Geotechnical Engineering Report

50-Ton Boat Hoist Replacement Project
Port of Port Orford
300 Dock Road
Port Orford, Oregon

Prepared for:
Port of Port Orford
300 Dock Road
Port Orford, Oregon 97465

June 20, 2024
PBS Project 74353.000

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1 INTRODUCTION

1.1 General

This report presents results of PBS Engineering and Environmental LLC (PBS) geotechnical engineering services for the proposed 50-ton boat hoist replacement project located at the Port of Port Orford in Port Orford, Oregon (site). The general site location is shown on the Vicinity Map, Figure 1. The locations of PBS' explorations in relation to existing and proposed site features are shown on the Site Plan, Figure 2.

1.2 Purpose and Scope

The purpose of PBS' services was to develop geotechnical design and construction recommendations in support of the planned crane replacement project. This was accomplished by performing the following scope of services.

1.2.1 Literature and Records Review

PBS reviewed various published geologic maps of the area for information regarding geologic conditions and hazards at or near the site. PBS also reviewed previously completed reports for the project site and vicinity.

1.2.2 Subsurface Explorations

Two borings were advanced to depths of approximately 81.5 feet below the existing ground surface (bgs) within the development footprint. The borings were logged and representative soil samples collected by a member of the PBS geotechnical engineering staff. The approximate boring locations are shown on the Site Plan, Figure 2. The interpreted boring logs are presented as Figures A1 and A2 in Appendix A, Field Explorations.

1.2.3 Soils Testing

Soil samples were returned to our laboratory and classified in general accordance with the Unified Soil Classification System (ASTM D2487) and/or the Visual-Manual Procedure (ASTM D2488). Laboratory tests included natural moisture contents, grain-size analyses, and Atterberg limits. Laboratory test results are included in the exploration logs in Appendix A, Field Explorations; and in Appendix B, Laboratory Testing.

1.2.4 Geotechnical Engineering Analysis

Data collected during the subsurface exploration, literature research, and testing were used to develop site-specific geotechnical design parameters and construction recommendations.

1.2.5 Report Preparation

This Geotechnical Engineering Report summarizes the results of our explorations, testing, and analyses, including information relating to the following:

- Field exploration logs and site plan showing approximate exploration locations
- Laboratory test results
- Groundwater levels and considerations
- Liquefaction potential
- Deep foundation recommendations:
  - Minimum depth of embedment
  - Axial compression and uplift capacity
  - Soil parameters for lateral analyses
  - Construction considerations

- Seismic design criteria in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD)

1.3 Project Understanding

The Port of Port Orford (The Port), located on open ocean, lifts their boats in and out of the water daily using two cranes, which currently have 15-ton and 25-ton capacities. The Port is planning to replace these cranes with higher capacity 50-ton cranes. The existing cranes will be abandoned after the new cranes are installed but will remain operational until then.

The existing cranes are supported on 48-inch diameter steel piles. Based on the as-built drawings, the steel piles are built within the permanent dock structure and primarily appear to be end-bearing and transfer the vertical forces to bedrock. The steel piles are installed close (approximately 2 feet) to the sheet piles of the permanent dock structure. A reinforced concrete slab (concrete apron) is built at ground level (base of the crane structure).

PBS understands the proposed 50-ton cranes will each be supported by one 9-foot-diameter drilled shaft and a reinforced concrete cap. Previous iterations of design considered 3-, 3.5-, and 4-foot-diameter drilled shafts.

2 SITE CONDITIONS

2.1 Surface Description

The dock at the Port is roughly rectangular and is bordered to the west by the ocean, to the south and east by the rock jetty extending into the ocean, and to the north by Dock Road and the adjacent beach. The crane replacement project is focused on the east side of the dock where the current cranes exist. The majority of the area behind the dock is surfaced with asphalt concrete (AC) pavement including access drives and car, boat, and trailer parking. Based on review of available Oregon Department of Geology and Mineral Industries (DOGAMI) lidar, the Port has an elevation of approximately 24 feet where the cranes are located (NAVD88, DOGAMI, 2023).

2.2 Geologic Setting

The project area is located at the northwestern extent of the Klamath Mountains physiographic province, a mountainous region along the Pacific Ocean that extends from the Coast Range to the north into California to the south, and east to the foothills of the Cascade Mountains. This province is situated along the Cascadia Subduction Zone (CSZ) where oceanic rocks of the Juan de Fuca Plate are subducting beneath the North American Plate, resulting in deformation and uplift of the Coast Range, volcanism in the Cascade Range, and a clockwise rotation of the North American Plate (Wells et al., 2002).

Uplift of the Coast Range and Klamath Mountains is expressed as a north-south oriented, north-plunging anticline, formed by east-west compression due to subduction (Yeats et al., 1996). Younger, more active northwest-trending faults accommodate the clockwise rotation of the North American Plate (Brocher et al., 2017; USGS, 2023).

Basement rocks in the Klamath Mountains are typically composed of metamorphic and igneous rocks that formed in an oceanic setting and subsequently collided with the North American continent about 150 million years ago. Within the Coast Range to the north, Paleocene to Eocene accreted oceanic island arcs and oceanic plate fragments described as submarine tholeiitic basalt, pillow basalts, and submarine and subaerial alkali basalts are more common. Along much of coastal Oregon, these older accreted mafic rocks are overlain by younger marine sequences of sandstone and siltstone ranging in age from middle to late Eocene (Walker et al., 1991).

2.3 Local Geology

The site is mapped as underlain by modern Anthropocene fill and construction material (McClaughry et al., 2013). These man-made deposits are described as poorly sorted and crudely layered mixed gravel, sand, clay, and other engineered fill usually containing rounded to angular clasts. Beneath the fill, geologic mapping indicates Anthropocene beach deposits likely exist, which are described as unconsolidated, well-sorted sand and gravel deposited along active ocean beaches. Based on the geologic mapping, underlying the beach deposits are sedimentary rocks of the lower Cretaceous to upper Jurassic aged Otter Point Formation. This unit is described as dark- to greenish-gray, well-indurated, fine- to coarse-grained volcaniclastic sandstone, mudstone, and siltstone.

2.4 Subsurface Conditions

The site was explored by drilling two borings, designated B-1 and B-2, to depths of approximately 81.5 feet bgs. The drilling was performed by Western States Soil Conservation, Inc., of Hubbard, Oregon, using a truck-mounted CME-75 drill rig and mud rotary drilling techniques.

PBS has summarized the subsurface units as follows:

ASPHALT:	Approximately 8 inches of AC pavement was encountered at the ground surface in both borings.
GRAVEL FILL:	Variable fill consisting of brown/gray to orange, fine to coarse, subangular to subrounded silty gravel with fine- to coarse-grained sand was encountered beneath the asphalt in both borings and extended to a depth of up to approximately 30 feet bgs. The fill was generally moist to wet and medium dense, with the fine-grained portion of the fill exhibiting low plasticity. Two samples in B-1 were loose.
Silty SAND (SM), Poorly Graded SAND with Silt (SP-SM):	Gray to black sand with varying amounts of silt was encountered below the gravel fill and continued to approximately 55 feet bgs in both borings. The sand was generally wet, medium dense to dense, and ranged from fine to coarse grained. Wood was also frequently mixed in with the sand, especially from approximately 43 to 53 feet bgs in boring B-1, which may have been remnants of old pilings from the previous dock at the port.
Weathered SILTSTONE BEDROCK:	Otter Point Formation siltstone/mudstone bedrock was encountered beneath the sand. The siltstone was weathered and could be manually manipulated to lean clay with varying amounts of fine- to coarse-grained sand and fine to coarse, subangular to subrounded gravel. The weathered siltstone was hard and exhibited low to medium plasticity. The degree of weathering decreased and the hardness increased with depth.
SILTSTONE BEDROCK:	Unweathered Otter Point Formation siltstone/mudstone bedrock was encountered beneath the weathered siltstone, typically at a depth of approximately 75 feet bgs. The siltstone consisted of hard, gravelly lean clay and exhibited low to medium plasticity.

2.5 Groundwater

Static groundwater was estimated to be at approximately 15 feet bgs based on the saturation of the SPT samples below this depth during our explorations. Groundwater is assumed to be correlated with ocean tides and sea levels adjacent to the port. Please note that levels can fluctuate during the year depending on climate, irrigation season, extended periods of precipitation, drought, and other factors.

3 CONCLUSIONS AND RECOMMENDATIONS

3.1 Geotechnical Design Considerations

The subsurface conditions at the site consist of gravel fill overlying sand and siltstone bedrock. Based on our observations and analyses, the new 50-ton cranes may be supported on drilled shaft foundations that extend through the potentially liquefiable soils and derive their capacity from embedded into the weathered siltstone and siltstone bedrock. Excavation of the surface soils (gravel fill) with conventional equipment is feasible at the site.

3.2 Seismic Design Considerations

3.2.1 Code-Based Seismic Design Parameters

Seismic design criteria for the project are based on the 2020 American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (BDS) (AASHTO, 2020).

The AASHTO BDS response spectrum for design is based on local seismicity and soil conditions. The seismicity is represented by the acceleration coefficient, A_s, which represents the peak ground acceleration (PGA) based on established seismic risk models adjusted for site conditions.

The USGS completed regional probabilistic ground motion studies to establish the PGA for various recurrence intervals equating to 7% occurrence in 75 years (approximately a 975-year return period event) (USGS, 2008).

The AASHTO BDS expresses the effects of site-specific subsurface conditions on the ground motion response in terms of *site coefficients*. The site coefficient accounts for the seismic response of the soil profile and is based on the density and stiffness of the soil profile underlying the site. The soil type can be correlated to the average standard penetration test (SPT) resistance (N-value) in the upper 100 feet of the soil profile. We characterize the site as AASHTO Site Class D.

AASHTO BDS site coefficients for Site Class D have been utilized to adjust the mapped PGA and spectral accelerations at periods of 0.2 (S_S) and 1.0 (S_1) seconds at the site, as shown in Table 1.

Table 1. 2020 AASHTO Seismic Design Parameters

Parameter	Short Period	1 Second
Spectral Acceleration	S_s = 1.717 g	S_1 = 0.629 g
Site Class	D	
Site Coefficient	F_a = 1.0	F_v = 1.5
Design Spectral Response Acceleration Parameters	S_{DS} = 1.717 g	S_{D1} = 0.944 g
Peak Ground Acceleration	PGA = 0.810 g	
Site Coefficient for Peak Ground Acceleration	F_{PGA} = 1.0	
Effective Peak Ground Acceleration	As = 0.810 g	

g= Acceleration due to gravity

3.2.2 Liquefaction and Lateral Spreading Evaluation

Liquefaction is defined as a decrease in the shear resistance of loose, saturated, cohesionless soil (e.g., sand) or low plasticity silt soils, due to the buildup of excess pore pressures generated during an earthquake. This results in a temporary transformation of the soil deposit into a viscous fluid. Liquefaction can result in ground settlement, foundation bearing capacity failure, and lateral spreading of ground.

Based on a review of the Oregon Statewide Geohazard Viewer (HazVu), the site is shown as having a high liquefaction hazard. Based on the results of our analyses, 6 to 12 inches of total liquefaction settlement, and approximately 3 to 6 inches of differential liquefaction settlement could occur as the result of a code-based earthquake. The resulting liquefaction will cause downdrag loads on drilled shaft foundations.

Assuming the sheet pile bulkhead fails, lateral spreading and flow failure, characterized as several inches to several feet of vertical and lateral movement toward the ocean, will occur after the code-based earthquake once liquefaction is initiated. The lateral movement of the non-liquefied soil crust and underlying liquefied soil will exert lateral pressures on the cap and drilled shafts. We recommend applying a triangular pressure distribution with equivalent fluid density of 375 pounds per cubic foot (pcf) to represent the non-liquefied crust layer acting against the full height and width of pile caps and over the full diameter of each drilled shaft, applied from top of shaft to a depth of 15 feet below the existing deck elevation. Below the non-liquefied crust layer, we recommend applying a trapezoidal pressure distribution with equivalent fluid density of 60 pcf acting over the diameter of each drilled shaft to represent liquefied soil flowing against and past the drilled shafts from a depth of 15 feet (top of the liquefied zone) to a depth of 50 feet (bottom of the liquefied zone). The lateral pressure ordinate at the top of the liquefied zone should be taken as 900 pounds per square foot (psf) and increase linearly to 3,000 psf at the bottom of the liquefied zone. This loading should be analyzed in conjunction with the post-inertial liquefaction lateral soil profile (LPILE) shown on Table 3.

3.2.3 Drilled Shafts

Due to the relatively high loads associated with support of the new cranes, the proposed new cranes should be supported on deep foundations that derive the majority of their capacity from the underlying weathered siltstone and siltstone bedrock. PBS completed analyses to evaluate the axial capacity of drilled shafts supporting the proposed 50-ton cranes. We considered a minimum pile embedment of 10 feet into the weathered siltstone with no capacity from the overlying potentially liquefiable soils. The actual length of the drilled shafts should also consider lateral pile loading (including lateral spreading loads) and the need to establish fixity, which may result in longer shafts than required for axial compressive or uplift resistance.

3.2.3.1 Axial Compressive Resistance for Drilled Shafts

We analyzed 9-foot-diameter drilled shafts. Detailed results of our analyses are presented as axial resistance versus depth on Figure 3. The results of our axial resistance analyses are presented for the *service, strength,* and *extreme limit* states for a single drilled shaft. The *service limit* state resistance shown assumes an axial compression settlement of up to 1 inch.

Downdrag is the force associated with negative skin friction on shafts. Downdrag resulting from liquefaction may develop along the shaft above depths of 50 feet and should be considered as a load for the *extreme limit* state cases. The unfactored downdrag load is estimated in Table 2 below.

Table 2. Downdrag Loads

Drilled Shaft Diameter (feet)	Unfactored Downdrag Load (kips)
9.0	525

The drilled shaft axial compressive and uplift resistances assume a minimum center-to-center spacing of 3 diameters. Calculated capacities are based on soil support capacities and do not consider the ultimate structural capacity of the drilled shaft; therefore, we recommend that the structural engineer check the allowable stress capacity of the shafts.

3.2.4 LPILE Parameters

We anticipate the lateral loading of drilled shafts will be evaluated using the software LPILE by Ensoft, assuming that the drilled shafts are spaced at least 3 diameters (center-to-center) apart. A summary of recommended input parameters for the static and inertial condition as well as the post-inertial (liquefied) condition are provided in Tables 3 and 4, respectively. The top of shaft elevations should be considered when developing the LPILE soil profile, as these were provided from the existing ground surface.

Table 3. Static and Inertial Condition LPILE Input Parameters

Soil Layer	Depth (feet bgs)	L-Pile Model	Effective Weight, γ (pcf)	Friction Angle, ϕ (deg)	p-y Modulus (pci)
GP-GM Fill	0 – 15	Sand (Reese)	115	32	75
Saturated GP-GM Fill	15 – 30	Sand (Reese)	57.6	32	50
SP-SM Alluvium	30 – 55	Sand (Reese)	57.6	30	25
Weathered Siltstone	55 – 81.5	Sand (Reese)	72.6	38	125

Table 4. Post-Inertial (Liquefied) Condition LPILE Input Parameters

Soil Layer	Depth (feet bgs)	L-Pile Model	Effective Weight, γ (pcf)	Friction Angle, ϕ (deg)	p-y Modulus (pci)	Soil Resistance, p (lbs/in)
GP-GM Fill	0 – 15	Sand (Reese)	115	32	75	NA
Saturated GP-GM Fill	15 – 30	User Input p-y Curves	57.6	NA	NA	0.1
SP-SM Alluvium	30 – 50	User Input p-y Curves	57.6	NA	NA	0.1
SP-SM Alluvium	50 – 55	Sand (Reese)	57.6	30	25	NA
Weathered Siltstone	55 – 81.5	Sand (Reese)	72.6	38	125	NA

The lateral resistance from the first three soil layers representing the upper 55 feet (GP-GM Fill, Saturated GP-GM Fill, and SP-SM Alluvium) should be disregarded for the front drilled shaft immediately adjacent to the existing sheet pile bulkhead. This will reduce the lateral pressures imparted by the crane foundations on the existing sheet piles. In our opinion, the reduction in lateral earth pressures by installing the 9-foot-diameter drilled shaft close to the sheets along with the limited anticipated deflection of the shafts should result in lateral loading of the existing sheet piles from the new crane foundations equal to or less than what the sheet piles have previously experienced.

To account for group effects, lateral resistance for single, isolated shafts should be reduced by applying the load-reduction factors summarized in the following Table 5. Lateral load reduction factors should be applied to shafts where the spacing between adjacent shafts is less than 5 shaft diameters (center-to-center) but greater than 3 diameters.

Table 5. Lateral Group Action Reduction Factors

Pile Spacing*	Load-Reduction Factor		
	Row 1	Row 2	Row 3 and Higher
3B	0.8	0.4	0.3
5B	1.0	0.85	0.7

* In the direction of loading
B=pile or shaft diameter
From AASHTO LRFD Table 10.7.2.4-1

3.3 New Pavement

Approximately 8 inches of AC pavement was encountered at the ground surface in both borings. We understand that new pavements will likely match existing AC pavement thicknesses.

The asphalt cement binder should be selected following ODOT SS 00744.11 – Asphalt Cement and Additives. The AC should consist of ½-inch hot mix asphalt concrete (HMAC) with a maximum lift thickness of 3 inches. The AC should conform to ODOT SS 00744.13 and 00744.14 and be compacted to 91% of the maximum theoretical density (Rice value) of the mix, as determined in accordance with ASTM D2041.

We recommend construction traffic not be allowed on new pavements, or that the contractor take appropriate precautions to protect the subgrade and pavement during construction.

4 CONSTRUCTION RECOMMENDATIONS

4.1 Site Preparation

Construction of the proposed new cranes may involve clearing demolition of possible existing structures. Demolition should include removing existing pavement, utilities, etc., throughout the proposed new crane area. Underground utility lines or other abandoned structural elements should also be removed. The voids resulting from removal of foundations or loose soil in utility lines should be backfilled with compacted structural fill. The base of these excavations should be excavated to stiff, native subgrade before filling, with sides sloped at a minimum of 1H:1V (horizontal to vertical) to allow for uniform compaction. Materials generated during demolition should be transported off site or stockpiled in areas designated by the owner's representative.

4.1.1 Proofrolling/Subgrade Verification

Following site preparation and prior to placing aggregate base over pavement subgrades, the exposed subgrade should be evaluated either by proofrolling or another method of subgrade verification. The subgrade should be proofrolled with a fully loaded dump truck or similar heavy, rubber-tire construction equipment to identify unsuitable areas. If evaluation of the subgrades occurs during wet conditions, or if proofrolling the subgrades will result in disturbance, they should be evaluated by PBS using a steel foundation probe. We recommend that PBS be retained to observe the proofrolling and perform the subgrade verifications. Unsuitable areas identified during the field evaluation should be compacted to a stiff condition or be excavated and replaced with structural fill.

4.1.2 Wet/Freezing Weather and Wet Soil Conditions

Due to the presence of fine-grained silt and sands in the near-surface materials at the site, construction equipment may have difficulty operating on the near-surface soils when the moisture content of the surface soil is more than a few percentage points above the optimum moisture required for compaction. Soils disturbed during site preparation activities, or unsuitable areas identified during proofrolling or probing, should be removed and replaced with compacted structural fill.

Site earthwork and subgrade preparation should not be completed during freezing conditions, except for mass excavation to the subgrade design elevations.

Protection of the subgrade is the responsibility of the contractor. Construction of granular haul roads to the project site entrance may help reduce further damage to the pavement and disturbance of site soils. The actual thickness of haul roads and staging areas should be based on the contractors' approach to site development, and the amount and type of construction traffic. The imported granular material should be placed in one lift over the prepared undisturbed subgrade and compacted using a smooth-drum, non-vibratory roller. A geotextile fabric should be used to separate the subgrade from the imported granular material in areas of repeated construction traffic. The geotextile should meet the specifications of ODOT SS Section 02320.10 and SS 02320.20, Table 02320-4 for soil separation. The geotextile should be installed in conformance with ODOT SS Section 00350 – Geosynthetic Installation.

4.2 Drilled Shafts

The installation procedures should follow the ODOT SS Section 00512 with appropriate special provisions to address the unique aspects of the site conditions and design approach for the drilled shaft foundations. The key issues for the drilled shaft installation are summarized below.

4.2.1 Soil and Rock Drilling

Drilled shafts will require drilling through soil and weathered rock. Drilling equipment and techniques need to be capable of excavating and removing variably weathered rock below groundwater. Temporary casing is not anticipated in the rock. However, local caving may occur, especially when penetrating soils below the groundwater table. This may require temporary casing above the rock, or drilling slurry, at the contractor's option.

4.2.2 Ground Disturbance

Effort must be implemented to prevent disturbance to the ground surface during drilling, and to remove disturbed rock and soil after drilling and prior to placement of concrete. As a minimum, either drilling slurry or temporary casing should be anticipated for shafts drilled in fill soils or soils below groundwater.

4.2.3 Shaft End Bearing Condition

Appropriate shaft construction should provide a reasonably clean bearing surface at the base of the shaft. If necessary, the contractor should use appropriate means such as a cleanout bucket or air lift to clean the bottom of the excavation of all shafts. No more than 2 inches of loose or disturbed material should be present at the bottom of the shaft prior to placing concrete. The excavated shaft should be inspected and accepted by the design engineer prior to proceeding with construction.

4.2.4 Shaft Quality Control

Methods to confirm shaft cross sectional integrity and tolerances along the full depth of shafts should be implemented. For in situ quality control testing, we recommend that ODOT Standard Specifications be followed, with special provisions for crosshole sonic log (CSL) testing in accordance with ASTM D6760 performed on each shaft. The requirement to test each shaft should be included in the special provisions. Per the procedures discussed in ASTM D6760, a minimum of one access duct for every 0.25 to 0.30 m (0.8 to 1.0 foot) of shaft diameter, with a minimum of three, spaced equally around the circumference, should be installed in each shaft. The testing and interpretation of results could be performed under the direction of the Construction Manager; however, we recommend that the testing be performed by a pre-approved CSL specialty subcontractor. During construction, we recommend full-time observation by a qualified representative from the design team to log the activities, observe subsurface conditions encountered, record and evaluate quantities of materials excavated and backfilled, and monitor key activities. We assume periodic visits of the design geotechnical and structural engineers of record will be made.

4.2.5 Shaft Casing

Based on the subsurface conditions present, casing may be necessary during excavation. The capacities provided in this report assume that if casing is used, it will be removed after drilled shaft installation. However, if casing is not removed, PBS should be consulted to update the drilled shaft capacities provided in this report. Permanent casing would reduce downdrag and lateral spreading loads.

4.3 Excavation

The near-surface soils at the site can be excavated with conventional earthwork equipment. Sloughing and caving should be anticipated. All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. The contractor is solely responsible for adherence to the OSHA requirements. Trench cuts should stand relatively vertical to a depth of approximately 4 feet bgs, provided no groundwater seepage is present in the trench walls. Open excavation techniques may be used provided the excavation is configured in accordance with the OSHA requirements, groundwater seepage is not present, and with the understanding that some sloughing may occur. Trenches/excavations should be flattened if sloughing occurs or seepage is present. Use of a trench shield or other approved temporary shoring is recommended if vertical walls are desired for cuts deeper than 4 feet bgs. If dewatering is used, we recommend that the type and design of the dewatering system be the responsibility of the contractor, who is in the best position to choose systems that fit the overall plan of operation.

4.4 Structural Fill

General site grading is not anticipated. Structural fill should be placed over subgrade that has been prepared in conformance with the Site Preparation and Wet/Freezing Weather and Wet Soil Conditions sections of this report. Structural fill material should consist of relatively well-graded soil, or an approved rock product that is free of organic material and debris, and contains particles not greater than 4 inches nominal dimension.

The suitability of soil for use as compacted structural fill will depend on the gradation and moisture content of the soil when it is placed. As the amount of fines (material finer than the US Standard No. 200 Sieve) increases, soil becomes increasingly sensitive to small changes in moisture content and compaction becomes more difficult to achieve. Soils containing more than about 5% fines cannot consistently be compacted to a dense, non-yielding condition when the water content is significantly greater (or significantly less) than optimum.

If fill and excavated material will be placed on slopes steeper than 5H:1V, these must be keyed/benched into the existing slopes and installed in horizontal lifts. Vertical steps between benches should be approximately 2 feet.

4.4.1 On-Site Soil

On-site soils encountered in our explorations are generally not suitable for placement as structural fill. The fine-grained fraction of the site soils are moisture sensitive, and may become unworkable because of excess moisture content.

4.4.2 Borrow Material

Borrow material for general structural fill construction should meet the requirements set forth in ODOT SS 00330.12 – Borrow Material. When used as structural fill, borrow material should be placed in lifts with a maximum uncompacted thickness of approximately 8 inches and compacted to not less than 92% of the maximum dry density, as determined by ASTM D1557.

4.4.3 Select Granular Fill

Selected granular backfill used during periods of wet weather for structural fill construction should meet the specifications provided in ODOT SS 00330.14 – Selected Granular Backfill. The imported granular material should be uniformly moisture conditioned to within about 2% of the optimum moisture content and compacted in relatively thin lifts using suitable mechanical compaction equipment. Selected granular backfill should be placed in lifts with a maximum uncompacted thickness of 8 to 12 inches and be compacted to not less than 95% of the maximum dry density, as determined by ASTM D1557.

4.4.4 Crushed Aggregate Base

Crushed aggregate base course below floor slabs, spread footings, and asphalt concrete pavements should be clean crushed rock or crushed gravel that contains no deleterious materials and meets the specifications provided in ODOT SS 02630.10 – Dense-Graded Aggregate, and has less than 5% by dry weight passing the US Standard No. 200 Sieve. The crushed aggregate base course should be placed in lifts with a maximum uncompacted thickness of 8 to 12 inches and be compacted to at least 95% of the maximum dry density, as determined by ASTM D1557.

4.4.5 Utility Trench Backfill

Pipe bedding placed to uniformly support the barrel of pipe should meet specifications provided in ODOT SS 00405.12 – Bedding. The pipe zone that extends from the top of the bedding to at least 8 inches above utility lines should consist of material prescribed by ODOT SS 00405.13 – Pipe Zone Material. The pipe zone material should be compacted to at least 90% of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer.

Under pavements, paths, slabs, or beneath building pads, the remainder of the trench backfill should consist of well-graded granular material with less than 10% by dry weight passing the US Standard No. 200 Sieve, and should meet standards prescribed by ODOT SS 00405.14 – Trench Backfill, Class B or D. This material should be compacted to at least 92% of the maximum dry density, as determined by ASTM D1557 or as required by the pipe manufacturer. The upper 2 feet of the trench backfill should be compacted to at least 95% of the maximum dry density, as determined by ASTM D1557. Controlled low-strength material (CLSM), ODOT SS 00405.14 – Trench Backfill, Class E, can be used as an alternative.

Outside of structural improvement areas (e.g., pavements, sidewalks, or building pads), trench material placed above the pipe zone may consist of general structural fill materials that are free of organics and meet ODOT SS

00405.14 – Trench Backfill, Class A. This general trench backfill should be compacted to at least 90% of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local jurisdictions.

4.4.6 Stabilization Material

Stabilization rock should consist of pit or quarry run rock that is well-graded, angular, crushed rock consisting of 4- or 6-inch-minus material with less than 5% passing the US Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material. ODOT SS 00330.16 – Stone Embankment Material can be used as a general specification for this material with the stipulation of limiting the maximum size to 6 inches.

5 ADDITIONAL SERVICES AND CONSTRUCTION OBSERVATIONS

In most cases, other services beyond completion of a final geotechnical engineering report are necessary or desirable to complete the project. Occasionally, conditions or circumstances arise that require additional work that was not anticipated when the geotechnical report was written. PBS offers a range of environmental, geological, geotechnical, and construction services to suit the varying needs of our clients.

PBS should be retained to review the plans and specifications for this project before they are finalized. Such a review allows us to verify that our recommendations and concerns have been adequately addressed in the design.

Satisfactory earthwork performance depends on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. We recommend that PBS be retained to observe general excavation, fill placement, and shaft installation. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

6 LIMITATIONS

This report has been prepared for the exclusive use of the addressee, and their architects and engineers, for aiding in the design and construction of the proposed development and is not to be relied upon by other parties. It is not to be photographed, photocopied, or similarly reproduced, in total or in part, without express written consent of the client and PBS. It is the addressee's responsibility to provide this report to the appropriate design professionals, building officials, and contractors to ensure correct implementation of the recommendations.

The opinions, comments, and conclusions presented in this report are based upon information derived from our literature review, field explorations, laboratory testing, and engineering analyses. It is possible that soil, rock, or groundwater conditions could vary between or beyond the points explored. If soil, rock, or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that PBS is notified immediately so that we may reevaluate the recommendations of this report.

Unanticipated fill, soil and rock conditions, and seasonal soil moisture and groundwater variations are commonly encountered and cannot be fully determined by merely taking soil samples or completing explorations such as soil borings. Such variations may result in changes to our recommendations and may require additional funds for expenses to attain a properly constructed project; therefore, we recommend a contingency fund to accommodate such potential extra costs.

The scope of work for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

If there is a substantial lapse of time between the submission of this report and the start of work at the site, if conditions have changed due to natural causes or construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, this report should be reviewed to determine the applicability of the conclusions and recommendations presented herein. Land use, site conditions (both on and off site), or other factors may change over time and could materially affect our findings; therefore, this report should not be relied upon after three years from its issue, or in the event that the site conditions change.

7 REFERENCES

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Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL
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ASSOCIATION

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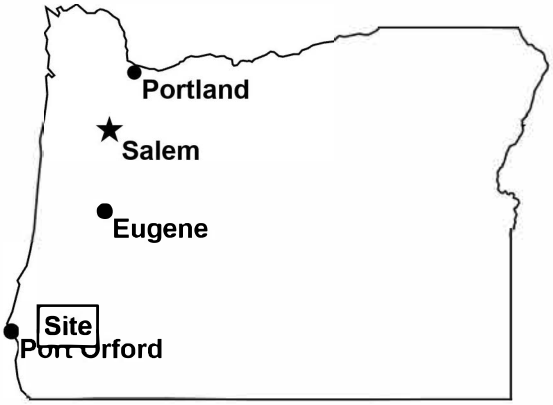
Figures



VICINITY MAP

50-TON BOAT HOIST REPLACEMENT PROJECT
PORT ORFORD, OREGON

DATE: JUN 2024 · PROJECT: 74353.000

PBS

FIGURE

1

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EXPLANATION

◐ B-1 - Boring name and approximate location

Notes: Google Earth 2019 imagery

Coordinate System: NAD 1983 2011 StatePlane
Oregon South FIPS 3602 Ft Intl

SCALE: 1 inch = 83 feet

0 50 100 200 feet

SITE PLAN

50-TON BOAT HOIST
REPLACEMENT PROJECT
PORT ORFORD, OREGON

DATE: JUN 2024 · PROJECT: 74353.000

PBS

FIGURE

2

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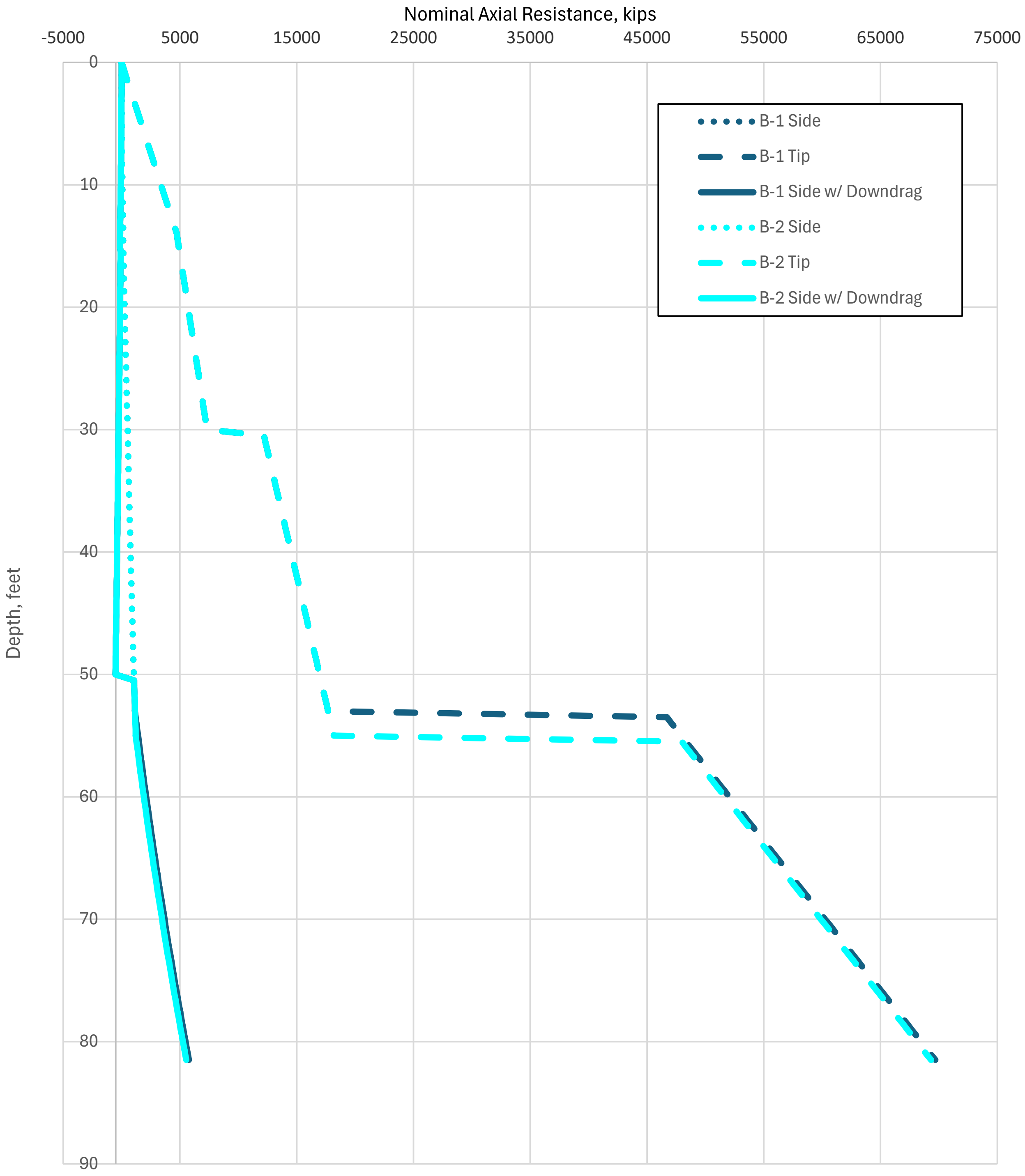


NOTES:

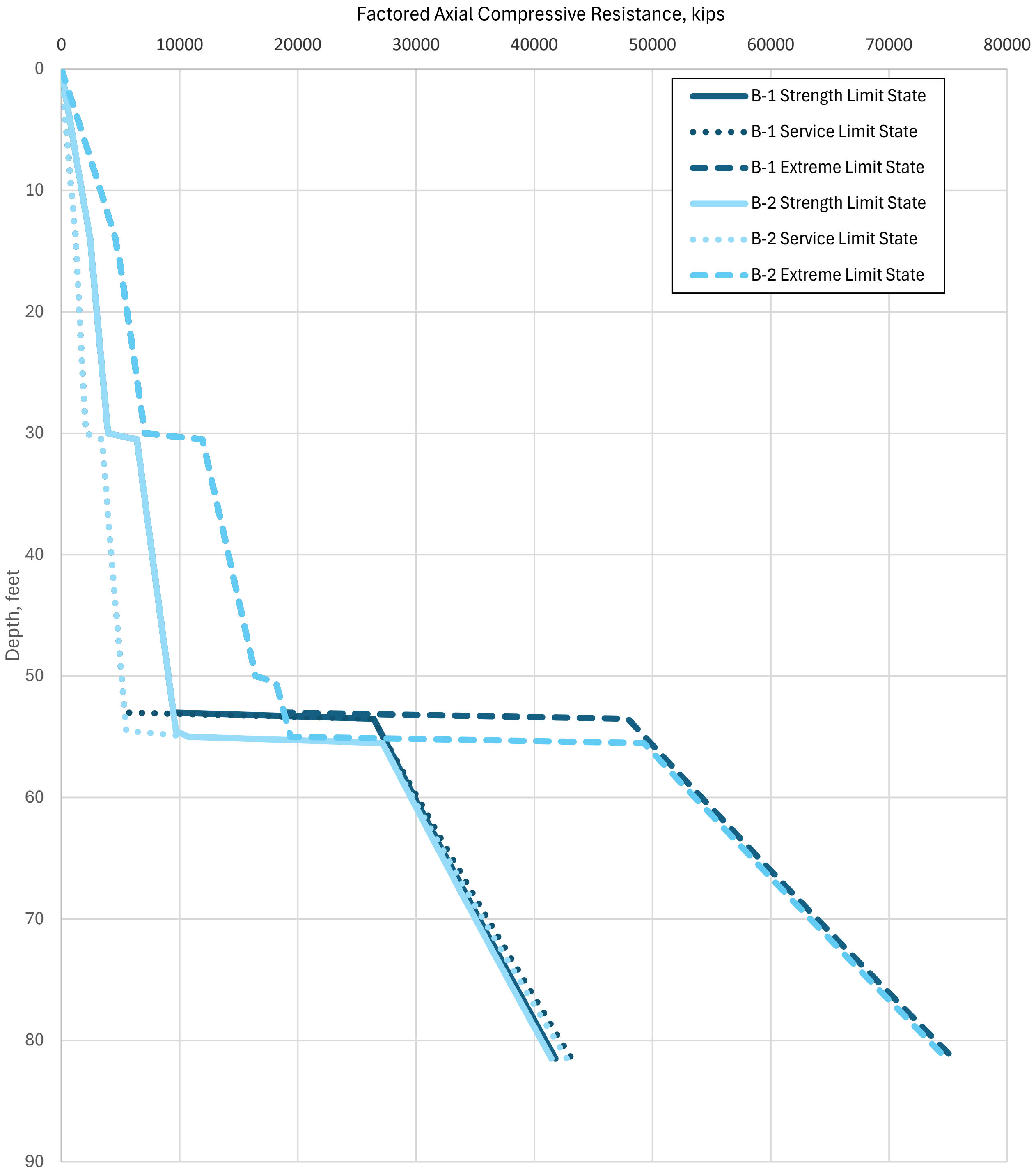
1. Piles must be embedded a minimum of 10 feet into the weathered siltstone, corresponding to minimum depths of 63 feet and 65 feet in borings B-1 and B-2, respectively.
2. A maximum downdrag load of 525 kips should be considered.

PBS
AN APEX COMPANY

9' DIAMETER DRILLED SHAFT NOMINAL AXIAL RESISTANCE

50-ton Boat Hoist Replacement Project
PORT ORFORD, OREGON

JUNE 2024
74353.000
FIGURE
3A



NOTES:

1. Piles must be embedded a minimum of 10 feet into the weathered siltstone, corresponding to minimum depths of 63 feet and 65 feet in borings B-1 and B-2, respectively.
2. A maximum downdrag load of 525 kips should be considered.

PBS
AN APEX COMPANY

9' DIAMETER DRILLED SHAFT AXIAL COMPRESSIVE RESISTANCE

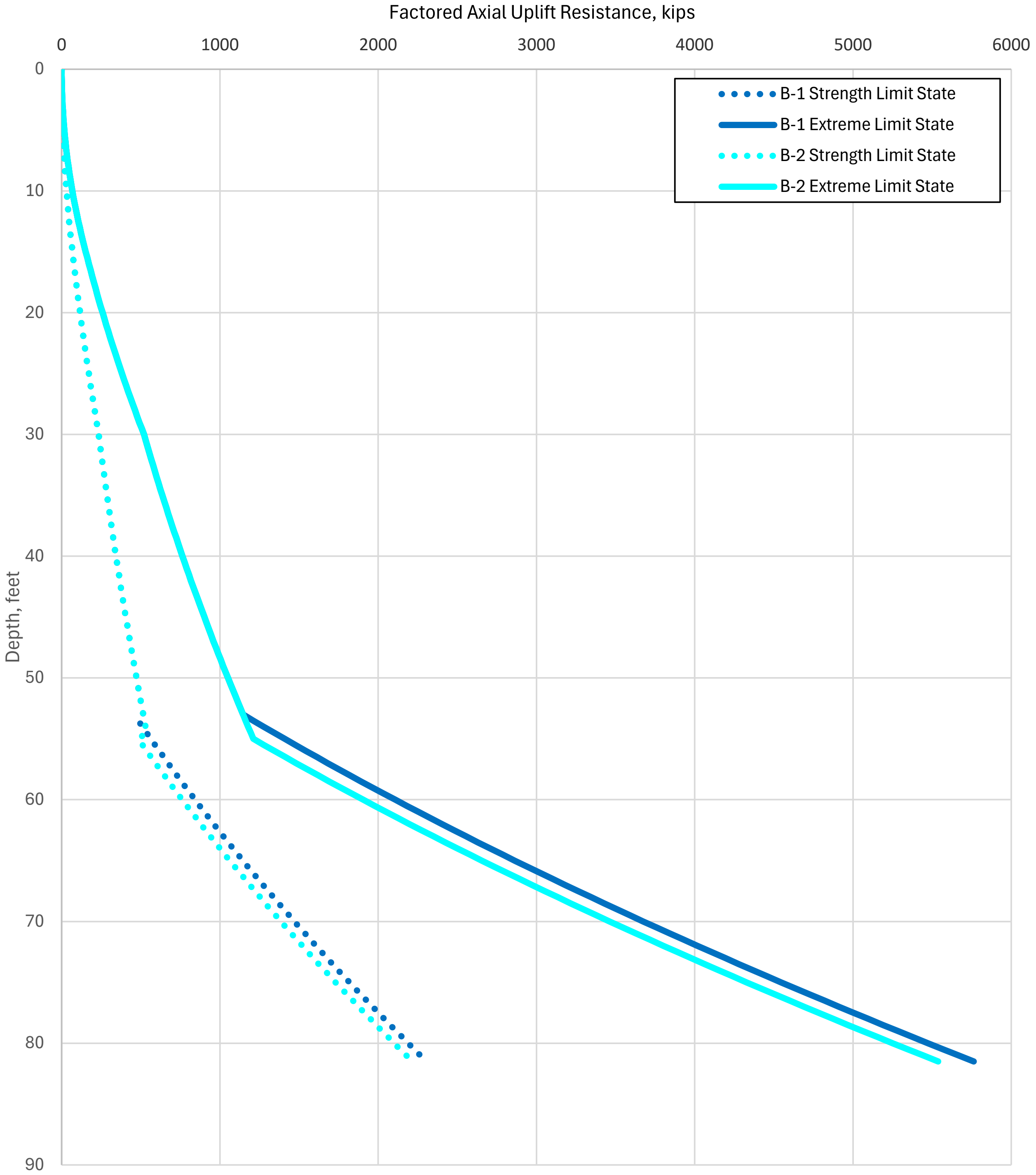
50-ton Boat Hoist Replacement Project

PORT ORFORD, OREGON

JUNE 2024
74353.000

FIGURE

3B



NOTES:

1. Piles must be embedded a minimum of 10 feet into the weathered siltstone, corresponding to minimum depths of 63 feet and 65 feet in borings B-1 and B-2, respectively.
2. A maximum downdrag load of 525 kips should be considered.

PBS
AN APEX COMPANY

9' DIAMETER DRILLED SHAFT AXIAL UPLIFT RESISTANCE

50-ton Boat Hoist Replacement Project

PORT ORFORD, OREGON

JUNE 2024
74353.000

FIGURE

3C

Appendix A

Field Explorations

Appendix A: Field Explorations

A1 GENERAL

PBS explored subsurface conditions at the project site by advancing two borings to depths of up to approximately 81.5 feet bgs on December 18 through 20, 2023. The approximate locations of the explorations are shown on Figure 2, Site Plan. The procedures used to advance the borings, collect samples, and other field techniques are described in detail in the following paragraphs. Unless otherwise noted, all soil sampling and classification procedures followed engineering practices in general accordance with relevant ASTM procedures. "General accordance" means that certain local drilling/excavation and descriptive practices and methodologies have been followed.

A2 BORINGS

A2.1 Drilling

Borings were advanced using a truck-mounted CME-75 drill rig provided and operated by Western States Soil Conservation, Inc., of Hubbard, Oregon, using mud rotary drilling techniques. The borings were observed by a member of the PBS geotechnical staff, who maintained a detailed log of the subsurface conditions and materials encountered during the course of the work.

A2.2 Sampling

Disturbed soil samples were taken in the borings at selected depth intervals. The samples were obtained using a standard 2-inch outside diameter, split-spoon sampler following procedures prescribed for the standard penetration test (SPT). Using the SPT, the sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance (N-value). The N-value provides a measure of the relative density of granular soils such as sands and gravels, and the consistency of cohesive soils such as clays and plastic silts. The disturbed soil samples were examined by a member of the PBS geotechnical staff and then sealed in plastic bags for further examination and physical testing in our laboratory.

A2.3 Boring Logs

The boring logs show the various types of materials that were encountered in the borings and the depths where the materials and/or characteristics of these materials changed, although the changes may be gradual. Where material types and descriptions changed between samples, the contacts were interpreted. The types of samples taken during drilling, along with their sample identification number, are shown to the right of the classification of materials. The N-values and natural water (moisture) contents are shown farther to the right.

A3 MATERIAL DESCRIPTION

Initially, samples were classified visually in the field. Consistency, color, relative moisture, degree of plasticity, and other distinguishing characteristics of the soil samples were noted. Afterward, the samples were reexamined in the PBS laboratory, various standard classification tests were conducted, and the field classifications were modified where necessary. The terminology used in the soil classifications and other modifiers are defined in Table A-1, Terminology Used to Describe Soil.

Table A-1
Terminology Used to Describe Soil
1 of 2

Soil Descriptions

Soils exist in mixtures with varying proportions of components. The predominant soil, i.e., greater than 50 percent based on total dry weight, is the primary soil type and is capitalized in our log descriptions (SAND, GRAVEL, SILT, or CLAY). Smaller percentages of other constituents in the soil mixture are indicated by use of modifier words in general accordance with the ASTM D2488-06 Visual-Manual Procedure. "General Accordance" means that certain local and common descriptive practices may have been followed. In accordance with ASTM D2488-06, group symbols (such as GP or CH) are applied on the portion of soil passing the 3-inch (75mm) sieve based on visual examination. The following describes the use of soil names and modifying terms used to describe fine- and coarse-grained soils.

Fine-Grained Soils (50% or greater fines passing 0.075 mm, No. 200 sieve)
The primary soil type, i.e., SILT or CLAY is designated through visual-manual procedures to evaluate soil toughness, dilatency, dry strength, and plasticity. The following outlines the terminology used to describe fine-grained soils, and varies from ASTM D2488 terminology in the use of some common terms.

Primary soil NAME, Symbols, and Adjectives			Plasticity Description	Plasticity Index (PI)
SILT (ML & MH)	**CLAY (CL & CH)**	**ORGANIC SOIL (OL & OH)**		
SILT		Organic SILT	Non-plastic	0 – 3
SILT		Organic SILT	Low plasticity	4 – 10
SILT/Elastic SILT	Lean CLAY	Organic SILT/ Organic CLAY	Medium Plasticity	10 – 20
Elastic SILT	Lean/Fat CLAY	Organic CLAY	High Plasticity	20 – 40
Elastic SILT	Fat CLAY	Organic CLAY	Very Plastic	>40

Modifying terms describing secondary constituents, estimated to 5 percent increments, are applied as follows:

Description	% Composition	
With Sand	% Sand ≥ % Gravel	15% to 25% plus No. 200
With Gravel	% Sand < % Gravel	
Sandy	% Sand ≥ % Gravel	≤30% to 50% plus No. 200
Gravelly	% Sand < % Gravel	

Borderline Symbols, for example CH/MH, are used when soils are not distinctly in one category or when variable soil units contain more than one soil type. **Dual Symbols,** for example CL-ML, are used when two symbols are required in accordance with ASTM D2488.

Soil Consistency terms are applied to fine-grained, plastic soils (i.e., PI ≥ 7). Descriptive terms are based on direct measure or correlation to the Standard Penetration Test N-value as determined by ASTM D1586-84, as follows. SILT soils with low to non-plastic behavior (i.e., PI < 7) may be classified using relative density.

Consistency Term	SPT N-value	Unconfined Compressive Strength	
		tsf	kPa
Very soft	Less than 2	Less than 0.25	Less than 24
Soft	2 – 4	0.25 – 0.5	24 – 48
Medium stiff	5 – 8	0.5 – 1.0	48 – 96
Stiff	9 – 15	1.0 – 2.0	96 – 192
Very stiff	16 – 30	2.0 – 4.0	192 – 383
Hard	Over 30	Over 4.0	Over 383

Soil Descriptions

Coarse - Grained Soils (less than 50% fines)

Coarse-grained soil descriptions, i.e., SAND or GRAVEL, are based on the portion of materials passing a 3-inch (75mm) sieve. Coarse-grained soil group symbols are applied in accordance with ASTM D2488-06 based on the degree of grading, or distribution of grain sizes of the soil. For example, well-graded sand containing a wide range of grain sizes is designated SW; poorly graded gravel, GP, contains high percentages of only certain grain sizes. Terms applied to grain sizes follow.

Material NAME	Particle Diameter	
	Inches	Millimeters
SAND (SW or SP)	0.003 – 0.19	0.075 – 4.8
GRAVEL (GW or GP)	0.19 – 3	4.8 – 75
Additional Constituents:		
Cobble	3 – 12	75 – 300
Boulder	12 – 120	300 – 3050

The primary soil type is capitalized, and the fines content in the soil are described as indicated by the following examples. Percentages are based on estimating amounts of fines, sand, and gravel to the nearest 5 percent. Other soil mixtures will have similar descriptive names.

Example: Coarse-Grained Soil Descriptions with Fines

>5% to < 15% fines (Dual Symbols)	≥15% to < 50% fines
Well graded GRAVEL with silt: GW-GM	Silty GRAVEL: GM
Poorly graded SAND with clay: SP-SC	Silty SAND: SM

Additional descriptive terminology applied to coarse-grained soils follow.

Example: Coarse-Grained Soil Descriptions with Other Coarse-Grained Constituents

Coarse-Grained Soil Containing Secondary Constituents	
With sand or with gravel	≥ 15% sand or gravel
With cobbles; with boulders	Any amount of cobbles or boulders.

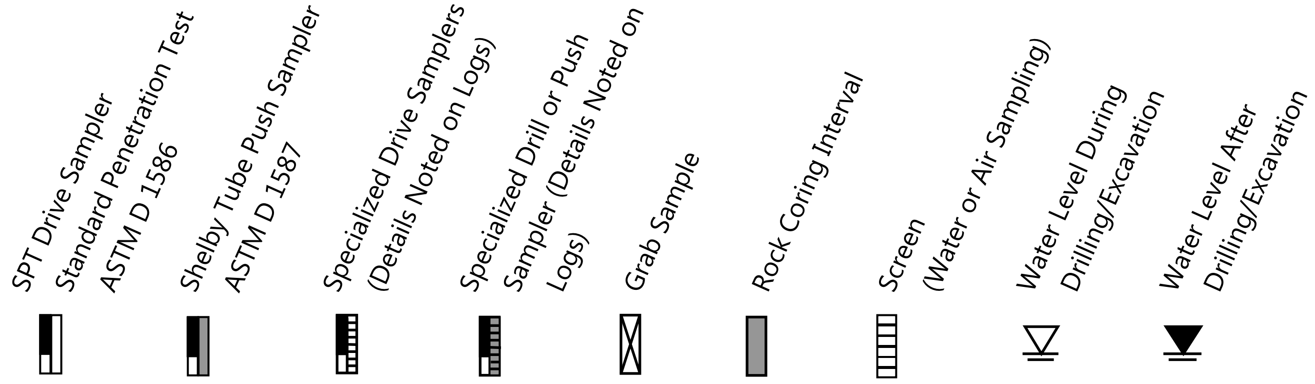
Cobble and boulder deposits may include a description of the matrix soils, as defined above.

Relative Density terms are applied to granular, non-plastic soils based on direct measure or correlation to the Standard Penetration Test N-value as determined by ASTM D1586-84.

Relative Density Term	SPT N-value
Very loose	0 – 4
Loose	5 – 10
Medium dense	11 – 30
Dense	31 – 50
Very dense	> 50

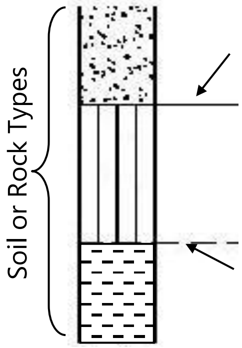
Table A-2
Key To Test Pit and Boring Log Symbols

SAMPLING DESCRIPTIONS



LOG GRAPHICS

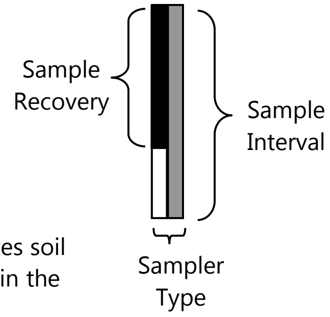
Soil and Rock



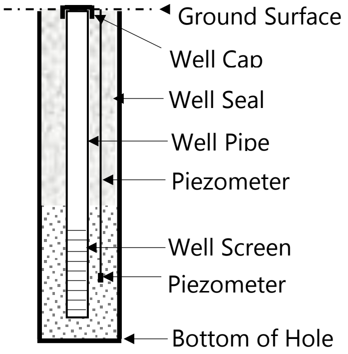
Lithology Boundary: separates distinct units (i.e., Fill, Alluvium, Bedrock) at approximate depths inciated

Soil-type or Material-type Change Boundary: separates soil and material changes within the same lithographic unit at approximate depth indicated

Sampling Symbols



Instrumentation Detail



Geotechnical Testing Acronym Explanations

PP	Pocket Penetrometer	HYD	Hydrometer Gradation
TOR	Torvane	SIEV	Sieve Gradation
DCP	Dynamic Cone Penetrometer	DS	Direct Shear
ATT	Atterberg Limits	DD	Dry Density
PL	Plasticity Limit	CBR	California Bearing Ratio
LL	Liquid Limit	RES	Resilient Modulus
PI	Plasticity Index	VS	Vane Shear
P200	Percent Passing US Standard No. 200 Sieve	bgs	Below ground surface
OC	Organic Content	MSL	Mean Sea Level
CON	Consolidation	HCL	Hydrochloric Acid
UC	Unconfined Compressive Strength		

Details of soil and rock classification systems are available on request.

Rev. 02/2017

PORT ORFORD CRANE REPLACEMENTS
PORT ORFORD, OREGON

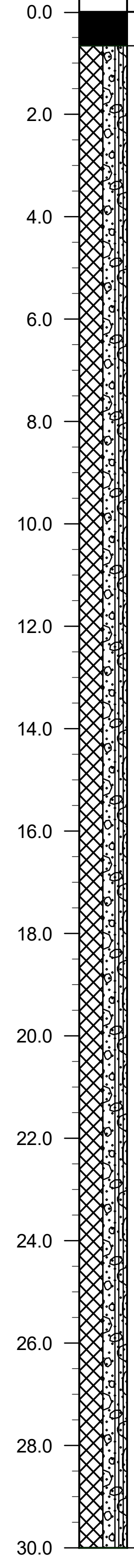
BORING B-1

PBS PROJECT NUMBER:
74353.000

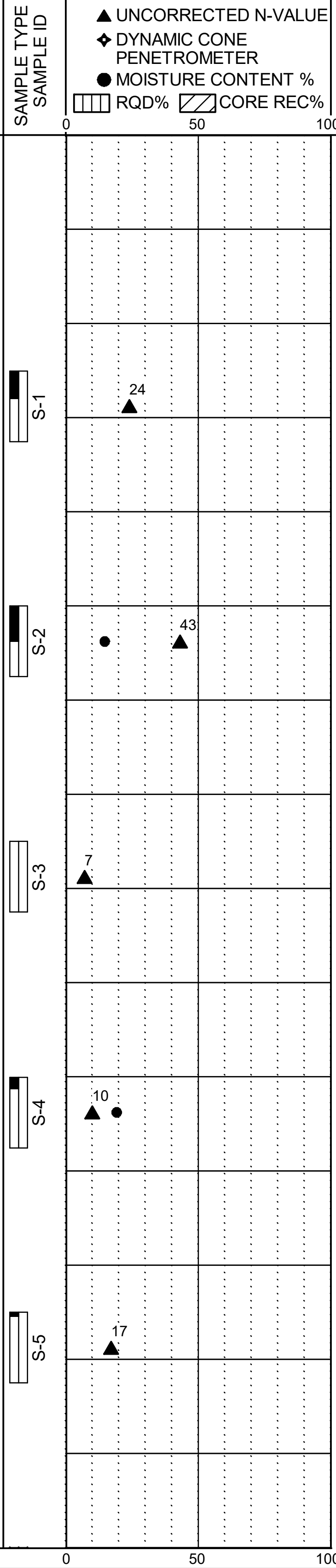
APPROX. BORING B-1 LOCATION:
(See Site Plan)
Lat: 42.73947 Long: -124.49850

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

Surface Conditions: Asphalt



MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	INSTALLATION AND COMMENTS
ASPHALT (~8 inches)	0.0			
Medium dense, brown, orange, and gray, poorly graded GRAVEL (GP-GM) with silt and sand; non-plastic; fine to coarse sand; fine to coarse, subrounded to subangular gravel; moist **FILL**	0.7		S-1	
becomes dense, increased orange color		P200 SIEV	S-2	P200 = 11%
becomes loose			S-3	No recovery; possible groundwater level based on wet soil cuttings
becomes wet			S-4	
becomes medium dense			S-5	Poor recovery



DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: J. Powell

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 85.3
LOGGING COMPLETED: 12/19/2023

FIGURE A1

PORT ORFORD CRANE REPLACEMENTS
PORT ORFORD, OREGON

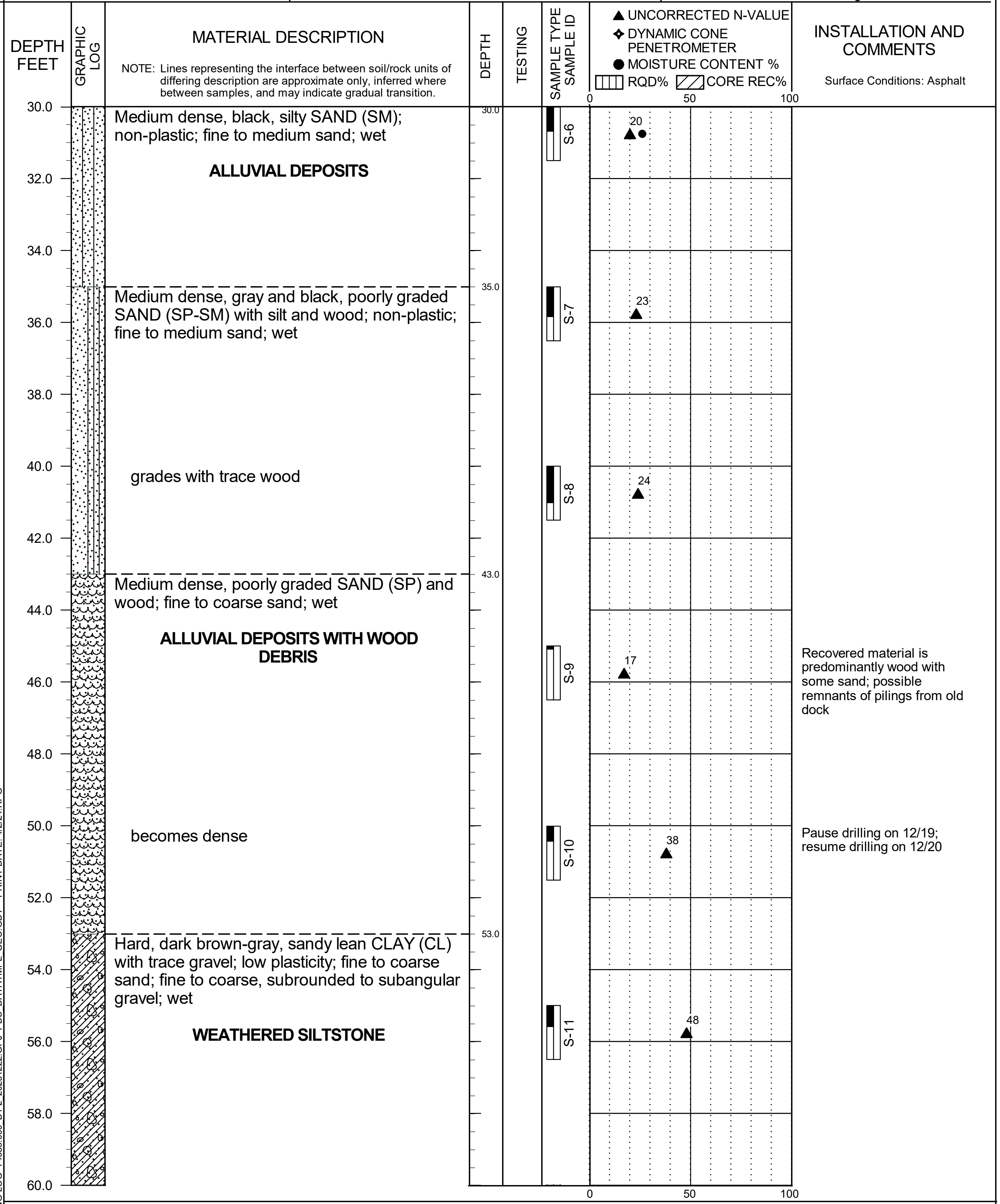
PBS PROJECT NUMBER:
74353.000

BORING B-1
(continued)

APPROX. BORING B-1 LOCATION:
(See Site Plan)
Lat: 42.73947 Long: -124.49850

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

Surface Conditions: Asphalt



30.0 – 35.0: Medium dense, black, silty SAND (SM); non-plastic; fine to medium sand; wet

ALLUVIAL DEPOSITS

35.0 – 43.0: Medium dense, gray and black, poorly graded SAND (SP-SM) with silt and wood; non-plastic; fine to medium sand; wet

grades with trace wood

43.0 – 53.0: Medium dense, poorly graded SAND (SP) and wood; fine to coarse sand; wet

ALLUVIAL DEPOSITS WITH WOOD DEBRIS

becomes dense

53.0 – 60.0: Hard, dark brown-gray, sandy lean CLAY (CL) with trace gravel; low plasticity; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet

WEATHERED SILTSTONE

Recovered material is predominantly wood with some sand; possible remnants of pilings from old dock

Pause drilling on 12/19; resume drilling on 12/20

DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: J. Powell

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 85.3
LOGGING COMPLETED: 12/19/2023

FIGURE A1

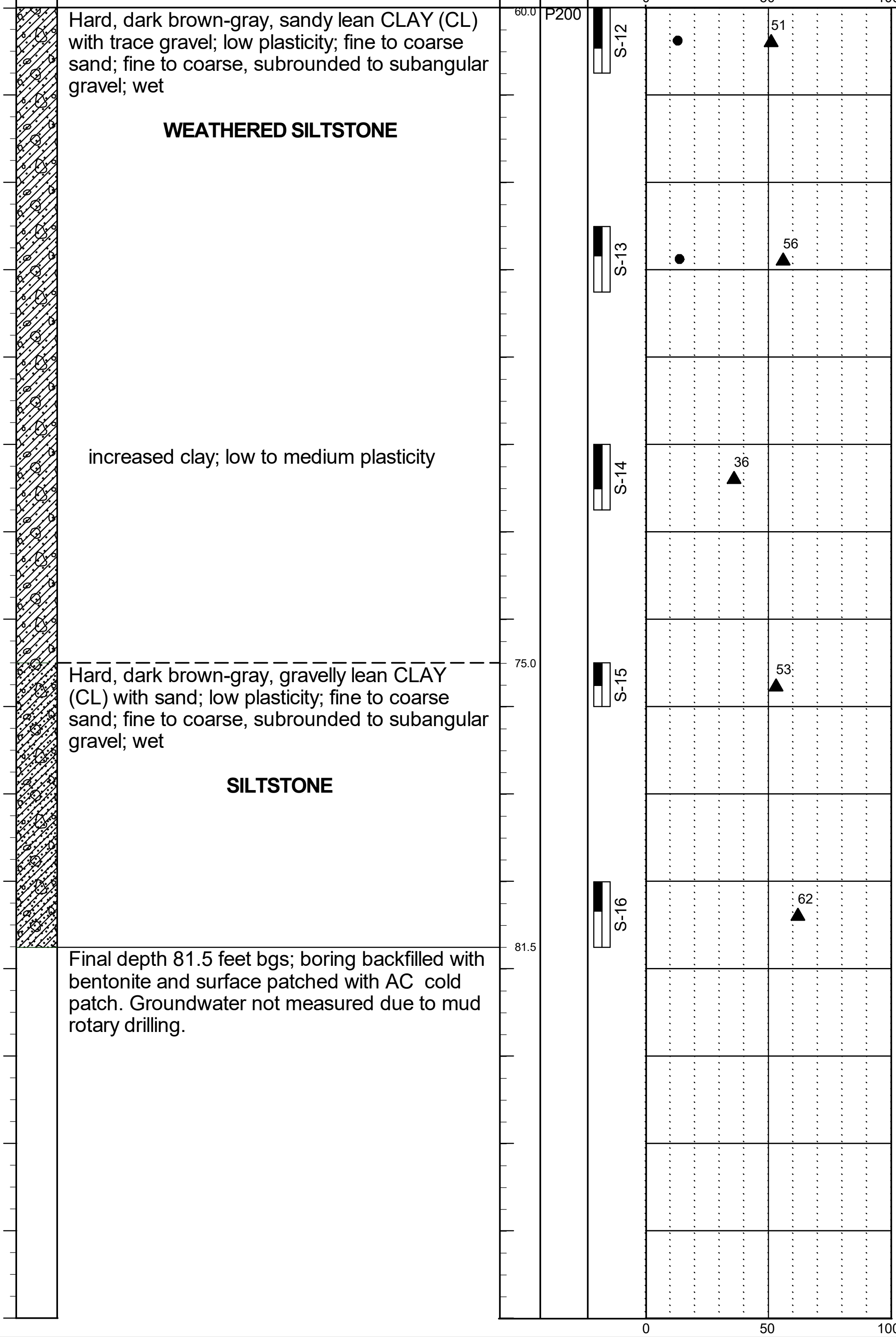
PBS	PORT ORFORD CRANE REPLACEMENTS PORT ORFORD, OREGON	**BORING B-1** (continued)
	PBS PROJECT NUMBER: 74353.000	APPROX. BORING B-1 LOCATION: (See Site Plan) Lat: 42.73947 Long: -124.49850

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

Legend: ▲ UNCORRECTED N-VALUE; ✧ DYNAMIC CONE PENETROMETER; ● MOISTURE CONTENT %; ▥ RQD%; ▨ CORE REC%

INSTALLATION AND COMMENTS — Surface Conditions: Asphalt

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	▲ UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
60.0	Hard, dark brown-gray, sandy lean CLAY (CL) with trace gravel; low plasticity; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet **WEATHERED SILTSTONE**	60.0	P200	S-12	51	P200 = 54%
				S-13	56	
70.0	increased clay; low to medium plasticity			S-14	36	
75.0	Hard, dark brown-gray, gravelly lean CLAY (CL) with sand; low plasticity; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet **SILTSTONE**	75.0		S-15	53	
				S-16	62	
81.5	Final depth 81.5 feet bgs; boring backfilled with bentonite and surface patched with AC cold patch. Groundwater not measured due to mud rotary drilling.	81.5				



DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: J. Powell

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 85.3
LOGGING COMPLETED: 12/19/2023

FIGURE A1

PORT ORFORD CRANE REPLACEMENTS
PORT ORFORD, OREGON

BORING B-2

PBS PROJECT NUMBER:
74353.000

APPROX. BORING B-2 LOCATION:
(See Site Plan)
Lat: 42.73916 Long: -124.49835

MATERIAL DESCRIPTION

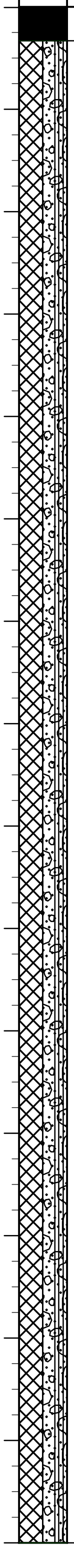
NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

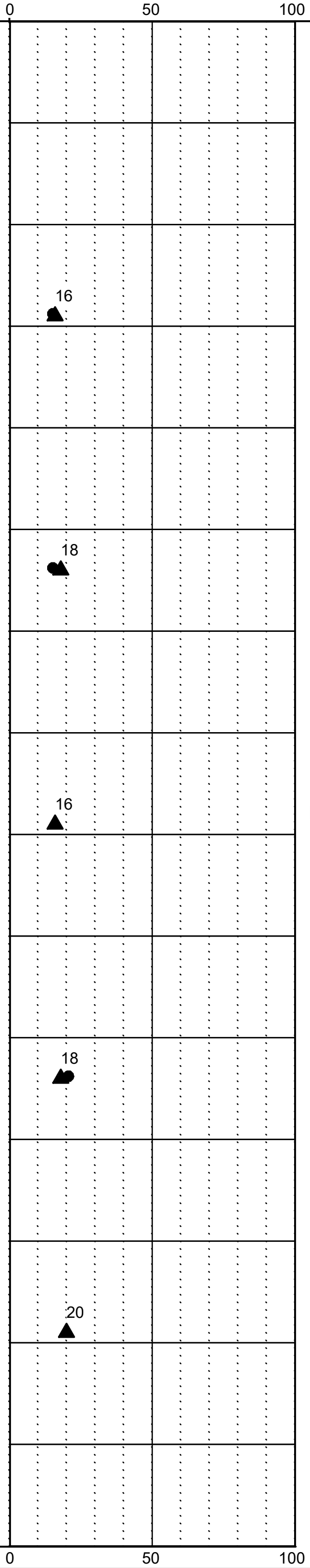
▲ UNCORRECTED N-VALUE
✧ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

INSTALLATION AND COMMENTS

Surface Conditions: Asphalt

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
0.0	ASPHALT (~8 inches)	0.0				
	Medium dense, light brown-gray, poorly graded GRAVEL (GP-GM) with silt and sand; non-plastic; fine to coarse sand; fine to coarse, subrounded to subangular gravel; moist to wet **FILL**	0.7				
6.0				S-1	16	
10.0	increased sand		P200	S-2	18	P200 = 12%
16.0	becomes wet			S-3	16	Possible groundwater level based on wet soil sample
20.0				S-4	18	
26.0	becomes medium dense			S-5	20	
30.0						





DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: J. Powell

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 85.3
LOGGING COMPLETED: 12/18/2023

FIGURE A2

PORT ORFORD CRANE REPLACEMENTS
PORT ORFORD, OREGON

PBS PROJECT NUMBER:
74353.000

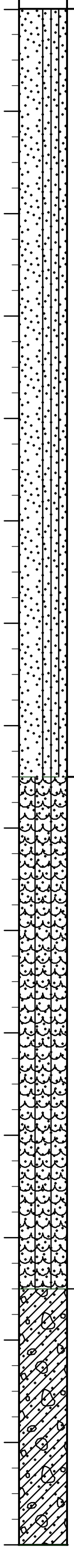
BORING B-2
(continued)

APPROX. BORING B-2 LOCATION:
(See Site Plan)
Lat: 42.73916 Long: -124.49835

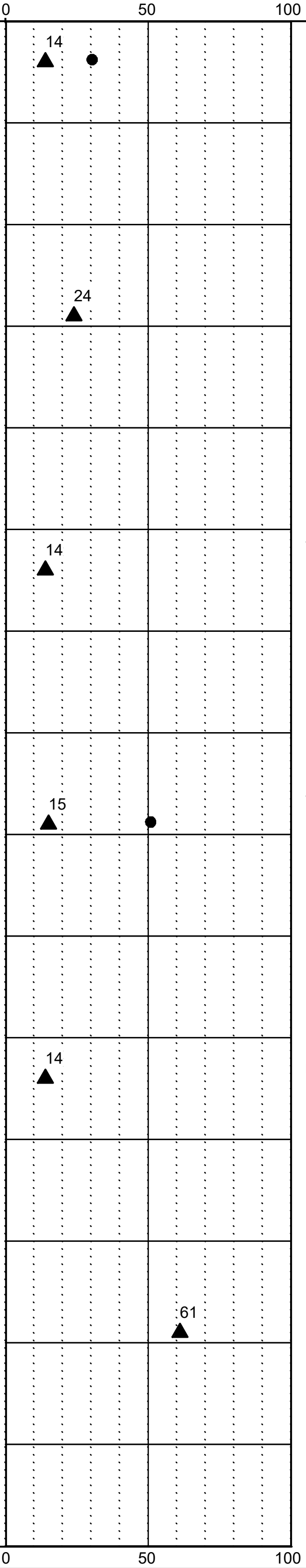
NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

▲ UNCORRECTED N-VALUE
✦ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

Surface Conditions: Asphalt



DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
30.0	Medium dense, black, poorly graded SAND (SP-SM) with silt; non-plastic; fine to medium sand; wet **ALLUVIAL DEPOSITS**	30.0	P200 HYD SIEV	S-6	14	P200 = 13%
				S-7	24	No recovery
40.0	becomes dark gray, with wood			S-8	14	Slight organic odor
	Medium dense, dark gray, silty SAND (SM) with wood; fine to medium sand; wet **ALLUVIAL DEPOSITS WITH WOOD DEBRIS**	45.0		S-9	15	Strong organic odor
50.0	grades with increased silt and decreased sand and wood			S-10	14	Pause drilling on 12/18; resume drilling on 12/19
	Hard, dark brown-gray, sandy lean CLAY (CL) with trace gravel; low to medium plasticity; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet **WEATHERED SILTSTONE**	55.0		S-11	61	



DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: J. Powell

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 85.3
LOGGING COMPLETED: 12/18/2023

FIGURE A2

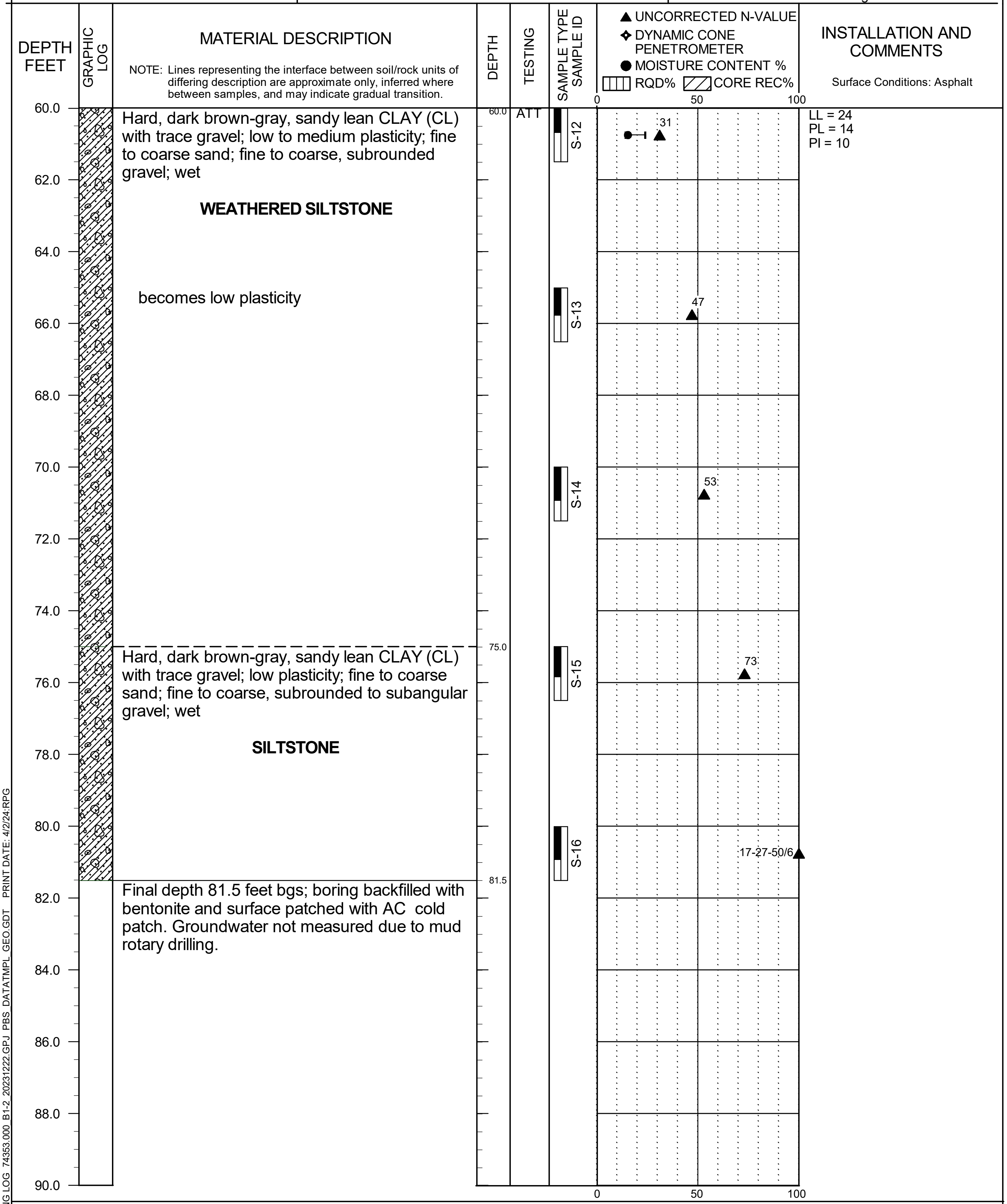
PORT ORFORD CRANE REPLACEMENTS
PORT ORFORD, OREGON

PBS PROJECT NUMBER:
74353.000

BORING B-2
(continued)

APPROX. BORING B-2 LOCATION:
(See Site Plan)

Lat: 42.73916 Long: -124.49835



Hard, dark brown-gray, sandy lean CLAY (CL) with trace gravel; low to medium plasticity; fine to coarse sand; fine to coarse, subrounded gravel; wet

WEATHERED SILTSTONE

becomes low plasticity

Hard, dark brown-gray, sandy lean CLAY (CL) with trace gravel; low plasticity; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet

SILTSTONE

Final depth 81.5 feet bgs; boring backfilled with bentonite and surface patched with AC cold patch. Groundwater not measured due to mud rotary drilling.

LL = 24
PL = 14
PI = 10

DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: J. Powell

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 85.3
LOGGING COMPLETED: 12/18/2023

FIGURE A2

Appendix B

Laboratory Testing

Appendix B: Laboratory Testing

B1 GENERAL

Samples obtained during the field explorations were examined in the PBS laboratory. The physical characteristics of the samples were noted and field classifications were modified where necessary. During the course of examination, representative samples were selected for further testing. The testing program for the soil samples included standard classification tests, which yield certain index properties of the soils important to an evaluation of soil behavior. The testing procedures are described in the following paragraphs. Unless noted otherwise, all test procedures are in general accordance with applicable ASTM standards. "General accordance" means that certain local and common descriptive practices and methodologies have been followed.

B2 CLASSIFICATION TESTS

B2.1 Visual Classification

The soils were classified in accordance with the Unified Soil Classification System with certain other terminology, such as the relative density or consistency of the soil deposits, in general accordance with engineering practice. In determining the soil type (that is, gravel, sand, silt, or clay) the term that best described the major portion of the sample is used. Modifying terminology to further describe the samples is defined in Table A-1, Terminology Used to Describe Soil, in Appendix A.

B2.2 Moisture (Water) Contents

Natural moisture content determinations were made on samples of the fine-grained soils (that is, silts, clays, and silty sands). The natural moisture content is defined as the ratio of the weight of water to dry weight of soil, expressed as a percentage. The results of the moisture content determinations are presented on the exploration logs in Appendix A and on Figure B3, Summary of Laboratory Data, in Appendix B.

B2.3 Atterberg Limits

Atterberg limits were determined on select samples for the purpose of classifying soils into various groups for correlation. The results of the Atterberg limits test, which included liquid and plastic limits, are plotted on Figure B1, Atterberg Limits Test Results, and on the exploration logs in Appendix A, where applicable.

B2.4 Grain-Size Analyses (Sieve)

Mechanical grain-size (sieve) analyses were performed on select samples to determine their particle size distribution. The results of the sieve analyses are presented on Figure B2, Particle-Size Analysis Test Results, in Appendix B.

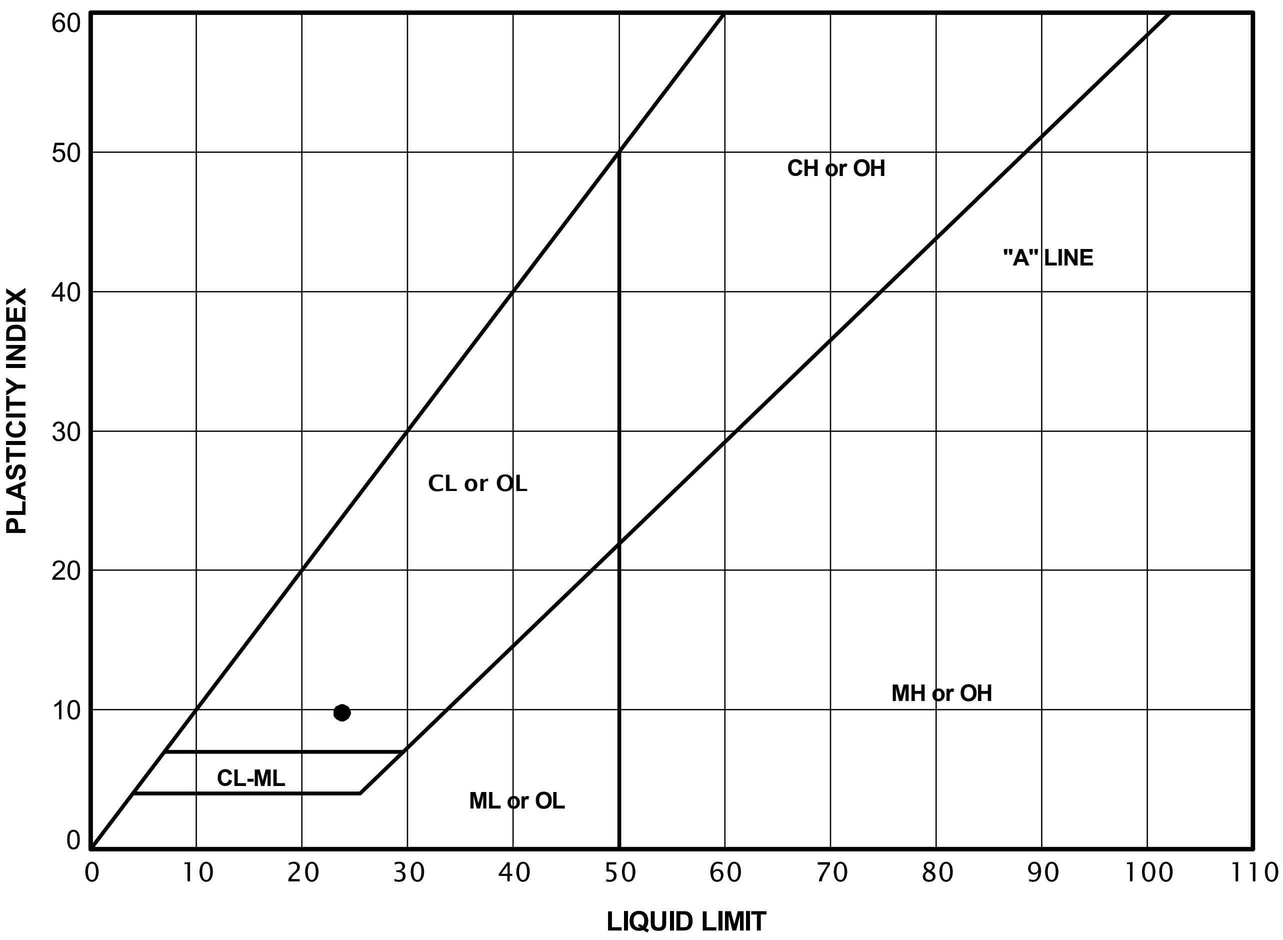
Washed sieve analyses (P200) were completed on samples to determine the portion of soil samples passing the No. 200 Sieve (i.e., silt and clay). P200 test results are presented on the exploration logs in Appendix A and on Figure B3, Summary of Laboratory Data, in Appendix B.

ATTERBERG LIMITS TEST RESULTS

PORT ORFORD CRANE REPLACEMENTS
PORT ORFORD, OREGON

PBS PROJECT NUMBER:
74353.000

TEST METHOD: ASTM D4318



KEY	EXPLORATION NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	NATURAL MOISTURE CONTENT (PERCENT)	PERCENT PASSING NO. 40 SIEVE (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-2	S-12	60.0	15.3	NA	24	14	10

FIGURE B1

__PARTICLE-SIZE ANALYSIS 74353.000_B1-2_20231222.GPJ PBS_DATATMPL_GEO.GDT PRINT DATE: 4/2/24:RPG

PBS

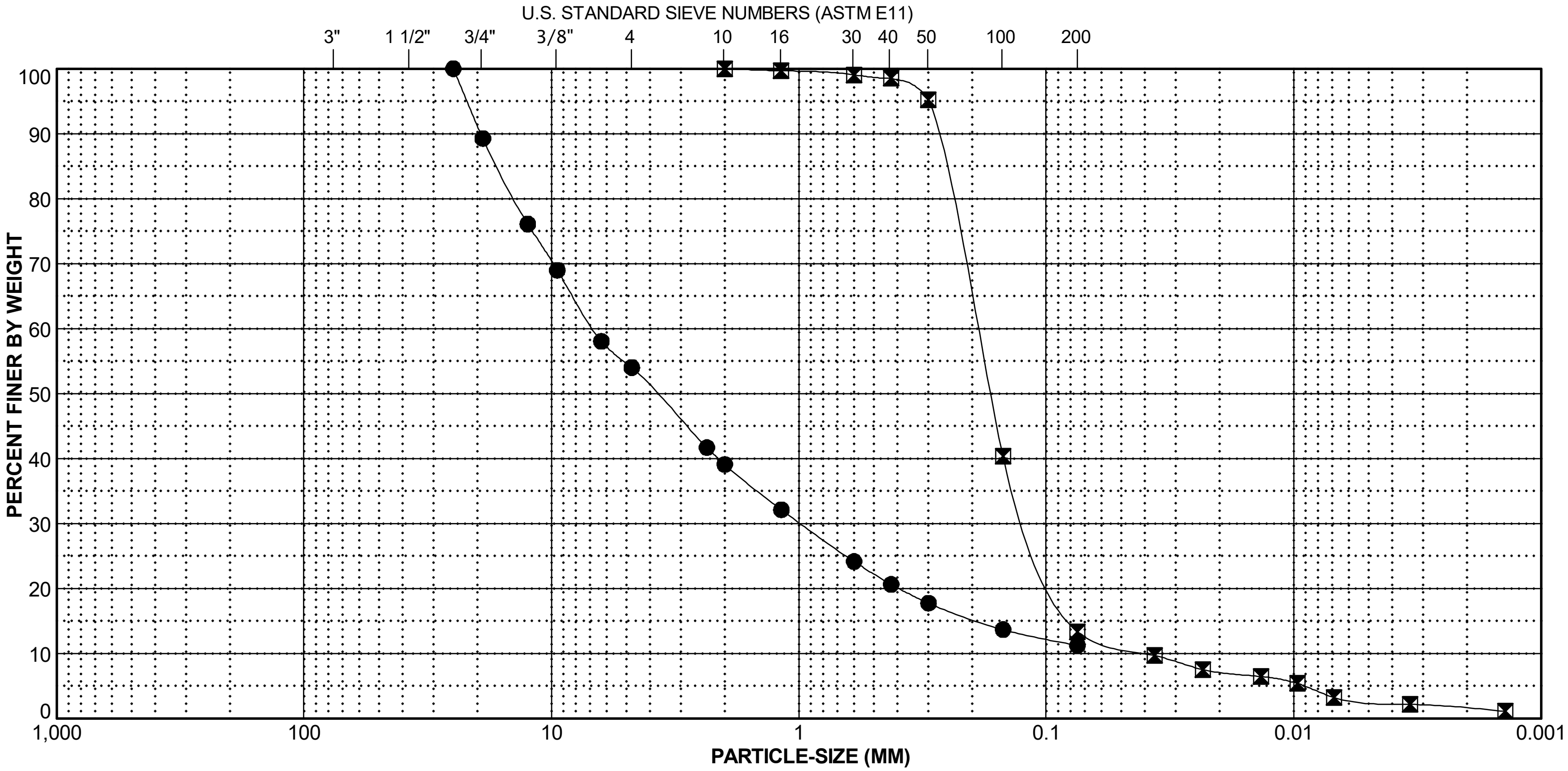
PARTICLE-SIZE ANALYSIS TEST RESULTS

PORT ORFORD CRANE REPLACEMENTS
PORT ORFORD, OREGON

PBS PROJECT NUMBER:
74353.000

TEST METHOD: ASTM C136/D422

BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY



KEY	EXPLORATION NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	D90 (MM)	D60 (MM)	D30 (MM)	D10 (MM)	D5 (MM)	GRAVEL (PERCENT)	SAND (PERCENT)	FINES (PERCENT)
●	B-1	S-2	10.0	15	19.4	6.8	1.0			46	43	11
☒	B-2	S-6	30.0	30	0.3	0.2	0.1	0.039	0.009		87	13

FIGURE B2
Page 1 of 1

SUMMARY OF LABORATORY DATA

PORT ORFORD CRANE REPLACEMENTS
PORT ORFORD, OREGON

PBS PROJECT NUMBER:
74353.000

SAMPLE INFORMATION				MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT (PERCENT)	PLASTIC LIMIT (PERCENT)	PLASTICITY INDEX (PERCENT)
B-1	S-2	10		14.6		46	43	11			
B-1	S-4	20		19.1							
B-1	S-6	30		26.0							
B-1	S-12	60		13.0				54			
B-1	S-13	65		13.7							
B-2	S-1	5		15.1							
B-2	S-2	10		15.1				12			
B-2	S-4	20		20.6							
B-2	S-6	30		30.3		0	87	13			
B-2	S-9	45		50.8							
B-2	S-12	60		15.3					24	14	10

LAB SUMMARY 74353.000_B1-2_20231222.GPJ PBS_DATATMPL_GEO.GDT PRINT DATE: 4/2/24:RPG

FIGURE B3