Seawall Phase II Investigation

Rogue Brewery Seawall
Newport, Oregon

Prepared for:

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PBS Project 74183.000

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1 EXECUTIVE SUMMARY

This report contains the results of the Phase II Investigation of the Rogue Brewery Seawall. This report builds on the Structural Evaluation Report, prepared by Berger/ABAM, Inc., dated December 2018. That document is attached as Appendix A. Refer to the Structural Evaluation Report for information related to the project background, seawall configuration, structural materials specifications (per design), and other information not repeated in this report.

We reviewed soil testing results provided to us by the Port of Newport (the Port). These tests were summarized in a report by Stantec, dated April 2, 2020. The tests showed that the sulfate and chloride levels of the soil were significantly below the threshold for a corrosive site, and that the pH level of the soil was significantly higher than the threshold. We therefore conclude that the soil conditions do not present a corrosive environment.

This report covers the measurements taken of the steel piles of the seawall, the concrete core samples taken from the seawall and the tie-back anchor investigation.

Based on the measurements taken of the steel piles, the worst-case piles have approximately 50% of the original moment capacity remaining.

The concrete core samples were taken from a concrete lagging panel above the splash zone. The strength and petrographic tests on the concrete cores showed the concrete has adequate strength and no signs of chemical degradation.

One of the tie-back anchors and its associated deadman were visually inspected through temporary excavation. The connection of the tie-back anchor to the steel pile and the connection to the deadman showed signs of corrosion but no significant loss of steel.

Two repair alternatives are included in this report. One involves adding steel plates to the existing steel piles to restore moment capacity of the piles. The other involves adding additional tie-back anchors and waler beams to reduce the moment demand on the existing steel piles. Both repair options include corrosion protection of the piles and soil stability improvements.

The corrosion protection will be provided by installing a coating system. The soil stability will be improved by polymer injection. There is historical video evidence of misaligned concrete lagging panels underwater and the assumption is that backfill material is pumping through the gaps in the panels. The polymer injection is likely to solve the problem of the gaps in the lagging panels, however this specific case should be considered during final design of the polymer injection system. If the polymer injection cannot self-seal gaps that large, containment plates may be required. The cost estimates for polymer injection include an allowance for minor underwater repair at these locations.

The report also includes a discussion of the various levels of structural system performance that can be expected with repair schemes as compared to seawall replacement options or whole-facility relocation.

2 INTRODUCTION

The Rogue Brewery Seawall is approximately 540 feet long and supports the Rogue World Headquarters building at 2320 SE Marine Science Drive in Newport, OR (44° 27′ 12″ N and 124° 3′ 8″ W). This report summarizes the findings from the tie-back anchor investigation, strength testing and petrographic analysis of the seawall lagging, and steel thickness measurements of the exposed flanges.

The condition of one tie-back anchor was investigated through exposing and observing the anchor. A vacuum excavator was used to expose a portion of the connection between the tieback rod and the bracket at the back of the pile, and the connection between the tieback rod and the deadman anchor.

The concrete lagging for the seawall was core sampled in two places and tested for concrete strength and petrographic analysis. The flanges of the steel piles of the seawall were measured to evaluate the level of corrosion.

Based on the results of the Phase II investigation, we have developed possible solutions that will address the deterioration of the structure and the leaking of the backfill material. The possible solutions developed will either extend the service life of the structure (repair option), result in an essentially new structure with 40+ year service life (replace option), or result in a relocated building and a demolished or abandoned seawall (relocate option).

The seawall is considered to have exceeded its useful design life. The wall was originally built circa 1979, so it has been in service for approximately 41 years. If repairs are not made to the corroded steel piles, eventually one of the piles will fail. The failure of the pile will almost certainly result in failure of a portion of the wall and significant damage to the building structure. The cost to repair the local failure would be comparable to the Option A repair cost presented in this report. The likelihood of a massive sudden failure with widespread damage across the entire seawall is low. However, continued corrosion will ultimately require ongoing major repairs.

3 LOADING EVALUATION FOR EXPANSIONS

Due to the corrosion of the seawall piles and the loss of backfill material, the seawall's ability to resist the original design loads has been compromised. The repairs outlined in this report intend to restore the capacity of the seawall to approximately its original strength by replacing or restoring damaged structural elements. There is no indication that the current loading configuration of the facility is overloading the structure. Based on the current condition of the seawall, we recommend that new equipment loading arrangements within 30 feet of the seawall be evaluated on a case-by-case basis. In general, load should not be added (storage, equipment, etc.) within 30 feet of the seawall until repairs have been made to the seawall, a replacement scheme has been constructed, or a specific loading evaluation has been conducted.

The original design live load of the facility is 125 pounds per square foot (psf). The repair schemes developed in this report will preserve the design live loading of 125 psf. Replacement schemes developed in future phases (if necessary) will likely enable higher distributed live load and possibly equipment loading due to the fact that the replacement will be required to be robust enough to resist seismic and liquefaction loads, which are much higher than the original design lateral loads on the wall. However, determination of the magnitude of the capacity increase is beyond the scope of this report.

4 GEOTECHNICAL CONSULTATION

GRI has provided geotechnical recommendations focused on a static evaluation of the existing wall. They have reviewed the as-built drawings of the wall, and the available geotechnical and geologic information for the site, including the recent explorations by Stantec. GRI was present and observed the excavation of the deadman anchor. GRI was consulted regarding the feasibility of additional drilled and grouted tie-back anchors for use as a repair scheme, and they were consulted regarding the applicability and feasibility of the polymer-injection soil stability technique.

A summary of their specific activities and recommendations is included as Appendix G.

5 ENVIRONMENTAL DATA RELATED TO SOIL AND WATER CHEMISTRY

To address the recommendations of the Berger/ABAM report (*See Appendix A, page 11, Possible Rehabilitation Methods and Approximate Costs*) regarding gathering additional information related to the potential for the site soil and water to constitute a corrosive environment, PBS has compiled soil sample information from a previous study.

Apex Laboratories in Tigard, Oregon performed tests on the soil samples for Stantec in Portland, Oregon (report dated April 2, 2020). Five soil samples (GP01-0-10 to GP04-0-10, GP0XC-0-10). The samples were tested for hydrocarbons, volatile organic compounds, polychlorinated biphenyls, organic pesticides, polyaromatic hydrocarbons, metals, anions and pH levels. No volatile organic compounds, polychlorinated biphenyls, organochlorine pesticides were detected in the samples. Excerpts from this report are reproduced below. The report is attached as Appendix B.

Table 1. Anion Levels (mg/kg dry)

Sample	Sulfate	Chloride
GP01-0-10	11.0	0.0
GP02-0-10	0.0	0.0
GP03-0-10	0.0	0.0
GP04-0-10	15.6	12.1

Table 2. pH Levels

Sample	pH Level
GP01-0-10	8.81
GP02-0-10	9.01
GP03-0-10	8.98
GP04-0-10	8.30
GP0XC-0-10	8.99

The California Department of Transportation publishes a document titled *Caltrans Corrosion Guidelines* which provides classification of a site as corrosive to structural elements if conditions exist as follows:

“For structural elements, the Department considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:
Chloride concentration of 500 ppm or greater, sulfate concentration is 1500 ppm or greater, or the pH is 5.5 or less.”

The tests showed that the sulfate and chloride levels of the soil were significantly below the threshold for a corrosive site, and that the pH level of the soil was significantly higher than the threshold. We therefore conclude that the soil conditions do not present a corrosive environment. Note that the sampling and testing program documented by the Stantec report was not a comprehensive site investigation for the purpose of corrosion potential. However, for the purposes of this evaluation it appears to be a reasonable indication that the soil itself is not corrosive.

The water from Yaquina Bay was not tested for salinity content. It should be assumed that the salinity content is high, likely nearly equivalent to seawater, considering the location of the South Beach Marina and the seawall to the mouth of the bay. Therefore, the environmental conditions should be considered corrosive to unprotected steel structure components. According to the Caltrans document, any structure located within 1000 feet of marine or brackish water is considered to be exposed to marine atmosphere. The Rogue Brewery Seawall is located in what we consider to be marine water.

6 MATERIAL TESTING

Tests and measurements were performed on the steel piles and concrete lagging of the seawall.

6.1 Steel Pile Measurements

The steel piles of the seawall were measured by ultrasonic testing (UT). The level of corrosion was determined by comparing the original dimensions of the piles to the measured dimensions. The steel piles of the seawall were measured in 18 locations.

UT thickness measurements are appropriate on sound steel or steel with limited corrosion and little to no delamination. In order to determine a base material loss that can reasonably be applied to the entire system, the thickness measurements were obtained at locations with little or no delamination (i.e., outside the splash zone). In other words, after 40 years of exposure to marine environment, what is the basic steel thickness loss of the piles above water and outside the splash zone? The minimum thickness obtained from this measurement will be used as the base material flange thickness for subsequent calculations.

The maximum thickness measured was 0.95 inches and the minimum thickness measured was 0.78 inches. The average thickness measurement is 0.84 inches. The piles are W18x97 wide flange steel shapes with a published flange thickness of 0.87 inches. See Appendix C for the UT testing report. The difference between the minimum measured thickness and the published flange thickness is considered to be the thickness lost. See the Generalized Loss Column in Table 3.

The thicknesses of the pile flanges measured in the previous Berger/ABAM report are used along with the expansion factor of 4 to determine the effective flange remaining. An expansion factor of 3 is also used to determine a worst case. The Effective Flange Remaining is determined by subtracting the Calculated Loss from the Original Base Thickness. The Corroded Section Thickness (Measured Thickness plus the Generalized Loss) is the Original Base Thickness plus the Corrosion Thickness minus the Calculated Loss. The Corrosion Thickness is the Expansion Factor multiplied by the Calculated Loss. The Calculated Loss is determined from the Corrosion Thickness and the Expansion Factor. The Moment Capacity is determined by calculating the moment of inertia of the W18x97 with one flange thickness as the Effective Flange Remaining. The percentage ratio is determined by comparing the moment capacity using the published moment of inertia value of the W18x97 to the reduced value.

Table 3. Capacity Remaining Due to Corrosion

	Original Base Thickness (inches)	Measured Thickness (inches)	Generalized Loss[1] (inches)	Corroded Section Thickness (inches)	Calculated Loss (inches)	Corrosion Thickness (inches)	Expansion Factor[2]	Effective Flange Remaining (inches)	Moment Capacity vs Original (%)
W18x97	0.87	1.25	0.09	1.34	0.157	0.627	4	0.713	86.0%
W18x97	0.87	2.00	0.09	2.09	0.407	1.627	4	0.463	64.5%
W18x97	0.87	1.25	0.09	1.34	0.235	0.705	3	0.635	79.4%
W18x97	0.87	2.00	0.09	2.09	0.610	1.83	3	0.26	46.3%

Notes 1) Determined from UT testing of non-delaminated steel sections. Assumed that all steel has lost 0.09 inches thickness that is no longer present as corrosion product.

2) Expansion factors from previous Berger/ABAM report and *Properties of Corrosion Production Used in Concrete Cover Cracking Model*.

Figure 1. Steel Pile at Splash Zone

6.2 Concrete Lagging Tests

Samples of the concrete lagging were taken to perform a strength test and a petrographic analysis. Two samples were taken from the concrete lagging at the west end of the seawall between piles 54 and 55 (pile numbering per as-built drawings). The cores were taken from the second to the top concrete lagging panel. The locations of the samples were limited by the accessibility of concrete coring equipment. The technicians were able to obtain samples from the concrete panels near the shore above the water elevation. The samples were taken above the water line to prevent seawater penetrating the patches used to fill the holes created by taking the concrete cores. One core was tested for compression strength and a petrographic analysis was performed on the other core. See Appendix D for the concrete test results.

The compression strength core ruptured at 6360 psi. The as-built drawings indicate that the concrete lagging 28-day design compressive strength (f'c) was 4000 psi. The compression strength of the tested sample is well above the design compressive strength. This indicates that the concrete lagging above the water line has retained its compression capacity.

The petrographic analysis of the concrete core showed one microcrack at the outside face. The microcrack is likely a shrinkage crack. No cracks were present in the core. No evidence of alkali-aggregate reaction nor chemical attack was observed. The other results of the petrographic analysis are consistent with pre-cast concrete lagging panels. Based on the absence of cracks and no evidence of chemical degradation, and visual observations of the lagging, we conclude that the concrete lagging is in generally good condition at the splash zone and above.

The generally good condition of the panels indicates that the concrete is providing adequate protection to the reinforcing steel in the panels.

7 DEADMAN ANCHORS

One tie-back anchor connecting the vertical steel pile and concrete deadman block was exposed for observation. On May 24, 2021, the tie-back anchor connected to pile number 50 was investigated to determine the level of corrosion at both the seawall anchor point and at the deadman anchor point. This tie-back anchor is located on the west end of the seawall. Refer to as-built drawing number 7-E-240 in Appendix E. The anchor at the seawall showed signs of corrosion (See Figure 2). However, the steel connection from the anchor to the pile looked to be intact. No significant loss of steel was observed. The anchor to deadman connection was observed on both sides of the deadman. The tendon is encased in a protective sleeve and the end of the anchor had a protective covering (See Figures 3 & 4). The corrosion level of the tendon could not be determined through observation. The plate that acts as a washer between the anchor end and the deadman showed signs of corrosion but did not have a significant loss of steel. The corrosion protection measures appeared to be intact. Note that we were only able to observe these connections from a distance of approximately 6 feet due to limited access due to the temporary excavation techniques. Only one tie-back out of 54 was observed. Defects in other elements that we were not able to observe may be present.

Based on this observation, it is our opinion that the tieback anchors can be considered to be capable of functioning as originally intended. As discussed elsewhere in this report, the original design did not explicitly consider seismic forces and is unlikely to be adequate to resist a modern design earthquake event.

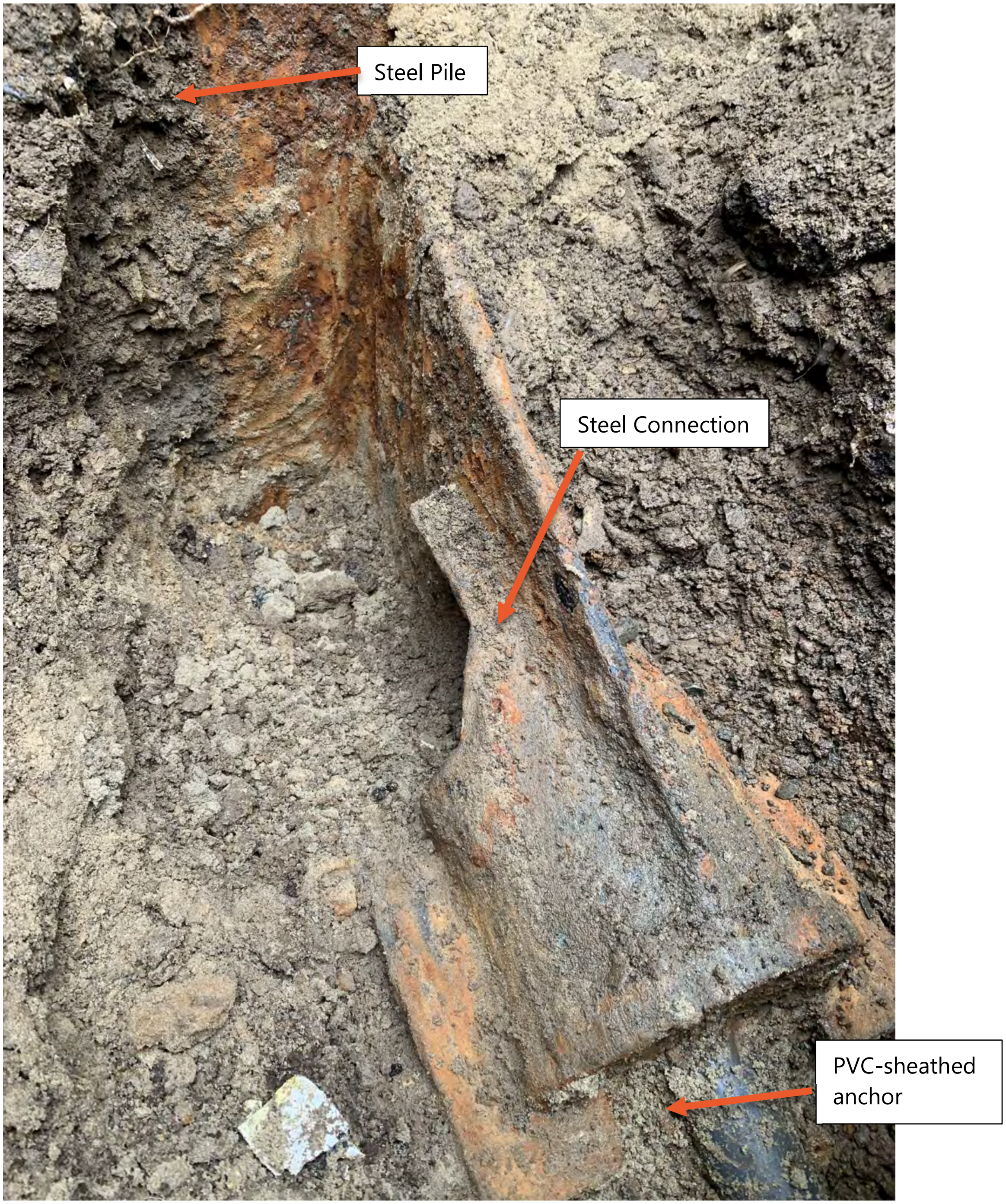

Figure 2. Seawall Tie-back Anchor Connection

Figure 3. Deadman Tie-back Anchor, Seawall Side

Figure 4. Deadman Tie-back Anchor, Upland Side

8 SOIL STABILIZATION INVESTIGATION

This report includes an evaluation of the logistics of high-density polymer injection soil stabilization as recommended by the previous report. Refer to that report for a description of the installation process. Loss of backfill material may have contributed to historic settlement issues. Continued loss of backfill is expected to result in future settlement concerns. Potential locations for the soil stabilization were identified during the site visit.

The west end of the seawall, outside the building, can be easily accessed for the installation of soil stabilization process (See Figure 5). This is the same location that was used for the deadman anchor observations, and a large vacuum excavator truck was able to operate in this area. The interior of the building next to the seawall has limited access points, however a few access points were installed in the building as part of prior investigations, and they would presumably be relatively accessible for the injection operation (See Figure 6). The east end of the seawall was not observed during the site visit due to kegs and other material obstructing access. The kegs and other materials would need to be relocated for the soil stabilization process. It is our opinion that the eastern area outside the building footprint would present no difficulties for the soil stabilization process.

This process will require environmental protection measures to contain any polymer that may seep through gaps in the seawall. The material is lighter than water and will float to the surface. Best management practices for containment of this material includes floating booms. This environmental protection will need to be included in the overall project permitting strategy.

The purpose of the polymer injection is to consolidate the soil near the back of the seawall, to eliminate backfill loss through the wall. As scoped in this report it is not intended as a general under-slab shoring throughout the footprint of the building. The scope of the injection program could be expanded, however, to include areas with suspect under-slab support, if it is desired to rehabilitate the building slab.

Figure 5. West End Soil Stabilization Access

Figure 6. Interior Soil Stabilization Access Location

PBS reviewed video of an underwater inspection conducted circa 2018. We observed misaligned concrete lagging panels that appear to have the potential for allowing backfill to pump through the gaps in the panels. Two screen captures from the video at the same wall location are given in Figures 7 & 8.

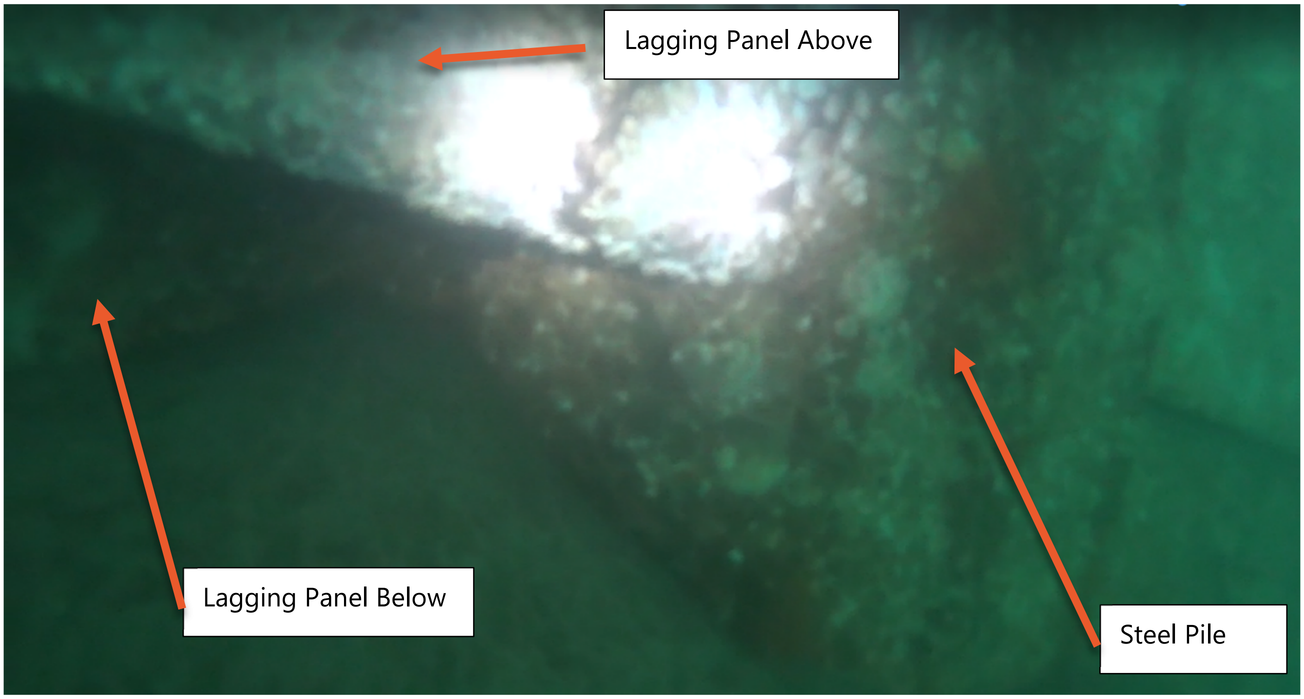

Figure 7. Underwater Concrete Lagging

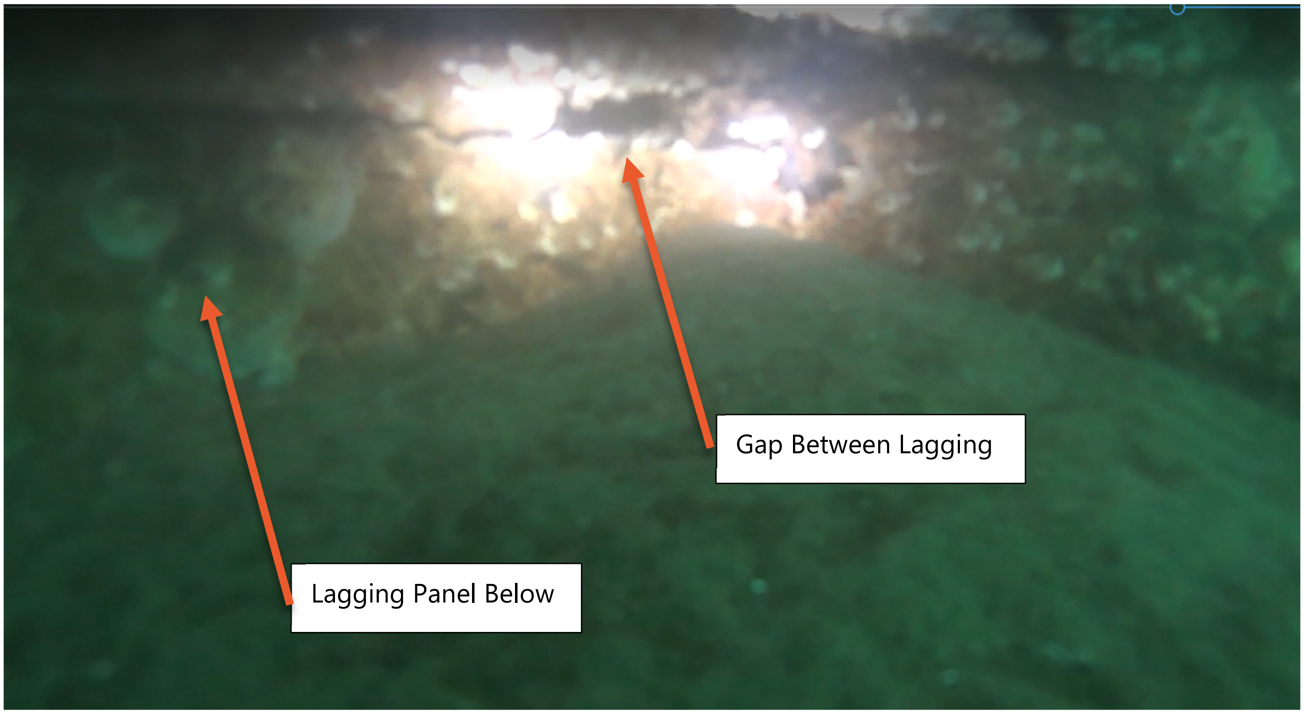

Figure 8. Underwater Concrete Lagging

9 BASIS OF DESIGN FOR MITIGATION SCHEMES

For the purposes of this report, the mitigation schemes discussed include the following:

Repair – The solution involves repairing the damaged elements of the structure such that an extended service life of 20 years or more can be expected.

Replace – The solution involves the installation of an essentially new bulkhead, such that a service life of 40 years or more can be expected.

Relocate – The solution involves relocation of the building and any operations that are supported by the seawall, with new facilities being constructed in a location that does not require a seawall as a structural support system. The solution may also include the demolition of the existing seawall, which may provide opportunity to be used as environmental mitigation for a future in-water project.

This report includes a full basis of design (BOD) for repair options, see Appendix F. Full BOD for replacement or relocation options are not in the scope of this report, but the following table indicates some of the major differences between the basis of design and performance expectations for the three general mitigation options.

Table 4. Basis of Design and Performance Expectations Comparison

Option	Service Life	Seismic Performance[1]	Backfill Stabilization Required?	Ground Improvement Required?
Repair	20 years	Low; no improvement over original design performance; would not be required to upgrade to current building seismic code; not “resilient”	Yes	No
Replace	40+ years	Would require upgrade to current seismic code; Life-safety performance level assumed; not “resilient”	Likely, unless a sheet pile bulkhead-type solution was employed	Likely not practical due to presence of building. Would require designing structure to withstand extreme forces.
Relocate	N/A (>50 years)	Would require design to current seismic code; not “resilient” unless premium paid for resilient construction (code does not require resilient design for this facility)	No	Likely, depending on location and ground conditions

Note 1) The term “resilient” refers to a structure designed to be operational and require little to no repair following the design earthquake and subsequent tsunami event.

10 REPAIR ALTERNATIVES ANALYSIS

The previous report suggested five repair methods (see Berger/ABAM report, Appendix A, Table 3), and further suggested that viable repairs schemes may include combinations of the methods suggested. Our analysis concludes that method number 1 (coating the piles to arrest corrosion) should be included in all repair schemes. Our analysis also concludes that method number 2 (adding lateral bracing to the compression flange behind the wall) would not increase the bending capacity enough to make it a viable option and should not be considered further.

Method number 3 (add section to the pile to replace lost section) is considered a viable option and is further developed herein and presented as Repair Option A.

We have determined that method numbers 3 and 4 are viable options when combined into a single repair scheme. We have developed Repair Option B, which comprises the installation of a horizontal wale beam along the pile face, with a second row of drilled and grouted tie-backs installed between existing piles. This scheme changes the bending moment of the pile such that the moment demand is reduced to at or below the capacity of the piles in their current state. These repair options are further discussed below.

In addition to the two major repair schemes, some general repair should be included in the project along with the pile coating and soil stabilization, regardless of which option is chosen. This includes repair of the concrete spalling of the pile cap and repair of localized lagging damage above and below the water.

In addition to the general seawall repair, the guide piles for the floating walkway are in need of repair. The video provided by the Port of Newport showed approximately 11 guide piles that were missing part of the cross section of the pile. Figure 10 shows one example. While not critical to the performance of the seawall, repairing the seawall and floating walkway concurrently can save mobilization costs for the dive crews performing the work. It is recommended that all 18 pipe piles acting as guides for the floating walkway be repaired or replaced.

The previous report indicated observations of deflection of the top of the steel piles. Our observations did not conclude that significant deflection had occurred. If the repair schemes described in this report are constructed, pile top deflection should no longer be a concern. The pile cap and deadman anchors limit the deflection of the steel piles.

Repair Option A - The loss of effective flange thickness due to corrosion in the steel piles can be made up by welding steel plates to the outside of the flanges. This will increase the bending capacity of the steel piles. The welds between the new steel and the existing steel piles will be at locations above and below the waterline. Underwater welding will be part of the installation of this repair option.

Repair Option B - Adding additional tie-backs will change the constraints of the pile and reduce the moment demand on the piles. To connect the new tie-backs to the existing steel piles a wale beam will be installed. The piles that support the floating walkway at the face of the wall and the movement of the walkway with the tides will interfere with the new wale beam. The walkway piles will need to be replaced or adjusted to accommodate the wale beam.

Figure 9. Concrete Spalling at Pile Cap

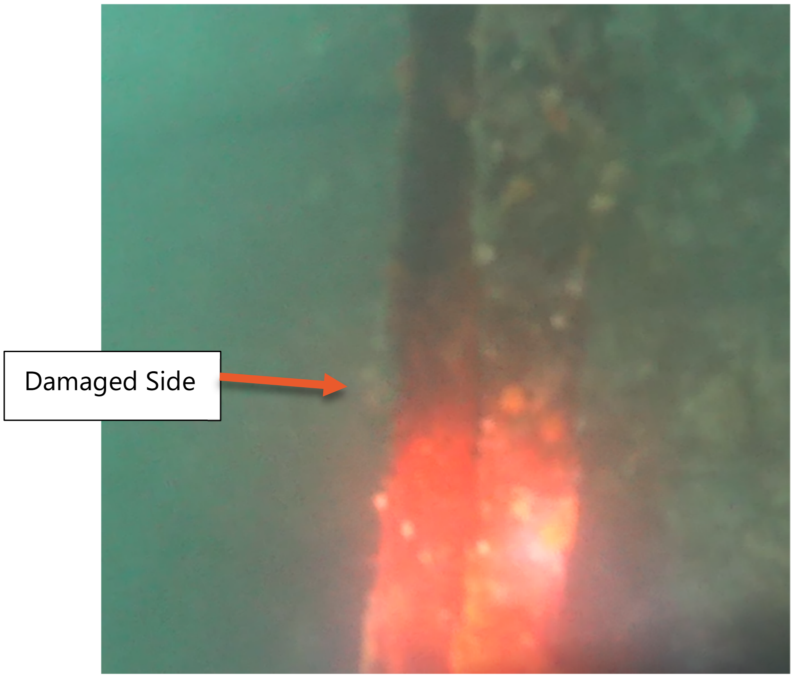

Figure 10. Damaged Pipe Pile

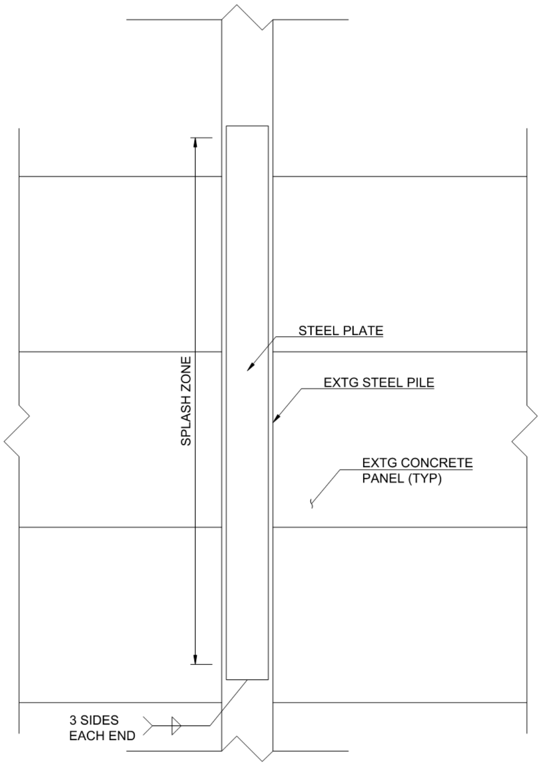

Figure 11 – Repair Option A, Elevation View

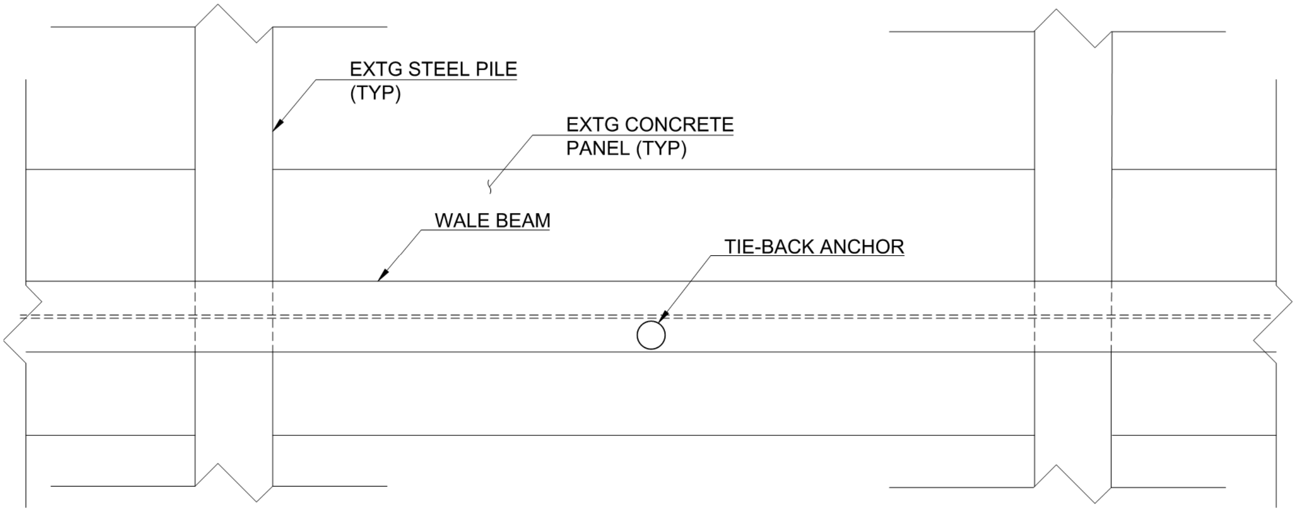

Figure 12 – Repair Option B, Wale Beam and Tieback Anchors, Elevation View

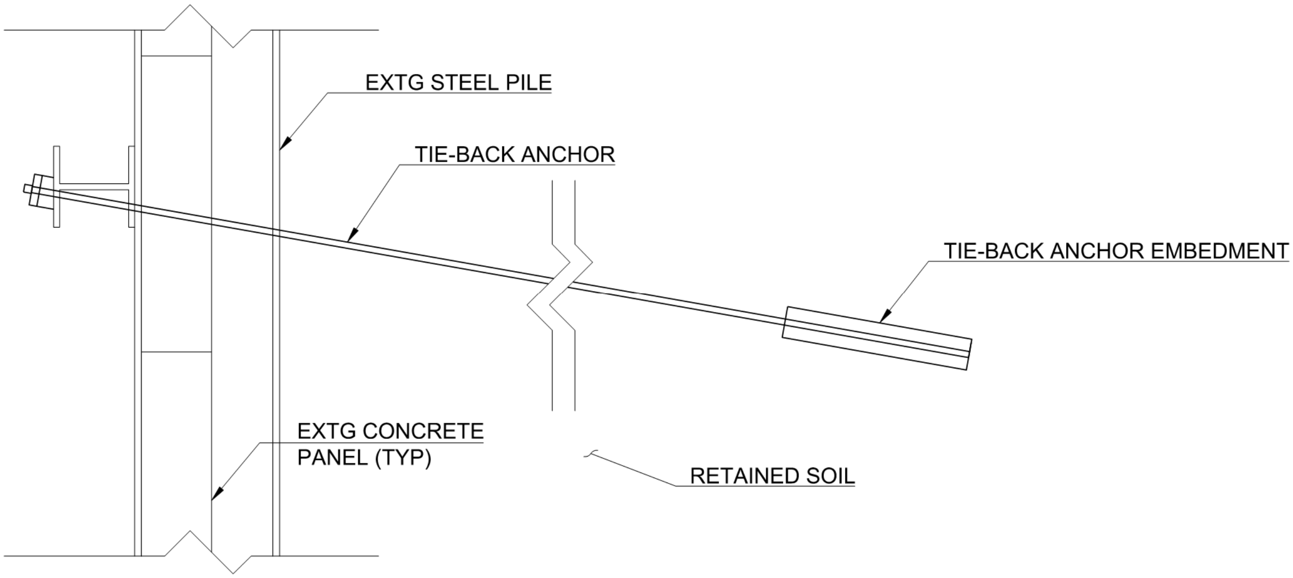

Figure 13 – Repair Option B, Cross-sectional View

The following recommendations are common to both repair options.

Soil stability polymer injection - The loss of soil through the seawall can be mitigated through injecting polymer behind the wall to stabilize the soil. The polymer will be injected at regular intervals behind the wall to fill the voids created from the soil loss and block the soil from leaking through the seawall.

This method was described in the previous report, and we consider it appropriate and feasible. Detailed design of this scheme would be performed by the selected contractor based on performance specifications provided in a construction contract. Significant environmental protections would be required by permit.

Pile corrosion protection - A coating system should be installed on the piles to slow further corrosion of the steel piles. This will consist of removing any delaminated material from the piles and applying the coating to the exterior flanges of the piles. The timing of this work will be tide dependent and significant environmental protections will be required by permit.

Guide piles for the floating dock – The floating dock at the face of the seawall is structurally connected to the seawall as the top of the dock guide piles are attached to the seawall pile cap. The floating dock is not part of the scope of this report, however through review of the underwater video of the seawall, it was noted that many of the guide piles exhibit significant deterioration. The condition of the guide piles does not affect the seawall, but a seawall repair program could include the repair of the guide piles and realize economy related to bundling similar work together into a single project.

These small diameter pipe piles could be replaced from above the splash zone to the mudline, and they could be attached to the seawall pilecap at the top and to the seawall soldier piles near the mudline rather than driven into the seafloor to potentially limit adverse environmental impacts.

See the table below for repair alternatives and their associated service lives. See Appendix H for detailed opinions of probable cost of repair for Options A & B and for the floating dock guide piles.

Table 5. Repair Alternatives

Repair	Service Life Extension	Probable Project Cost[1]	Notes
Option A - Weld additional steel to piles with Soil Stability Polymer Injection and Pile Corrosion Protection	20 years	$1,420,000	Will require underwater welding.
Option B - Additional Tie-backs with Soil Stability Polymer Injection and Pile Corrosion Protection	20 years	$2,320,000	Would likely provide more capacity than the Option A. Waler beam will conflict with floating walkway and piles. Environmental containment for drilling operation will be significant.

Note 1: Cost includes installation, permitting and design.

11 REFERENCES

American Concrete Institute [ACI] (2014). *Building Code Requirements for Structural Concrete*, ACI 318-14.

IBC. (2018). *International Building Code*. Country Club Hills, IL: International Code Council, Inc.

American Institute of Steel Construction [AISC] (2016), *Specification for Structural Steel Buildings*, ANSI/AISC 360-16.

California Department of Transportation Division of Engineering Services Materials Engineering and Testing Services Corrosion Branch, Corrosion Guidelines, Version 3.2, May 2021

Properties of Corrosion Production Used in Concrete Cover Cracking Model; Yuxi Zhao, Haiyang Ren, Hong Dai, Weiliang Jin; International Conference on Durability of Building Materials and Components; April 12th-15th, 2011

Appendix A

Structural Evaluation Report

BergerABAM
December 2018

Structural Evaluation Report

Port of Newport
Rogue Brewery Seawall

Submitted to

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STRUCTURAL EVALUATION REPORT

Port of Newport
Rogue Brewery Seawall
Newport, Oregon

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STRUCTURAL EVALUATION

INTRODUCTION

Background

The Port of Newport retained BergerABAM to perform a limited structural condition assessment and evaluation of the Rogue Brewery Seawall located at the South Beach Marina in Newport, Oregon. Rogue Brewery Seawall is approximately 540 feet long and supports the Rogue World Headquarters building at 2320 SE Marine Science Drive in Newport (44° 37′ 12″ N and 124° 3′ 8″ W).

Purpose

The overall purpose of the project is to provide an assessment of the current structural conditions and service life of the seawall and provide possible solutions and associated costs with repair approaches. The results of this report are intended to assist the Port of Newport in developing plans for maintenance and rehabilitation in order to maintain the long-term functionality of the seawall.

Documents Reviewed

BergerABAM reviewed the following documents as part of the basis for this condition assessment.

- Original as-built drawings for the seawall and the superstructure shelter, dated 1 February 1979.
- Original as-built drawings for the Rogue Ales Brewery building (formerly the Dry Moorage Building), dated 1 February 1979.
- Evaluation of slab-on-grade floor – Letter report, BergerABAM No. PAPOR-04-053, dated 3 October 2003.
- Rogue Ales Tasting Room Addition, Job No. 91-96, Engineering Concepts Inc., dated 1 December 1997.
- Original geotechnical report: Soils Investigation, South Beach Marina on Yaquina Bay, Newport, Oregon, Dames and Moore, dated 8 March 1978.

The following references were used to check the soldier piles:

- Retaining Wall Design Guide, U.S. Department of Agriculture, FHWA-FLP 94006, September 1994.
- Heavy Construction Costs with RSMeans Data, 75th Annual Edition, 2017.

Description

The seawall supports the Port's tenant, the Rogue Ales Brewery facilities. The Rogue Ales Brewery building was built in 1980 and is currently being supported by the seawall on its north side. The building is approximately 98 feet by 240 feet with a maximum roof height of 46 feet. This building was first occupied by the Rogue Ales Brewery in 1992 and is currently being used for beer production and packing activities. It also contains a restaurant.

SEAWALL CONFIGURATION

The Rogue Brewery Seawall comprises steel soldier piles and concrete lagging panels tied back with steel rods to a deadman anchor (see Appendix B). The W18 soldier piles were spaced 10-feet on center and supported about 4 feet 6 inches below the pile top by deadman anchor tie-backs. According to the as-built drawings, the tie-back anchors consist of 1-1/4–inch-diameter, high-strength steel rods, coated in mastic and covered with extruded polyethylene. The anchors are connected to 5-foot square by 1-foot thick precast concrete deadman slabs. Tie-back lengths are variable but mostly 60 feet. A 2-foot-8-inch by 1-foot-11-inch pile cap embraces all piles tips. The seawall involves 56 soldier piles as detailed in Table 1. Concrete lagging was used between soldier piles to support the backfill.

Table 1. Pile Data for Rogue Brewery Seawall

Pile No.	Tip Elevation	Length
1 & 55	-14'-4"	30'
2	-19'-4"	35'
3 & 54	-24'-4"	40'
4	-29'-4"	45'
5 & 53	-36'-4"	50'
6	-39'-4"	55'
7 - 52	-44'-4"	60'
6	-4'-4"	20'

Note: The pile top elevation is 14 feet 6 inches. Data provided on the as-built drawings was not independently verified. Mean lower low water (MLLW) is 0'-0".

Structural Materials

The material data are derived from the as-built drawings. The soldier piles conform to ASTM-A588 Grade B steel with yield stress of 50 ksi. The drawings indicate that the tie-back rods have an ultimate strength of 150 ksi. All hardware and bolts were hot dip galvanized. The concrete reinforcement was A-615 Grade 40 and the concrete minimum 28-day strength was 4,000 psi, with cement Type II as noted in ASTM C-150 and aggregate per ASTM C-33.

INSPECTION METHODOLOGY

BergerABAM visited the site of the Rogue Brewery Seawall on 27 February 2018 and 8 October 2018. Howard Wells, PE, senior project manager, led the inspection with assistance from engineer Vahid J. Azad (present only in the second inspection). Also present at the second visit were Aaron Bretz and Chris Urbach with the Port of Newport. The first inspection was performed in a near-low tide condition while the second happened at a near-high tide condition. The inspection was conducted in general conformance with a Routine Above-Water Inspection as set forth by the American Society of Civil Engineers (ASCE) *Waterfront Facilities Inspection and Assessment* manual.

Additionally, the superstructure (Rogue Ales Brewery building) was inspected from inside for possible damage due to backfill instabilities. Due to considerable settlements under the building slabs, a local repair along the seawall was performed about 10 years ago. The slabs on grade were generally inspected for additional damage after the local repair on 8 October 2018.

The inspection was limited to accessible components of the structure. Inspection methods were visual. Underwater inspection and destructive testing were not in the scope of this work. The inspection assessed the general condition of the whole soldier pile wall with the intent of providing recommendations for future maintenance and rehabilitation according to the ASCE manual.

EXISTING CONDITIONS

The four decades of exposure to the marine environment have resulted in visible deterioration of many of the seawall major structural elements. This deterioration includes corrosion of the steel soldier piles and spalling of the concrete beam/pile cap. In addition, some loss of backfill material through gaps in the concrete lagging panel is apparent as material can be seen in front at the base of the wall. It is suspected that some historical settlement of the interior floor slab of the brewery may be due to this material loss. Finally, the wall appears to be deflecting outward in some places, although this deflection may have occurred at the time of construction rather than gradually over time. While a detailed description of possible damage mechanisms is provided hereafter, Appendix A presents more informative visual inspection pictures taken in both visits.

Soldier Piles

Soldier piles are the major structural components in the seawall, and their performance can directly affect the superstructure. There are visible misalignments, cracks, and corrosion damage as described hereafter.

Visible Corrosion Damage

Figure 1 (a through e) shows the typical damage to the soldier piles. There is corrosion damage visible as laminated rust in two zones (a) the tide splash zone (elevation -3 feet to elevation +10 feet on average) and (b) below the cap beam on the all soldier piles. Considerable expansion was observed on the seaside pile flange showing a thickness increasing to approximately 1-1/4 to 2 inches (originally 0.87 inch for W18x97).

1(a). Corrosion on soldier piles
(27 February 2018)

1(b). Typical chloride-induced corrosion damage in splash zone
(27 February 2018)

1(c). Typical chloride-induced corrosion damage in splash zone
(8 October 2018)

1(d). Typical chloride induced corrosion damage under the pile cap
(8 October 2018)

1(e). Formation of calcium carbonate shows the possibility of carbon-induced corrosion (27 February 2018)

Figure 1. Observed corrosion damage on soldier pile flanges

The damage is also severe below the pile cap where there is no direct water contact. This is due to the geometry of the corroded area, where the pitting and crevice corrosion possibilities are higher than smooth areas. The chloride-induced corrosion is more probable in locations where the access to oxygen is more limited because of specific geometric configurations like corners, etc.

There are various locations where the pile cap concrete has cracked or spalled (as will be discussed later in this report). This may be due to pile tip outward deformations, especially on the western side, caused by corrosion damage, extra surcharge, etc.

Deadman Anchors

The as-built drawings indicate that, based on ASTM standards, 1-1/4-inch-diameter anchors with a 2-inch sleeve and corrosion protection were installed at the time of construction. The anchors and connections were not checked during the site visits. The existing misalignments in the wall profile may indicate some tie-back insufficiencies, but from the overall wall stability, it does not appear they are in a critical situation. There might be other reasons behind this outward deformation in addition to tie-backs, such as imperfect alignment during original construction.

Concrete Lagging

The concrete laggings are in generally good condition in terms of concrete surface quality (cracks, spalling, etc.) and vertical alignments. Figure 2 shows a typical lagging condition.

Figure 2. Concrete lagging existing conditions
(There are surface effects from water; however, the overall visual inspection seemed acceptable at this point.)

Some minor sulfate attack and carbonation issues were found during the visual inspections. The corrosion or degradation due to carbon or sulfates can be monitored and prevented with service-life modeling and design, probably with coating. This is a less severe damage mechanism than chloride-induced corrosion, but can be resolved when corrosion inhibitors are applied.

Concrete Pile Cap

There is visible damage on the intersection of pile cap and solder piles in many locations. At some points, as shown in Figure 3(a), concrete spalling is evident. The spalling is most probably related to minor tension happening on pile cap face due to lateral pile deformations (i.e., minor axis bending on the pile cap). There are many other locations where small repairs have been performed over time for outer cracking on pile cap, shown in Figure 3(b).

3(a). Concrete spalling on pile cap (Pile No. 42, see as-built drawings)

3(b). Repairs for cracks on pile cap face

Figure 3. Concrete spalling on pile cap (Pile No. 40)

Backfill Material

According to Mr. Bretz, the backfill materials are continuously leaking into water from the concrete lagging joints in some location. This issue may be the reason behind the historical slab on grade settlements in the Rogue Ales Brewery.

Several years ago, a repair program was performed by the tenant to attempt to arrest slab settlements on the interior of the building. The repair scheme involved cutting 3-foot-diameter holes in the slab approximately 5 feet behind the seawall. These holes were spaced approximately 20-feet on center for the full length of the seawall. Flowable concrete or grout was placed through these slab penetrations to fill voids between the slab and the soil below. It is our understanding that this concrete or grout was not installed under mechanical pressure. It was placed in a flowable state, and travelled beyond the slab opening only as far as the material was able to flow under the influence of gravity. The extent of the void filling is unknown. The slab openings were sealed with manhole lids.

The repair appears to have arrested the settlement, but it was not possible for us to determine how well the repair is performing in light of the continued loss of backfill material that has been observed. There may also be areas of slab that are not continuously supported by soil or grout. These "soft spots" may be functioning because of the small inherent bending resistance of the slab, rather than continuous bearing support, as intended by design. If this is the case, the slab could be at risk for localized cracking, settlement, or collapse under concentrated loading, or possibly, under distributed uniform loading, if the backfill loss continues.

CODE BASED ANALYSIS OF ROGUE SEAWALL (SOLDIER PILES AND TIE-BACKS)

To obtain a preliminary evaluation of soldier pile structural *initial* and *existing* performance, a stress analysis was conducted based on the following assumptions:

1. The geotechnical parameters were provided by GRI based on typical average soil types in South Beach Marina, Newport, Oregon (including friction angle as 35 degrees and soil density as 110 pounds per cubic foot).
2. Full drainage was assumed resulting in no hydrostatic pressure behind the wall.
3. The soil was considered saturated below the water level at elevation 0 feet (MLLW) as shown on the as-built drawings.

Figure 4 shows the loading assumption on the soldier pile with tie-back wall. According to as-built drawings, the piles were not driven to bedrock. The tie-back and W-sections will be rechecked based on AISC-ASD for the tallest piles (Pile Nos. 7 through 52).

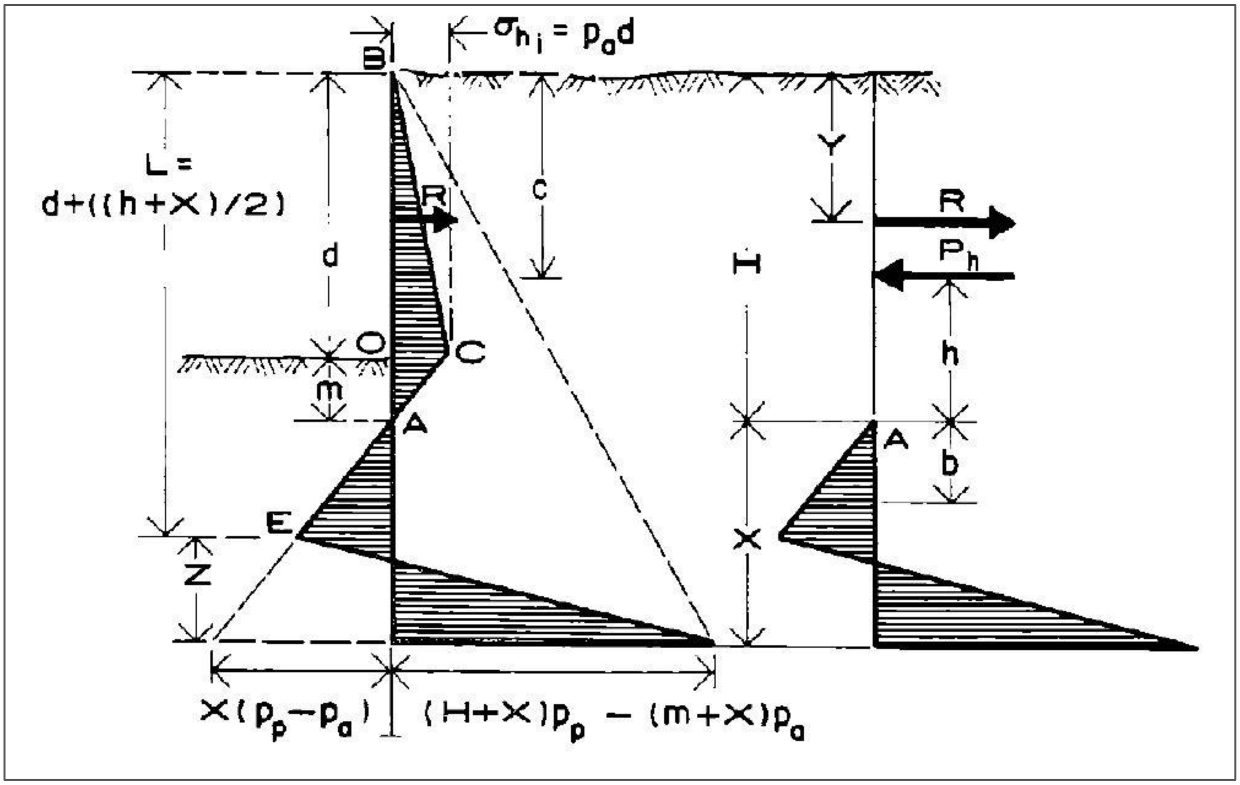


Figure 4. Backfill loading on soldier piles
(Reference: Retaining Wall Design Guide, FHWA-FLP 94006)

The live load on the building slab was assumed per ASCE 7-10: light manufacturing as 125 psf. This preliminary assessment report is concentrated on the results for gravity loads and backfill pressures and excludes seismic loading. A complete repair and rehabilitation should include all possible load combinations including seismic events.

Initial Structural Code Based Design Recheck

The goal of this recheck is to reproduce the structural design calculations and compare the existing degraded structure. The following design assumptions were held:

1. Tie-back tension capacity was calculated from F_{ult} = 150 ksi.
2. No aboveground lateral bracing is assumed for the piles; i.e., the laterally unbraced length is approximately 30 feet.

Current Structural Code Based Check

Corrosion products take more volume compared to initial iron material. The measurements from the site visit indicated flange thicknesses of approximately 1-1/4 to 2 inches. Assuming an average of four times volume expansion during steel corrosion due to corrosion products formation, increasing the flange thickness from 0.87 inch to approximately 1-1/4 to 2 inches can be translated to a flange thickness reduction of about 0.14 to 0.38 inch. Table 3 shows the existing pile brief analysis results using a 0.87- 0.38 inch (or 0.14 inch) equals 0.49 inch (or 0.73 inch) splash zone flange thickness. The reduced section assumes a uniform damage to the exposed flange *only*.

Table 2. Analysis Results for Tallest Soldier Piles: Current Corroded Conditions

Pile No.	Spacing (ft)	Surcharge	Tie-back force	Maximum pile moment in corroded area	Tie-back capacity		Maximum W section capacity	
					Initial	Existing	Initial	Existing
7-52	10	125 psf	110 kips	406 ft-kips	92 kips	*unknown*	410 ft-kips	352-410 ft-kips

Based on the initial calculations in Table 2, it seems the selected sections at construction time were economically chosen. This calculation is assumed as a base for the next section where the corrosion effects are considered.

According to Table 2, a significant moment capacity decrease is seen compared to the existing loads and previous calculations. The maximum moment happens on the lower part of the corroded area, underwater, where the corrosion damage is slightly less than the upper part. These calculations show the need for possible repairs, which should be based on more accurate structural analyses using valid input data taken from the site, as discussed later in this report.

POSSIBLE REHABILITATION METHODS AND APPROXIMATE COSTS

Our limited investigation and analysis suggests various issues from a structural and material standpoint where further in-depth analysis based on field testing is warranted. The possible repair costs cover a large range because of the limited nature of this initial assessment. This report will provide cost ranges assuming different repair levels.

Accurate performance-based analyses and repair design will provide extended service life of the Rogue Brewery Seawall at minimum cost. The provided data should involve:

- geotechnical data for backfill mechanics during normal strength and extreme seismic events;
- material and dimensional data for concrete lagging, soldier pile reduced sections, pile cap and their components;
- tie-backs connections and anchorage data; and
- superstructure surcharge estimations and geometry of the considerable loadings.

We also recommend a continuous service life prediction. Establishing the chemical composition of the soil and water (sulfate amounts, pH, carbon, and chloride content) will be useful in the service life analysis.

Soldier Piles

The initial step will be the protection of current piles against further corrosion using coating materials according to NACE and ASTM standards for highways and bridges. Table 3 provides different proposed methods and the approximate involved costs. The final decisions on the methods require in-depth analyses that need accurate site data as explained previously.

The final design will likely include multiple methods provided in Table 3, because the damage extent over the structure is variable. The calculations in the table are simply assuming a uniform damage level.

Table 3. Possible Soldier Pile Repair Methods and Approximate Associated Costs (30 percent contingency was applied).

Method No.	Repair Method	Work items	Approximate Cost per pile	Conditions	Description
1	Pile corrosion protection using coating. (This method is required with all other methods.)	• Cleaning of structural metal framing • Coating	$700	All.	A basic coating protection method is assumed here.
2	Lateral bracing for existing soldier piles.	• Local lagging demolition (112 #.) • Bracing material (W8x15: 860 LF) • Welding • Cleaning of structural metal framing • Coating	$1400	Low corrosion damage and short piles.	This method will slightly increase pile bending capacity. It requires local lagging demolition to access pile compression flange. Material cost details from a quote from Skyline Steel and labor from RSMeans Data.
3	Adding another section on each pile and providing welding connections.	• Additional pile (W18x50: 3155 LF) • Welding • Cleaning of structural metal framing • Coating	$4800	The existing pile capacity is not enough versus demands. Also, connections to existing piles are possible.	This method will require a permit to extend the structure into water. Cost details from a quote from Skyline Steel.
4	Horizontal component (e.g., truss or waler) at the maximum force locations.	• Truss material (HSS 6x5x3/8: 2500 LF) • Tie-backs (20 #) • Welding • Cleaning of structural metal framing • Coating	$5200	In addition to method 3, plus if there are minor issues with tie-backs.	The horizontal member can connect piles faces and be supported in few locations using additional tie-backs. This method will require in-water permits. Material cost details from a quote from Skyline Steel and labor from RSMeans Data.
5	Second level tie-back.	• Tie-backs (56 #) • Cleaning of structural metal framing • Coating	$6250	When the existing pile capacity is too low compared to demands and water work permits are not available.	Cost details from U.S. Department of Transportation Bid Item Unit Price Average.

Deadman Anchors

There was no access to deadman anchoring systems; therefore, any repair suggestion is dependent on further in-depth investigations. We suggest gaining access to the connections, at least where the misalignments have happened, to make sure the connections and tie-back are stable.

Concrete Lagging

The lagging system is not in a critical situation. The surface conditions do not show significant damage at this point; however, the structural damage usually becomes evident well after the initiation of corrosion. Therefore, the service-life predictions will be very useful for concrete lagging as important structural components. Core sampling at different zones is suggested for the overall prediction of long-term lagging performance. The possibility of sulfate attack should also be determined.

Concrete Pile Cap

Local repairs are needed for the pile cap after the overall soldier pile tip deformation is resolved. The associated repair includes resolving the deformation issue independently and repairing the spall damage on pile cap. The cost associated with this repair is quite low compared to other structural issues and is ignored at this stage.

Backfill Material

Soil stabilization is recommended to prevent more backfill loss into water to increase the superstructure service life. According to Mr. Urbach, the sinkholes due to vertical settlement on the superstructure subgrade soil were about a foot deep in a very wide area close to the seawall. The sinkholes were filled with aggregates and cement mortar about 10 years ago (but not mud jacking). The previous repairs have helped the performance of the floor, but the remaining structural life is unknown. In addition, the current stable conditions may be due to bending action of floor slabs.

For the repair, high-density polymer injection is suggested. The low viscosity polymer resin components are injected underground using small holes in the floor (5/8- to 2-inches diameter). The polymer material flows into the voids and weak zones in the soil mass. Then, the polymer starts reacting and results in an expanded reaction product that can influence 8 to 10 feet around it. The material can drive out water and seal the backfill from the entry of water into subsurface soil pockets. A patterned injection is used by the technicians so that all voids can be filled. The process can be monitored under and above water using divers and live-stream video.

The associated cost for soil stabilization ranges from $580,000 to $715,000 (Ref: quote from Uretek, with a 30 percent contingency), assuming the whole wall length requires polymer materials. Different factors can affect this pricing, including spot treatment (reduces the costs) and superstructure subgrade stabilization requirements (increases the costs). There might be a considerable variance in the costs based on the amount of material loss under the superstructure slabs, which is currently unknown.

NEXT STEPS

We recommend the following in-depth investigations as the next step for final repair design and predicted service life of the seawall structure. Together, these activities can be thought of as the Phase 2 Investigation.

- Perform thorough condition assessment and document current damaged structural system, to a level of detail sufficient to enable selection of the repair schemes and to enable production of construction contract documents.
- Prepare superstructure loading evaluations for probable future extensions.
- Prepare a geotechnical report involving backfill pressures, site seismologic data, tide information, etc.
- Review environmental data on soil/water chemistry and environmental factor histories (temperature, wind, etc.).
- Perform sampling from the concrete lagging and steel piles and the required chemical and mechanical tests in laboratories.
- Perform inspections for soil stabilization;
- Obtain access to inaccessible portions of the structure, such as deadman anchor connections.

The final repair recommendations (Phase 3 Final Design) will be performed using the results of these investigations.

CONCLUSION

This report provides an objective evaluation of current structural performance of the Rogue Brewery Seawall. With existing loading, the seawall structure is not facing a short-term safety problem; however, the future service life of the structure is unknown and there are two major problems that need to be addressed: backfill stabilization and soldier pile repairs.

Before we can provide final detailed repair recommendations, we recommend investigations, including a more in-depth data-gathering program, service-life analysis, and repair alternatives analysis. This study should be performed in conjunction with an economic evaluation of the facility by the Port in order to determine cost-benefit ratios associated with various repair and replacement schemes.

The final repair recommendations will be based on the damage extents provided by the in-depth investigations. The repair method may be variable over the seawall and will range from minor to major repair methods. The following approximate costs are associated with the repair phase:

Engineering and Permitting: $265,000

Soil Stabilization: $715,000

Soldier Piles Repair: $350,000

The final optimized seawall repair will likely be a mixture of methods in Table 3 over the structure because the damage is not uniform. The above cost may change with further assessments and over time.

In addition, there might be extra repairs required for other structural elements that were visually inaccessible during the site visits, including deadman anchors, anchor connections, concrete lagging reinforcement, etc.

The provided service life of the repaired structure will completely depend on the repair methods and structural evaluation intervals. An extension of 20 years or more to the current service life is possible with regular structural evaluations and maintenance. At this point, BergerABAM cannot provide an opinion on the serviceable future of the seawall and fill, given current loading. The extended service life can be determined after in-depth investigations and repair methods are finalized.

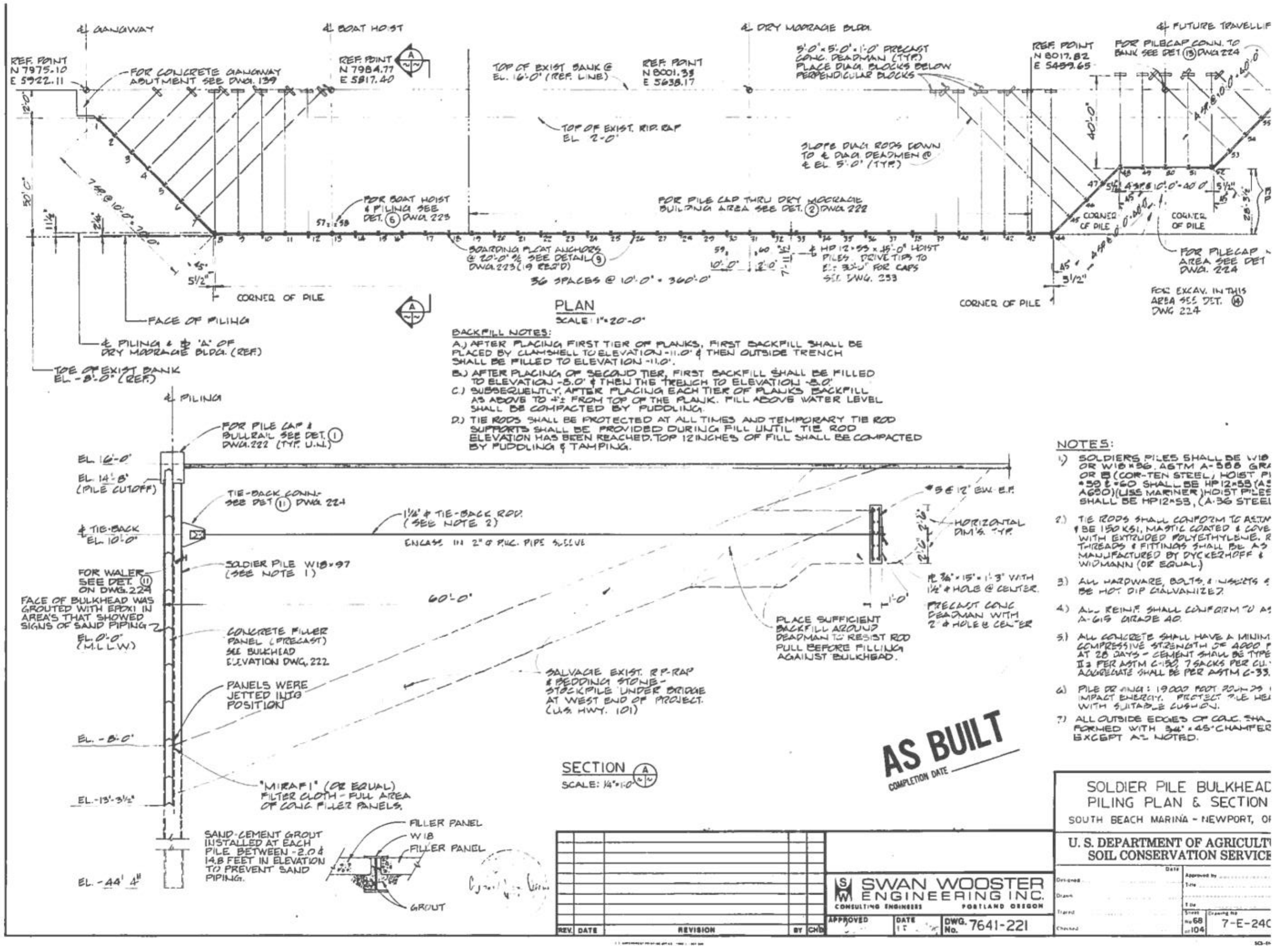
Structural Evaluation Report
Port of Newport Rogue Brewery Seawall
Newport, Oregon

Appendix A
Additional Photographs


SEA VENTURE I

Structural Evaluation Report
Port of Newport Rogue Brewery Seawall
Newport, Oregon

Appendix B
Rogue Brewery Seawall Drawings



BACKFILL NOTES:

A.) AFTER PLACING FIRST TIER OF PLANKS, FIRST BACKFILL SHALL BE PLACED BY CLAMSHELL TO ELEVATION -11.0' & THEN OUTSIDE TRENCH SHALL BE FILLED TO ELEVATION -11.0'.

B.) AFTER PLACING OF SECOND TIER, FIRST BACKFILL SHALL BE FILLED TO ELEVATION -8.0' & THEN THE TRENCH TO ELEVATION -8.0'.

C.) SUBSEQUENTLY, AFTER PLACING EACH TIER OF PLANKS BACKFILL AS ABOVE TO 4'± FROM TOP OF THE PLANK. FILL ABOVE WATER LEVEL SHALL BE COMPACTED BY PUDDLING.

D.) TIE RODS SHALL BE PROTECTED AT ALL TIMES AND TEMPORARY TIE ROD SUPPORTS SHALL BE PROVIDED DURING FILL UNTIL TIE ROD ELEVATION HAS BEEN REACHED. TOP 12 INCHES OF FILL SHALL BE COMPACTED BY PUDDLING & TAMPING.

NOTES:

1.) SOLDIERS PILES SHALL BE W18 OR W18 x 56, ASTM A-588 GRA OR B (COR-TEN STEEL, HOIST PI #59 & #60 SHALL BE HP 12 x 53 (AS A690) (USS MARINER) HOIST PILES SHALL BE HP12 x 53, (A-36 STEEL

2.) TIE RODS SHALL CONFORM TO ASTM & BE 150 KSI, MASTIC COATED & COVE WITH EXTRUDED POLYETHYLENE. R THREADS & FITTINGS SHALL BE AS MANUFACTURED BY DYCKERHOFF & WIDMANN (OR EQUAL.)

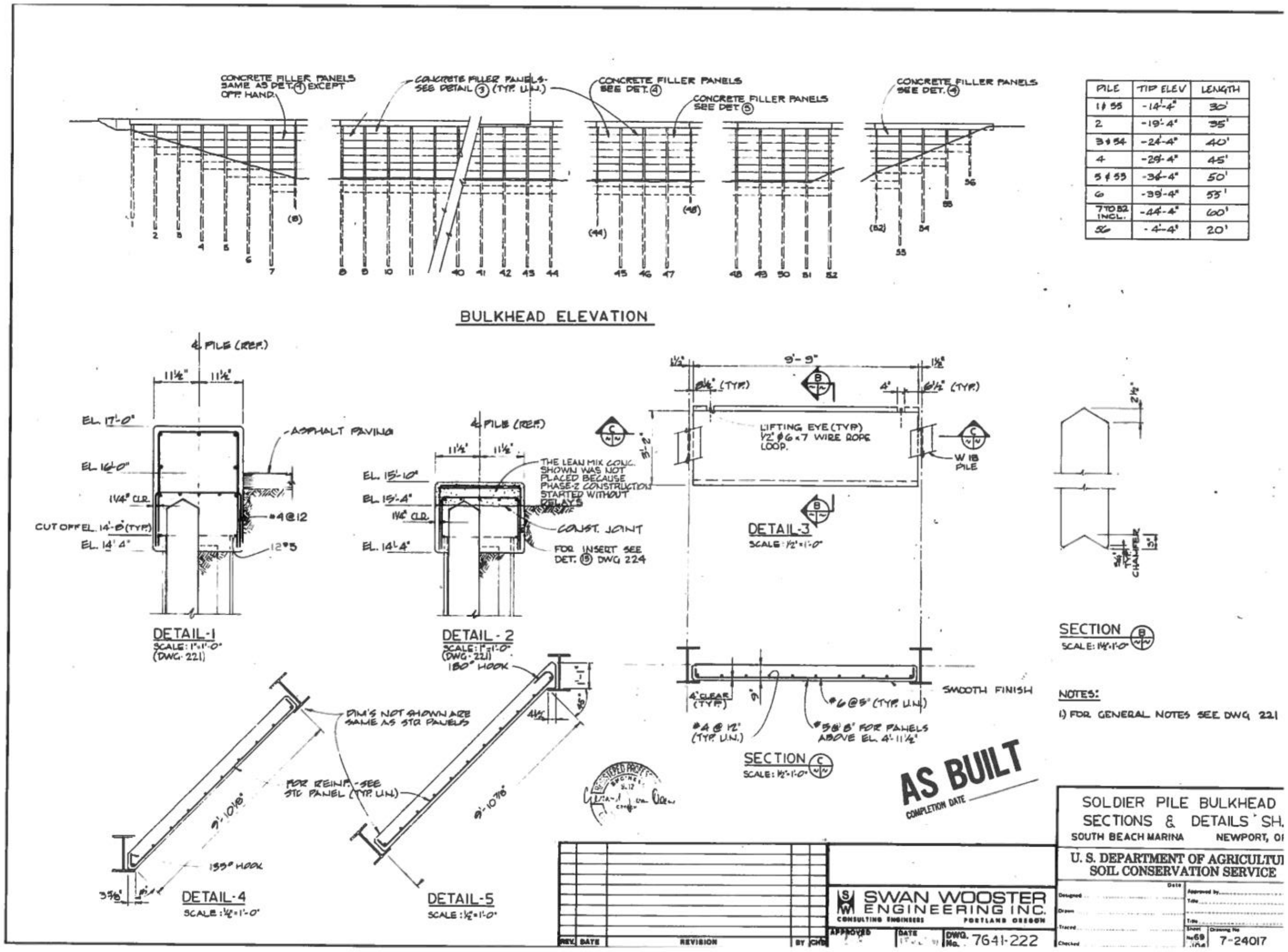
3.) ALL HARDWARE, BOLTS, & INSERTS S BE HOT DIP GALVANIZED.

4.) ALL REINF. SHALL CONFORM TO AS A-615 GRADE 40.

5.) ALL CONCRETE SHALL HAVE A MINIM COMPRESSIVE STRENGTH OF 4000 P AT 28 DAYS - CEMENT SHALL BE TYPE II a PER ASTM C-150, 7 SACKS PER CU. AGGREGATE SHALL BE PER ASTM C-33.

6.) PILE DRIVING: 19000 FOOT POUNDS IMPACT ENERGY. PROTECT PILE HEA WITH SUITABLE CUSHION.

7.) ALL OUTSIDE EDGES OF CONC. SHA FORMED WITH ¾" x 45° CHAMFER EXCEPT AS NOTED.



PILE	TIP ELEV	LENGTH
1 & 55	-14'-4"	30'
2	-19'-4"	35'
3 & 54	-24'-4"	40'
4	-29'-4"	45'
5 & 53	-34'-4"	50'
6	-39'-4"	55'
7 TO 52 INCL.	-44'-4"	60'
56	-4'-4"	20'

NOTES:

1) FOR GENERAL NOTES SEE DWG 221

SOLDIER PILE BULKHEAD
SECTIONS & DETAILS SH.
SOUTH BEACH MARINA NEWPORT, O

U. S. DEPARTMENT OF AGRICULTU
SOIL CONSERVATION SERVICE

SWAN WOOSTER ENGINEERING INC.
CONSULTING ENGINEERS PORTLAND OREGON

REV.	DATE	REVISION	BY	CHK

APPROVED
DATE
DWG. No. 7641-222

Designed
Drawn
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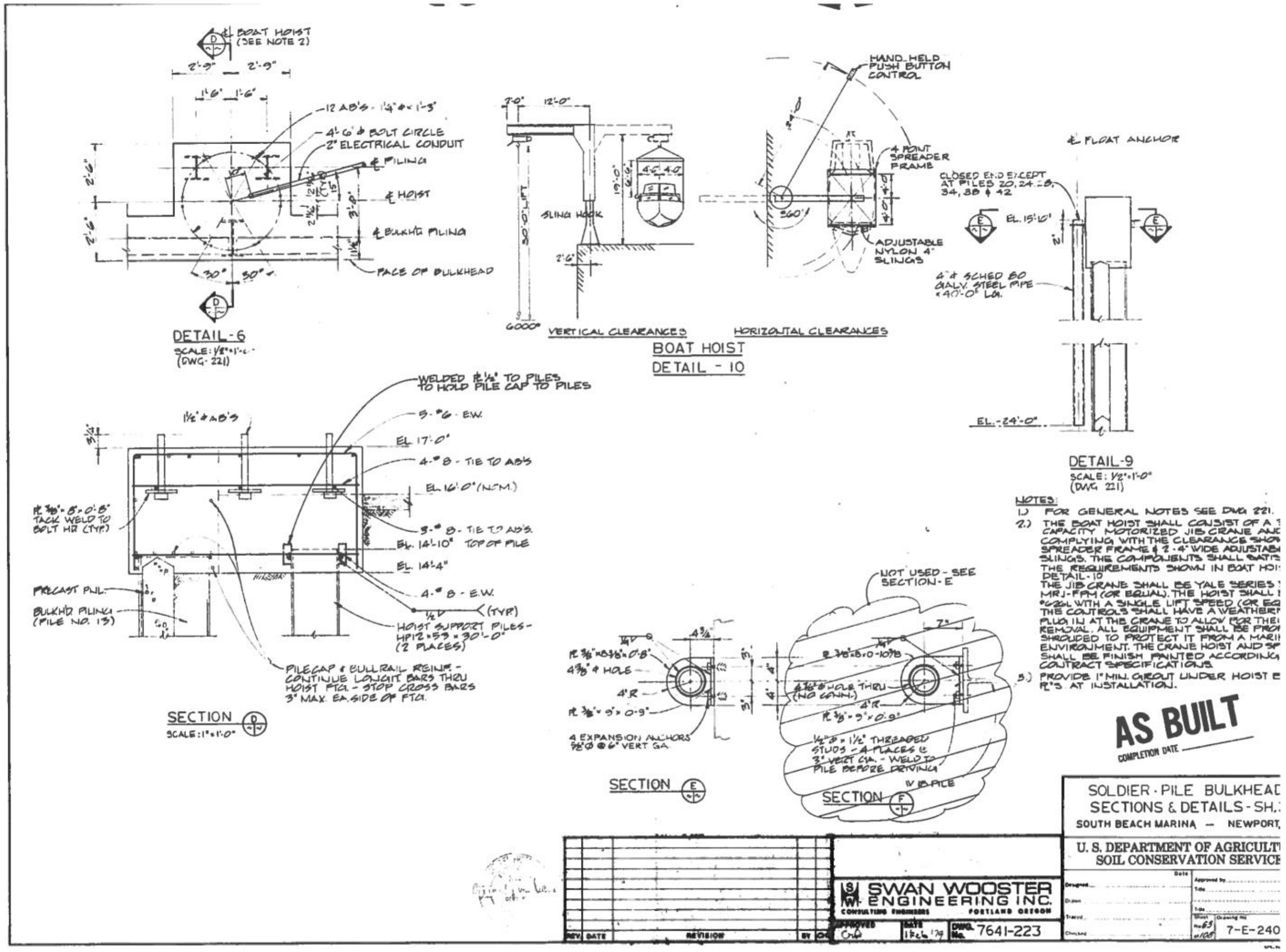
Date
Approved by
Title
Sheet No 69 / 104
Drawing No 7-24017

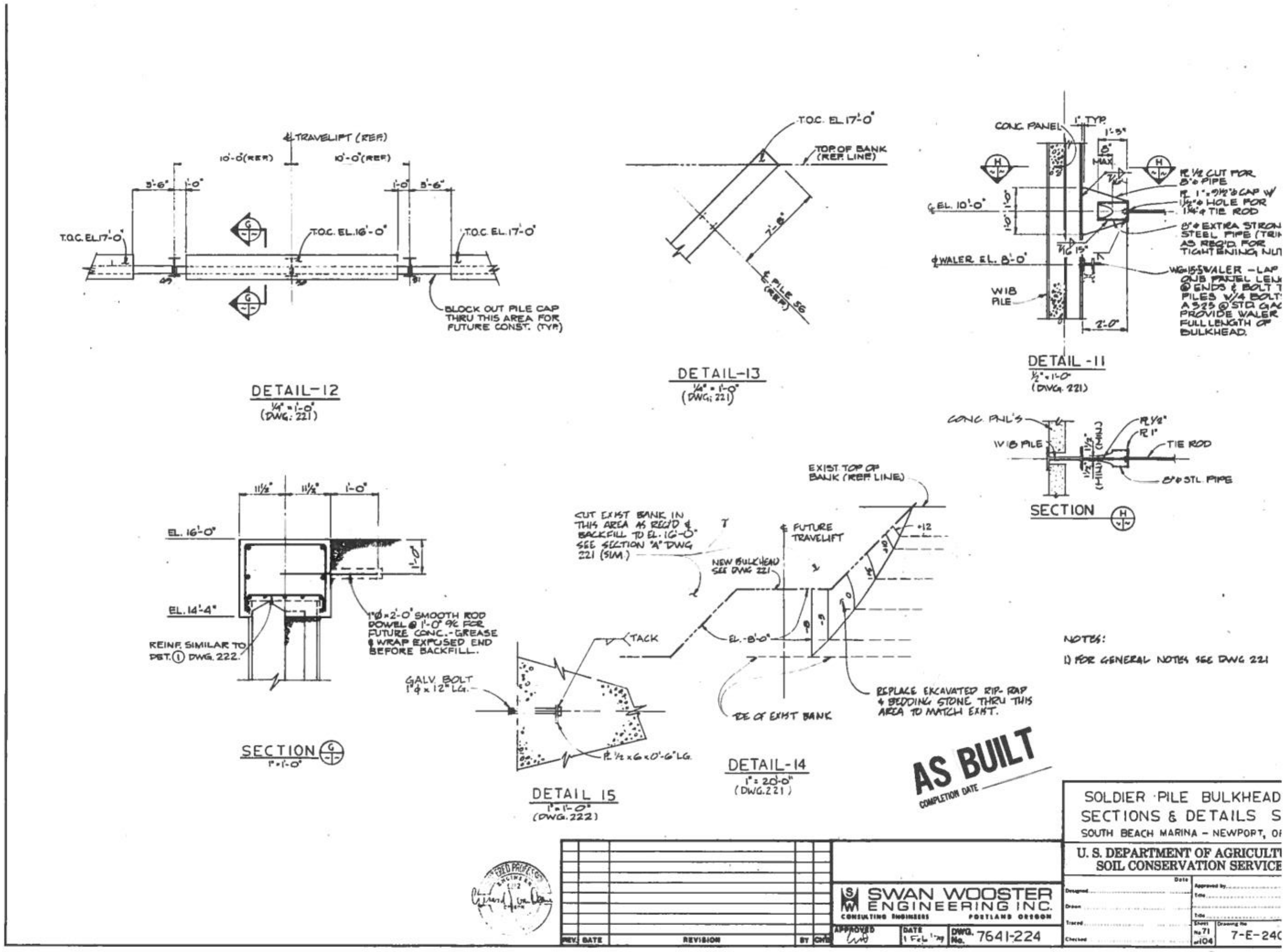


NOTES:

1.) FOR GENERAL NOTES SEE DWG 221.

2.) THE BOAT HOIST SHALL CONSIST OF A CAPACITY MOTORIZED JIB CRANE AND COMPLYING WITH THE CLEARANCE SHOWN SPREADER FRAME & 2 - 4" WIDE ADJUSTAB SLINGS. THE COMPONENTS SHALL SATIS THE REQUIREMENTS SHOWN IN BOAT HOI DETAIL - 10
THE JIB CRANE SHALL BE YALE SERIES MRJ-FPM (OR EQUAL). THE HOIST SHALL #6261 WITH A SINGLE LIFT SPEED (OR EQ THE CONTROLS SHALL HAVE A WEATHER PLUG IN AT THE CRANE TO ALLOW FOR THE REMOVAL. ALL EQUIPMENT SHALL BE PRO SHROUDED TO PROTECT IT FROM A MARI ENVIRONMENT. THE CRANE HOIST AND SP SHALL BE FINISH PAINTED ACCORDING CONTRACT SPECIFICATIONS.

3.) PROVIDE 1" MIN. GROUT UNDER HOIST E PL'S AT INSTALLATION.



NOTES:

1) FOR GENERAL NOTES SEE DWG 221

Structural Evaluation Report
Port of Newport Rogue Brewery Seawall
Newport, Oregon

Appendix C

American Society of Civil Engineers (ASCE)

Waterfront Facilities Inspection and Assessment: Section A.5: Seawalls and Revetments

A.5 SEAWALLS AND REVETMENTS

A.5.1 General

Seawalls and revetments function as barriers against the sea to prevent erosion of land area or damage to structures (Fig. A-18). Typically, this type of structure needs to be substantial to resist wind, wave, and ice forces. The outside shape of seawalls varies and can be designed to reflect or redirect the energy of the waves away from the shoreline. Revetments are protected slopes typically consisting of riprap or gabions (rock-filled wire baskets).

Types of structures used to build seawalls include gravity retaining walls, cantilever retaining walls, and pile-supported retaining walls. Many seawalls have a sheet pile cutoff wall incorporated into their foundations to prevent undermining and to maintain stability. The design also accounts for overtopping of waves and the associated drainage issues to allow water to drain back to the sea without causing damage to the structure. Many seawalls incorporate several types of construction such as a combination of a gravity retaining wall and armor stone at the toe.

The most common material used to build seawalls is concrete. In the past, stone was used extensively due to its durability. Stone is also used at the toe of many seawalls to prevent scour and dissipate wave energy. Alternatives to armor stone are often precast concrete shapes that are placed at the toe of a seawall.

Fig. A-18. Mass concrete seawall
Source: Courtesy of Childs Engineering Corp., reproduced with permission.

A.5.2 Typical Components and Problem Areas

The inspection of seawalls and revetments should be performed using a method similar to that of the inspection of retaining walls and bulkheads, by inspecting as much of the structure as possible during the above water inspection at low tide and performing an underwater inspection of the remainder. Make a general observation of the wall for misalignment of the overall structure and plumbness of individual elements making up the bulkhead or wall system. Note differential settlement between elements and displacement or severe damage by vessel impact or other means. The general observation of the wall should include an observation of the fill behind the wall, noting any signs of loss of fill such as depressions or sinkholes. Perform a general inspection of the revetment slope for alignment, signs of settlement or instability (slip failures), areas missing the protection layer, and signs of erosion at the toe of the slope. Where gabions are used, note the general condition of the wire baskets. The baskets are susceptible to corrosion and abrasion, potentially causing unraveling of the revetment. Table A-7 summarizes what to look for when inspecting the condition of these structures.

A.5.2.1 Access Many seawalls are located in very exposed locations, subject to significant wind, current, and wave action. Underwater inspection of these structures can be extremely hazardous, requiring specialized diving techniques.

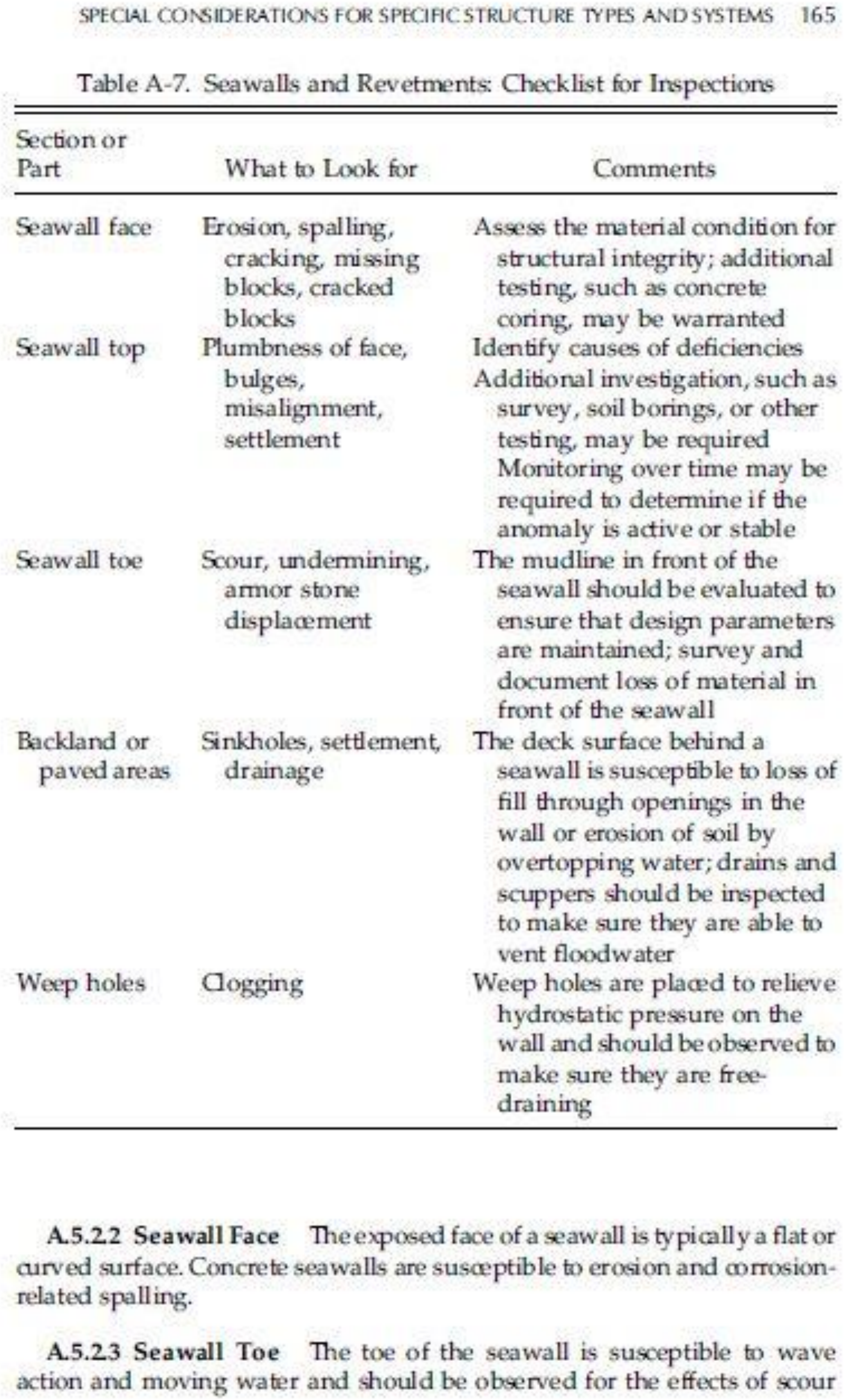
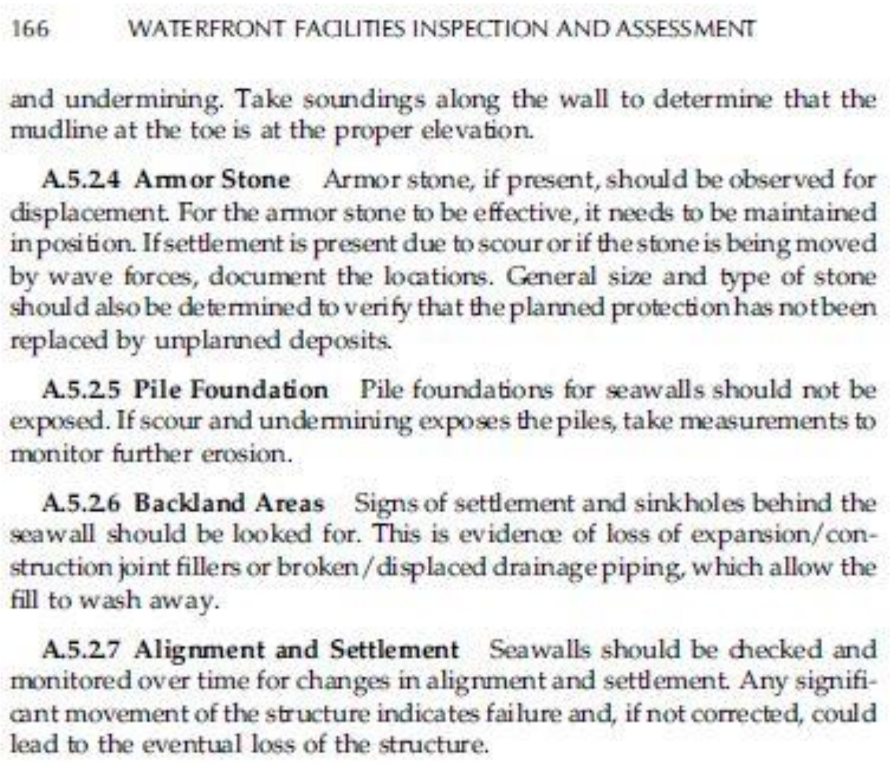


Table A-7. Seawalls and Revetments: Checklist for Inspections

Section or Part	What to Look for	Comments
Seawall face	Erosion, spalling, cracking, missing blocks, cracked blocks	Assess the material condition for structural integrity; additional testing, such as concrete coring, may be warranted
Seawall top	Plumbness of face, bulges, misalignment, settlement	Identify causes of deficiencies Additional investigation, such as survey, soil borings, or other testing, may be required Monitoring over time may be required to determine if the anomaly is active or stable
Seawall toe	Scour, undermining, armor stone displacement	The mudline in front of the seawall should be evaluated to ensure that design parameters are maintained; survey and document loss of material in front of the seawall
Backland or paved areas	Sinkholes, settlement, drainage	The deck surface behind a seawall is susceptible to loss of fill through openings in the wall or erosion of soil by overtopping water; drains and scuppers should be inspected to make sure they are able to vent floodwater
Weep holes	Clogging	Weep holes are placed to relieve hydrostatic pressure on the wall and should be observed to make sure they are free-draining

A.5.2.2 Seawall Face The exposed face of a seawall is typically a flat or curved surface. Concrete seawalls are susceptible to erosion and corrosion-related spalling.

A.5.2.3 Seawall Toe The toe of the seawall is susceptible to wave action and moving water and should be observed for the effects of scour



and undermining. Take soundings along the wall to determine that the mudline at the toe is at the proper elevation.

A.5.2.4 Armor Stone Armor stone, if present, should be observed for displacement. For the armor stone to be effective, it needs to be maintained in position. If settlement is present due to scour or if the stone is being moved by wave forces, document the locations. General size and type of stone should also be determined to verify that the planned protection has not been replaced by unplanned deposits.

A.5.2.5 Pile Foundation Pile foundations for seawalls should not be exposed. If scour and undermining exposes the piles, take measurements to monitor further erosion.

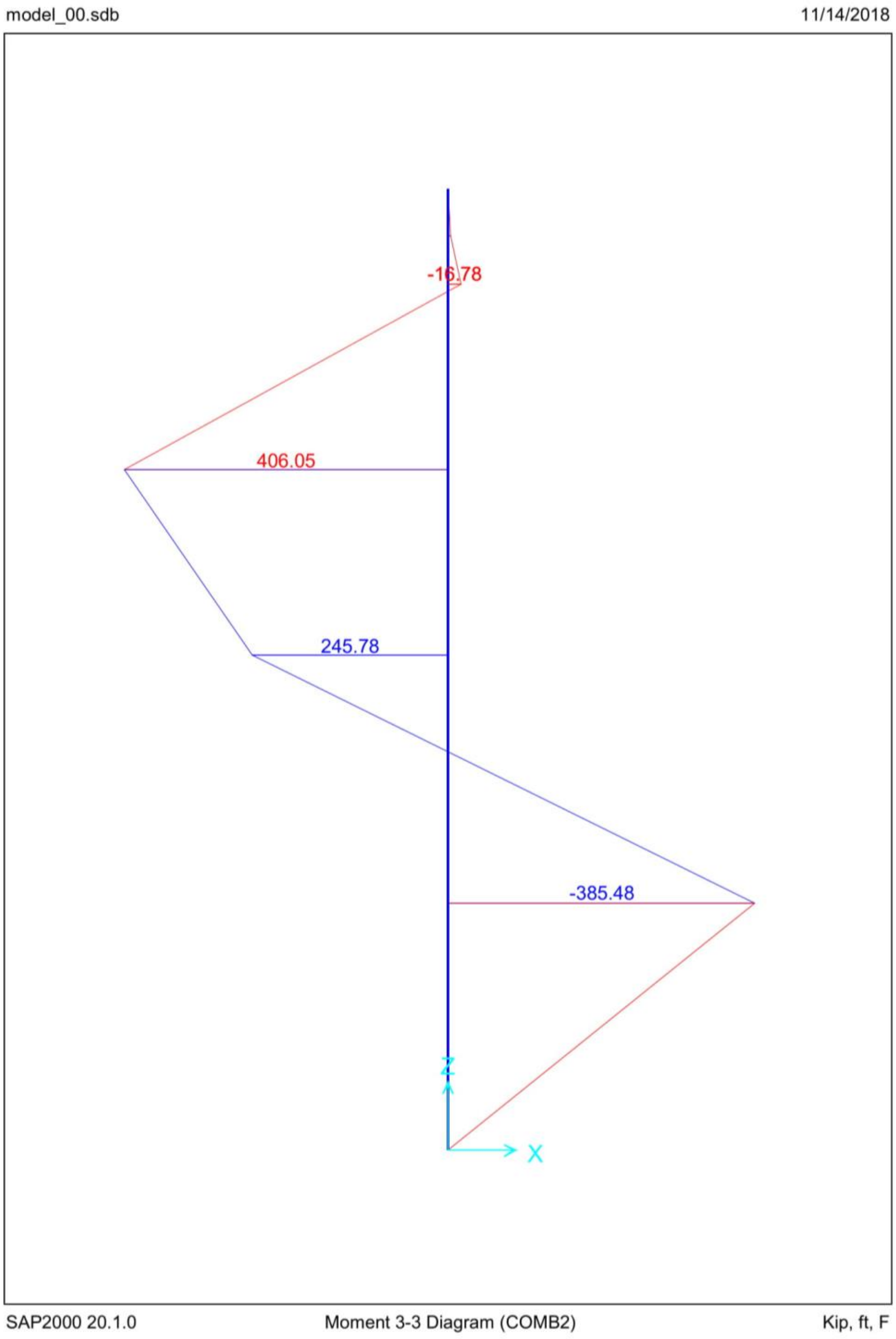
A.5.2.6 Backland Areas Signs of settlement and sinkholes behind the seawall should be looked for. This is evidence of loss of expansion/construction joint fillers or broken/displaced drainage piping, which allow the fill to wash away.

A.5.2.7 Alignment and Settlement Seawalls should be checked and monitored over time for changes in alignment and settlement. Any significant movement of the structure indicates failure and, if not corrected, could lead to the eventual loss of the structure.

Structural Evaluation Report
Port of Newport Rogue Brewery Seawall
Newport, Oregon

Appendix D
SOLDIER PILE WALL MODELING RESULTS

Assuming the backfill active and passive pressures and a 3-foot unbalanced water level behind the wall, the following moment diagram is obtained for the soldier pile. The maximum moment for this diagram is used for checking the initial and existing pile and tie-back.



Appendix B

Excerpt of Soil / Water Chemical Analysis

Stantec / Apex Laboratories
April 2, 2020

Apex Laboratories, LLC

6700 S.W. Sandburg Street
Tigard, OR 97223
503-718-2323
EPA ID: OR01039

Thursday, April 2, 2020

Graeme Taylor
Stantec Portland
601 SW 2nd Ave Suite 1400
Portland, OR 97204

RE: A0C0717 - Rogue Brewery - 185750579

Thank you for using Apex Laboratories. We greatly appreciate your business and strive to provide the highest quality services to the environmental industry.

Enclosed are the results of analyses for work order A0C0717, which was received by the laboratory on 3/19/2020 at 3:05:00PM.

If you have any questions concerning this report or the services we offer, please feel free to contact me by email at: ldomenighini@apex-labs.com, or by phone at 503-718-2323.

Please note: All samples will be disposed of within 30 days of sample reciept, unless prior arrangements have been made.

Cooler Receipt Information

(See Cooler Receipt Form for details)

Cooler#1 3.6 degC

This Final Report is the official version of the data results for this sample submission, unless superseded by a subsequent, labeled amended report.
All other deliverables derived from this data, including Electronic Data Deliverables (EDDs), CLP-like forms, client requested summary sheets, and all other products are considered secondary to this report.

Lisa A Domenighini

Stantec Portland	Project:	**Rogue Brewery**	
601 SW 2nd Ave Suite 1400	Project Number:	**185750579**	**Report ID:**
Portland, OR 97204	Project Manager:	**Graeme Taylor**	**A0C0717 - 04 02 20 0852**

ANALYTICAL REPORT FOR SAMPLES

SAMPLE INFORMATION

Client Sample ID	Laboratory ID	Matrix	Date Sampled	Date Received
GP01-0-10	**A0C0717-01**	**Soil**	**03/18/20 09:30**	**03/19/20 15:05**
GP02-0-10	**A0C0717-02**	**Soil**	**03/17/20 11:10**	**03/19/20 15:05**
GP03-0-10	**A0C0717-03**	**Soil**	**03/17/20 10:05**	**03/19/20 15:05**
GP04-0-10	**A0C0717-04**	**Soil**	**03/17/20 13:10**	**03/19/20 15:05**
GP0XC-0-10	**A0C0717-05**	**Soil**	**03/17/20 10:30**	**03/19/20 15:05**
EB01-031720	**A0C0717-06**	**Water**	**03/17/20 17:00**	**03/19/20 15:05**
EB02-031820	**A0C0717-07**	**Water**	**03/18/20 14:30**	**03/19/20 15:05**
TB01-031720	**A0C0717-08**	**Water**	**03/17/20 00:00**	**03/19/20 15:05**

Lisa A Domenighini

Stantec Portland
601 SW 2nd Ave Suite 1400
Portland, OR 97204

Project: **Rogue Brewery**
Project Number: **185750579**
Project Manager: **Graeme Taylor**

Report ID:
A0C0717 - 04 02 20 0852

ANALYTICAL SAMPLE RESULTS

Anions by Ion Chromatography

Analyte	Sample Result	Detection Limit	Reporting Limit	Units	Dilution	Date Analyzed	Method Ref.	Notes
GP01-0-10 (A0C0717-01)				**Matrix: Soil**				
Batch: 0030739								
Chloride	ND	---	10.3	mg/kg dry	1	03/20/20 14:32	EPA 9056A	
Sulfate	**11.0**	---	10.3	mg/kg dry	1	03/20/20 14:32	EPA 9056A	
GP02-0-10 (A0C0717-02)				**Matrix: Soil**				
Batch: 0030739								
Chloride	ND	---	10.2	mg/kg dry	1	03/20/20 15:37	EPA 9056A	
Sulfate	ND	---	10.2	mg/kg dry	1	03/20/20 15:37	EPA 9056A	
GP03-0-10 (A0C0717-03)				**Matrix: Soil**				
Batch: 0030739								
Chloride	ND	---	10.4	mg/kg dry	1	03/20/20 15:58	EPA 9056A	
Sulfate	ND	---	10.4	mg/kg dry	1	03/20/20 15:58	EPA 9056A	
GP04-0-10 (A0C0717-04)				**Matrix: Soil**				
Batch: 0030739								
Chloride	**12.1**	---	10.8	mg/kg dry	1	03/20/20 16:20	EPA 9056A	
Sulfate	**15.6**	---	10.8	mg/kg dry	1	03/20/20 16:20	EPA 9056A	
GP0XC-0-10 (A0C0717-05)				**Matrix: Soil**				
Batch: 0030739								
Chloride	ND	---	10.3	mg/kg dry	1	03/20/20 16:42	EPA 9056A	
Sulfate	ND	---	10.3	mg/kg dry	1	03/20/20 16:42	EPA 9056A	

Lisa A Domenighini

Stantec Portland	Project: **Rogue Brewery**	
601 SW 2nd Ave Suite 1400	Project Number: **185750579**	**Report ID:**
Portland, OR 97204	Project Manager: **Graeme Taylor**	**A0C0717 - 04 02 20 0852**

ANALYTICAL SAMPLE RESULTS

Conventional Chemistry Parameters

Analyte	Sample Result	Detection Limit	Reporting Limit	Units	Dilution	Date Analyzed	Method Ref.	Notes
GP01-0-10 (A0C0717-01)				**Matrix: Soil**				
Batch: 0030737								
Soil pH (measured in H2O)	**8.81**	---		pH Units	1	03/20/20 11:38	EPA 9045D	**pH_S**
pH Temperature (deg C)	**22.7**	---		pH Units	1	03/20/20 11:38	EPA 9045D	**pH_S**
GP02-0-10 (A0C0717-02)				**Matrix: Soil**				
Batch: 0030737								
Soil pH (measured in H2O)	**9.01**	---		pH Units	1	03/20/20 11:40	EPA 9045D	**pH_S**
pH Temperature (deg C)	**22.5**	---		pH Units	1	03/20/20 11:40	EPA 9045D	**pH_S**
GP03-0-10 (A0C0717-03)				**Matrix: Soil**				
Batch: 0030737								
Soil pH (measured in H2O)	**8.98**	---		pH Units	1	03/20/20 11:41	EPA 9045D	**pH_S**
pH Temperature (deg C)	**22.5**	---		pH Units	1	03/20/20 11:41	EPA 9045D	**pH_S**
GP04-0-10 (A0C0717-04)				**Matrix: Soil**				
Batch: 0030737								
Soil pH (measured in H2O)	**8.30**	---		pH Units	1	03/20/20 11:42	EPA 9045D	**pH_S**
pH Temperature (deg C)	**22.5**	---		pH Units	1	03/20/20 11:42	EPA 9045D	**pH_S**
GP0XC-0-10 (A0C0717-05)				**Matrix: Soil**				
Batch: 0030737								
Soil pH (measured in H2O)	**8.99**	---		pH Units	1	03/20/20 11:43	EPA 9045D	**pH_S**
pH Temperature (deg C)	**22.4**	---		pH Units	1	03/20/20 11:43	EPA 9045D	**pH_S**

QUALITY CONTROL (QC) SAMPLE RESULTS

Anions by Ion Chromatography

Analyte	Result	Detection Limit	Reporting Limit	Units	Dilution	Spike Amount	Source Result	% REC	% REC Limits	RPD	RPD Limit	Notes
Batch 0030739 - DI Leach							**Soil**					
Blank (0030739-BLK1)		Prepared: 03/20/20 09:51	Analyzed: 03/20/20 13:49									
EPA 9056A												
Chloride	ND	---	10.0	mg/kg wet	1	---	---	---	---	---	---	
Sulfate	ND	---	10.0	mg/kg wet	1	---	---	---	---	---	---	
LCS (0030739-BS1)		Prepared: 03/20/20 09:51	Analyzed: 03/20/20 14:11									
EPA 9056A												
Chloride	78.8	---	10.0	mg/kg wet	1	80.0	---	99	90 - 110%	---	---	
Sulfate	80.3	---	10.0	mg/kg wet	1	80.0	---	100	90 - 110%	---	---	
Duplicate (0030739-DUP1)		Prepared: 03/20/20 09:51	Analyzed: 03/20/20 14:54									
QC Source Sample: GP01-0-10 (A0C0717-01)												
EPA 9056A												
Chloride	ND	---	10.2	mg/kg dry	1	---	ND	---	---	---	15%	
Sulfate	**11.5**	---	10.2	mg/kg dry	1	---	11.0	---	---	4	15%	
Matrix Spike (0030739-MS1)		Prepared: 03/20/20 09:51	Analyzed: 03/20/20 15:15									
QC Source Sample: GP01-0-10 (A0C0717-01)												
EPA 9056A												
Chloride	88.4	---	10.7	mg/kg dry	1	86.0	ND	103	80 - 120%	---	---	
Sulfate	98.0	---	10.7	mg/kg dry	1	86.0	11.0	101	80 - 120%	---	---	

Stantec Portland	Project:	**Rogue Brewery**	
601 SW 2nd Ave Suite 1400	Project Number:	**185750579**	**Report ID:**
Portland, OR 97204	Project Manager:	**Graeme Taylor**	**A0C0717 - 04 02 20 0852**

QUALITY CONTROL (QC) SAMPLE RESULTS

Conventional Chemistry Parameters

Analyte	Result	Detection Limit	Reporting Limit	Units	Dilution	Spike Amount	Source Result	% REC	% REC Limits	RPD	RPD Limit	Notes
Batch 0030737 - DI Leach							**Soil**					
Duplicate (0030737-DUP1)		Prepared: 03/20/20 09:36 Analyzed: 03/20/20 11:39										
QC Source Sample: GP01-0-10 (A0C0717-01)												
EPA 9045D												
Soil pH (measured in H2O)	**8.90**	---		pH Units	1	---	8.81	---	---	1	5%	pH_S
pH Temperature (deg C)	**22.5**	---		pH Units	1	---	22.7	---	---	0.9	30%	pH_S
Reference (0030737-SRM1)		Prepared: 03/20/20 09:36 Analyzed: 03/20/20 11:36										
EPA 9045D												
Soil pH (measured in H2O)	6.03	---		pH Units	1	6.00		100	98.33333 - 101.6667%	---	---	
pH Temperature (deg C)	21.8	---		pH Units	1	20.0		109	50 - 200%	---	---	
Reference (0030737-SRM2)		Prepared: 03/20/20 09:36 Analyzed: 03/20/20 11:44										
EPA 9045D												
Soil pH (measured in H2O)	7.98	---		pH Units	1	8.00		100	98.75 - 101.25%	---	---	
pH Temperature (deg C)	21.9	---		pH Units	1	20.0		110	50 - 200%	---	---	

Stantec Portland	Project: **Rogue Brewery**	
601 SW 2nd Ave Suite 1400	Project Number: **185750579**	**Report ID:**
Portland, OR 97204	Project Manager: **Graeme Taylor**	**A0C0717 - 04 02 20 0852**

QUALITY CONTROL (QC) SAMPLE RESULTS

Percent Dry Weight

Analyte	Result	Detection Limit	Reporting Limit	Units	Dilution	Spike Amount	Source Result	% REC	% REC Limits	RPD	RPD Limit	Notes
Batch 0030740 - Total Solids (Dry Weight)							**Soil**					
No Client related Batch QC samples analyzed for this batch. See notes page for more information.												

SAMPLE PREPARATION INFORMATION

Hydrocarbon Identification Screen by NWTPH-HCID

Prep: EPA 3510C (Fuels/Acid Ext.)

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030801							
A0C0717-06	Water	NWTPH-HCID	03/17/20 17:00	03/23/20 12:54	880mL/5mL	1000mL/5mL	1.14
A0C0717-07	Water	NWTPH-HCID	03/18/20 14:30	03/23/20 12:54	1020mL/5mL	1000mL/5mL	0.98

Prep: NWTPH-HCID (Soil)

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030775							
A0C0717-01	Soil	NWTPH-HCID	03/18/20 09:30	03/23/20 12:57	10.87g/10mL	10g/10mL	0.92
A0C0717-02	Soil	NWTPH-HCID	03/17/20 11:10	03/23/20 12:57	10.21g/10mL	10g/10mL	0.98
A0C0717-03	Soil	NWTPH-HCID	03/17/20 10:05	03/23/20 12:57	10.43g/10mL	10g/10mL	0.96
A0C0717-04	Soil	NWTPH-HCID	03/17/20 13:10	03/23/20 12:57	10.3g/10mL	10g/10mL	0.97
A0C0717-05	Soil	NWTPH-HCID	03/17/20 10:30	03/23/20 12:57	10.51g/10mL	10g/10mL	0.95

Diesel and/or Oil Hydrocarbons by NWTPH-Dx

Prep: EPA 3546 (Fuels)

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030823							
A0C0717-02	Soil	NWTPH-Dx	03/17/20 11:10	03/24/20 13:04	10.67g/5mL	10g/5mL	0.94
A0C0717-04	Soil	NWTPH-Dx	03/17/20 13:10	03/24/20 13:04	10.46g/5mL	10g/5mL	0.96

Volatile Organic Compounds by EPA 8260C

Prep: EPA 5030B

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030828							
A0C0717-08	Water	EPA 8260C	03/17/20 00:00	03/24/20 09:43	5mL/5mL	5mL/5mL	1.00

Polychlorinated Biphenyls by EPA 8082A

Prep: EPA 3510C (Neutral pH)

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030749							
A0C0717-06	Water	EPA 8082A	03/17/20 17:00	03/20/20 12:23	830mL/5mL	1000mL/5mL	1.20
A0C0717-07	Water	EPA 8082A	03/18/20 14:30	03/20/20 12:23	880mL/5mL	1000mL/5mL	1.14

SAMPLE PREPARATION INFORMATION

Polychlorinated Biphenyls by EPA 8082A

Prep: EPA 3546

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030757							
A0C0717-01	Soil	EPA 8082A	03/18/20 09:30	03/20/20 12:30	10.16g/5mL	10g/5mL	0.98
A0C0717-02	Soil	EPA 8082A	03/17/20 11:10	03/20/20 12:30	10.78g/5mL	10g/5mL	0.93
A0C0717-03	Soil	EPA 8082A	03/17/20 10:05	03/20/20 12:30	10.78g/5mL	10g/5mL	0.93
A0C0717-04	Soil	EPA 8082A	03/17/20 13:10	03/20/20 12:30	10.18g/5mL	10g/5mL	0.98
A0C0717-05	Soil	EPA 8082A	03/17/20 10:30	03/20/20 12:30	10.68g/5mL	10g/5mL	0.94

Organochlorine Pesticides by EPA 8081B

Prep: EPA 3510C (Neutral pH)

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030826							
A0C0717-06	Water	EPA 8081B	03/17/20 17:00	03/24/20 07:16	1000mL/5mL	1000mL/5mL	1.00
A0C0717-07	Water	EPA 8081B	03/18/20 14:30	03/24/20 07:16	930mL/5mL	1000mL/5mL	1.08

Prep: EPA 3546/3640A (GPC)

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030762							
A0C0717-01RE1	Soil	EPA 8081B	03/18/20 09:30	03/20/20 07:18	10.44g/10mL	10g/5mL	1.92
A0C0717-02RE1	Soil	EPA 8081B	03/17/20 11:10	03/20/20 07:18	10.45g/10mL	10g/5mL	1.91
A0C0717-03RE1	Soil	EPA 8081B	03/17/20 10:05	03/20/20 07:18	10.63g/10mL	10g/5mL	1.88
A0C0717-04RE1	Soil	EPA 8081B	03/17/20 13:10	03/20/20 07:18	10.12g/10mL	10g/5mL	1.98
A0C0717-05RE1	Soil	EPA 8081B	03/17/20 10:30	03/20/20 07:18	10.37g/10mL	10g/5mL	1.93

Polyaromatic Hydrocarbons (PAHs) by EPA 8270D SIM

Prep: EPA 3510C (Acid Extraction)

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030806							
A0C0717-06	Water	EPA 8270D (SIM)	03/17/20 17:00	03/23/20 12:02	1010mL/2mL	1000mL/2mL	0.99
A0C0717-07	Water	EPA 8270D (SIM)	03/18/20 14:30	03/23/20 12:02	970mL/2mL	1000mL/2mL	1.03

Prep: EPA 3546

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030726							
A0C0717-01	Soil	EPA 8270D (SIM)	03/18/20 09:30	03/20/20 07:17	10.67g/5mL	10g/5mL	0.94
A0C0717-02	Soil	EPA 8270D (SIM)	03/17/20 11:10	03/20/20 07:17	10.67g/5mL	10g/5mL	0.94

SAMPLE PREPARATION INFORMATION

Polyaromatic Hydrocarbons (PAHs) by EPA 8270D SIM

Prep: EPA 3546

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
A0C0717-03	Soil	EPA 8270D (SIM)	03/17/20 10:05	03/20/20 07:17	10.47g/5mL	10g/5mL	0.96
A0C0717-04	Soil	EPA 8270D (SIM)	03/17/20 13:10	03/20/20 07:17	10.63g/5mL	10g/5mL	0.94
A0C0717-05	Soil	EPA 8270D (SIM)	03/17/20 10:30	03/20/20 07:17	10.09g/5mL	10g/5mL	0.99

Total Metals by EPA 6020A (ICPMS)

Prep: EPA 3015A

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030853							
A0C0717-06	Water	EPA 6020A	03/17/20 17:00	03/24/20 11:24	45mL/50mL	45mL/50mL	1.00
A0C0717-07	Water	EPA 6020A	03/18/20 14:30	03/24/20 11:24	45mL/50mL	45mL/50mL	1.00

Prep: EPA 3051A

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030741							
A0C0717-01	Soil	EPA 6020A	03/18/20 09:30	03/20/20 10:31	0.468g/50mL	0.5g/50mL	1.07
A0C0717-02	Soil	EPA 6020A	03/17/20 11:10	03/20/20 10:31	0.484g/50mL	0.5g/50mL	1.03
A0C0717-03	Soil	EPA 6020A	03/17/20 10:05	03/20/20 10:31	0.495g/50mL	0.5g/50mL	1.01
A0C0717-04	Soil	EPA 6020A	03/17/20 13:10	03/20/20 10:31	0.487g/50mL	0.5g/50mL	1.03
A0C0717-05	Soil	EPA 6020A	03/17/20 10:30	03/20/20 10:31	0.464g/50mL	0.5g/50mL	1.08

Anions by Ion Chromatography

Prep: DI Leach

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030739							
A0C0717-01	Soil	EPA 9056A	03/18/20 09:30	03/20/20 09:51	5.2145g/50mL	5g/50mL	0.96
A0C0717-02	Soil	EPA 9056A	03/17/20 11:10	03/20/20 09:51	5.2583g/50mL	5g/50mL	0.95
A0C0717-03	Soil	EPA 9056A	03/17/20 10:05	03/20/20 09:51	5.0811g/50mL	5g/50mL	0.98
A0C0717-04	Soil	EPA 9056A	03/17/20 13:10	03/20/20 09:51	5.1238g/50mL	5g/50mL	0.98
A0C0717-05	Soil	EPA 9056A	03/17/20 10:30	03/20/20 09:51	5.1313g/50mL	5g/50mL	0.97

Conventional Chemistry Parameters

Prep: DI Leach

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030737							

SAMPLE PREPARATION INFORMATION

Conventional Chemistry Parameters

Prep: DI Leach

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
A0C0717-01	Soil	EPA 9045D	03/18/20 09:30	03/20/20 09:36	10.0486g/10mL	10g/10mL	NA
A0C0717-02	Soil	EPA 9045D	03/17/20 11:10	03/20/20 09:36	10.1126g/10mL	10g/10mL	NA
A0C0717-03	Soil	EPA 9045D	03/17/20 10:05	03/20/20 09:36	10.3777g/10mL	10g/10mL	NA
A0C0717-04	Soil	EPA 9045D	03/17/20 13:10	03/20/20 09:36	10.2969g/10mL	10g/10mL	NA
A0C0717-05	Soil	EPA 9045D	03/17/20 10:30	03/20/20 09:36	10.2884g/10mL	10g/10mL	NA

Percent Dry Weight

Prep: Total Solids (Dry Weight)

Lab Number	Matrix	Method	Sampled	Prepared	Sample Initial/Final	Default Initial/Final	RL Prep Factor
Batch: 0030740							
A0C0717-01	Soil	EPA 8000C	03/18/20 09:30	03/20/20 10:14			NA
A0C0717-02	Soil	EPA 8000C	03/17/20 11:10	03/20/20 10:14			NA
A0C0717-03	Soil	EPA 8000C	03/17/20 10:05	03/20/20 10:14			NA
A0C0717-04	Soil	EPA 8000C	03/17/20 13:10	03/20/20 10:14			NA
A0C0717-05	Soil	EPA 8000C	03/17/20 10:30	03/20/20 10:14			NA

Stantec Portland
601 SW 2nd Ave Suite 1400
Portland, OR 97204

Project: **Rogue Brewery**
Project Number: **185750579**
Project Manager: **Graeme Taylor**

Report ID:
A0C0717 - 04 02 20 0852

QUALIFIER DEFINITIONS

Client Sample and Quality Control (QC) Sample Qualifier Definitions:

Apex Laboratories

C-05	Extract has undergone a GPC (Gel-Permeation Chromatography) cleanup per EPA 3640A. Reporting levels may be raised due to dilution necessary for cleanup. Sample Final Volume includes the GPC dilution factor, see the Prep page for details.
C-07	Extract has undergone Sulfuric Acid Cleanup by EPA 3665A, Sulfur Cleanup by EPA 3660B, and Florisil Cleanup by EPA 3620B in order to minimize matrix interference.
pH_S	Method recommends preparation 'as soon as possible'. See Sample Preparation Information section of report for details. Consult regulator or permit manager to determine the usability of data for intended purpose.
Q-05	Analyses are not controlled on RPD values from sample and duplicate concentrations that are below 5 times the reporting level.
Q-19	Blank Spike Duplicate (BSD) sample analyzed in place of Matrix Spike/Duplicate samples due to limited sample amount available for analysis.
Q-41	Estimated Results. Recovery of Continuing Calibration Verification sample above upper control limit for this analyte. Results are likely biased high.
Q-42	Matrix Spike and/or Duplicate analysis was performed on this sample. % Recovery or RPD for this analyte is outside laboratory control limits. (Refer to the QC Section of Analytical Report.)
R-02	The Reporting Limit for this analyte has been raised to account for interference from coeluting organic compounds present in the sample.

Stantec Portland **601 SW 2nd Ave Suite 1400** **Portland, OR 97204**	Project: **Rogue Brewery** Project Number: **185750579** Project Manager: **Graeme Taylor**	**Report ID:** **A0C0717 - 04 02 20 0852**

REPORTING NOTES AND CONVENTIONS:

Abbreviations:

DET	Analyte DETECTED at or above the detection or reporting limit.
ND	Analyte NOT DETECTED at or above the detection or reporting limit.
NR	Result Not Reported.
RPD	Relative Percent Difference. RPDs for Matrix Spikes and Matrix Spike Duplicates are based on concentration, not recovery.

Detection Limits: Limit of Detection (LOD)

Limits of Detection (LODs) are normally set at a level of one half the validated Limit of Quantitation (LOQ).
If no value is listed ('-----'), then the data has not been evaluated below the Reporting Limit.

Reporting Limits: Limit of Quantitation (LOQ)

Validated Limits of Quantitation (LOQs) are reported as the Reporting Limits for all analyses where the LOQ, MRL, PQL or CRL are requested. The LOQ represents a level at or above the low point of the calibration curve, that has been validated according to Apex Laboratories' comprehensive LOQ policies and procedures.

Reporting Conventions:

Basis: Results for soil samples are generally reported on a 100% dry weight basis.
The Result Basis is listed following the units as " dry", " wet", or " " (blank) designation.

" dry"	Sample results and Reporting Limits are reported on a dry weight basis. (i.e. "ug/kg dry") See Percent Solids section for details of dry weight analysis.
" wet"	Sample results and Reporting Limits for this analysis are normally dry weight corrected, but have not been modified in this case.
" "	Results without 'wet' or 'dry' designation are not normally dry weight corrected. These results are considered 'As Received'.

QC Source:

In cases where there is insufficient sample provided for Sample Duplicates and/or Matrix Spikes, a Lab Control Sample Duplicate (LCS Dup) may be analyzed to demonstrate accuracy and precision of the extraction batch.

Non-Client Batch QC Samples (Duplicates and Matrix Spike/Duplicates) are not included in this report. Please request a Full QC report if this data is required.

Miscellaneous Notes:

" --- " QC results are not applicable. For example, % Recoveries for Blanks and Duplicates, % RPD for Blanks, Blank Spikes and Matrix Spikes, etc.

" *** " Used to indicate a possible discrepancy with the Sample and Sample Duplicate results when the %RPD is not available. In this case, either the Sample or the Sample Duplicate has a reportable result for this analyte, while the other is Non Detect (ND).

Blanks:

Standard practice is to evaluate the results from Blank QC Samples down to a level equal to ½ the Reporting Limit (RL).
-For Blank hits falling between ½ the RL and the RL (J flagged hits), the associated sample and QC data will receive a 'B-02' qualifier.
-For Blank hits above the RL, the associated sample and QC data will receive a 'B' qualifier, per Apex Laboratories' Blank Policy.
For further details, please request a copy of this document.

REPORTING NOTES AND CONVENTIONS (Cont.):

Blanks (Cont.):

Sample results flagged with a 'B' or 'B-02' qualifier are potentially biased high if the sample results are less than ten times the level found in the blank for inorganic analyses, or less than five times the level found in the blank for organic analyses.

‘B’ and ‘B-02’ qualifications are only applied to sample results detected above the Reporting Level.

Preparation Notes:

Mixed Matrix Samples:

Water Samples:

Water samples containing significant amounts of sediment are decanted or separated prior to extraction, and only the water portion analyzed, unless otherwise directed by the client.

Soil and Sediment Samples:

Soil and Sediment samples containing significant amounts of water are decanted prior to extraction, and only the solid portion analyzed, unless otherwise directed by the client.

Sampling and Preservation Notes:

Certain regulatory programs, such as National Pollutant Discharge Elimination System (NPDES), require that activities such as sample filtration (for dissolved metals, orthophosphate, hexavalent chromium, etc.) and testing of short hold analytes (pH, Dissolved Oxygen, etc.) be performed in the field (on-site) within a short time window. In addition, sample matrix spikes are required for some analyses, and sufficient volume must be provided, and billable site specific QC requested, if this is required. All regulatory permits should be reviewed to ensure that these requirements are being met.

Data users should be aware of which regulations pertain to the samples they submit for testing. If related sample collection activities are not approved for a particular regulatory program, results should be considered estimates. Apex Laboratories will qualify these analytes according to the most stringent requirements, however results for samples that are for non-regulatory purposes may be acceptable.

Samples that have been filtered and preserved at Apex Laboratories per client request are listed in the preparation section of the report with the date and time of filtration listed.

Apex Laboratories maintains detailed records on sample receipt, including client label verification, cooler temperature, sample preservation, hold time compliance and field filtration. Data is qualified as necessary, and the lack of qualification indicates compliance with required parameters.

Apex Laboratories, LLC

6700 S.W. Sandburg Street
Tigard, OR 97223
503-718-2323
EPA ID: OR01039

LABORATORY ACCREDITATION INFORMATION

TNI Certification ID: OR100062 (Primary Accreditation) - EPA ID: OR01039

All methods and analytes reported from work performed at Apex Laboratories are included on Apex Laboratories' ORELAP Scope of Certification, with the exception of any analyte(s) listed below:

Apex Laboratories

Matrix	Analysis	TNI_ID	Analyte	TNI_ID	Accreditation

All reported analytes are included in Apex Laboratories' current ORELAP scope.

Secondary Accreditations

Apex Laboratories also maintains reciprocal accreditation with non-TNI states (Washington DOE), as well as other state specific accreditations not listed here.

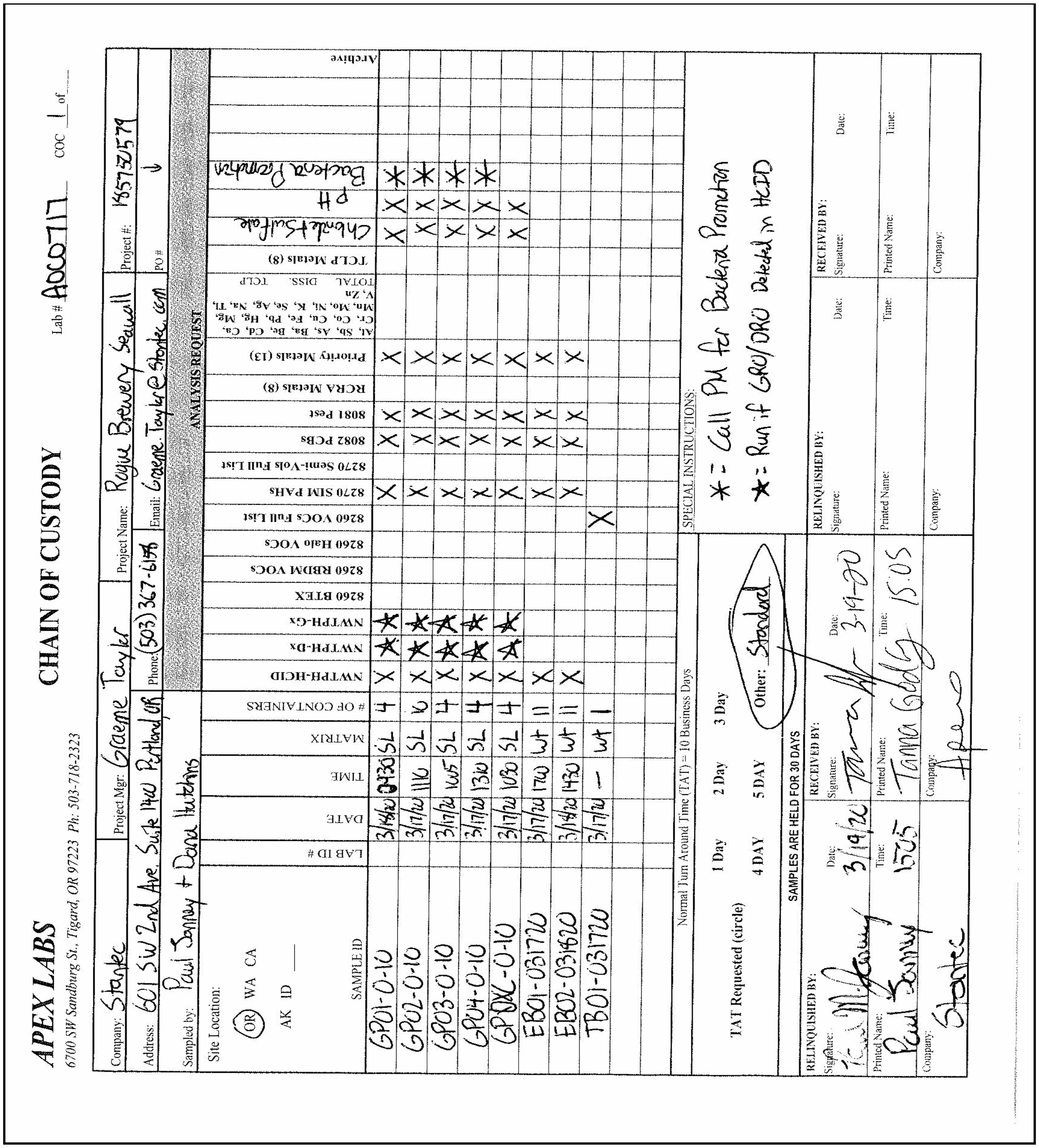
Subcontract Laboratory Accreditations

Subcontracted data falls outside of Apex Laboratories' Scope of Accreditation.
Please see the Subcontract Laboratory report for full details, or contact your Project Manager for more information.

Field Testing Parameters

Results for Field Tested data are provded by the client or sampler, and fall outside of Apex Laboratories' Scope of Accreditation.

Stantec Portland	Project: **Rogue Brewery**	
601 SW 2nd Ave Suite 1400	Project Number: **185750579**	**Report ID:**
Portland, OR 97204	Project Manager: **Graeme Taylor**	**A0C0717 - 04 02 20 0852**



APEX LABS

6700 SW Sandburg St., Tigard, OR 97223 Ph: 503-718-2323

CHAIN OF CUSTODY

Lab # A0C0717 COC 1 of ___

Company: Stantec
Project Mgr: Graeme Taylor
Project Name: Rogue Brewery Seawall
Project #: 185750579
Address: 601 SW 2nd Ave. Suite 1400 Portland OR
Phone: (503) 367-6156
Email: Graeme.Taylor@Stantec.com
PO # ↓
Sampled by: Paul Sonney + Dana Hutchins
Site Location: [✓ OR] WA CA AK ID ___

ANALYSIS REQUEST

SAMPLE ID	LAB ID #	DATE	TIME	MATRIX	# OF CONTAINERS	NWTPH-HCID	NWTPH-Dx	NWTPH-Gx	8260 BTEX	8260 RBDM VOCs	8260 Halo VOCs	8260 VOCs Full List	8270 SIM PAHs	8270 Semi-Vols Full List	8082 PCBs	8081 Pest	RCRA Metals (8)	Priority Metals (13)	Al, Sb, As, Ba, Be, Cd, Ca, Cr, Co, Cu, Fe, Pb, Hg, Mg, Mn, Mo, Ni, K, Se, Ag, Na, Tl, V, Zn TOTAL DISS. TCLP	TCLP Metals (8)	Chloride+Sulfate	pH	Bacteria Promotion				Archive
GP01-0-10		3/16/20	0930	SL	4	X	★	★					X		X	X		X			X	X	✶				
GP02-0-10		3/17/20	1110	SL	10	X	★	★					X		X	X		X			X	X	✶				
GP03-0-10		3/17/20	1005	SL	4	X	★	★					X		X	X		X			X	X	✶				
GP04-0-10		3/17/20	1310	SL	4	X	★	★					X		X	X		X			X	X	✶				
GPDUC-0-10		3/17/20	1030	SL	4	X	★	★					X		X	X		X			X	X					
EB01-031720		3/17/20	1700	Wt	11	X							X		X	X		X									
EB02-031620		3/16/20	1430	Wt	11	X							X		X	X		X									
TB01-031720		3/17/20	—	Wt	1							X															

Normal Turn Around Time (TAT) = 10 Business Days

TAT Requested (circle): 1 Day 2 Day 3 Day 4 DAY 5 DAY [✓ Other: Standard]

SAMPLES ARE HELD FOR 30 DAYS

SPECIAL INSTRUCTIONS:
✶ = Call PM for Bacteria Promotion
★ = Run if GRO/DRO Detected in HCID

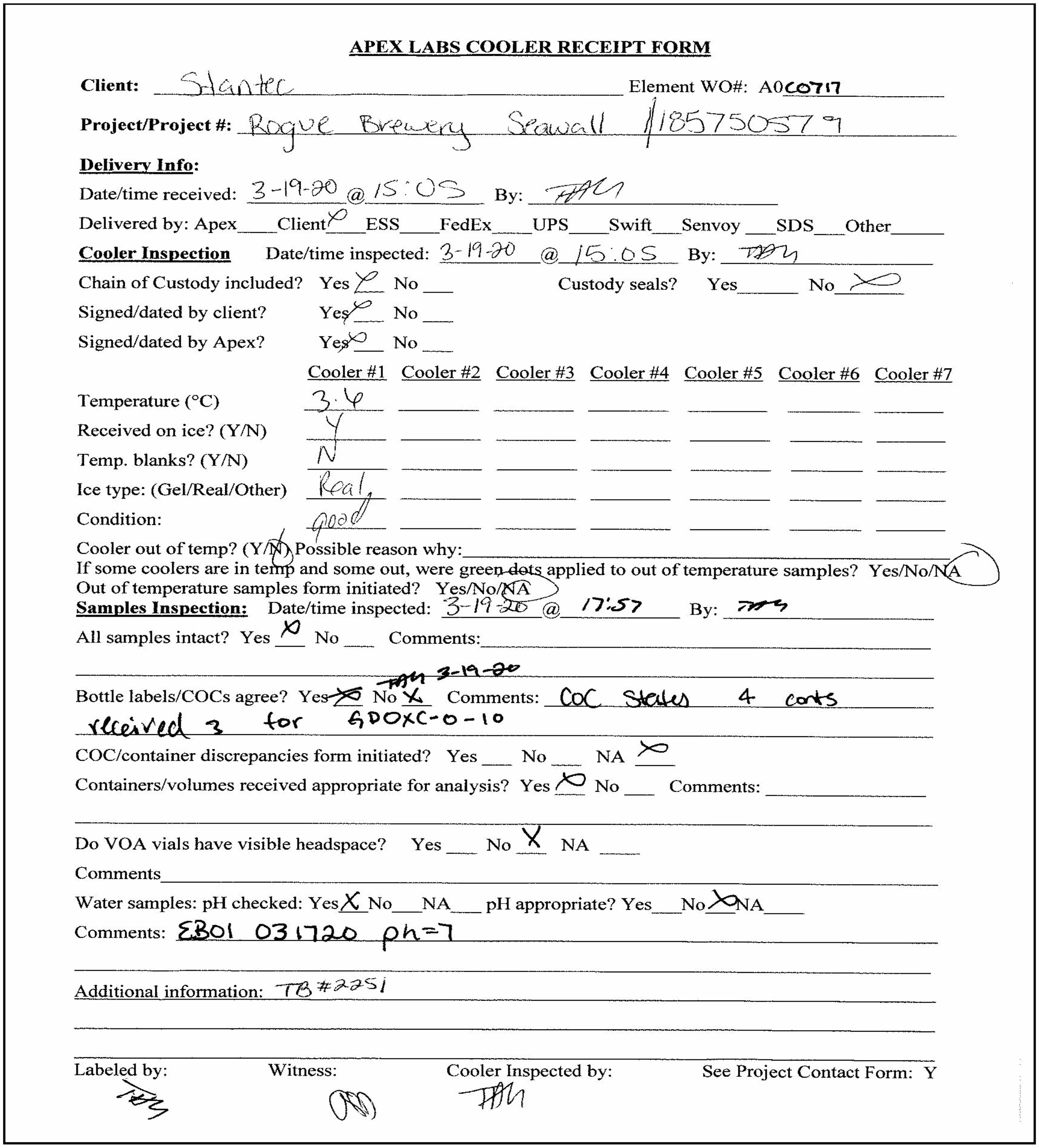
RELINQUISHED BY:	RECEIVED BY:	RELINQUISHED BY:	RECEIVED BY:
Signature: Paul Sonney Date: 3/19/20	Signature: Tanna Date: 3-19-20	Signature: Date:	Signature: Date:
Printed Name: Paul Sonney Time: 1505	Printed Name: Tanna Goddy Time: 15:05	Printed Name: Time:	Printed Name: Time:
Company: Stantec	Company: Apex	Company:	Company:

Apex Laboratories

Lisa A Domenighini

Lisa Domenighini, Client Services Manager

The results in this report apply to the samples analyzed in accordance with the chain of custody document. This analytical report must be reproduced in its entirety.



APEX LABS COOLER RECEIPT FORM

Client: Stantec Element WO#: A0C0717

Project/Project #: Rogue Brewery Seawall /185750579

Delivery Info:

Date/time received: 3-19-20 @ 15:05 By:

Delivered by: Apex____Client X ESS____FedEx____UPS____Swift___Senvoy ___SDS___Other______

Cooler Inspection Date/time inspected: 3-19-20 @ 15:05 By:

Chain of Custody included? Yes X No ___ Custody seals? Yes______ No X

Signed/dated by client? Yes X No ___

Signed/dated by Apex? Yes X No ___

	Cooler #1	Cooler #2	Cooler #3	Cooler #4	Cooler #5	Cooler #6	Cooler #7
Temperature (°C)	3.4						
Received on ice? (Y/N)	Y						
Temp. blanks? (Y/N)	N						
Ice type: (Gel/Real/Other)	Real						
Condition:	good						

Cooler out of temp? (Y/[✓ N]) Possible reason why:________________

If some coolers are in temp and some out, were green dots applied to out of temperature samples? Yes/No/[✓ NA]

Out of temperature samples form initiated? Yes/No/[✓ NA]

Samples Inspection: Date/time inspected: 3-19-20 @ 17:57 By:

All samples intact? Yes X No ___ Comments:________________

3-19-20

Bottle labels/COCs agree? Yes ~~X~~ No X Comments: CoC States 4 conts received 3 for GDOXC-0-10

COC/container discrepancies form initiated? Yes ___ No ___ NA X

Containers/volumes received appropriate for analysis? Yes X No ___ Comments: ________________

Do VOA vials have visible headspace? Yes ___ No X NA ____

Comments________________

Water samples: pH checked: Yes X No___NA___ pH appropriate? Yes___No X NA____

Comments: EB01 031720 ph=7

Additional information: TB #2251

Labeled by: Witness: Cooler Inspected by: See Project Contact Form: Y

Appendix C

Ultrasonic Test Report

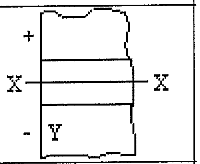
Carlson Testing, Inc.
June 11, 2021

Bend Office (541) 330-9155
Geotechnical Office (503) 601-8250
Eugene Office (541) 345-0289
Salem Office (503) 589-1252
Tigard Office (503) 684-3460

Carlson Testing, Inc.

Ultrasonic Test Report

Client: PBS ENGINEERING & ENVIRONMENTAL - HOWARD WELLS **Date:** 06/11/2021
Project: ROGUE BREWERY SEAWALL **CTI Job #:** S2101564.
Job Address: 2320 SE MARINE SCIENCE DR NEWPORT **PO #:**
Permit (s): X
Fab Shop Inspection At: N/A
Drawing #: PER VERBAL REQUEST **Material:** CARBON STEEL
Test Method Standard: CTI-UT-4 **Acceptance Standard:** AWS D1.1 TABLE 6.2
Ultrasonic Unit SN: 130501703 **Ultrasonic Unit Model:** EPOCH 600
Transducer SN: 578915 **Joint Description:** THICKNESS TEST

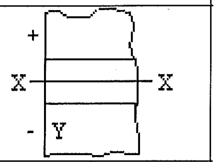
Test No.	Weld Identification	Acceptable	Rejectable	Transducer Angle	Form Face	Leg 1-2-3	Decibels				Defect			Distance	
							Indicatior Level (dB)	Reference Level (dB)	Attenuation Factor (dB)	Indication Rating	Length (in.)	Angular Distance (in.)	Depth from "A" Surface		
	Elevation, Grid & Orientation, Specific Weld						a	b	c	d				From "X"	From "Y"
1	TEST 1	X		.0	A	1		22.5			0.90				
2	TEST 2	X		.0	A	1		22.5			0.85				
3	TEST 3	X		.0	A	1		22.5			0.85				
4	TEST 4	X		.0	A	1		22.5			0.83				
5	TEST 5	X		.0	A	1		22.5			0.84				
6	TEST 6	X		.0	A	1		22.5			0.78				
7	TEST 7	X		.0	A	1		22.5			0.86				
8	TEST 8	X		.0	A	1		22.5			0.90				
9	TEST 9	X		.0	A	1		22.5			0.80				
10	TEST 10	X		.0	A	1		22.5			0.79				
11	TEST 11	X		.0	A	1		22.5			0.88				

Please see reverse side for additional information.

☐ Distribute attachments.

Job Number: S2101564.

Date Of Test: 06/11/2021

Test No.	Weld Identification	Acceptable	Rejectable	Transducer Angle	Form Face	Leg 1-2-3	Decibels				Defect			Distance	
							Indicatior Level (dB)	Reference Level (dB)	Attenuation Factor (dB)	Indication Rating	Length (in.)	Angular Distance (in.)	Depth from "A" Surface		
	Elevation, Grid & Orientation, Specific Weld						a	b	c	d				From "X"	From "Y"
12	TEST 12	X		.0	A	1		22.5			0.90				
13	TEST 13	X		.0	A	1		22.5			0.85				
14	TEST 14	X		.0	A	1		22.5			0.78				
15	TEST 15	X		.0	A	1		22.5			0.78				
16	TEST 16	X		.0	A	1		22.5			0.79				
17	TEST 17	X		.0	A	1		22.5			0.80				
18	TEST 18	X		.0	A	1		22.5			0.95				

Lamination scans were performed: ☒ Yes ☐ No ☐ N/A

Typist: GM

Remarks:

CC: PBS ENGINEERING & ENVIRONMENTAL - HOWARD WELLS HOD.WELLS@PBSUSA.COM

PORT OF NEWPORT - PAULA MIRANDA PMIRANDA@PORTOFNEWPORT.COM

Test results were brought to the attention of: TIM (Name) **with:** CIVIL WEST (Company)

Inspector: M. DAGGETT (Name) II (Level) 524 (Certification Number)

Project Mgr.: Steven Hoppe **Reviewed By:** Project Manager On 06/15/2021

Appendix D

Concrete Strength and Petrographic Analysis Test Report

Carlson Testing
June 15, 2021 (strength)
July 12, 2021 (petrographic)

Carlson Testing, Inc.

Bend Office	**(541) 330-9155**
Geotechnical Office	**(503) 601-8250**
Eugene Office	**(541) 345-0289**
Salem Office	**(503) 589-1252**
Tigard Office	**(503) 684-3460**

June 15, 2021
S2101564
Lab Log #21-7669

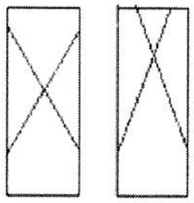
PBS Engineering and Environmental – Howard Wells
4412 S Corbett Ave
Portland, OR 97239

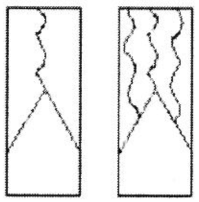
RE: **ROGUE BREWERY SEAWALL**
2320 SE MARINE SCIENCE DR. NEWPORT
COMPRESSIVE STRENGTH OF DRILLED CONCRETE CORES (ASTM C42)

As requested, Carlson Testing Inc. has completed compression testing on one (1) concrete core specimens that were extracted from the above-mentioned project. The samples were obtained by core drilling on June 8, 2021 by our representative from various locations of the structure. Please refer to the second page for coring locations. The ends of the cores were trimmed using a wet diamond blade sawing process. The core specimens were placed into sealed bags on June 8, 2021 where they remained for a minimum of 5 days prior to testing. Testing was completed on June 14, 2021 and the results are as follows:

Register #_21-7669_ Specimen number	**1**
Age of Specimen (days)	6
Date and Time tested	6/14/2021
Nominal Maximum Aggregate Size (in.)	3/4"
Length of Specimen as Received (in.)	8.00
Length of specimen prior to capping (in.)	5.55
Length of specimen after capping (in.)	5.70
Direction of load in respect to placement	Perp
Moisture condition at time of testing	MT
Average diameter of core specimen (in.)	2.83
Length to diameter ratio (l/d) *	2.01
Applied load at specimen failure (lbs.)	39975
Specimen area (sq.in.)	6.29
Uncorrected unit (psi)	6360
Strength correction factor *	N/A
Corrected unit psi (psi)	6360
Type of Fracture	4
Density lb/ft^3	N/A

*P - Perpendicular *Strength correction factor applied when length to diameter ratio is less than 1.75
*L – Parallel *N/R – Not Requested


Type 1
Reasonable well-formed cones on both ends, less than 1 in. [25 mm] of cracking through caps


Type 2
Well-Formed cone on one end, vertical cracks running through caps, no well-defined cone on other end


Type 3
Columnar vertical cracking through both ends, no well-formed cones


Type 4
Diagonal fracture with no cracking through ends; tap with hammer to distinguish from Type 1

Core Specimen Location

Specimen No. 1	***West End Seawall***

Our reports pertain to the material tested/inspected only. Information contained herein is not to be reproduced, except in full, without prior authorization from this office. Under all circumstances, the information contained in this report is provided subject to all terms and conditions of CTI's General Conditions in effect at the time this report is prepared. No party other than those to whom CTI has distributed this report shall be entitled to use or reply upon the information contained in this document.

If there are any further questions regarding this matter, please do not hesitate to contact this office.

Respectfully submitted,
CARLSON TESTING, INC.

DK for

Steve Hoppe
Salem Branch Manager

gm

cc: PBS ENGINEERING & ENVIRONMENTAL - HOWARD WELLS E-Mailed
PORT OF NEWPORT - PAULA MIRANDA E-Mailed

Carlson Testing, Inc.

Bend Office	(541) 330-9155
Geotechnical Office	(503) 601-8250
Eugene Office	(541) 345-0289
Salem Office	(503) 589-1252
Tigard Office	(503) 684-3460

July 12, 2021
S2101564

PBS Engineering & Environmental – Howard Wells
4412 SW Corbett Ave
Portland, OR 97239

Re: **Rogue Brewery Seawall**
2320 SE Marine Science Dr. Newport, OR

Project Team:

Attached please find Petrographic Examination for concrete core obtained on June 8, 2021, submitted by CTI representative for the above referenced project. The testing was completed by CTL Group.

Our reports pertain to the material tested/inspected only. Information contained herein is not to be reproduced, except in full, without prior authorization from this office. Under all circumstances, the information contained in this report is provided subject to all terms and conditions of CTI's General Conditions in effect at the time this report is prepared. No party other than those to whom CTI has distributed this report shall be entitled to use or rely upon the information contained in this document.

If there are any further questions regarding this matter, please do not hesitate to contact this office.

Respectfully submitted,
CARLSON TESTING, INC.

Steve Hoppe
Salem Branch Manager

gm

Attachment: Petrographic Examination of Concrete Core from Rogue Brewery Seawall Project, Newport, Oregon

cc: PBS ENGINEERING & ENVIRONMENTAL - HOWARD WELLS E-Mailed
PORT OF NEWPORT - PAULA MIRANDA E-Mailed

Copy No. 1

Report for
Carlson Testing
4060 Hudson Avenue, Salem, Oregon 97301

CTLGroup Project No. 157708

Petrographic Examination of Concrete Core from Rogue Brewery Seawall Project, Newport, Oregon

July 8, 2021

Submitted by:
Jaclyn Ferraro

CTLGroup is a registered d/b/a of Construction Technology Laboratories, Inc.

REPORT OF PETROGRAPHIC EXAMINATION

Date: July 8, 2021

CTLGroup Project No.: 157708

Petrographic Examination of Concrete Core from Rogue Brewery Seawall Project, Newport, Oregon

One concrete core, identified as S2101564 (Fig. 1), was received on June 22, 2021, from Mr. Steve Hoppe, Carlson Testing, Salem, Oregon. Reportedly, the concrete core was horizontally obtained from a concrete seawall at the Rogue Brewery in Newport, Oregon. The age of the concrete is unknown. Petrographic examination, ASTM C856, of the core was requested to evaluate constituents and properties, as well as the condition of the concrete.

This report presents the details and results of the petrographic examination of the concrete in Core S2101564. For convenience, descriptions of larger scale features are given in inches and smaller scale features are given in millimeters in the text of this report.

FINDINGS AND CONCLUSIONS

The concrete represented by Core S2101564 is good quality and is in good condition. The majority of paste is dense and hard; the outermost 1 mm of the paste is moderately soft, which is considered a minimal depth. No cracks are present in the core. One short microcrack, perpendicular to the core outer surface, is present at the core outer surface, extending to a depth of 4 mm (Fig. 2); this is likely a shrinkage microcrack. No other deterioration or distress is observed in the core. No evidence of alkali-aggregate reaction, chemical attack, or other deleterious mechanism is observed.

The outer end of Core S2101564 is a fairly flat concrete surface covered by a thin, black coating (Fig. 1a). The concrete consists of natural gravel coarse aggregate and natural sand fine aggregate within an air-entrained portland cement paste (Figs. 2 and 3). The properties of the concrete are summarized in the following:

- Coarse aggregate. The concrete contains a natural gravel coarse aggregate consisting mostly of various igneous and metaigneous rock types including basalt, andesite, and gabbro. Much lesser amounts of other various rock types are present as well. The coarse aggregate is rounded to subrounded in shape, with an observed top size of 0.5 in. The aggregate is uniformly distributed throughout the core (Fig. 2)
- Fine aggregate. The fine aggregate is a natural sand consisting mainly of various igneous and metaigneous rock types including basalt, andesite, and gabbro; with lesser amounts of quartz, feldspar, pyroxene, chert and/or chalcedony, ironstone, and other various rocks and minerals. The fine aggregate is rounded to angular in shape and uniformly distributed throughout the core.
- Air-void system. The air content is estimated at 2.5 to 3.5%. The hardened concrete is considered not air entrained, based upon scarcity of air voids. Most air voids are spherical in shape.
- Paste. Paste in the outer 1 mm of the core is light gray and moderately soft, following by a thin (less than 0.1 mm (0.004 in.) thick) dark gray, very hard line; this line appears to be a water infiltration line. Paste in the remainder of the core is medium-dark gray and hard. The paste-to-aggregate bond is weak; this is typical of concrete with rounded natural gravel. The depth of paste carbonation is 1.5 mm from the core outer surface. Ettringite is present, lightly lining many air voids. The residual amount of portland cement grains is estimated at 15 to 18%, by volume of paste. No supplementary cementitious materials are observed. The water-to-cement ratio (w/c) in the concrete is estimated at 0.34 to 0.40; this estimate is based upon the interpretation of the petrographically observed paste properties (Fig. 3)

We reserve the right to amend this report, should additional cores be provided or information about the concrete and/or service conditions be made available.

All information obtained in the petrographic examination is presented in the petrographic data form at the end of this report.

METHODS OF TEST

Petrographic examination of Core S2101564 was performed in accordance with ASTM C856-20, "Standard Practice for Petrographic Examination of Hardened Concrete." The core was visually inspected and photographed as received. The core was saw-cut in half longitudinally through the depth of the concrete, and one of the resulting saw-cut surfaces was ground (lapped) to produce a smooth, flat, semi-polished surface. Lapped and freshly broken surfaces of the concrete were examined using a stereomicroscope at magnifications up to 45X.

For thin-section study, a small 36 mm (1.4 in.) rectangular block was cut from the outer portion of the core, and one side of the block was lapped to produce a smooth, flat surface. The block was cleaned and dried, and the prepared surface was mounted on a separate ground glass microscope slide with epoxy. After the epoxy hardened, the thickness of the mounted block was reduced to approximately 20 μm (0.0008 in.). The resulting thin section was examined using a polarized-light (petrographic) microscope at magnifications up to 400X to study aggregate and paste mineralogy and microstructure.

Estimated w/c is based on observed concrete and paste properties including, but not limited to: 1) relative amounts of residual (unhydrated and partially hydrated) portland cement clinker particles, 2) amount and size of calcium hydroxide crystals, 3) paste hardness, color, and luster, and, 4) paste-aggregate bond. These techniques have been widely used by industry professionals to estimate w/c.

Depth and pattern of paste carbonation was initially determined by application of a pH indicator solution (phenolphthalein) to freshly cut or fractured concrete surfaces. The solution imparts a deep magenta stain to high pH, non-carbonated paste. Carbonated paste does not change color. The extent of paste carbonation was confirmed in thin-section.

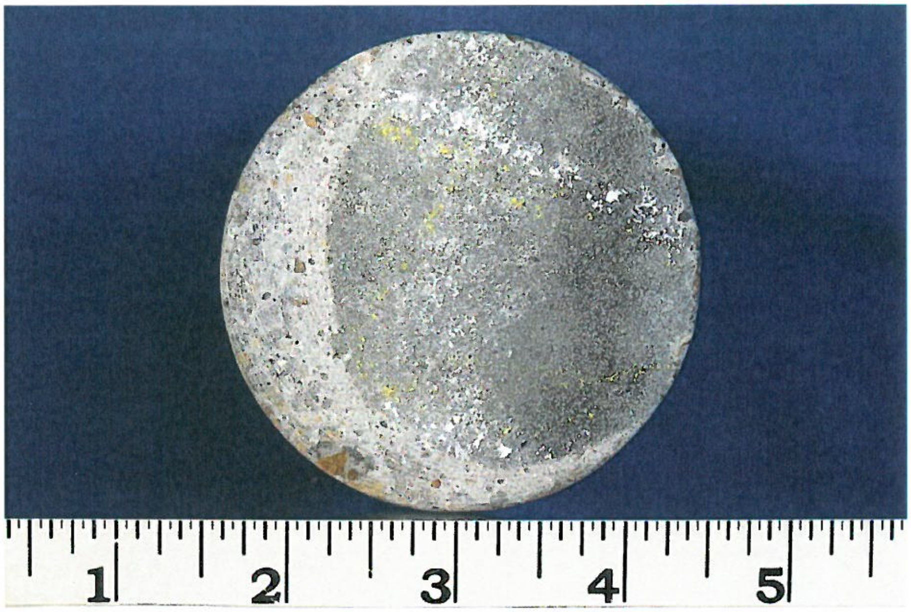
Jaclyn Ferraro

Jaclyn Ferraro
Group Director &Petrographer III
Petrography Group

JMF/

Attachment

Notes:
1. Results refer specifically to the sample submitted.
2. This report may not be reproduced except in its entirety.
3. The sample will be retained for 30 days, after which it will be discarded unless we hear otherwise from you.



1a. **Core outer surface. The surface is a fairly flat concrete surface covered by a thin, black coating.**

1b. **Side view of core.**

Fig. 1 **Core S2101564, as received for testing. Scale is in inches.**

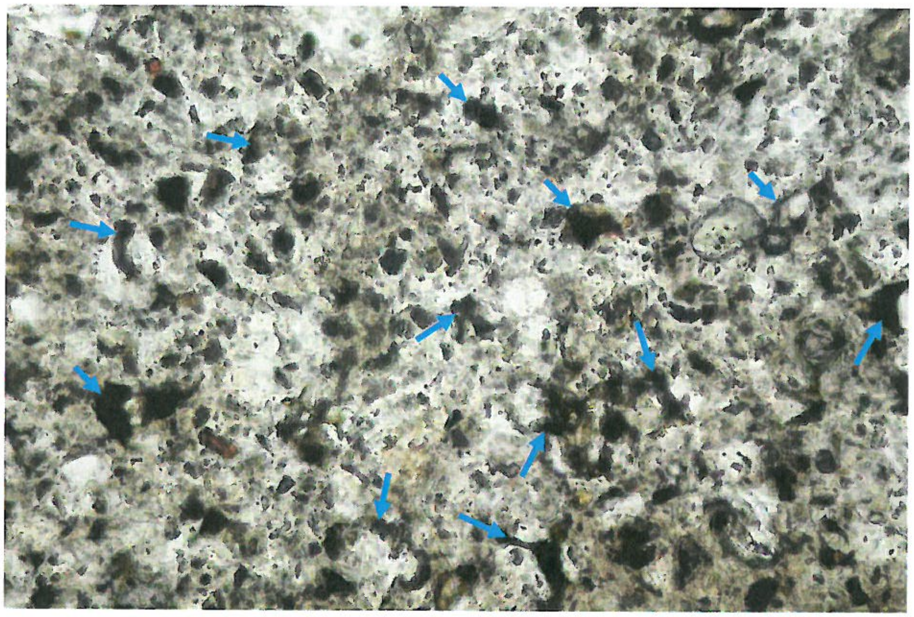
Fig. 2 Lapped, cross-sectional concrete surface of Core S2101564.

No cracks are present. One short microcrack (outline in red), perpendicular to the core outer surface, is present at the core outer surface, extending to a depth of 4 mm (0.02 in.).

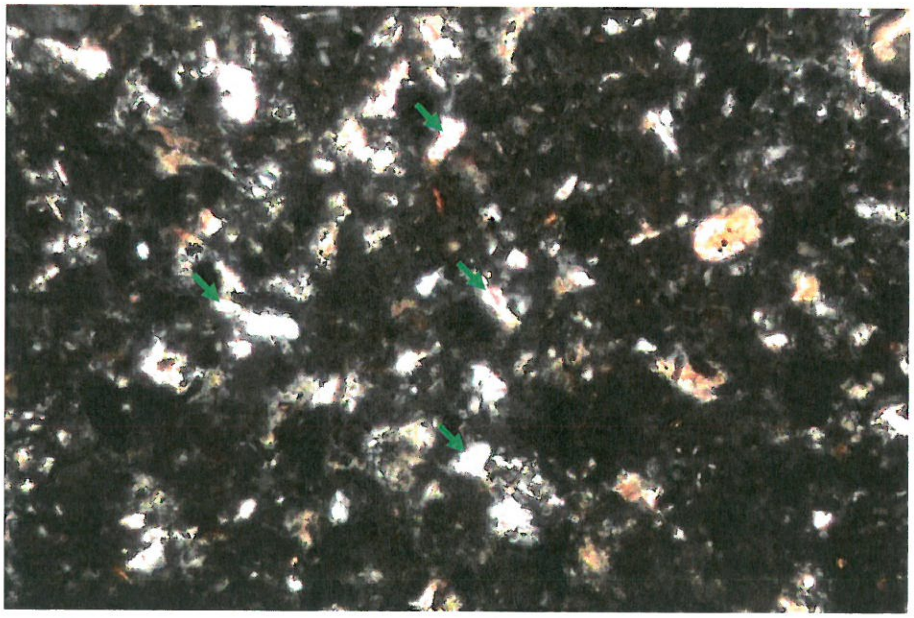
Paste in the outer 1 mm (0.04 in.) of the core is light gray (white arrow), following by a thin (less than 0.1 mm (0.004 in.) thick) dark gray line (yellow arrow). Paste in the remainder of the core is medium-dark gray.

The natural gravel coarse aggregate has an observed top size of 0.5 in., is rounded to subrounded in shape and uniformly distributed throughout the core.

Scale is in inches.



3a. Plane-polarized light. Blue arrows designate residual portland cement grains, estimated at 15 to 18%, by volume of paste.



3b. Cross-polarized light. Calcium hydroxide (green arrows) is coarse and patchy.

Fig. 3 Thin section micrographs of Core S2101564. The photos show the same field using different lighting. Field of view, left to right, is approximately 0.4 mm (0.016 in.).

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE, ASTM C856

STRUCTURE: Seawall
LOCATION: Newport, Oregon

DATE RECEIVED: June 22, 2021
EXAMINED BY: Jaclyn Ferraro

SAMPLE

Client Identification: S2101564.

CTLGroup Identification: 5304901.

Dimensions: Core diameter = 72 mm (2.8 in.). Core length = 91 to 101 mm (3.6 to 4.0 in.); partial structure thickness. The core has a black coating on the outer surface, measured at less than 0.1 mm (0.004 in.) thick.

Core Outer Surface: Fairly flat concrete surface covered by a thin, black coating.

Core Inner Surface: Broken concrete surface, extending around all aggregate particles.

Cracks, Joints, Large Voids: One large, irregularly shaped void is present, measured at 6 by 10 mm (0.2 by 0.4 in.). No cracks or joints are observed.

Reinforcement: None present.

AGGREGATES

Coarse: Natural gravel consisting mostly of various igneous and metaigneous rock types including basalt, andesite, and gabbro. Much lesser amounts of other various rock types are present as well.

Fine: Natural sand consisting mainly of various igneous and metaigneous rock types including basalt, andesite, and gabbro; with lesser amounts of quartz, feldspar, pyroxene, chert and/or chalcedony, ironstone, and other various rocks and minerals.

Gradation & Top Size: Visually appears evenly graded to an observed top size of 13 mm (0.5 in.).

Shape, Texture, Distribution: Coarse- Rounded to subrounded, mostly equant in shape (very few elongate); slightly irregular texture; uniform distribution. Fine- Rounded to angular; uniform distribution.

PASTE

Color: Paste in the outer 1 mm (0.04 in.) of the core is light gray, following by a thin (less than 0.1 mm (0.004 in.) thick) dark gray line. Paste in the remainder of the core is medium-dark gray.

Hardness: Paste in the outer 1 mm (0.04 in.) of the core is moderately soft, following by a thin (less than 0.1 mm (0.004 in.) thick) very hard line. Paste in the remainder of the core is hard.

Luster: Paste in the outer 1 mm (0.04 in.) of the core is somewhat dull, following by a thin (less than 0.1 mm (0.004 in.) thick) vitreous line. Paste in the remainder of the core is vitreous to subvitreous.

Paste-Aggregate Bond: Weak. When struck with a geology hammer in the laboratory, the concrete fractured around most, but through a couple, coarse aggregate particles.

Air Content: Estimated at 2.5 to 3.5%. The hardened concrete is considered not air entrained, based upon scarcity of air voids. Most air voids are spherical in shape.

Depth of Carbonation: 2 mm (0.08 in.) from the concrete outer surface.

Calcium Hydroxide*: Estimated 20 to 25%; Coarse, patchy crystallinity. Small areas of amorphous paste surround some aggregate particles.

Residual Portland Cement Clinker Particles*: Estimated at 15 to 18%.

Supplementary Cementitious Materials: None observed.

Secondary Deposits: Ettringite is present, lightly lining many air voids.

MICROCRACKS: One short microcrack, perpendicular to the core outer surface, is present at the core outer surface, extending to a depth of 4 mm (0.02 in.).

ESTIMATED WATER-TO-CEMENT RATIO: Estimated at 0.34 to 0.40 in the body of the concrete; this estimate is based upon the interpretation of the petrographically observed paste properties.

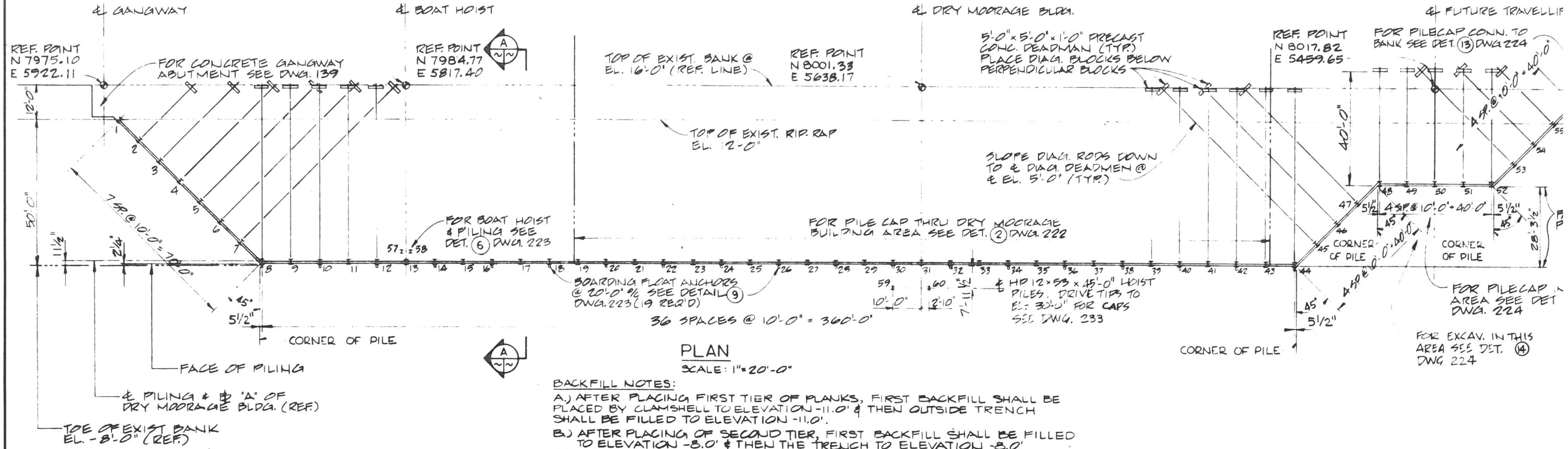
---

*percent by volume of paste

Appendix E

Soldier Pile Bulkhead Piling Plan & Section As Built Drawing

Swan Wooster Engineering
Drawing 7-E-240



PLAN

SCALE: 1"=20'-0"

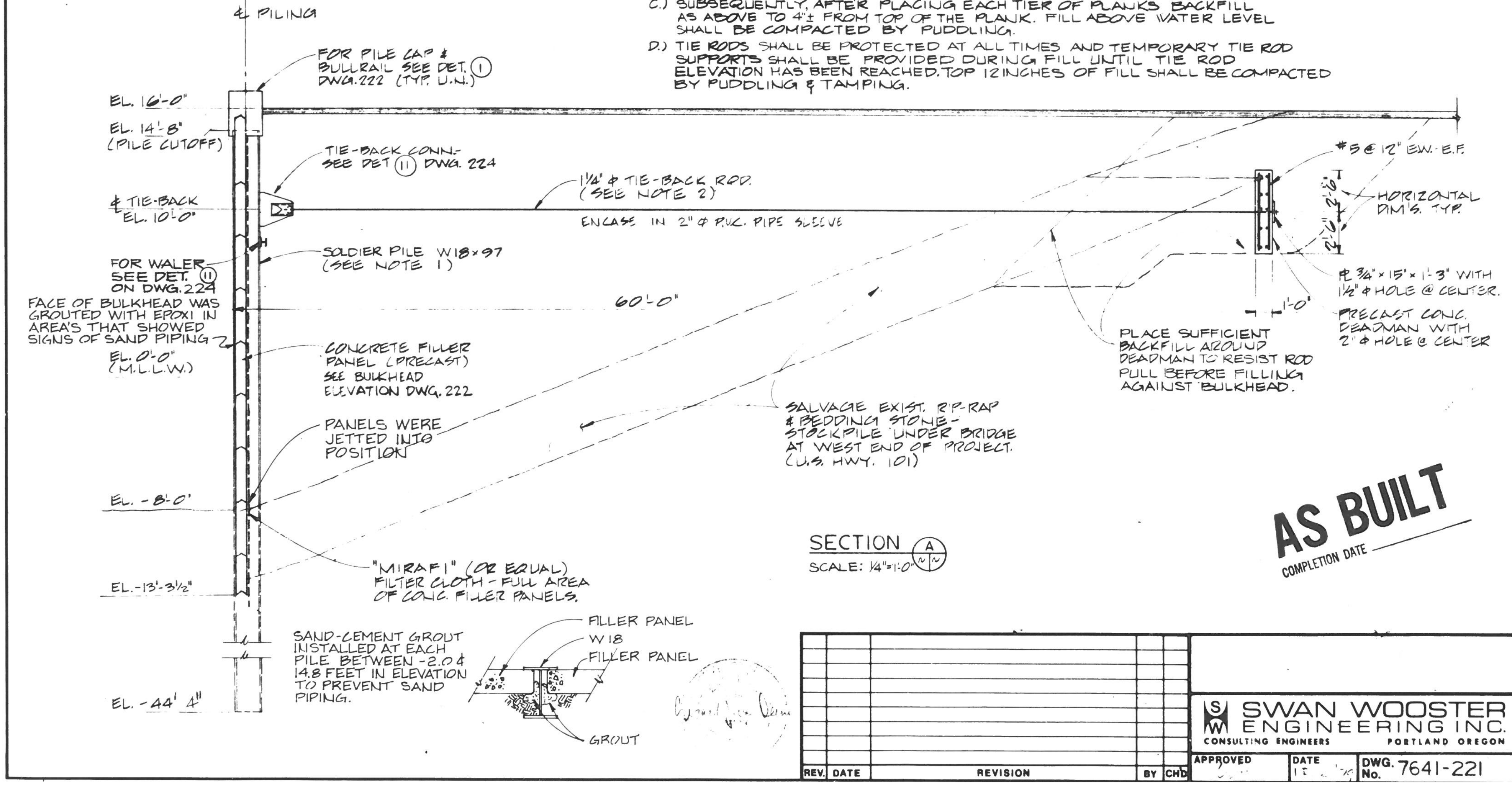
BACKFILL NOTES:

A.) AFTER PLACING FIRST TIER OF PLANKS, FIRST BACKFILL SHALL BE PLACED BY CLAMSHELL TO ELEVATION -11.0' & THEN OUTSIDE TRENCH SHALL BE FILLED TO ELEVATION -11.0'.

B.) AFTER PLACING OF SECOND TIER, FIRST BACKFILL SHALL BE FILLED TO ELEVATION -8.0' & THEN THE TRENCH TO ELEVATION -8.0'.

C.) SUBSEQUENTLY, AFTER PLACING EACH TIER OF PLANKS BACKFILL AS ABOVE TO 4"± FROM TOP OF THE PLANK. FILL ABOVE WATER LEVEL SHALL BE COMPACTED BY PUDDLING.

D.) TIE RODS SHALL BE PROTECTED AT ALL TIMES AND TEMPORARY TIE ROD SUPPORTS SHALL BE PROVIDED DURING FILL UNTIL TIE ROD ELEVATION HAS BEEN REACHED. TOP 12 INCHES OF FILL SHALL BE COMPACTED BY PUDDLING & TAMPING.



SECTION Ⓐ

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

NOTES:

1.) SOLDIERS PILES SHALL BE W18 OR W18 x 36, ASTM A-588 GRA OR B (COR-TEN STEEL) HOIST PI #59 & #60 SHALL BE HP 12 x 53 (A5 A690) (USS MARINER) HOIST PILES SHALL BE HP12 x 53, (A-36 STEEL

2.) TIE RODS SHALL CONFORM TO ASTM & BE 150 KSI, MASTIC COATED & COVE WITH EXTRUDED POLYETHYLENE. R THREADS & FITTINGS SHALL BE AS MANUFACTURED BY DYCKERHOFF & WIDMANN (OR EQUAL.)

3.) ALL HARDWARE, BOLTS, & INSERTS S BE HOT DIP GALVANIZED.

4.) ALL REINF. SHALL CONFORM TO AS A-615 GRADE 40.

5.) ALL CONCRETE SHALL HAVE A MINIM COMPRESSIVE STRENGTH OF 4000 P AT 28 DAYS - CEMENT SHALL BE TYPE II a PER ASTM C-150, 7 SACKS PER CU. AGGREGATE SHALL BE PER ASTM C-33.

6.) PILE DRIVING: 19000 FOOT-POUNDS IMPACT ENERGY. PROTECT PILE HEA WITH SUITABLE CUSHION.

7.) ALL OUTSIDE EDGES OF CONC. SHAL FORMED WITH 3/4" x 45° CHAMFER EXCEPT AS NOTED.

AS BUILT
COMPLETION DATE

REV.	DATE	REVISION	BY	CHD

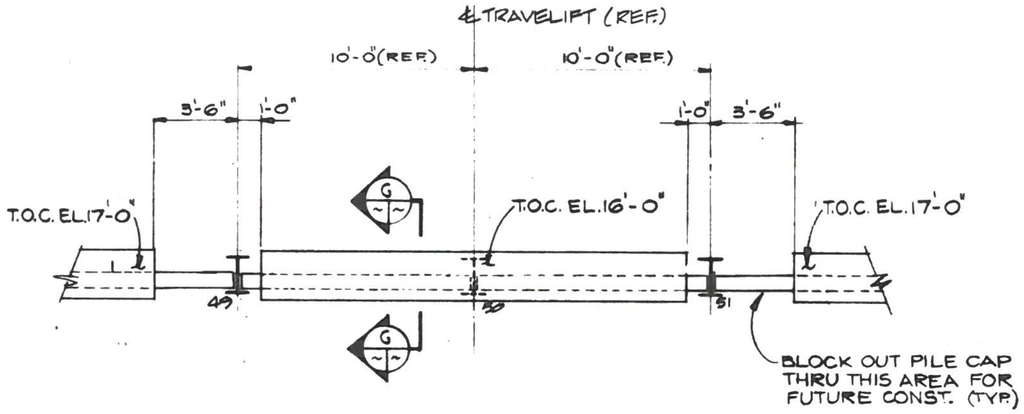
SWAN WOOSTER
ENGINEERING INC.
CONSULTING ENGINEERS PORTLAND OREGON

APPROVED
DATE
DWG. No. 7641-221

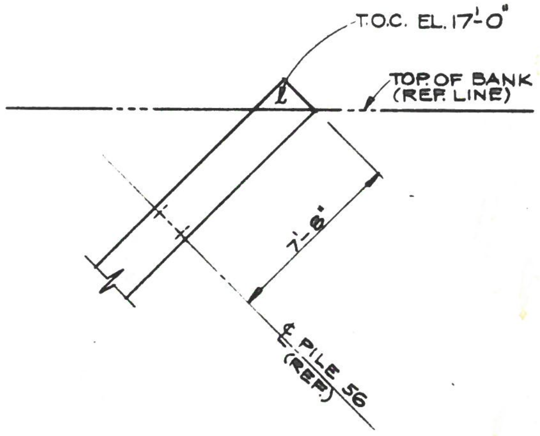
SOLDIER PILE BULKHEAD
PILING PLAN & SECTION
SOUTH BEACH MARINA - NEWPORT, OR

U. S. DEPARTMENT OF AGRICULT
SOIL CONSERVATION SERVICE

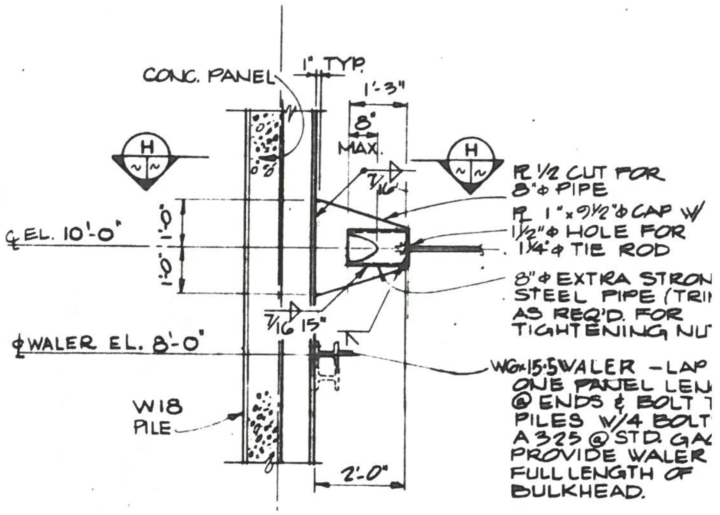
Designed
Drawn
Traced
Checked
Date
Approved by
Title
Title
Sheet No 68 of 104
Drawing No 7-E-240



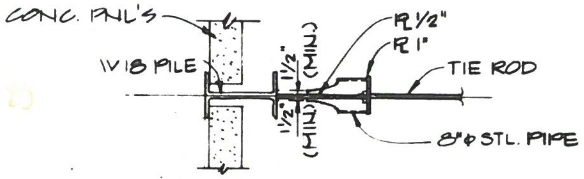
DETAIL-12
1/4" = 1'-0"
(DWG. 221)



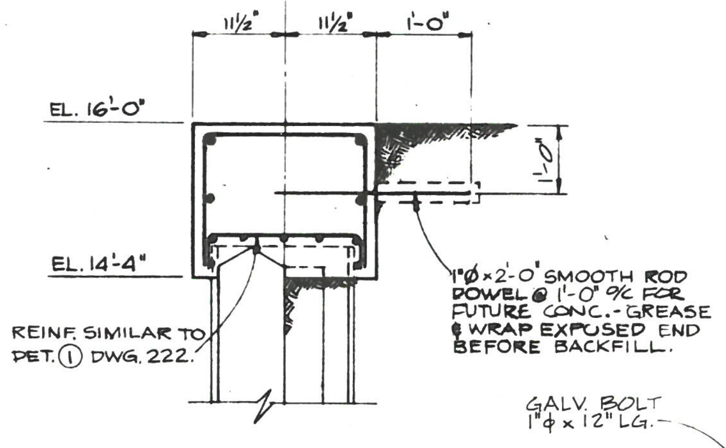
DETAIL-13
1/4" = 1'-0"
(DWG. 221)



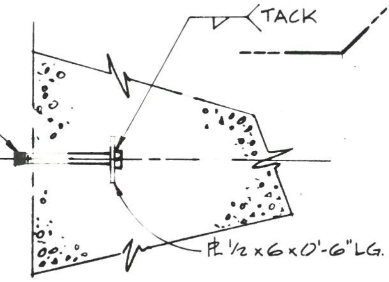
DETAIL -11
1/2" = 1'-0"
(DWG. 221)



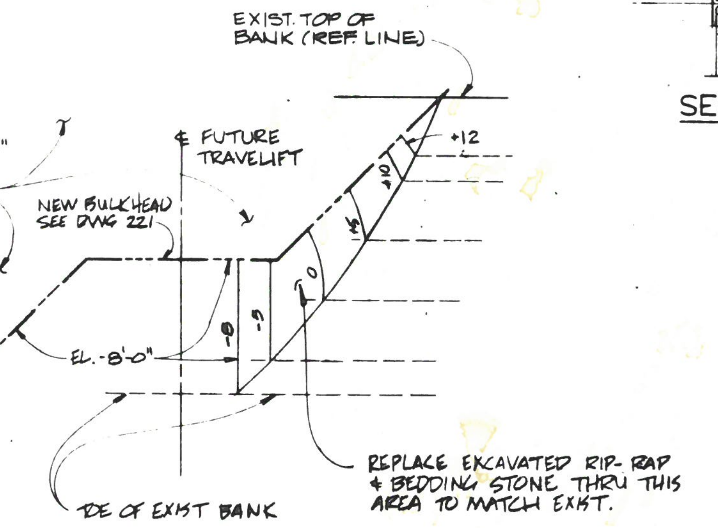
SECTION H



SECTION G
1" = 1'-0"



DETAIL 15
1" = 1'-0"
(DWG. 222)



DETAIL-14
1" = 20'-0"
(DWG. 221)

NOTES:

1.) FOR GENERAL NOTES SEE DWG 221

AS BUILT
COMPLETION DATE

SOLDIER PILE BULKHEAD
SECTIONS & DETAILS S
SOUTH BEACH MARINA – NEWPORT, OR

U. S. DEPARTMENT OF AGRICULTU
SOIL CONSERVATION SERVICE

SWAN WOOSTER ENGINEERING INC.
CONSULTING ENGINEERS PORTLAND OREGON

REV.	DATE	REVISION	BY	CHK

APPROVED
DATE 1 Feb '79
DWG. No. 7641-224

Designed
Drawn
Traced
Checked
Date
Approved by
Title
Title
Sheet No 71
Drawing No 7-E-240

Appendix F

Basis of Repair Design

PBS Engineering and Environmental
September 17, 2021

Basis of Repair Design

Rogue Brewery Seawall
Newport, Oregon

Prepared for:

Port of Newport
600 SE Bay Boulevard
Newport, Oregon 97365

September 17, 2021
PBS Project 74183.000

PBS

4412 SW CORBETT AVENUE
PORTLAND, OR 97239
503.248.1939 MAIN
866.727.0140 FAX
PBSUSA.COM

Table of Contents

1 PROJECT DESCRIPTION

The Rogue Brewery Seawall is approximately 540 feet long and supports the Rogue World Headquarters building at 2320 SE Marine Science Drive in Newport, OR (44° 27′ 12″ N and 124° 3′ 8″ W). The seawall that retains the soil that the brewery rests on is deteriorated. In addition, the soil is seeping out of the seawall through the concrete lagging panels in some locations.

This Basis of Design (BOD) describes the criteria for repairing the seawall to continue its original function and extend the service life. The BOD for replacement of the seawall or relocation of the building will differ from this BOD. The repair BOD does not include any seismic upgrades or retrofits, nor are there any considerations for liquefaction of the soil. The repair of the seawall will include stabilization of the soil for the purpose of arresting the seeping of the soil through the panels. Also, it will accommodate the original design loading and/or the current loading configuration of the buildings, whichever produces the largest load effect.

1.1 Location

The project site (see Figure 1-1, Rogue Brewery) is in Newport, Oregon at the following location:

Address	Coordinates for Center of Project
2320 SE Marine Science Dr.	44° 37' 12" N
Newport, Oregon 97365	124° 3' 8" W

Figure 1-1 Site Location

1.2 Scope

The scope of this basis of design is for the repair of the seawall. Only the current loading and the original design loading are considered. Seismic upgrades to current code requirements are not included in this basis of design. Soil liquefaction is also excluded from this basis of design.

1.3 Existing Conditions

The seawall supports the Rogue Ales Brewery facilities. The building was built in 1980. The seawall on the north side of the building retains the soil supporting the building. The building is approximately 98 feet by 240 feet with a roof height of 96 feet. Rogue Ales is a tenant of the Port of Newport.

The facilities are being used for beer production and packing. There is also a restaurant in the building.
The seawall is a soldier pile retaining wall with tieback rods connected to concrete deadman anchors that are in various states of deterioration. The steel piles show signs of corrosion and soil is seeping through the concrete lagging.

1.4 Geotechnical Conditions

Our understanding of subsurface conditions at the site is based on our review of available reports summarized in Section 2 and our observations of shallow vacuum truck explorations on May 24, 2021.

Available subsurface information indicates the site is surfaced with asphaltic concrete pavement underlain by sand fill that extends to depths of about 12 feet underlain by sand to depths of 76.5 feet, the maximum depth explored in the Stantec borings. The sand fill is tan to light gray, fine-grained, and contains up to a trace of silt and man-made debris/garbage. Based on SPT N-values the sand fill is typically loose to medium dense. The sand fill was dense at a depth of 10 feet in boring GP-03. A 6-inch-thick layer of gravelly clay fill was encountered within the sand fill at a depth of 3 feet in boring GP-04. Sand was encountered below the fill at a depth of about 12 feet and is typically tan to light gray or gray, fine-grained, and contains up to some silt. The sand in boring GP01 was dark gray to black at a depth of 17 feet. Gravel was encountered in the sand in boring GP-04 between depths of 15 feet and 25 feet. Wood fragments were encountered in the sand in GP-01 and GP-04 at depths of 35 feet and 36.5 feet, respectively. The sand is clayey from a depth of 16 feet to 17 feet in boring GP01. Based on SPT N-values the sand is typically medium dense to very dense below the fill to a depth of 40 feet and dense to very dense below 40 feet. The sand in GP-04 was loose at a depth of 20 feet.

The sand in borings GP-01 and GP-04 were observed to be wet to saturated below depths of 16 feet and 11.5 feet, respectively at the time of drilling, indicating possible groundwater depth. Groundwater levels at the site fluctuate in response to precipitation and the level of the nearby bay.

2 GOVERNING CODES AND REFERENCE DOCUMENTS

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

Principal General Design Standard:

- Oregon Structural Specialty Code 2019 (OSSC)
- American Society of Civil Engineers (ASCE), "Minimum Design Loads for Buildings and Other Structures," ASCE/SEI 7-16

In situations where OSSC or ASCE 7-16 do not cover a design situation, the applicable design practices and guidelines may include, but are not limited, to the following:

- American Concrete Institute (ACI), "Building Code Requirements for Structural Concrete," ACI 318-14.

- American Institute of Steel Construction (AISC), "Specification for Structural Steel Buildings," AISC 360-16.
- AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, 8th Edition
- International Code Council, "2018 International Building Code (IBC)," 2018.
- American Society of Civil Engineers (ASCE), "Seismic Design of Piers and Wharves", ASCE 61-14
- American Society of Civil Engineers (ASCE), "Waterfront Facilities Inspection and Assessment," MOP 130

The following information was reviewed for the geotechnical conditions of the site:

- Stantec, August 20, 2021, Limited Phase II Environmental Site Assessment and Geotechnical Evaluation, Port of Newport Rogue Brewery Property, 2320 SE Marine Science Drive, Newport, Oregon 97365; prepared for Oregon Cascades West Council of Governments.

- Northwest Testing, Inc., May 7, 2020, Laboratory Testing – Rogue Brewery; prepared for Stantec.

3 GENERAL REQUIREMENTS

3.1 Seawall Description

The Rogue Brewery Seawall is approximately 540 feet long and supports the Rogue World Headquarters building. The seawall is a soldier pile wall with steel piles and concrete lagging. The seawall consists of 55 steel piles. The steel piles are approximately 60 feet in length with approximately 16 feet above the mean lower low water level (MLLW). The pile size is W18x97. The pile spacing is 10 feet. The concrete lagging consists of 9-inch-thick precast panels. Tieback rods are installed 6 feet below grade and connect the piles to precast concrete deadmen. The deadmen are 40-60 feet away from the face of the seawall, depending on location. The tops of the soldier piles are encased by a concrete grade beam that runs the length of the wall.

A floating dock is located along most of the face of the seawall. The dock is supported by pipe and timber piles located between the dock and the seawall. The tops of the piles are connected to the concrete grade beam.

Figure 3-1 Rogue Brewery Seawall

3.2 Function

The seawall retains the soil underneath the building of the Rogue Brewery. The seawall also provides support for the top of the floating dock guide piles.

3.3 Units

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

Length:	Feet and/or inches
Force:	Tons, Kips (kilopounds), or pounds
Time	Seconds and/or minutes/hour/days/months/years
Temperature:	Degrees Fahrenheit

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

4 DESIGN CRITERIA

4.1 Dead Loads (D)

Dead loads consist of the self-weight of the structure (i.e., weight of permanent structure, reinforced concrete, structural steel, permanent equipment, utilities, etc.).

4.1.1 Unit Weights

The following unit weights shall be used in the calculations:

Reinforced concrete:	150 lb/ft^3 (normal weight)
Reinforced concrete:	120 lb/ft^3 (light weight)
Structural steel:	490 lb/ft^3
Asphalt concrete:	145 lb/ft^3
Water:	62.4 lb/ft^3

4.1.2 Superimposed Dead Loads

Superimposed dead loads include the weight of the buildings.

4.2 Live Loads (L)

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

4.2.1 Uniform Live Loads (Lu)

Uniform live loads are the maximum distributed loads expected by intended use. The live load surcharge varies by location. The general uniform live load based on ASCE 7-16 for a light manufacturing facility is 125 psf. A larger live load shall not be within 30 feet of the seawall.

4.2.2 Vehicle Live Load (Lv)

Vehicle live load is the maximum load expected for the largest vehicle anticipated to access a given area. Uniform live loadings and concentrated live loading from pneumatic-tired equipment will not be applied simultaneously in the same area. Vehicle live loads are present during shipments to and from the brewery and moving of material as part of operating activities.

4.3 Load Combinations

4.3.1 General

The structure shall be analyzed to safely resist the load combinations represented in section 4.3.2. Each component of the structures and the foundation elements shall be analyzed for all the applicable combinations.

4.3.2 Design Methods

Load combinations and load factors used for load factor design are presented in this section. Concrete and steel structures shall be designed using the load resistance factor design (LRFD) method. For the geotechnical demands of the piles, allowable design (ASD) shall be used.

For LRFD method, the following load combination shall be used:

1. 1.2D+1.6Lu+1.6Lv (max compression)
2. 0.9D+1.6Lu+1.6Lv (max tension)

For ASD method (for geotechnical demands), the following load combination shall be used:

1. 1.0D+1.0Lu+1.0Lv

Where

D: Dead Load
Lu: Uniform Live Load
Lv: Vehicle Live Load

Appendix G

Geotechnical Report

GRI
October 7, 2021

GRI
9750 SW Nimbus Avenue
Beaverton, OR 97008-7172
503-641-3478 | www.gri.com

MEMORANDUM

To: Howard Wells / PBS

Date: October 7, 2021

GRI Project No.: 6179-B

From: Scott Schlechter, PE, GE; and Brian Bayne, PE

Re: Preliminary Geotechnical Consultation
Rogue Brewery Seawall
Port of Newport
Newport, Oregon

At your request, GRI has completed a geotechnical consultation to assist PBS in the preliminary evaluation of potential repair schemes for the existing seawall versus replacement options. The primary purpose of our consultation was to evaluate static lateral earth pressures on the existing wall, evaluate potential seismic considerations for wall replacement, and provide constructability considerations for different wall alternatives.

The following information for the project site was reviewed:

> BergerABAM, December 2018, "Structural Evaluation Report, Port of Newport, Rogue Brewery Seawall; prepared for Port of Newport."
>
> Stantec, August 20, 2021, "Limited Phase II Environmental Site Assessment and Geotechnical Evaluation, Port of Newport Rogue Brewery Property, 2320 SE Marine Science Drive, Newport, Oregon 97365," prepared for Oregon Cascades West Council of Governments.
>
> Northwest Testing, Inc., May 7, 2020, "Laboratory Testing – Rogue Brewery; prepared for Stantec."

SITE DESCRIPTION

The Vicinity Map, Figure 1, shows the general location of the site and previous explorations in the area. The site is located on the south side of Yaquina Bay, south of an existing marina. The seawall is approximately 540 feet long and consists of W18x97 steel piles at about 10-foot spacing with concrete lagging between piles. A deadman anchor system with an anchor connection at about an elevation of 10 feet [Mean Lower Low Water (MLLW)] provides lateral support for the wall. The seawall supports the Rogue Work Headquarters building (Rogue Brewery) and a relatively flat asphalt concrete (AC) parking lot/storage area at about an elevation of 16 feet MLLW. Based on recent Army Corps of Engineers bathymetric data, the mudline on the marina side of the seawall is at about elevation -8 feet to -10 feet MLLW and is relatively flat.

PROJECT DESCRIPTION

As discussed in BergerABAM's 2018 report, corrosion of the steel soldier piles and spalling of the concrete beam/pile cap was observed for the existing seawall. In addition, the report discussed the loss of backfill material through gaps in the concrete lagging, which may have led to the historical settlement of the interior floor slab of the Rogue Brewery. During dropping tide conditions, relatively heavy seepage can be observed between the piles and concrete lagging, which supports the risk of backfill piping through these joints during the tidal differential head conditions at the site. These conditions have decreased the serviceable life of the existing seawall. PBS was contracted by the Port of Newport further to evaluate the remaining service life of the seawall and develop structure repair alternatives or replacement options initially discussed in BergerABAM's 2018 report and associated cost estimates.

SUBSURFACE CONDITIONS

Our understanding of subsurface conditions at the site is based on our review of available reports summarized above and our observations of shallow-vacuum truck explorations on May 24, 2021.

Available subsurface information indicates the site is surfaced with AC pavement underlain by sand fill that extends to depths of about 12 feet underlain by sand to depths of 76.5 feet, the maximum depth explored in the Stantec borings. The sand fill is tan to light gray, fine grained, and contains up to a trace of silt and man-made debris/garbage. Based on SPT N-values, the sand fill is typically loose to medium dense. The sand fill was dense at a depth of 10 feet in boring GP-03. A 6-inch-thick layer of gravelly clay fill was encountered within the sand fill at a depth of 3 feet in boring GP-04. Sand was encountered below the fill at a depth of about 12 feet and is typically tan to light gray or gray, fine grained, and contains up to some silt. The sand in boring GP-01 was dark gray to black at a depth of 17 feet. Gravel was encountered in the sand in boring GP-04 between depths of 15 feet and 25 feet. Wood fragments were encountered in the sand in borings GP-01 and GP-04 at depths of 35 feet and 36.5 feet. The sand is clayey from a depth of 16 feet to 17 feet in boring GP-01. Based on SPT N-values, the sand is typically medium dense to very dense below the fill to a depth of 40 feet and dense to very dense below 40 feet. The sand in boring GP-04 was loose at a depth of 20 feet.

The sand in borings GP-01 and GP-04 were observed to be wet to saturated below depths of 16 feet and 11.5 feet, respectively, at the time of drilling, indicating possible groundwater depth. Groundwater levels at the site fluctuate in response to precipitation and the level of the nearby bay.

PRELIMINARY GEOTECHNICAL CONSIDERATIONS

Static Lateral Earth Pressures

Static lateral earth pressures on the existing tied-back seawall can be evaluated using the lateral earth pressure criteria provided on Figure 2. Additional loading due to surcharge loads should be added in accordance with the criteria shown on Figure 3.

It is our understanding corrosion of the existing soldier piles has caused a reduction in their moment capacities. To reduce moment demand on the existing piles, PBS has considered installing a row of tieback anchors to supplement the existing deadman anchors at about elevation 5 feet. Based on a preliminary evaluation of subsurface conditions behind the wall, we estimate a tieback anchor can develop an ultimate capacity on the order of 100 kips to 150 kips. It should be noted that the installation of a row of tieback anchors would likely modify the loading pattern on the retained earth portion of Figure 2 to reflect a more traditional apparent earth-pressure diagram for multiple anchor levels. While this modification could increase the assumed overall lateral loading on the wall, we do not anticipate the assumed additional load would exceed the substantial additional resistance provided by a tieback.

To reduce the risk of future loss of soil through the concrete lagging, we understand the team is considering installing high-density polymer injection behind the face of wall. The installation of high-density polymer would reduce the permeability of the existing sand soils behind the wall causing a potential hydraulic pressure gradient between water levels on the front and backsides of the wall following tidal fluctuations. If high-density polymer injection is used extensively behind the wall, the Figure 2 lateral earth pressure diagram would likely require modification to account for the additional differential head, unless a suitable drainage system is installed concurrently with the polymer injection. The need for weep holes or other drainage improvements will need to be evaluated further during the next phase of design if this alternative is advanced.

Seismic Considerations

Our preliminary analysis indicates that during a current code-based earthquake, there is a potential for liquefaction of the submerged loose to medium-dense sand encountered in the recent Stantec borings. Associated liquefaction-induced lateral spreading will result in significant lateral loading on the seawall. We estimate lateral spreading deformations could be in excess of 5 feet to 10 feet during a code-based earthquake. Based on our experience in the area, we anticipate replacement of the wall would require significant effort and costs to mitigate the lateral spreading hazard with ground improvement or similar alternatives. Repair alternatives are less likely to trigger the consideration of seismic mitigation.

Based on our experience in the area, there is a risk of tsunami inundation at the site following a code-based earthquake, which may need to be considered in a replacement alternative.

Preliminary Construction Considerations

As part of the repair alternatives, the construction of a row of tieback anchors installed at about elevation 5 feet is being considered. Installation of a row of tieback anchors would require barge access in the marina due to a lack of drill-rig access to the top of the wall. Containment of drill spoils to prevent them from entering the marina will be an important and likely costly construction consideration and will likely require environmental permitting. Tieback anchors will also require the construction of a waler system on the front of the wall, which may impact the existing floating walkway.

If the wall replacement option is considered, the wall would likely require design to the current seismic code and mitigation of the lateral spreading hazard. The use of ground improvement is commonly used to mitigate lateral spreading hazards in waterfront environments and mitigation of the hazard with only structural improvements at this site would likely be challenging or impractical. Ground improvement would likely require creating a block of improved soil behind the back of the wall either through densification of the existing sand or mixing an additive into the soil to improve its seismic performance. Due to the Rogue Brewery location, installing ground improvement beneath the building would be costly and potentially unfeasible and may require relocation of the brewery. Installation of ground improvement behind the seawall and adjacent to existing deadman anchors may cause damage to the wall and should be further evaluated if the wall replacement option is considered.

LIMITATIONS

This memorandum has been prepared to aid the project team in the conceptual alternatives of the project and associated cost estimates. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to wall design. The comments, conclusions, and site-development guidelines presented in this memorandum are preliminary. Depending on the design approach selected, additional subsurface explorations, laboratory testing, and engineering studies are required to provide suitable criteria for the final design.

The conclusions and recommendations submitted in this memorandum are based on geotechnical data obtained by others at the locations indicated on Figure 1 and from other sources of information discussed in this memorandum. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between exploration locations. This memorandum does not reflect any variations that may occur between these locations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions are different from those described in this report or are observed or encountered, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Please contact the undersigned if you have any questions.

Submitted for GRI,

REGISTERED PROFESSIONAL ENGINEER
74883PE
OREGON
May 11, 2004
SCOTT M. SCHLECHTER

Renews: 6-2022

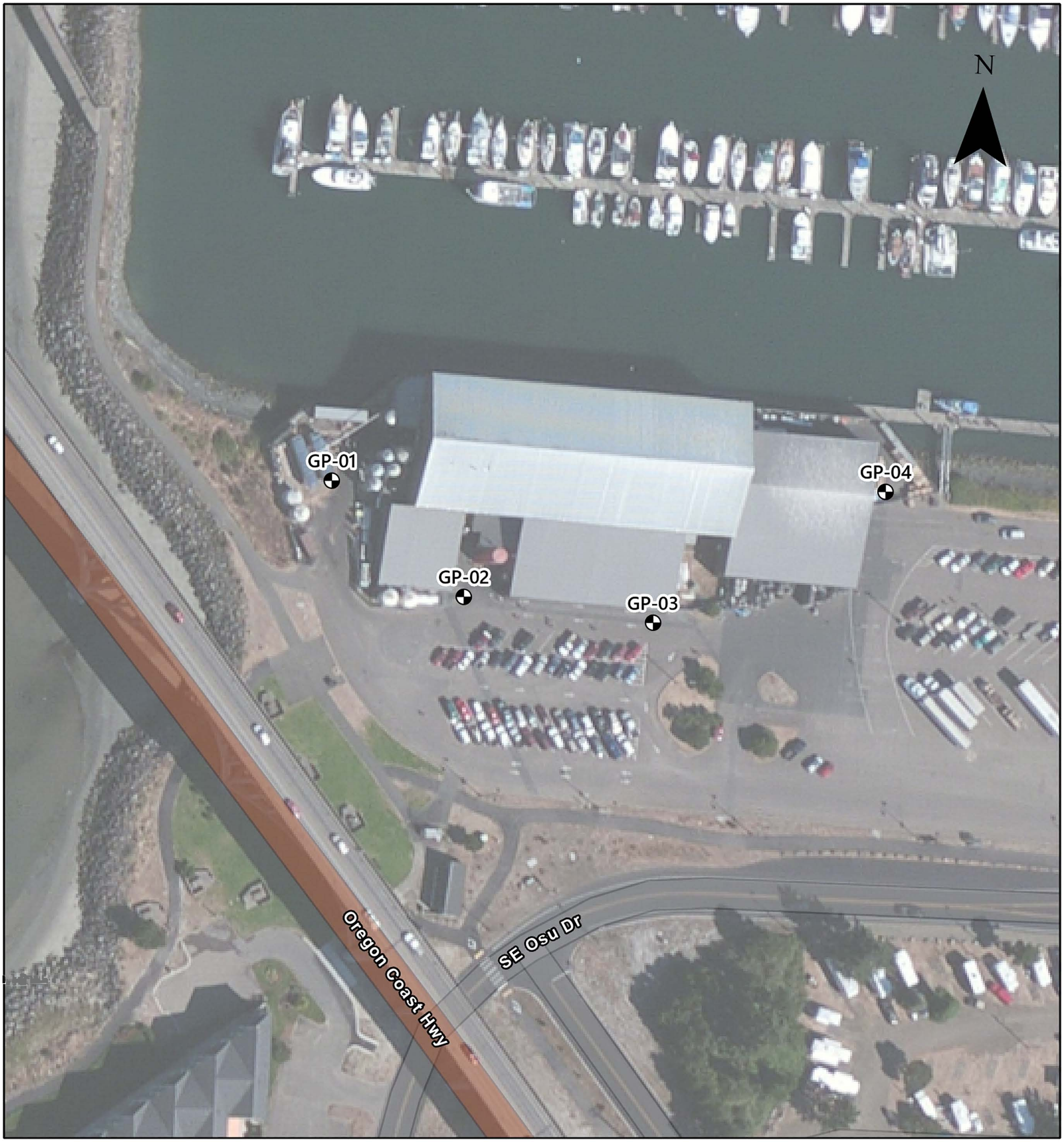
Brian Bayne

Scott M. Schlechter, PE, GE
Principal

Brian J. Bayne, PE
Senior Engineer

This document has been submitted electronically.

6179-B GEOTECHNICAL CONSULTATION MEMORANDUM



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◒ BORING COMPLETED BY STANTEC (2020)

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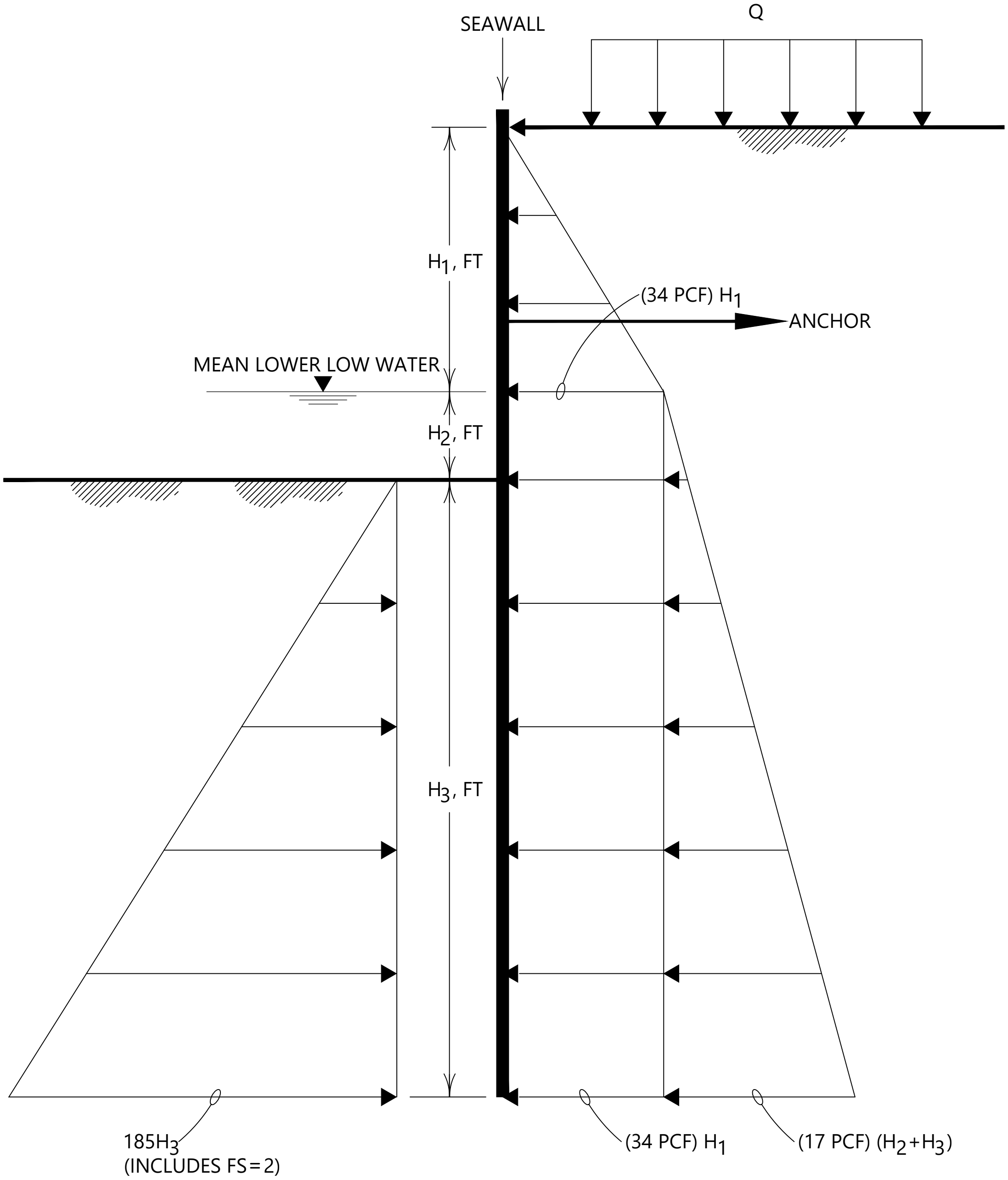
ROGUE BREWERY SEAWALL
NEWPORT, OREGON

VICINITY MAP

OCT. 2021 JOB NO. 6179-B FIG. 1



NOTE:
1) ADD A MINIMUM 0.27Q (PSF) HORIZONTAL SURCHARGE PRESSURE TO LAGGED PORTION OF WALL TO ACCOUNT FOR SURCHARGE EFFECTS FROM TRAFFIC AND OTHER LIVE (STORAGE LOADS).

2) ASSUMES HORIZONTAL SLOPE BEHIND AND IN FRONT OF WALL.

3) EARTH PRESSURES ACT OVER ENTIRE LAGGED PORTION OF WALL.

4) EARTH PRESSURES ACT OVER TWO PILE DIAMETERS BELOW LAGGED PORTION OF WALL.

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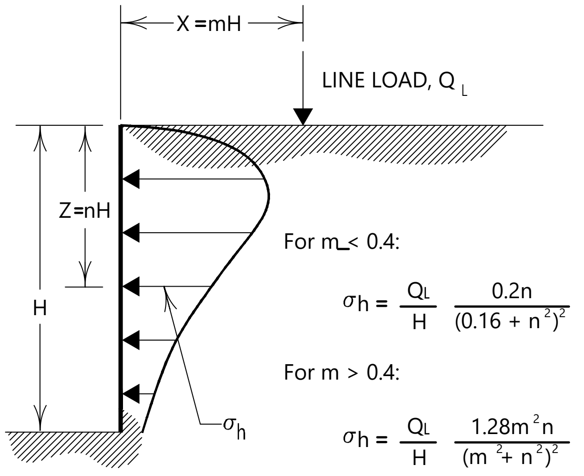
LATERAL EARTH PRESSURES (TIED-BACK WALL - STATIC)

OCT. 2021 JOB NO. 6179-B FIG. 2



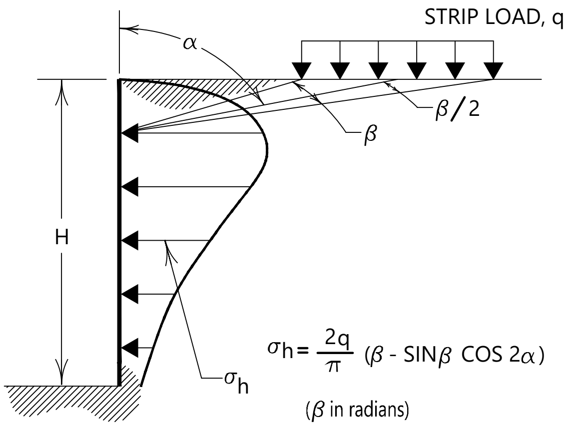
For $m \leq 0.4$:

$$\sigma_h = \frac{Q_L}{H} \frac{0.2n}{(0.16 + n^2)^2}$$

For $m > 0.4$:

$$\sigma_h = \frac{Q_L}{H} \frac{1.28m^2n}{(m^2 + n^2)^2}$$

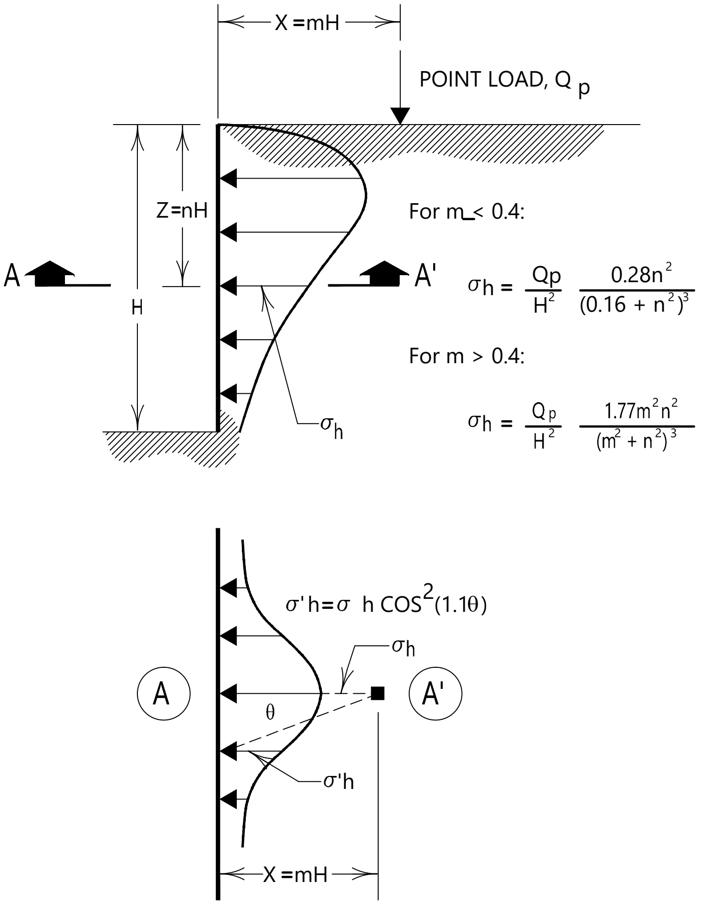
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{\pi} (\beta - \mathrm{SIN}\beta \ \mathrm{COS}\ 2\alpha)$$

(β in radians)

STRIP LOAD PARALLEL TO WALL



For $m \leq 0.4$:

$$\sigma_h = \frac{Q_p}{H^2} \frac{0.28n^2}{(0.16 + n^2)^3}$$

For $m > 0.4$:

$$\sigma_h = \frac{Q_p}{H^2} \frac{1.77m^2n^2}{(m^2 + n^2)^3}$$

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

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SURCHARGE-INDUCED LATERAL PRESSURE

OCT. 2021 JOB NO. 6179-B FIG. 3

Appendix H

Cost Estimates

PBS Engineering and Environmental
October 18, 2021

PRELIMINARY - COST ESTIMATE

Option A

SECTION
Rogue Brewery Seawall

COUNTY

DESCRIPTION OF WORK
Seawall Repair

DESIGNER
Nick Mincks

Reference	ITEM DESCRIPTION	UNIT	AMOUNT	UNIT COST	SUBTOTAL	TOTAL
ADDING STEEL PLATES					**$191,416.21**	
	MOBILIZATION	L.S.				$ 45,000.00
	7/8" Steel Plates	lbs.	9826	$ 3.00		$ 29,476.56
	Welding above & below water	Ton	4.91	$ 5,850.00		$ 28,739.65
	Dive crew	Days	15	$ 5,000.00		$ 75,000.00
	Barge rental	Month	1	$ 13,200.00		$ 13,200.00
OTHER ITEMS					**$686,518.40**	
	Soil stabilization	L.S.	1	$ 650,000		$ 650,000.00
	Pile corrosion coating	S.F.	622	$ 4.00		$ 2,486.40
	Pile surface preparation	S.F.	622	$ 20.00		$ 12,432.00
	Environmental controls	L.S.				$ 20,000.00
	Concrete repair - Pile Cap Spalling w/o Rebar	S.F.	20	$ 80.00		$ 1,600.00
ADDING STEEL OPTION						**$877,934.61**
	Permitting	L.S.				$ 50,000.00
	Design Engineering			15.0%		$ 131,690.19
	Engineering Support During Construction			5.0%		$ 43,896.73
	Construction Management & Inspection			6.0%		$ 52,676.08
	CONTINGINCIES			30.0%		$ 263,380.38
TOTAL CONSTRUCTION COST						**$1,419,577.99**

PRELIMINARY - COST ESTIMATE

Option B

SECTION
Rogue Brewery Seawall

COUNTY

DESCRIPTION OF WORK
Seawall Repair

DESIGNER
Nick Mincks

Reference	ITEM DESCRIPTION	UNIT	AMOUNT	UNIT COST	SUBTOTAL	TOTAL
ADDING TIE-BACK ANCHORS					**$747,196.56**	
	MOBILIZATION	L.S.	10%			$ 67,926.96
	Wale beam	L.F.	540	$ 275.00		$ 148,500.00
	Tie-back installation	EA	55	$ 7,000.00		$ 385,000.00
	Walkway pile alteration	EA	18	$ 1,987.20		$ 35,769.60
	Environmental controls	L.S.				$ 50,000.00
	Barge rental	Months	2	$ 30,000.00		$ 60,000.00
OTHER ITEMS					**$686,518.40**	
	Soil stabilization	L.S.	1	$ 650,000		$ 650,000.00
	Pile corrosion coating	S.F.	622	$ 4.00		$ 2,486.40
	Pile surface preparation	S.F.	622	$ 20.00		$ 12,432.00
	Environmental controls	L.S.				$ 20,000.00
	Concrete repair - Pile Cap Spalling w/o Rebar	S.F.	20	$ 80.00		$ 1,600.00
TIE-BACK OPTION						**$1,433,714.96**
	Permitting	L.S.				$ 50,000.00
	Design Engineering			15.0%		$ 215,057.24
	Engineering Support During Construction			5.0%		$ 71,685.75
	Construction Management & Inspection			8.0%		$ 114,697.20
	CONTINGINCIES			30.0%		$ 430,114.49
TOTAL CONSTRUCTION COST						**$2,315,269.64**

PRELIMINARY - COST ESTIMATE

SECTION **Rogue Brewery Seawall**				COUNTY		
	DESCRIPTION OF WORK **Seawall Repair (Dock Guide Piles)**			DESIGNER **Nick Mincks**		
Reference	**ITEM DESCRIPTION**	**UNIT**	**AMOUNT**	**UNIT COST**	**SUBTOTAL**	**TOTAL**
REPLACING GUIDE PILES FOR DOCK					**$132,415.00**	
	MOBILIZATION	L.S.				$ 40,000.00
	Steel Pipes	Each	18	$ 3,500.00		$ 63,000.00
	Dive crew	Days	5	$ 5,000.00		$ 25,000.00
	Barge rental	Week	1	$ 4,415.00		$ 4,415.00
SUB TOTAL						**$132,415.00**
	Permitting	LS				$ 15,000.00
	Design Engineering			10.0%		$ 13,241.50
	Engineering Support During Construction			5.0%		$ 6,620.75
	Construction Management & Inspection			6.0%		$ 7,944.90
	CONTINGINCIES			15.0%		$ 19,862.25
TOTAL CONSTRUCTION COST						**$195,084.40**