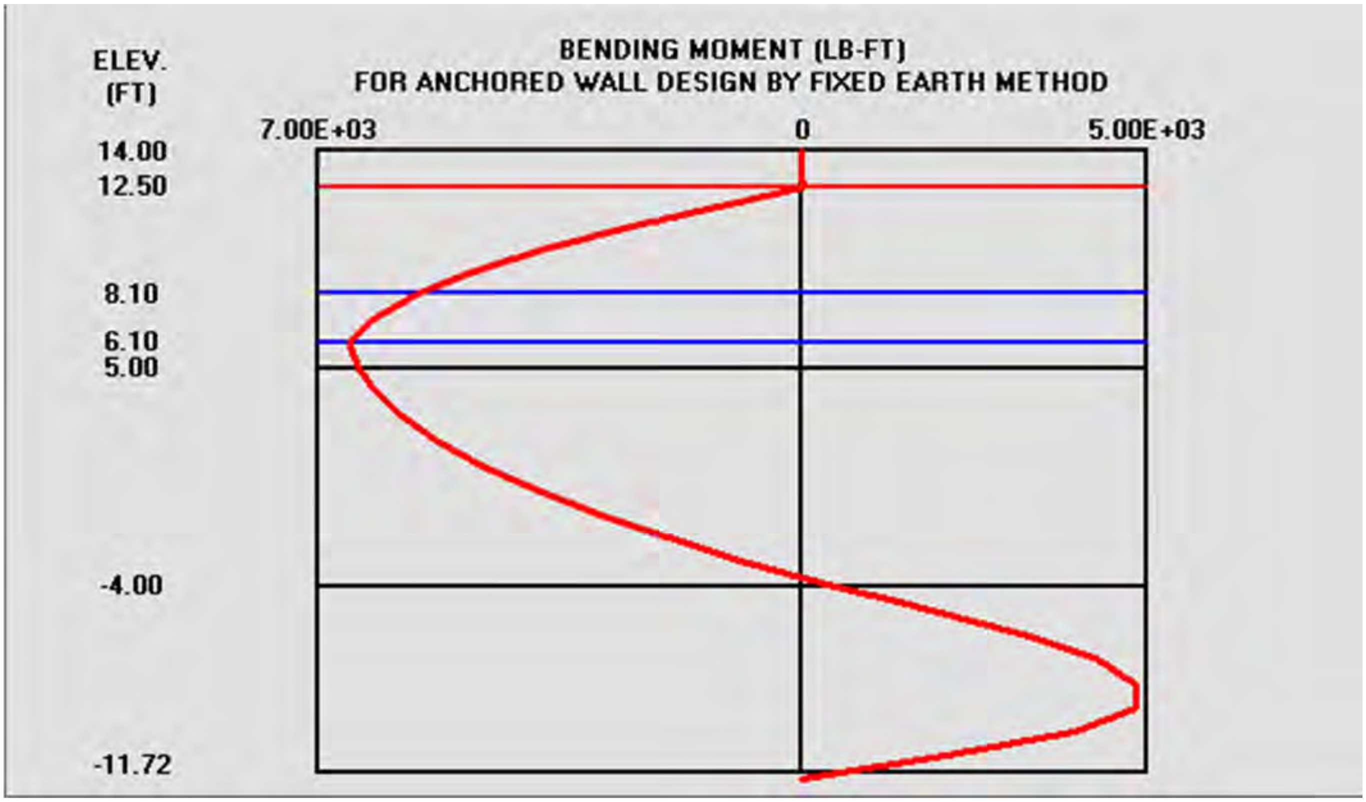


Figure C-10 – Ground Improvement

Load Case 3 – 250 psf Live Load

Water Elevations: Land Side = +8.1, Water Side = +6.1



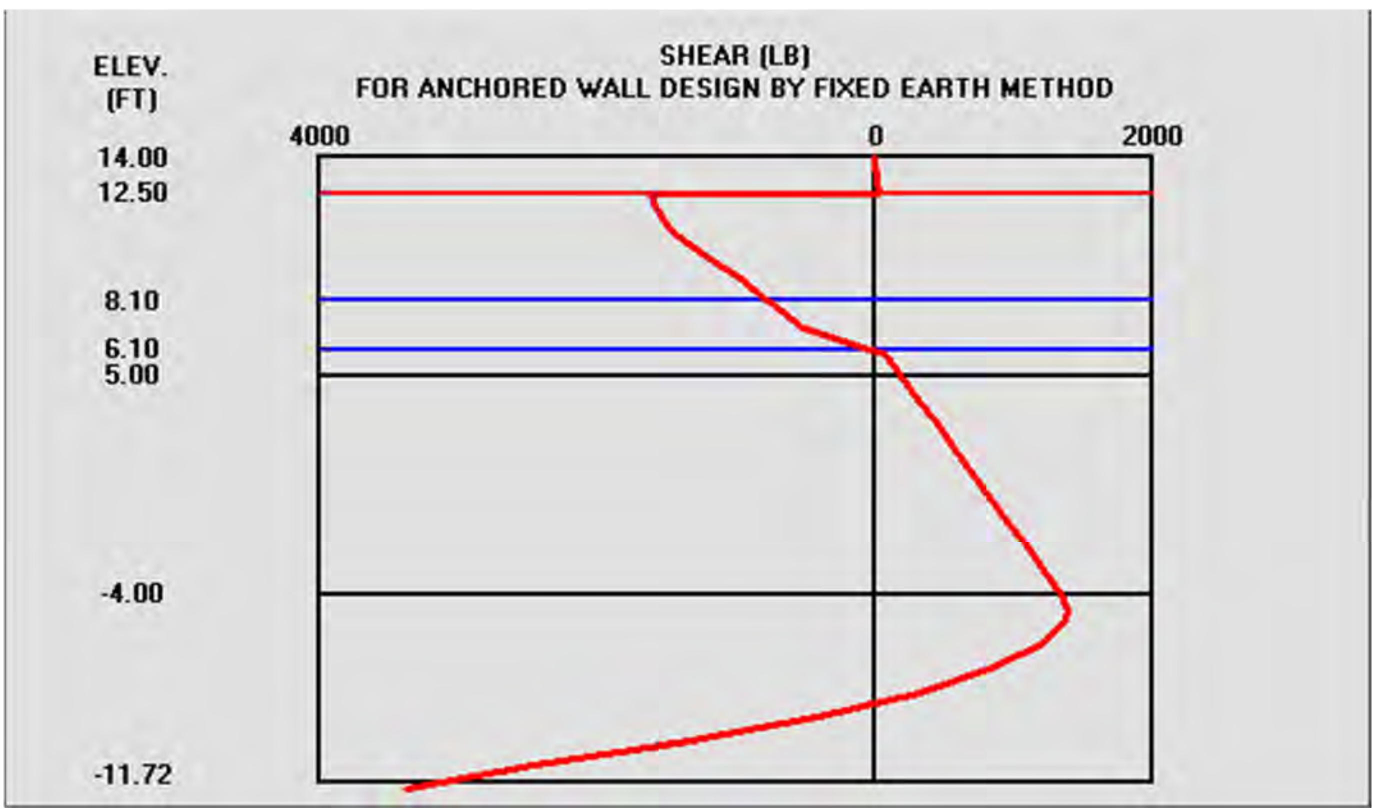
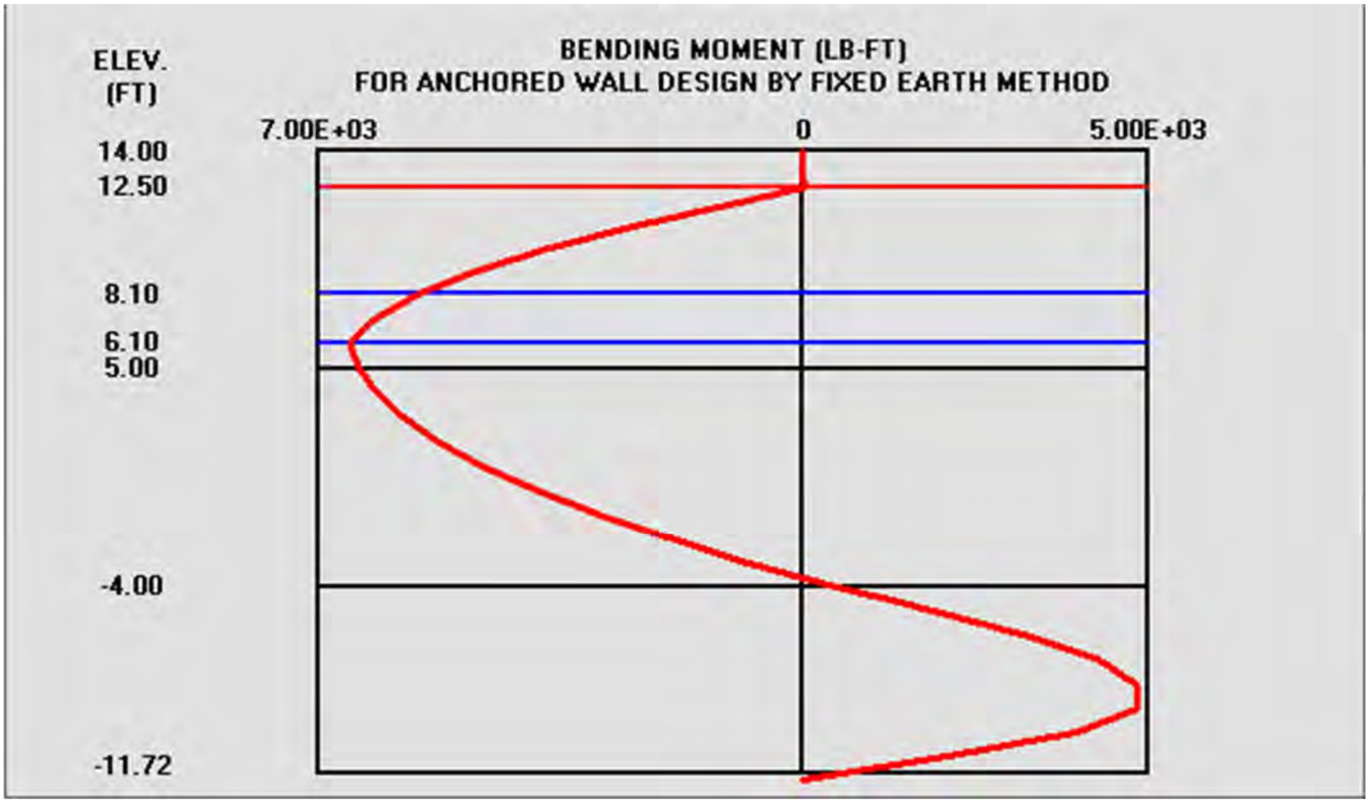


Figure C-11 – Ground Improvement

Load Case 4 – HS20 Live Load

Water Elevations: Land Side = +8.1, Water Side = +6.1



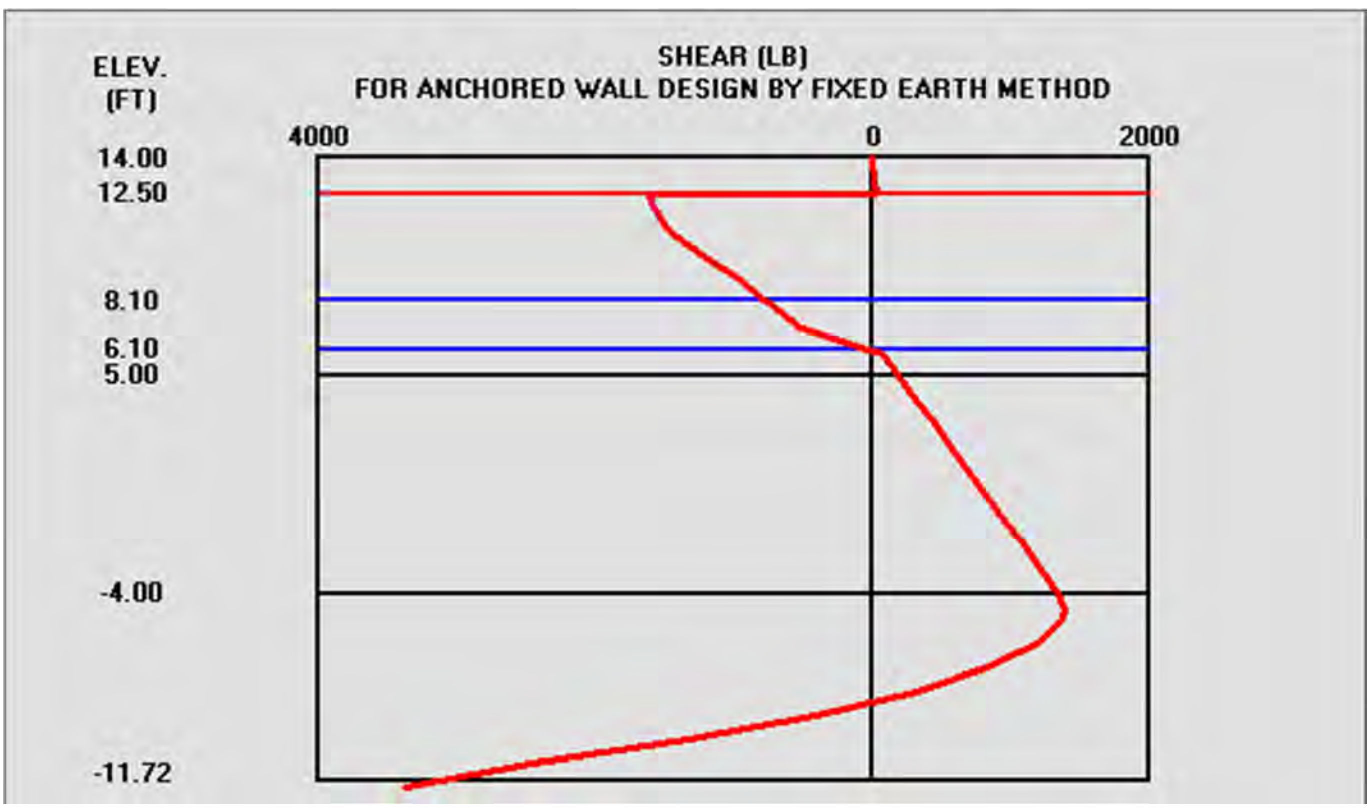
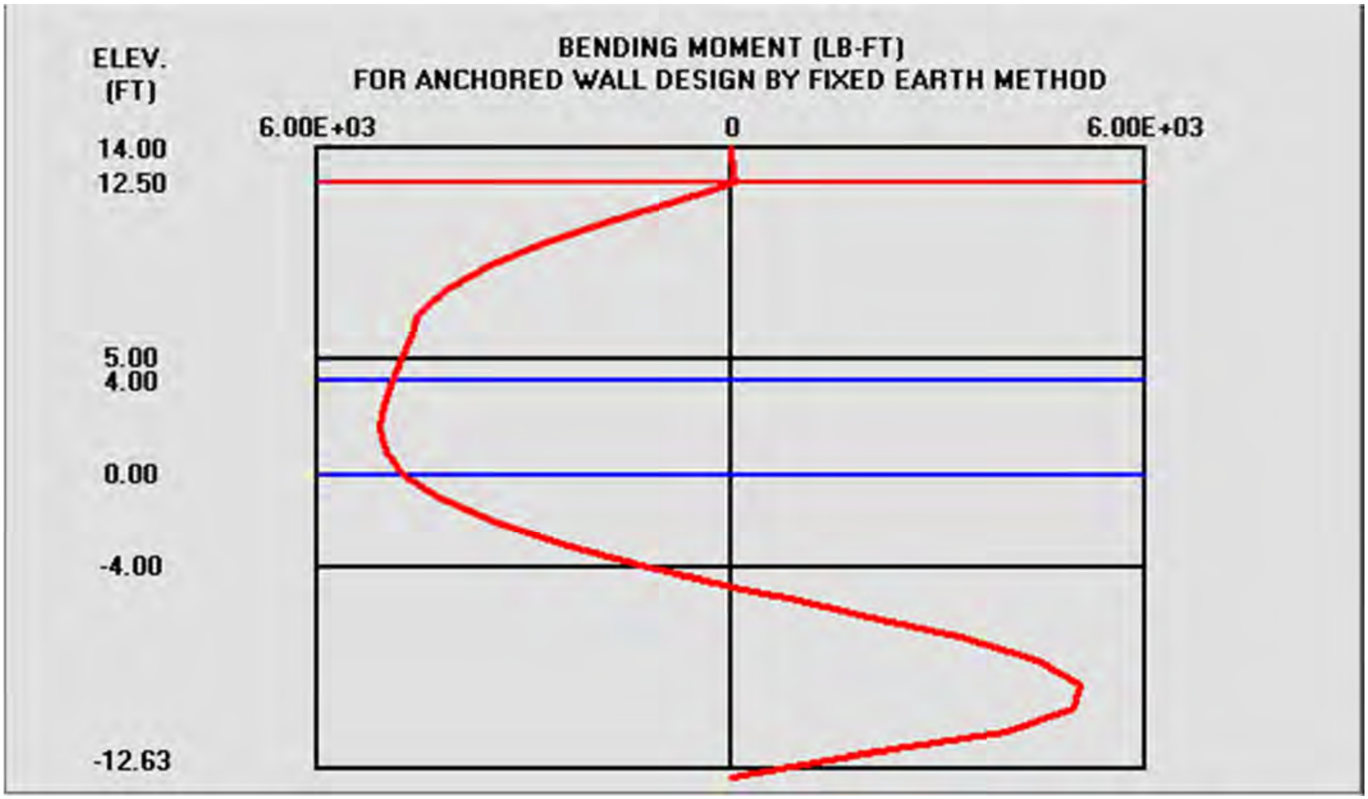


Figure C-12 – Ground Improvement

Load Case 5 – Drawdown



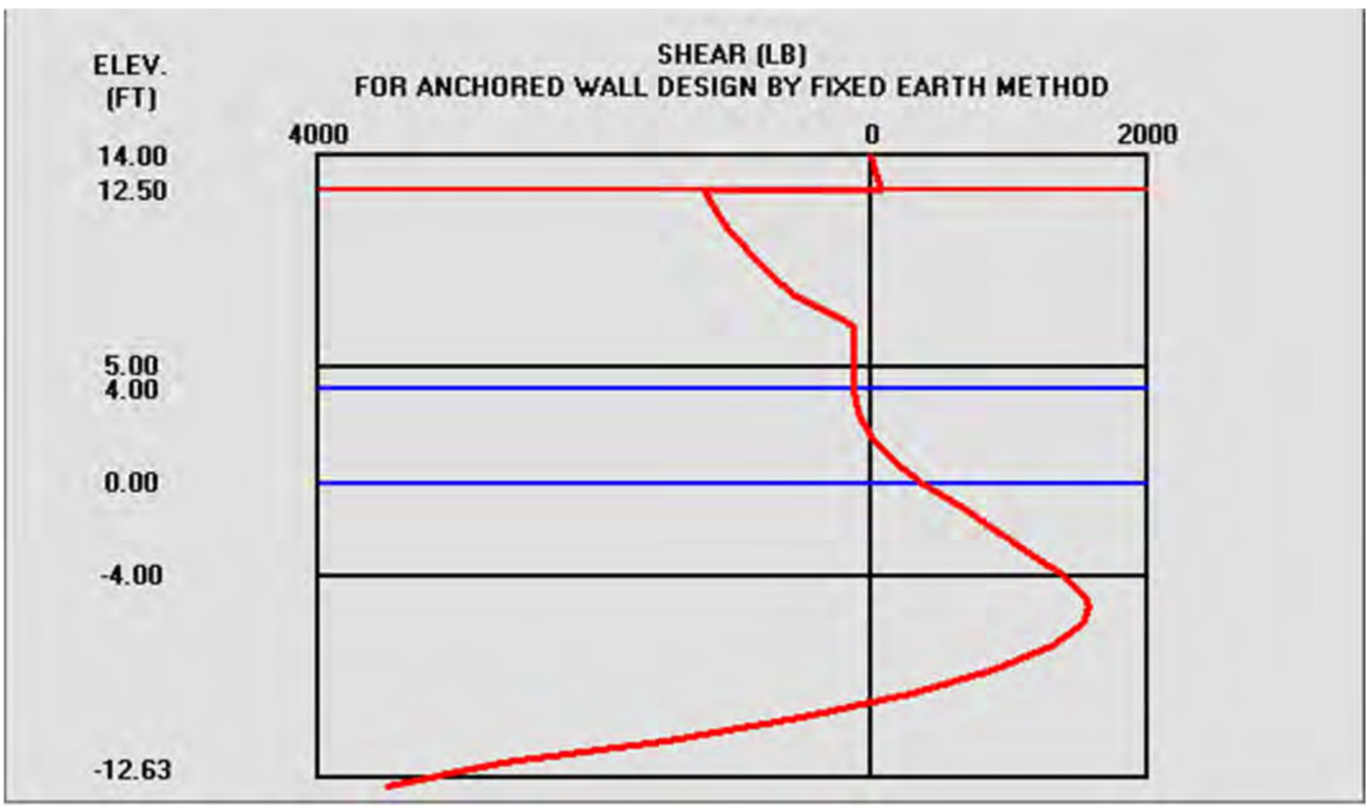
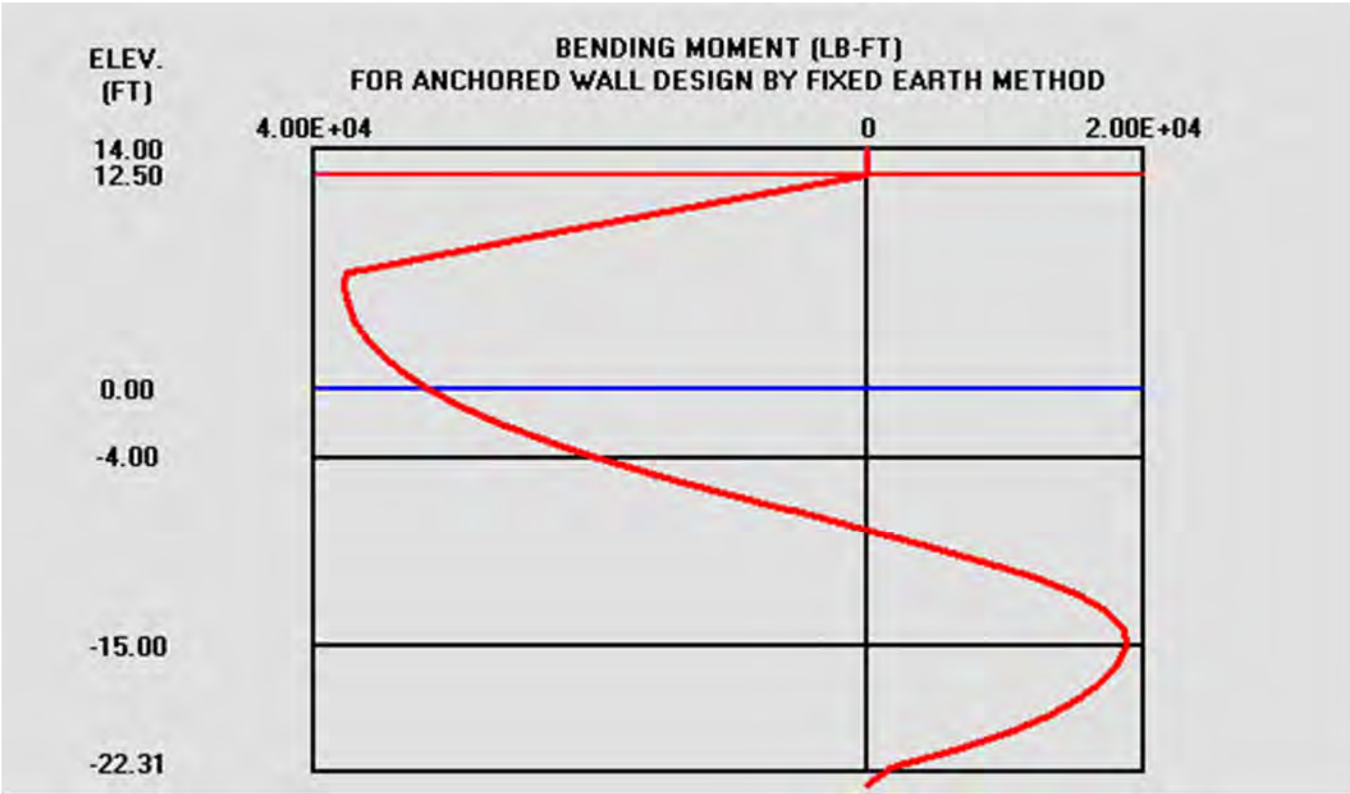
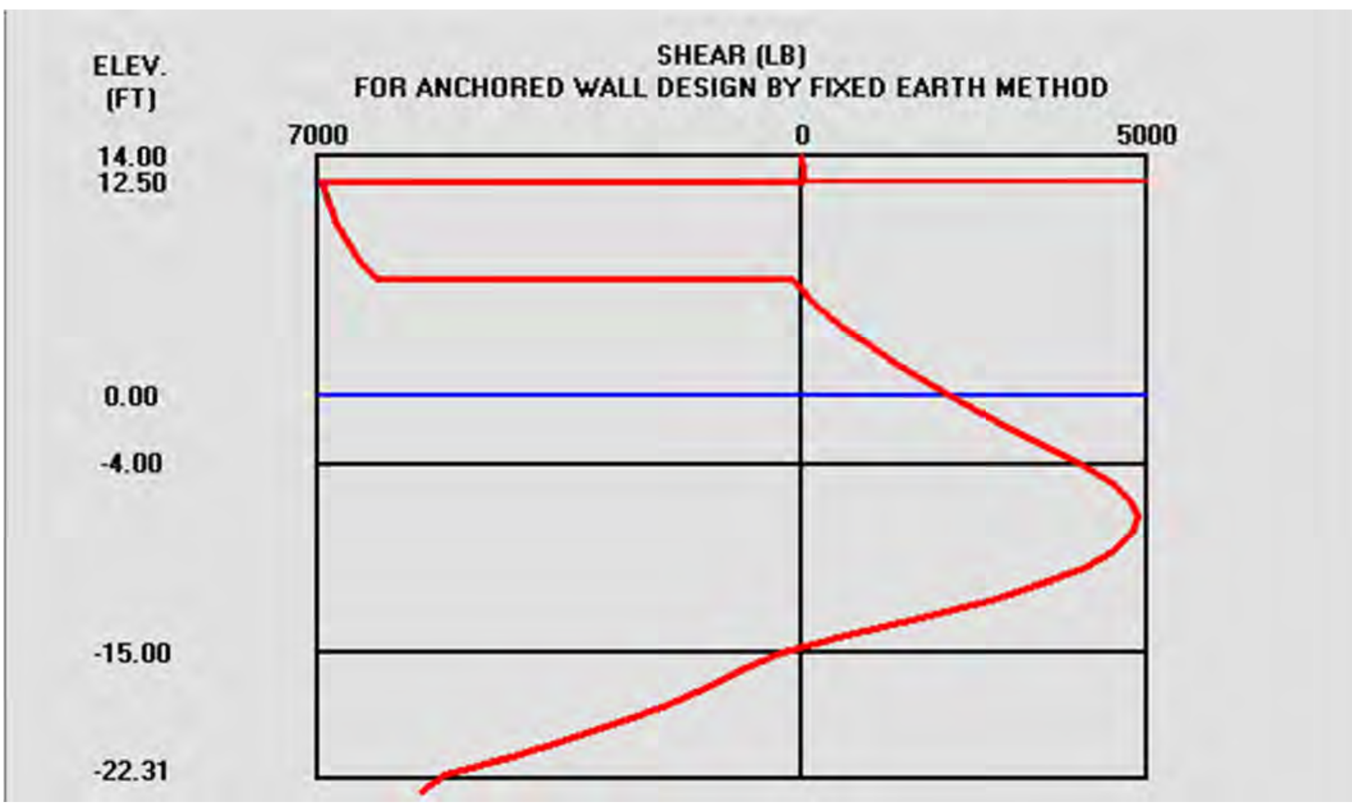


Figure C-13 – Ground Improvement

Load Case 6 – Seismic





APPENDIX D
Report Limitations and Guidelines for Use

APPENDIX D
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 Port of Ilwaco, Moffatt & Nichol, and their agents 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 Port of Ilwaco dated November 17, 2020, 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 proposed Port of Ilwaco Marina Repairs Conceptual Design Project 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.

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

- the function of the proposed structure;

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

- elevation, configuration, location, orientation or weight of the proposed structure;

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.

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 in the vicinity 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.

Appendix D – Ilwaco Marina Condition Assessment Report (Moffatt & Nichol)

moffatt & nichol

600 University St, Suite 610
Seattle, WA 98101

(206) 622-0222
www.moffattnichol.com

November 4, 2020

Port of Ilwaco
165 Howerton Avenue
Ilwaco, WA 98624

Attn: Mr. Guy Glenn, Port Manager

Subject: Port of Ilwaco Marina Repairs Condition Assessment Letter Report - DRAFT
M&N Project No. 11172

Mr. Glenn,

Moffatt & Nichol (M&N) conducted an above-water inspection and condition assessment of the Port of Ilwaco Marina on October 15, 2020. This letter report includes: a summary of M&N's scope of work, description of the facilities, inspection methodology, observed conditions, and recommendations.

1. Scope of Work

Moffatt & Nichol (M&N) was retained by the Port of Ilwaco (the Port) to provide a condition assessment of the TraveLift Pier, Jessie's Wharf East Bulkhead, and the West Access Pier at the Port of Ilwaco in Ilwaco, Washington. The scope of work included above-water inspection of facilities, perimeter mapping, and preliminary bathymetric measurements. The observations noted in the field were analysed to ascertain a condition assessment rating for the structures and determine repair or replacement recommendations.

2. Facility Description

The Port of Ilwaco is located in Ilwaco, WA on the north bank of Baker Bay, immediately east of the mouth of the Columbia River. The Port area generally consists of a marina used for year-round moorage of recreational and commercial fishing vessels, upland commercial buildings and a boatyard. The boatyard is located at the northwest end of the marina and includes the TraveLift Pier. The TraveLift Pier was constructed in the late 1970's and primarily consists of concrete structural components and is used to haul-out large vessels. The TraveLift equipment currently in use at the facility has a 50-ton capacity. Several commercial buildings, including the Jessie's Ilwaco Fish Co. building, are located along the length of the marina's northern shoreline. The buildings associated with Jessie's are located east of the boatyard and TraveLift Pier. The Jessie's Ilwaco Fish Co. buildings are located on an earth-filled and timber pile supported wharf structure and includes a timber bulkhead along the eastern limits of the wharf. The timber pile supported wharf structure along the west side of the Jessie property is not included as part of this condition assessment.

The marina is accessed from the shoreline by four timber access piers. The West Access Pier is one of the main entry points for the marina for both pedestrians and utilities. The general arrangement of the Port and the facilities inspected is shown in Figure 1.

Figure 1: Port of Ilwaco General Arrangement

3. Inspection Methodology

The above-water inspection methodology was based on the ASCE Manuals and Reports on Engineering Practice Number 130, "Waterfront Facilities Inspection and Assessment", 2015 Edition (ASCE 130). ASCE 130 describes the types of inspections and specific structure considerations depending on objectives, frequency of inspection and the level of damage.

Three basic levels of inspection are used for inspecting waterfront facilities. The type and extent of damage/deterioration that can be detected depends on the level of inspection performed. The following general descriptions for Levels I through III comply with ASCE 130. This inspection included a Level I inspection of timber piling. Level II inspections were not necessary as marine growth was not required to be removed for visual or tactile inspection. Level III inspections are typically not performed unless the findings of a Level I or Level II inspection indicate that the components being inspected may have additional damage or deterioration not readily quantifiable from a tactile inspection. A Level III inspection was performed above-water on select timber elements suspected of having internal decay and that were not readily identifiable as being in major or severe condition.

Level I - Visual and tactile inspection of components without the removal of marine growth. This level of inspection generally serves as a confirmation of as-built conditions and detects obvious damage or deterioration to the structure.

Level II - Partial marine growth removal of a statistically representative sample – for piling, this is typically 10 percent of the visually inspected piles, or roughly 1 in every 10 piles. The procedure requires that removal occur at three distinct bands for a distance of 1 foot at each band. The bands are located near the mudline, at mid-depth, and near the waterline. This level of inspection is intended to detect and identify damage and deterioration that may be hidden by surface biofouling.

Level III - Non-destructive testing (NDT) or partially destructive testing (PDT) of a statistically representative sample. These procedures are conducted to detect any hidden internal damage or deterioration. For the purpose of this inspection, suspect above water components were drilled (PDT) to determine the presence and extent of internal rot. The drilled hole was then filled with a treated dowel to prevent water and insect entry.

The field inspection consisted of observing the structural elements of the TraveLift pier and the east bulkhead of Jessie's wharf. The inspection also included observing the shoreline, slope protection, and gathering preliminary bathymetric and perimeter mapping data for use in conceptual design development. Photographs of typical components and conditions as well as deteriorated components and conditions were taken.

The above-water visual and tactile inspection of accessible above-water components was conducted on October 15, 2020. The above-water inspection included an inspection of the following:

TraveLift - The above-water, above-deck, and under-deck inspection of the TraveLift pier included inspection of the piles, pile caps, concrete deck, handrails, abutment, and the adjacent shoreline. Concrete surfaces of the TraveLift Pier were sounded with a hammer in areas of damage and suspected deterioration.

Jessie's Wharf East Bulkhead - The above-water and above-deck inspection of the east bulkhead included an inspection of the timber piles, timber lagging, timber walers, and accessible portions of the steel-cable tiebacks. A hammer was used to sound the timber components. Suspect components were drilled to determine the presence and extent of internal rot. The drilled hole was then filled with a treated dowel to prevent water and insect entry.

West Access Pier - The above-water, above-deck, and under-deck inspection of the access pier included the above water portions of the piles, pile caps, stringers, cross-bracing, decking, and handrails. A hammer was used to sound the timber components. Suspect components were drilled to determine the presence and extent of internal rot. The drilled hole was then filled with a treated dowel to prevent water and insect entry.

The water level at the time of the inspection, as observed at the Cape Disappointment, WA Tide Station (NOAA Station ID: 9440581) was approximately +3.0-feet in the morning, rising to +7.9-feet mid-day, and then dropping to+2.4-feet in the afternoon. The water levels noted are relative to the mean low lower water vertical datum.

4. Observations

4.1. TraveLift Pier

The TraveLift Pier consists of prestressed-concrete piles, a steel H-pile, reinforced concrete runway beams, a reinforced-concrete abutment, and steel handrails along the outer perimeter of the runway beams. A timber catwalk is located at approximately mid-length of the runway beams and consists of timber stringers, and decking. Timber-pile dolphins are located at the southern end of each runway. The adjacent shoreline consists of various sized rock armouring. The general arrangement of the TraveLift Pier is shown in Photograph 1.

Photograph 1: TraveLift Pier

4.1.1. Piles

The TraveLift Pier is supported by 27 prestressed-concrete piles and one steel H-pile. The concrete piles measure 12-inches square, and the H-pile is assumed to be a HP12.

No damage or deterioration is reported for 15 of the 27 concrete piles (56%). These piles visually appear sound with no cracking, delaminations, or spalling. Photograph 2 shows a typical concrete pile.

Minor to moderate damage, including cracks up to 1/16th inch wide, is present on 12 of the 27 concrete pile (44%). Photograph 3 shows a concrete pile with typical cracking approximately 10-inches below the runway beam soffit.

Four of the 27 concrete piles (15%) have corner spalls on the upper corners. The spalls have all been previously repaired and are believed to be from the original construction. Photograph 4 shows a typical observed corner spall with repair mortar.

The steel H-Pile exhibits moderate corrosion over the entire surface area above water. The corrosion does not appear to have resulted in measurable section loss. Photograph 6 shows the steel H-Pile.

Photograph 2: Typical Concrete Piles

Photograph 3: Moderate Cracking on Pile 2:A

Photograph 4: Corner Spall Repair on Pile 1:A

Photograph 5: Corner Spall Repair on Pile 2:A

Photograph 6: Steel H-Pile at Bent 5:A

4.1.2. Runway Beams

The runway beams span across the top of the piles functioning as a both a pile cap and a travel path for each side of the lift during operation. The beams measure 20-inches deep and 42-inches wide consisting of reinforced concrete. The upper inside corner of the beams has a steel angle iron wheel guide/kick plate. The beams also support steel handrails, a timber catwalk spanning between the beams and a timber walkway cantilevered off the outside of the beams near the southern ends. A topside view of the runway beams is shown in Photograph 7.

Photograph 7: Runway Beams from Atop Bent 1:A

Minor vertical hairline cracks measuring one-inch to nine-inches long are present along the full length of the beams. These hairline cracks are along the connection interface between the beam and the wheel guide/kick plate. Photograph 8 and Photograph 9 show typical examples of the hairline cracks.

One major crack is present on the southwest corner of the east runway beam. The crack is at a handrail post connection which appears to have been impacted during operation but does not affect the capacity of the structure.

Photograph 8: Typical Cracking Beneath Wheel Guide/Kick Plate

Photograph 9: Typical Cracking and Rust Staining

4.1.3. Abutment

A reinforced concrete abutment is located immediately north of the runway beams at the landside connection. A typical section of the abutment from the available as-built drawings is shown in Figure 2 below. In general, no damage or deterioration was observed on the concrete elements of the abutment, however, the concrete abutment shows signs of localized settlement. The top of the concrete abutment is approximately two-inches lower than the top of the adjacent runway beam surface. Wood wedges with a steel covering have been placed at the transition to compensate for the height difference.

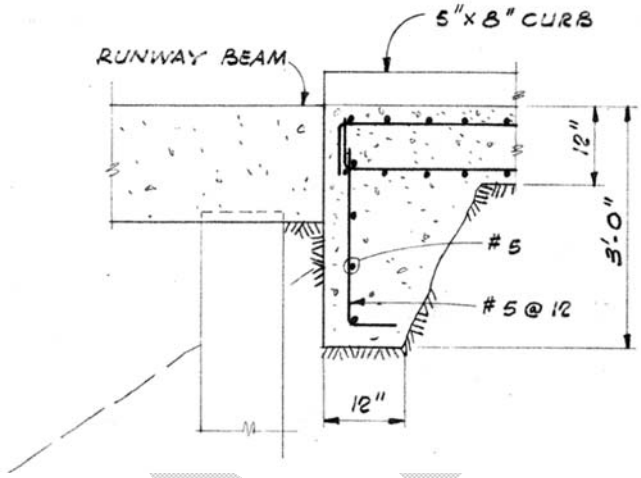
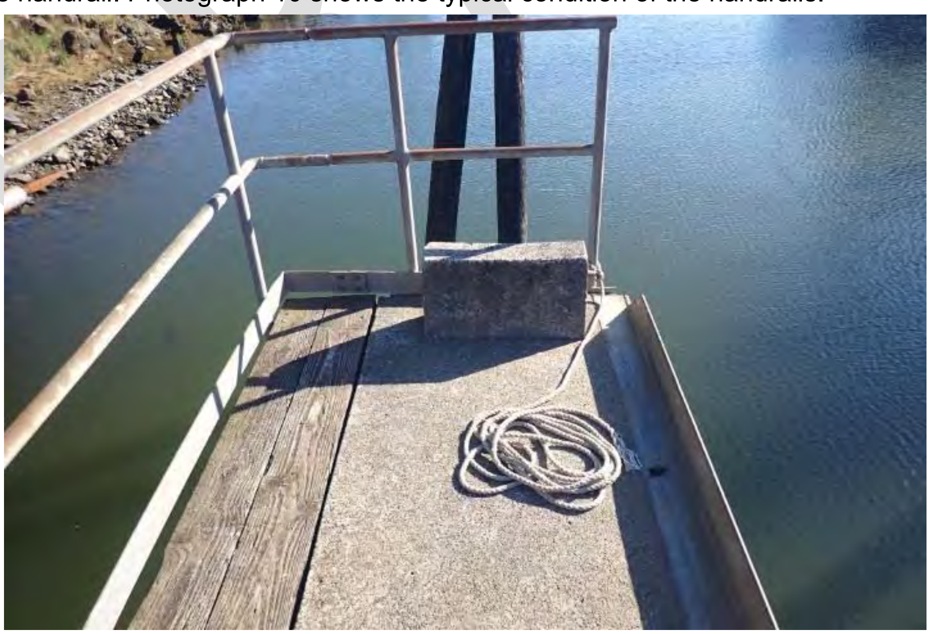


Figure 2: TraveLift Pier Abutment

4.1.4. Handrails

Galvanized steel handrails are located along the outer perimeter of the runway beams. The handrails consist of steel posts at six-foot spacing with top rails, mid rails, and toe plates. The handrail posts are attached to the runway beams with base plates and anchor bolts. Minor corrosion is present on isolated portions of the handrail. Photograph 10 shows the typical condition of the handrails.



Photograph 10: Typical Handrail with Minor Corrosion

4.1.5. Catwalk

A timber catwalk, oriented perpendicular to the runway beams, is located at approximately mid-length of the runways. The catwalk consists of pressure treated timber stringers and timber decking. The stringers are connected to the runway beams with steel brackets and anchor bolts. The minor checking and weathering of the timber elements is present. Photograph 11 shows the typical condition of the catwalk.

Photograph 11: Timber Catwalk

4.1.6. Dolphins

Two timber dolphins, each consisting of three creosote-treated batter piles, are located at the southern end of each runway. Minor damage, including checking and splitting is present. The east timber dolphin is shown Photograph 12.

Photograph 12: Typical Timber Pile Dolphin

4.1.7. Shoreline and Slope Protection

The shoreline immediately adjacent to the TraveLift Pier consists of mix-sized riprap and sparse vegetation. Evidence of erosion of the slope is present, most notable beneath the concrete vault located between the runway beam. Loss of material is estimated to be approximately one-foot thick, over an area of 1,000 square feet. The inconsistency of riprap size along the shoreline suggests that material has moved or been displaced since the original construction. This is further evident with the difference in elevation between the abutment and the runway beams, discussed previously. Photograph 13 and Photograph 14 show the typical condition of the shoreline. Photograph 15 shows the erosion below the concrete vault.

Photograph 13: Shoreline West of TraveLift Pier

Photograph 14: Shoreline East of TraveLift Pier

Photograph 15: Erosion Evident Below Concrete Vault

4.1.8. Preliminary Bathymetry

Water depth measurements were taken along the length of the two runway structures using a handheld digital depth sounder at each of the pile locations. Measurements were corrected for tides and summarized in Table 1.

Table 1: TraveLift Preliminary Bathymetry

West Runway		East Runway	
Location (Pile Bent:Pile Row)	**Mudline Elevation (ft, MLLW)**	**Location (Pile Bent:Pile Row)**	**Mudline Elevation (ft, MLLW)**
2:A	+4.0	2:G	+5.0
3:A	-0.6	3:G	-0.6
4:A	-3.3	4:G	-2.2
5:A	-3.8	5:G	-4.1
6:A	-5.3	6:G	-4.1
7:A (West Dolphin)	-5.1	7:G (East Dolphin)	-4.1

4.2. Jessie's Wharf East Bulkhead

The East 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 in the upland area. Three steel pipe piles are located along the face of the bulkhead and are assumed to be used for mooring of vessels. The adjacent northern shoreline consists of a vegetated slope and a timber pile bulkhead retaining structure. The shoreline south of the East Bulkhead consists of various sized rocks and concrete debris. Timber pile stubs are located south of the southern shoreline. The general arrangement of the East Bulkhead is shown in Photograph 16.

Photograph 16: Jessie's Wharf East Bulkhead

4.2.1. Piles

Minor damage, including checking and splitting less than ½-inch wide is present on all of the East Bulkhead timber piling. Moderate to major rot is present in the upper 12-inches of 26 of the 118 timber piling (22-percent). Severe damage and deterioration, including section loss greater than 50-percent and missing or broken piling is present on 18 of the 118 timber piling (15-percent). Photograph 17 shows several of the piles with severe damage.

Photograph 17: East Bulkhead Piles

Minor damage, including checking and splitting less than ½-inch wide is present on all of the North Bulkhead timber piling that were visible at the time of inspection. The northern bulkhead is shown in Photograph 18.

Photograph 18: North Bulkhead Piles (Water Level Approx. +6.3-feet MLLW)

4.2.2. Lagging, Walers, and Tiebacks

4.2.2.1. East Bulkhead

Minor damage, including checking and splitting is present throughout the lagging and waler members. Additionally, moderate to severe deterioration of the lagging, including section loss, and breakage is present. The deterioration has led to gaps between adjacent lagging members, measuring up to several inches wide and allows for loss of backfill material. This loss of material has led to subsidence of the upland area. The East Bulkhead lagging, walers, and tieback strands are shown in Photograph 19 through Photograph 21.

Photograph 19: East Bulkhead Lagging

Photograph 20: East Bulkhead Waler

Photograph 21: East Bulkhead Tieback Strands

4.2.2.2. North Bulkhead

The North Bulkhead lagging consists of timber piles laid horizontally behind the vertical bulkhead piles. The lagging measures approximately 14-inches in diameter. Minor damage, including checking and splitting is present. The north bulkhead lagging is shown in Photograph 22.

Photograph 22: North Bulkhead Lagging

4.2.3. Shoreline and Slope Protection

The shoreline immediately south of the East Bulkhead consists of mix-sized riprap, broken concrete debris and sparse vegetation. No evidence of advanced erosion of sloughing is present, however, the inconsistency of riprap size suggests that some material may have moved or been displaced since the original construction of the shoreline. Photograph 23 shows the typical condition of the south shoreline.

Photograph 23: East Bulkhead South Shoreline

The shoreline northeast of the East Bulkhead and adjacent to the North Bulkhead consists of a vegetated slope and timber debris. No evidence of advanced erosion of sloughing is present. Photograph 24 show the typical condition of the north shoreline.

Photograph 24: North Bulkhead Shoreline, Looking Northwest

4.2.4. Preliminary Bathymetry

Water depth measurements were taken along the length of the structure using a handheld digital depth sounder at approximate 10-foot intervals along three transects, with Station 0+00 being the northern corner of the bulkhead. Transects were located along the face of the bulkhead, 15-feet offset, and 30-feet offset from the bulkhead face. The measurements were corrected for tides and mudline elevations along the transects summarized in Table 2.

Table 2: East Bulkhead Preliminary Bathymetry

Station	Mudline Elevation (ft, MLLW)		
	Transect 1 (Face of Bulkhead)	Transect 2 (15ft Offset)	Transect 3 (30ft Offset)
0+00	7.0	4.5	2.0
0+10	4.1	2.6	0.8
0+20	1.9	0.3	-0.2
0+30	0.4	-1.7	-3.4
0+40	-0.7	-4.2	-5.2
0+50	-1.0	-4.9	-7.2
0+60	-9.2	-6.1	-8.3
0+70	-1.4	-5.5	-8.7
0+80	-1.9	-7.5	-8.7
0+90	-2.8	-7.5	-8.6
1+00	-2.9	-9.1	-9.0
1+10	-4.0	-9.0	-10.2
1+20	-3.6	-9.6	-10.2
1+30	-6.4	-9.4	-9.7
1+40	-3.8	-9.6	-9.8
1+50	-3.7	-9.0	-10.4
1+60	-2.8	-8.8	-10.4
1+70 (End of Timber Wall)	0.9	-4.9	-10.3

4.2.5. Upland Area

The upland area of the East Bulkhead consists of a paved driving surface used for accessing portions of the Jessie's Fish Co. building. The pavement is generally free of damage or significant deterioration, however, the pavement appears to have localized areas of subsidence adjacent to the bulkhead. Photograph 25 shows the typical condition of the upland area.

Photograph 25: East Bulkhead Upland

4.3. West Access Pier

The West Access Pier consists of treated timber components including piles, pile caps, stringers, cross bracing, and decking. A concrete abutment connects the pier to the upland area. Water and electrical utilities which service the marina are supported along the pier and transition onto an aluminium gangway. The adjacent shoreline consists of a vegetated slope. Photograph 26 shows an elevation view of the West Access Pier.

Photograph 26: West Access Pier, Looking Northeast (Water Level Approx. +6.3-feet MLLW)

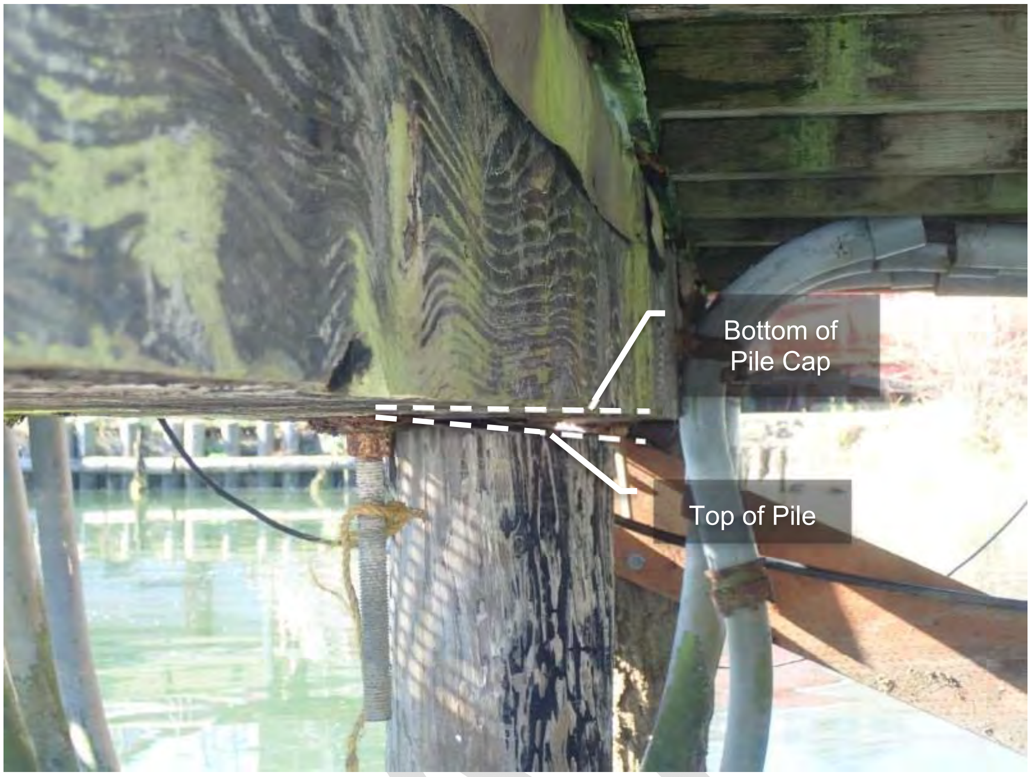
4.3.1. Piles

The West Access Pier is supported by nine creosote treated timber piles. The timber pile diameter is nominally 14-inches.

Major damage including diameter loss greater than 15-percent as a result of delamination of the outer shell and piles partially supporting the pile cap was observed on eight of the nine timber piles (89-percent). Photograph 27 shows diameter loss of Pile 3:A. Photograph 28 shows Pile 4:B partially supporting the pile cap.

Severe damage was observed on one of the nine timber piles (11-percent). Photograph 29 shows Pile 4:C with severe cross-section area loss.

Photograph 27: West Access Pier Typical Timber Pile with Shell Delamination



Photograph 28: West Access Pier Timber Pile 4:B, Partially Non-Bearing

Photograph 29: West Access Pier Timber Pile 4:C with Severe Section Loss

4.3.2. Pile Caps

The West Access Pier pile caps are creosote treated 12x12 timber members. Typically, there is no damage or deterioration of the pile caps. Photograph 30 shows a typical pile cap.

Photograph 30: West Access Pier Timber Pile Cap

4.3.3. Abutment

A reinforced concrete abutment provides a transition to the landside area of the West Access Pier. In general, no damage or deterioration was observed on the concrete elements of the abutment, and no signs of localized settlement or erosion are present. Photograph 31 shows the typical condition of the abutment.

Photograph 31: West Access Pier Abutment

4.3.4. Stringers

The stringers are pressure treated 4x12 timber members at 16-inch spacing. The stringers are double span, and lapped at the pile caps with toenail connections to the pile cap. The pile caps exhibit no damage or deterioration. Photograph 32 shows the typical condition of the stringers at Bent 3.

Photograph 32: West Access Pier Timber Stringers

4.3.5. Cross Bracing

The timber cross bracing for the West Access Pier is pressure treated 2x12 members positioned longitudinally along Row A and Row C and transversely along Bent 3 and Bent 4. The bracing exhibits no damage or deterioration. Photograph 33 shows the typical condition of the cross bracing.

Photograph 33: West Access Pier Timber Cross Bracing

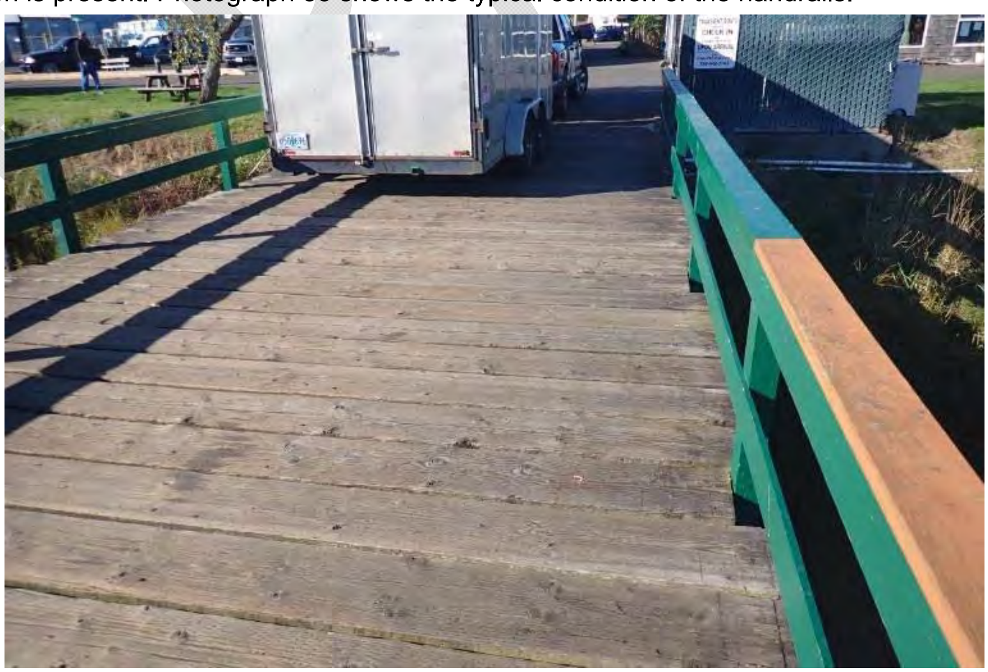
4.3.6. Decking

The timber decking for the West Access Pier is comprised of 3x12 timbers oriented 90-degrees to the stringers. The decking typically has no damage or deterioration. Photograph 34 shows the typical condition of the decking.

Photograph 34: West Access Pier Timber Decking

4.3.7. Handrail

Timber handrails are located along the outer perimeter of the West Access Pier and consist of timber posts, top rails, and mid rails. The handrail posts are attached to the stringers with lag bolts. No damage or deterioration is present. Photograph 35 shows the typical condition of the handrails.

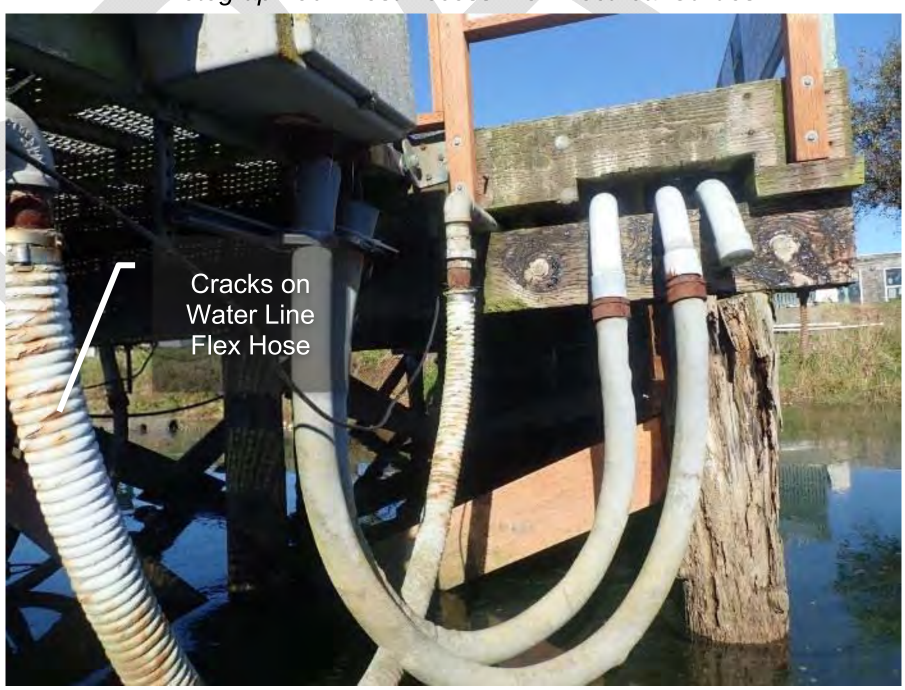


Photograph 35: Typical Handrail

4.3.8. Utilities

Utility conduit and piping for electrical and water systems are hung from the West Access Pier and transition onto an aluminium gangway. The electrical flex connection on the east side of the gangway has separated from the junction box and the water line flex hose is cracked near the southeast corner of the pier. Photograph 36 and Photograph 37 show the typical condition of the utilities.

Photograph 36: West Access Pier Electrical Utilities



Photograph 37: West Access Pier Water Flex Hose

4.3.9. Shoreline and Slope Protection

The shoreline east and west of the West Access Pier consists of vegetated slopes and small timber debris. No evidence of advanced erosion or sloughing is present. Photograph 38 and Photograph 39 show the typical condition of the shoreline.

Photograph 38: West Access Pier East Shoreline, Looking East

Photograph 39: West Access Pier West Shoreline, Looking East

4.3.10. Preliminary Bathymetry

Water depth measurements were taken along the length of the structure using a handheld digital depth sounder at each of the pile locations. Measurements were corrected for tides and summarized in Table 3.

Table 3: West Access Pier Preliminary Bathymetry

West		East	
Location (Pile Bent:Pile Row)	**Mudline Elevation (ft, MLLW)**	**Location (Pile Bent:Pile Row)**	**Mudline Elevation (ft, MLLW)**
2:A	5.3	2:C	5.9
3:A	3.3	3:C	3.3
4:A	1.2	4:C	-1.1

5. Recommendations & Closing

An overall Condition Assessment Rating (CAR) is assigned to the three facilities. The CARs are based on the findings of the visual observations. The condition assessment scale includes the following six categories: Good, Satisfactory, Fair, Poor, Serious, and Critical. The six CARs and descriptions are provided as an attachment to this report.

5.1. TraveLift Pier

The TraveLift Pier is rated as "*Fair*". All primary structural elements are sound; but minor to moderate defects and deterioration are observed. Localized areas of moderate deterioration are present but do not significantly reduce the structural capacity. The localized deterioration is affecting the concrete apron structure and does not affect the structural capacity of the TraveLift itself. The following repairs are recommended:

- Install shoreline protection along the shoreline
- Restore fill supporting the concrete vault

5.2. Jessie's Wharf East Bulkhead

The East Bulkhead is rated as "Serious". Advanced deterioration and breakage have affected the load-bearing capacity of the bulkhead. Additionally, the bulkhead experiences overtopping during extreme storm and tidal events. Due to the extent and nature of the deterioration, as well as overtopping of the bulkhead, it is assumed that repairing the structure in-kind is cost prohibitive, therefore, alternatives for replacement of the bulkhead should be evaluated.

5.3. West Access Pier

The West Access Pier is rated as "*Serious*". Advanced deterioration of the timber piling has affected the load bearing capacity of the pier. The following repairs are recommended:

- Install structural pile jackets on all piles (nine total)
- Restore Piles 4:B and Pile 3:C to full bearing, and install steel straps to secure to pile cap
- Repair broken electrical fittings
- Replace cracked water flex hose

Thank you for the opportunity to work with you on this project. If you have any questions or need any further information, please do not hesitate to contact me.

Sincerely,

moffatt & nichol

Aaron Patterson, PE
Inspection Team Leader

Attachments:

- Attachment A - Component Rating System
- Attachment B - Condition Assessment Ratings

Attachment A - Component Damage Rating System

Individual components were categorized into six damage ratings based on the observations and the component damage rating descriptions per ASCE 130[1]. Each component rating is defined in Table A-1.

Table A-1: Component condition rating descriptions

DAMAGE RATING	DESCRIPTION
Not Inspected (NI)	Component was inaccessible or not included in the scope.
No Damage (ND)	Component had a sound material surface.
Minor (MN)	*Timber:* Checks, splits, and gouges less than 0.5 inches wide. *Steel:* Less than 50% of perimeter or circumference affected by corrosion at any elevation or cross-section; loss of thickness up to 15% of nominal thickness at any location. *Reinforced Concrete:* Mechanical abrasion or impact dents; general cracks up to 1/16-inch wide; occasional corrosion stain or small pop-out corrosion spall. *Prestressed Concrete:* Minor mechanical or impact spalls up to 1/2-inch deep.
Moderate (MD)	*Timber:* Checks and splits greater than 0.5 inches wide; diameter loss up to 15%; cross-section area loss up to 25%; corroded hardware; marine borer infestation. *Steel:* Greater than 50% of surface at any elevation/cross-section affected by corrosion; 15% to 30% loss of nominal thickness at any location. *Reinforced Concrete:* Structural cracks up to 1/16-inch wide; corrosion cracks up to ¼-inch wide; chemical deterioration; random cracks up to 1/16-inch wide; soft concrete and rounding corners up to 1-inch deep; frequent corrosion stain or medium pop-out corrosion spall. *Prestressed Concrete:* Structural cracks up to 1/32-inch in width; Chemical deterioration: random cracks up to 1/32-inch in width.
Major (MJ)	*Timber:* Checks and splits through full depth of cross-section; diameter loss 15% to 30%; cross-section loss 25% to 50%; heavily corroded hardware; displacement, misalignments at connections. *Steel:* Partial loss of flange edges or visible reduction of wall thickness; 30% to 50% loss of nominal thickness, any location. *Reinforced Concrete:* Structural cracks 1/16-inch to ¼-inch wide; partial breakage (spalls); corrosion cracks greater than ¼-inch wide; multiple cracking and disintegration of surface due to chemical deterioration. *Prestressed Concrete:* Structural cracks 1/32-inch to 1/8-inch in width; Any corrosion cracks generated by strands or cables; Chemical deterioration: cracks wider than 1/8-inch; “Softening” of concrete up to 1-inch deep.
Severe (SV)	*Timber:* Diameter loss greater than 30%; cross-section area loss greater than 50%; loss of connections and/or fully non-bearing; partial or complete breakage.

[1] Damage Rating Descriptions from ASCE Manuals and Reports on Engineering Practice Number 130, "Waterfront Facilities Inspection and Assessment" (ASCE 130), Table 2-4, Table 2-5, Table 2-6, and Table 2-7.

DAMAGE RATING	DESCRIPTION
	Steel: Structural bends or buckling, breakage and displacement at supports, loose or lost connections; greater than 50% loss of nominal thickness, any location. *Reinforced Concrete:* Structural cracks greater than ¼-inch wide; breakage; loss of bearing and displacement at connections; reinforcing steel w/cover loss and greater than 30% diameter loss for any main bar; exposed steel due to chemical deterioration; cross section loss greater than 30% of any component for any reason. *Prestressed Concrete:* Structural cracks wider than 1/8-inch and at least partial breakage or loss of bearing; Corrosion spalls over any prestressing steel; Partial spalling and loss of cross section due to chemical deterioration.

Attachment B – Condition Assessment Ratings

Overall Condition Assessment Ratings (CAR), as defined by ASCE 130[2], are assigned to each structure and primary component. The CARs are based on the findings of the visual and tactile observations. The condition assessment scale includes the six categories described in Table B-1.

Table B-1: Condition Assessment Rating Descriptions

CAR	DESCRIPTION
"Good"	No visible damage or only minor damage noted. Structural elements may show very minor deterioration, but no overstressing observed. *No repairs are required.*
"Satisfactory"	Limited minor to moderate defects or deterioration observed but no overstressing observed. *No repairs are required.*
"Fair"	All primary structural elements are sound but minor to moderate defects or deterioration observed. Localized areas of moderate to advanced deterioration may be present but do not significantly reduce the loadbearing capacity of the structure. *Repairs are recommended, but the priority of the recommended repairs is low.*
"Poor"	Advanced deterioration or overstressing observed on widespread portions of the structure but does not significantly reduce the load-bearing capacity of the structure. *Repairs may need to be carried out with moderate urgency.*
"Serious"	Advanced deterioration, overstressing, or breakage may have significantly affected the load-bearing capacity of primary structural components. Local failures are possible and loading restrictions may be necessary. *Repairs may need to be carried out on a high-priority basis with urgency.*
"Critical"	Very advanced deterioration, overstressing, or breakage has resulted in localized failure(s) of primary structural components. More widespread failures are possible or likely to occur, and load restrictions should be implemented as necessary. *Repairs may need to be carried out on a very high-priority basis with strong urgency.*

[2] CAR Descriptions from ASCE Manuals and Reports on Engineering Practice Number 130, "Waterfront Facilities Inspection and Assessment" (ASCE 130), Table 2-14