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GEOTECHNICAL INVESTIGATION REPORT

PGE Tonquin Substation

TUALATIN, OREGON

SHANNON & WILSON

January 2021

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Submitted To: POWER Engineers, Inc.
3940 Glenbrook Drive
Hailey, ID 83333
Attn: Kurt Penberthy, P.E.

Subject: GEOTECHNICAL INVESTIGATION REPORT, PGE TONQUIN SUBSTATION, TUALATIN, OREGON

This report presents the results of Shannon & Wilson's geotechnical study to support engineering, design, and construction of the Portland General Electric (PGE) Tonquin Substation Project. Shannon & Wilson prepared this report and participated in this project as a subconsultant to POWER Engineers, Incorporated. Our scope of services was specified in General Services Agreement HLY 007-10327 for Geotechnical Services, dated October 23, 2020.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON, INC.

REGISTERED PROFESSIONAL ENGINEER
77330PE
OREGON
Jan. 10, 2006
ELLIOTT CARL MECHAM
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ACRONYMS

ASCE	American Society of Civil Engineers
bgs	below ground surface
bpf	blows per foot
CSZ	Cascadia Subduction Zone
CRBG	Columbia River Basalt Group
IBC	International Building Code
FEMA	Federal Emergency Management Agency
OSHA	Occupational Safety and Health Administration
PCA	Power Control Assembly
pcf	pounds per cubic foot
psf	pounds per square foot
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
USGS	United States Geological Survey

1 PROJECT UNDERSTANDING

1.1 Project Description

PGE is planning the construction of a new 115kV/13kV substation and access road on a forested site at approximately 122°48′16.02″W, 45°21′56.31″N, near Tualatin, Oregon. The proposed substation is approximately 184 feet by 515 feet in plan, and involves site grading, an access roadway, standard substation equipment, a control enclosure, and stormwater elements. We understand substation equipment support structures are typically founded on lightly loaded cast-in-drilled-hole piers of 30-inch to 54-inch diameters, and dead-end H-frame structures may have pier foundations up to 84-inches in diameter. The lateral loading will generally dictate pier design. The dead-end A-frame structures have significantly higher axial loads, including uplift. We understand the other equipment such as the circuit breakers and transformers will be placed on concrete pads and the proposed control enclosure may be founded on drilled piers, a thickened slab on grade, or spread footings. The estimated foundation loads and allowable settlements provided by POWER Engineers on September 29, 2020 are included in Exhibit 1-1 below:

Exhibit 1-1: Estimated Foundation Loads and Allowable Settlements provided by POWER Engineers, Inc.

Strcture	Axial (kips)	Uplift (kips)	Shear (kips)	Moment (ft-k)	Foundation Type	Total Allowable Settlement (inch)
Small Equipment	2 - 15	-5	2 - 15	5 - 50	Shallow	0.5 - 1.0
Large Equipment	15 - 50	-15	15 - 50	50 - 300	Shallow	0.5 - 1.0
Control Building	55	---	---	---	Slab	0.5
Transformer	350	---	---	---	Mat	1.0
Deadend Structure	25 - 180	25 - 150	5 - 150	50 - 500	Shallow or Deep	1.0
Transmission Structure	25 - 180	25 - 150	5 - 150	50 - 500	Drilled Pier	1.0

1.2 Site Description

The proposed PGE Tonquin Substation is located approximately 1.8 miles northeast of the City of Sherwood, and 2.3 miles southwest of the City of Tigard in Washington County, Oregon. The project location is shown on the Vicinity Map, Figure 1. The substation site is situated on the east side of SW 124th Ave between SW Tualatin Sherwood Road and SW Tonquin Road. The site topography is slightly undulating with some small mounds and small berm sections along the south side of the site and a high point along the west side of the site which gently slopes to the northeast. The site itself is located on the north side, of the apex of a long gently sloping hill which slopes down to SW Tualatin Sherwood Road. Elevations at the site vary between approximately 254 and 238 feet (NAVD88). The site is moderately vegetated with low lying small white oak shrubs, scotch broom, poison oak, and blackberry brambles and forested by Douglas fir and Pacific Madrone trees. Numerous small boulders and small boulder piles are exposed at the ground surface. There are no streams or water bodies on the site.

Exhibit 1-2 through Exhibit 1-5 present site photographs showing several views of the site and existing features.

Exhibit 1-2: View south along SW 124th Ave, near northwest corner of site.

Exhibit 1-3: View north at the location of boring B-3, note Pacific Madrone trees and boulders at surface.

Exhibit 1-4: View south at the location of boring B-5, note berm along south edge of property.

Exhibit 1-5: View south at the location of boring B-6, note boulders at the surface.

1.3 Scope of Services

Our scope of services included the following tasks:

- A surface reconnaissance and desk study of the substation area to evaluate proposed exploration locations and assess geologic hazards;
- A subsurface exploration program including borings, dynamic cone penetration testing, infiltration testing, and laboratory testing;
- Evaluation of the proposed substation expansion with general site construction considerations; and
- Development of this geotechnical investigation report.

2 GEOLOGY AND SEISMIC SETTING

2.1 Regional Geology

The proposed PGE Tonquin Substation site resides at the southeastern margin of the Tualatin Basin, an approximately 35-mile-long by 20-mile-wide, northwest-trending, gently sloping synclinal valley (Madin, 1990). The Tualatin Basin is one of several localized subbasins within the Willamette Lowland, a broader regional geologic depression (Gannett and Caldwell, 1998). The basins are structural depressions, created by complex folding and faulting of the basement rocks (Schlicker and Deacon, 1967). The basement, or floor, of the basins is made up of lava flows collectively referred to as the Columbia River Basalt Group (CRBG), which flowed into the area in the middle Miocene epoch, between about 17 and 6 million years ago. Over the span of geologic time, sedimentary deposits consisting of clay, silt, sand, and gravel eroded from the surrounding uplands and settled into the basins formed on the CRBG surface. In the Tualatin Basin, these sediments have historically been referred to by several names, including Troutdale Formation (Schlicker and Deacon, 1967), Sandy River Mudstone equivalent (Madin, 1990), and Hillsboro Formation (Wilson, 1998). The basin-fill sediments are thickest near the center of the basin and thin toward the margins.

More recent sedimentation and, in some areas, morphology of the basin was heavily influenced by a series of late-Pleistocene glacial outburst floods. During the late stages of the last great ice age, between about 18,000 and 15,000 years ago, a lobe of the continental ice sheet repeatedly blocked and dammed the Clark Fork River in western Montana, which formed an immense glacial lake called Lake Missoula. The lake grew until its depth was sufficient to buoyantly lift and rupture the ice dam, allowing the entire massive lake to empty catastrophically. Once the lake had emptied, the ice sheet again gradually dammed the Clark Fork Valley and the lake refilled, leading to 40 or more repetitive outburst floods at intervals of decades (Allen and others, 2009). During each short-lived episode, floodwaters washed across the Idaho panhandle, through the eastern Washington scablands, and through the Columbia River Gorge. When the floodwater emerged from the western end of the gorge, it spread out over the Portland and Tualatin Basins and up the Willamette Valley as far south as Junction City, depositing a tremendous load of sediment (O'Conner and others, 2001). The catastrophic floods deposited extensive gravel bars across east Portland and up to 50 feet of micaceous clay to fine sandy silt in the Tualatin Basin. In mapping by O'Connor and others (2001), these sediments are referred to as Fine-Grained Missoula Flood Deposits.

2.2 Local Geology

Geologic mapping of the proposed substation site by Ma and others (2012) and Schlicker and Deacon (1967) indicate the surface of the site is made up of Fine-grained Missoula Flood Deposits underlain by Basalt of the CRBG. Additionally, the site is located in a unique geologic area on the edge of the Tualatin Basin commonly referred to as the Tonquin Scablands. During the catastrophic late-Pleistocene glacial outburst floods, flood water levels in the Tualatin Basin grew high enough to overtop the southeastern rim of the basin at the present-day location of the scablands. As the water overtopped the highlands and spilled south into the Willamette Valley, it locally eroded the basin rim, leaving deep channels, kolk ponds, and erosional remnants of basalt ridges. As the floods passed through, the area was scoured and stripped bare to rock. Today, only a thin veneer of sediment lies on top of much of the native ground surface. Scablands are generally identified in the Portland area by a dominance of Pacific Madrone trees, numerous relatively drought tolerant plants such as scotch broom and poison oak and are at an elevation of 300 feet or less. There are multiple rock quarries operating in the Tonquin Scablands because relatively fresh rock is so readily accessible. One such quarry is located on the property south of the proposed substation site and is owned and operated by Tigard Sand and Gravel, LLC. Based on aerial imagery the property immediately south of the site had previously been quarried up to around 2011 but has since been backfilled and sloped to an approximate 2H:1V slope along the property line adjacent to the substation.

2.3 Seismic Setting

Oregon is subject to seismic events from three major sources: (1) Cascadia Subduction Zone (CSZ) Megathrust earthquakes at the interface of the Juan de Fuca and North American Plates; (2) deep-focus, CSZ intraplate earthquakes (within the Juan de Fuca and North American Plates); and (3) shallow-focus earthquakes in local and regional continental crustal faults. The maximum magnitude for a CSZ Megathrust event is expected to be in the range of Moment Magnitude (M) 8 to 9, with a possible reoccurrence interval of 500 to 600 years. Intraslab events have occurred on a frequent basis in the Puget Sound area, but there is no strong historical evidence for such events in Oregon and southern Washington.

2.3.1 Local Faults and Folds

Faults and associated folds which reveal geological evidence of coseismic surface deformation in large earthquakes occurring during the Quaternary period in Oregon, have been located and characterized by the United States Geological Survey (USGS). The USGS provides approximate fault locations and a detailed summary of available fault information in the USGS Quaternary Fault and Fold Database. The database defines four categories of faults, Class A through D, based on evidence of tectonic movement known or presumed to be associated with large earthquakes during Quaternary time (within the last 2.6 million years). For Class A faults, geologic evidence demonstrates that a tectonic fault exists and that it has likely been active within the Quaternary period. For Class B faults, there is equivocal geologic evidence of Quaternary tectonic deformation, or the fault may not extend deep enough to be considered a source of significant earthquakes. Class C and D faults lack convincing geologic evidence of Quaternary tectonic deformation or have been studied carefully enough to determine that they are not likely to generate significant earthquakes.

According to the USGS Quaternary Fault and Fold database (USGS, 2020), there are 12 Class A features within approximately 30 miles of the project site. Their names, general locations relative to the site, and the time since their most recent deformation are summarized in Exhibit 2-1. The CSZ itself is approximately 110 miles west of the project site, with an average slip rate of approximately 40 millimeters (1.5 inches) per year and the most recent deformation occurring about 300 years ago (Personius and Nelson, 2006).

Exhibit 2-1: USGS Class A Faults Within an Approximate 30-mile Radius of the Project Site

Fault Name	USGS Fault Number	Approximate Length	Approximate Distance and Direction from Project Site[1]	Slip Rate Category[2]	Time Since Last Deformation[3]
Canby-Molalla Fault	716	31.1 miles	3.3 miles ENE	< 0.2 mm/yr	< 15 ka
Beaverton Fault Zone	715	9.3 miles	7.5 miles NNE	< 0.2 mm/yr	< 750 ka
Oatfield Fault	875	18.0 miles	8.0 miles NE	< 0.2 mm/yr	< 1.6 Ma
Newberg Fault	717	3.1 miles	9.3 miles SW	< 0.2 mm/yr	< 1.6 Ma
Portland Hills Fault	877	30.4 miles	9.9 miles NE	< 0.2 mm/yr	<1.6 Ma
Damascus-Tickle Creek Fault	879	9.9 miles	12.3 miles ENE	< 0.2 mm/yr	< 750 ka
East Bank Fault	876	18.0 miles	12.9 miles NE	< 0.2 mm/yr	< 750 ka
Helvetia Fault	714	4.3 miles	13.3 miles NW	< 0.2 mm/yr	< 1.6 Ma
Gales Creek Fault Zone	718	45.4 miles	13.86 miles WNW	< 0.2 mm/yr	< 1.6 Ma
Grant Butte Fault	878	6.2 miles	14.4 miles NE	< 0.2 mm/yr	< 750 ka
Mount Angel Fault	873	18.6 miles	16.9 miles SSW	< 0.2 mm/yr	< 15 ka
Lacamas Lake Fault	880	14.9 miles	25.4 miles NE	< 0.2 mm/yr	< 750 ka

NOTES:
1 Approximate distance between project site and nearest extent of fault mapped at the ground surface.
2 mm = millimeters; yr = year.
3 Ma = "Mega-annum" or million years ago; ka = "Kilo-annum" or one thousand years ago.

3 EXPLORATION PROGRAM

3.1 Field Explorations

Shannon & Wilson explored subsurface conditions at the site with six geotechnical borings, designated B-1 through B-6, and one infiltration test, designated IN-1. The geotechnical borings were completed between November 18 and November 20, 2020 by a CME-850 track mounted rig provided and operated by Western States Soil Conservation, Inc., out of Hubbard, Oregon. The borings were advanced to depths ranging from 16.5 to 30.4 feet below ground surface (bgs) using a combination of open-hole mud rotary drilling to advance the boring through soil and weathered rock and continuous HQ3-wireline rock coring technique to advance the boring through competent bedrock. The approximate exploration locations are shown on Figure 2, Site and Exploration Plan.

A qualified Shannon & Wilson geologist was on site throughout our exploration program to locate the borings, observe the drilling, collect samples, and log the materials encountered. Details of the exploration program, including descriptions of the techniques used to advance and sample the borings, logs of the materials encountered, and backfill details are presented in Appendix A, Field Explorations.

3.2 In Situ Infiltration Testing

An in situ infiltration test, designation IN-1, was completed at one location specified by POWER Engineers in an emailed Google Earth kmz file received by Shannon & Wilson on November 10, 2020 and at the approximate location shown on Figure 2. The test was performed to determine a representative infiltration rate of water into the onsite soils. The test was performed in accordance with the Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer per ASTM D3385-18. Details about the infiltration testing and test results are presented in Appendix A.

3.3 Laboratory Testing

The samples we obtained during our field explorations were transported to our laboratory for further observation. We then selected representative samples for laboratory tests. The testing program included moisture content tests, Atterberg limits tests, and unconfined compressive strength of intact rock core testing. Testing was performed by Northwest Testing, Inc. (NTI), of Wilsonville, Oregon, and by Shannon & Wilson. All tests were performed in accordance with applicable American Society for Testing and Materials (ASTM) International standards. The results of the laboratory tests and brief descriptions of the test procedures are presented in Appendix B, Laboratory Test Results.

4 SUMMARY OF SUBSURFACE CONDITIONS

The explorations and laboratory testing were performed to evaluate geotechnical soil, rock and groundwater conditions for the PGE Tonquin Substation Project. This section describes the general geotechnical units encountered in our subsurface exploration program and includes an overview of our interpreted geologic conditions at the project site.

Our observations presented in this report are specific to the locations, depths, and times noted on the logs and may not be applicable to all areas of the site. No amount of explorations or testing can precisely predict the characteristics, quality, or distribution of subsurface and site conditions. Potential variation includes, but is not limited to, the following:

- The conditions between and below explorations may be different.
- The passage of time or intervening causes (natural and manmade) may result in changes to site and subsurface conditions.
- Groundwater levels and flow directions may fluctuate due to seasonal, irrigation-related, and recharge source variations.

If conditions different from those described herein are encountered during construction, we should review our description of the subsurface conditions and reconsider our conclusions and recommendations.

4.1 Geotechnical Units

We grouped the materials encountered in our field explorations into four geotechnical units, as described below. Our interpretation of the subsurface conditions is based on the explorations and regional geologic information from published sources described herein and in Appendix A. General descriptions of the geotechnical units encountered in our subsurface explorations are as follows:

- **Fine-grained Missoula Flood Deposits**: very stiff, Lean Clay with Sand (CL);
- **Residual Soil**: medium dense to very dense, Gravel with varying amounts of clay, silt, and sand (GC, GM, GP-GM), with cobbles and boulders; dense, Silt Sand with Gravel to Silty Gravel with Sand (SM/GM), and hard, Elastic Silt with Sand (MH);
- **Weathered Columbia River Basalt**: extremely weak to medium strong (R0-R3), moderate to highly weathered and occasionally highly to completely weathered basalt; and
- **Columbia River Basalt**: weak to very strong (R2-R5), fresh to slightly weathered basalt.

These geotechnical units were grouped based on their engineering properties, geologic origins, and their distribution in the subsurface. Contacts between units may be more gradational than shown on the Logs of Borings in Appendix A. The SPT blow counts shown on the Logs of Borings, and discussed below, are as counted in the field (uncorrected). The following sections provide additional details for each of the individual geotechnical units listed previously.

4.1.1 Fine-grained Missoula Flood Deposits

Fine-grained Missoula Flood Deposits represent sediments deposited during the catastrophic Missoula Floods described in Section 2.1. Fine-grained Missoula Flood

Deposits were encountered in boring B-5, from the surface to a depth of approximately 4.5 feet. Although not specifically called out in the other borings, the material likely thinly blankets the surface elsewhere around the site. The material encountered in boring B-5, consisted of very stiff, light brown and brown, Lean Clay with Sand (CL) with trace amounts of gravel. The gravel constituent was fine to coarse and angular to subangular. The sand constituent was fine to medium and the fines content was medium plasticity. As with elsewhere around the Tualatin area, mica flakes were observed in the Missoula Flood Deposit material. One SPT N-value in the unit was 15 bpf. A moisture content of a tested sample indicated a moisture content of 28 percent. An Atterberg limits test on the same sample indicated a liquid limit of 42 and a plasticity index of 25, with a USCS group designation of CL.

4.1.2 Residual Soil

The Columbia River Basalt Group flows were deeply weathered after their deposition, causing a thick layer of Residual Soil to form within the uppermost flows. Significant changes in rock hardness, strength, compressibility and permeability occur due to the weathering and alteration over time, and the rock mass is predominantly decomposed to soil. Residual Soil was typically encountered at the surface in all borings except boring B-5 which encountered a thick layer of Missoula Flood Deposits overlying the Residual Soil. Generally, the unit occurred as a layer approximately 1.5 to 5 feet thick overlying less weathered basalt assigned to the Weathered Columbia River Basalt unit. In boring B-4, the unit was also encountered as an interbed within the Weathered Columbia River Basalt unit which was interpreted to be an interflow zone.

The Residual Soil observed in project boreholes consisted predominantly of medium dense to very dense, yellow, red, orange-brown, brown and gray, Gravel with variable amounts of clay, silt, sand, cobbles and boulders (USCS group designations GC, GM, GP-GM) with lesser amounts of hard, Elastic Silt with Sand (MH), with trace gravel and dense, Silty Sand with Gravel (SM). The gravel constituent is generally fine to coarse and angular to subangular. Cobbles and boulders were occasionally encountered within the unit and cobbles and boulders were often observed at the surface throughout the project site. Five of the seven SPTs attempted in the Residual Soil met refusal, where more than 50 blows were required to drive the sampler through a 6-inch interval. The two non-refusal SPT N-values in the Residual Soil were 43 and 23 bpf. Moisture contents of two samples, one from boring B-1 and one from boring B-5 indicated moisture contents of 23 and 33 percent, respectively.

4.1.3 Weathered Columbia River Basalt

Weathered Columbia River Basalt was observed in borings B-1, B-2 and B-5 overlying the Columbia River Basalt bedrock and boring B-4 was terminated within the unit. In borings

B-1, B-2 and B-5, the unit ranged in thickness from approximately 2.5 to 6.5 feet and in boring B-4 the boring was terminated within the unit at a depth of 30.4 feet. The Weathered Basalt observed in project boreholes consisted of moderate to highly weathered and occasionally highly to completely weathered, extremely weak to medium strong (R0-R3) basalt. The unit was often highly fractured and joint spacing may be highly variable with estimated extremely close to close joint spacing based on observed drill action, SPT samples and SPT N-values, but based off previous nearby explorations may include moderate joint spacing. The material was often described as orange-brown, brown and gray-brown. Weathered Basalt observed in boring B-4 between 18 and 23 feet consisted of highly to completely weathered basalt, which exhibits a relict basalt texture and structure which remolds under finger pressure to Silty Sand (USCS group designation SM) and was included in the unit since its original rock fabric was intact. All SPTs attempted within the unit met refusal.

4.1.4 Columbia River Basalt

The Columbia River Basalt unit underlies the entire project area beneath the Missoula Flood Deposits, Residual Soil, and Weathered Columbia River Basalt. Except for boring B-4, the borings were terminated within the unit at depths ranging from 16.5 to 21 feet. Basalt observed in the project boreholes consisted of fresh to slightly weathered, weak to very strong (R2-R5) basalt.

The basalt was generally observed to be aphanitic to slightly porphyritic, and with trace vesicles. Jointing was generally close to moderate spaced with occasional zones of very close to close spaced or moderate to wide spaced jointing. Wide spaced jointing was observed in sample C-1 of boring B-6 where a solid (no joints observed) rock core sample was acquired between 6.5 and 11.5 feet. Joints were generally rough, undulating to stepped, and low to high (0°-90°) angle, with iron oxide staining and joint coting 1- to 4-mm thick and occasional localized zones of joints with hard clay infilling 1- to 9-mm thick. Paleosol interflow layers were encountered within the Columbia River Basalt in boring B-3 from 16.2 to 16.4 feet and in boring B-5 from 15.5 to 15.7 feet. The basalt encountered underlying the paleosol layers were generally moderately vesicular and weak to medium strong (R2-R3). All SPTs attempted within the Columbia River Basalt met refusal, typically in the first 2-inches. The Rock Quality Designation (RQD) of unit in the boreholes ranged from 0 to 100 percent and averaged 26 percent. The unconfined compressive strength (UCS) of the basalt in tested samples generally ranged from 10,070 to 26,954 psi; however, one sample of moderately vesicular basalt from boring B-3 sheared at its end early during testing under a stress of 4,402 psi.

4.2 Groundwater

The geotechnical borings performed by Shannon & Wilson were advanced using mud rotary and HQ3-wireline rock coring techniques which make it difficult to discern the depth to groundwater if it is encountered due to the introduction of artificial drilling fluids into the borehole. Hollow-stem auger drilling, which generally allows the observation of groundwater during drilling was not used due to the shallow bedrock and the presence of cobbles and boulders in the upper subsurface. In an attempt observe perched groundwater conditions at the site the boreholes were left open over night or for up to approximately 48 hours depending on borehole completion. Water level measurements were performed in the completed open boreholes on the morning of November 20, 2020 prior to backfilling the boreholes and are presented in Exhibit 4-1.

Exhibit 4-1: Measurements of Water in Boreholes

Boring	Borehole Water Depth Below Ground Surface (ft)[2]
B-1	5.4
B-2	Not Measured[1]
B-3	9.3
B-4	6.6
B-5	10.4
B-6	10.5

NOTES:

1. Boring was completed on same day borehole water measurements were taken.
2. See discussion of water measurements in completed boreholes below.

Loss of drilling fluids, and drilling fluid circulation loss in the boreholes was not encountered during drilling. Drilling fluid loss and loss of fluid circulation may indicate an open-matrix gravel containing limited matrix material, voids, wide aperture joints in bedrock with little to no infilling, or open fracture zones in bedrock. During water depth measurements in the boreholes, artesian conditions and water seepage into the boreholes from the borehole sides was not observed. It is our opinion the water levels measured in the boreholes was perched ground water and may have been influenced by remaining drilling fluid which had not yet infiltrated into the surrounding subsurface material. Regional groundwater mapping (Snyder 2008) indicates the depth to the static ground water table at the proposed substation is greater than 70 feet. Perched groundwater locally tends to occur at the top of bedrock on the decomposed layer or upper weathered layer. Perched groundwater at the site will generally vary based on precipitation since the substation site is located near the apex of a gently sloping hill. We expect groundwater levels throughout the site should be expected to vary seasonally and with changes in topography and precipitation. Locally, groundwater highs typically occur in the late fall to spring and groundwater lows typically occur in the late summer and early fall.

5 GEOLOGIC AND SEISMIC HAZARDS

Shannon & Wilson reviewed publicly available geologic and technical publications along with data collected from the site reconnaissance and subsurface explorations to assess the landslide, flooding, and seismic (including liquefaction) hazards at the site. The following paragraphs discuss these hazards in greater detail.

These assessments are summaries of the potential geologic hazards at the site and are not meant to fully characterize each hazard. Further research, explorations, and analysis may be necessary to properly characterize the hazard and its impact on the substation.

5.1 Landslide Hazard

There are no mapped landslides upslope or near the proposed substation site. According to the Oregon Department of Geology and Mineral Industries Statewide Geohazards Viewer (DOGAMI, 2020), landslide hazard is low with landsliding unlikely for the site.

5.2 Flooding Hazard

The project site is located near the apex of a broad hill at approximate elevation 245 feet. The Tualatin River is approximately 1.7 miles north of the site and is at an approximate elevation of 110 feet. The proposed substation site is located outside the 100-year flood (1% annual chance of flooding) extent, according to Federal Emergency Management Agency (FEMA, 2015) National Flood Insurance Program Flood Insurance Rate Maps (FIRM) and is located in a FEMA Flood Zone X, area of minimal flood hazard.

5.3 Seismic Design Parameters

For the proposed substation expansion, we obtained seismic design parameters from the 2018 International Building Code (IBC) and subsequently the 2016 edition of ASCE 7 (ASCE 7-16). The maximum considered earthquake (MCE) ground motions for the new structure are obtained from the USGS's US Seismic Design Maps Web Application, which considers a target risk of structural collapse of 1 percent in 50 years (2,475-year return period) (USGS, 2020). Exhibit 5-1 provides the recommended seismic design parameters for the site.

Exhibit 5-1: Recommended Seismic Design Parameters

Parameter	Symbol	ASCE 7-16
Site Class	-	C
Mapped Zero Period Spectral Acceleration	PGA	0.38g
Mapped Short Period Spectral Acceleration	S_S	0.833g
Mapped 1-Second Period Spectral Acceleration	S_1	0.389g
Zero Period Site Factor	F_{PGA}	1.2
Short Period Site Factor	Fa	1.2
1-Second Period Site Factor	Fv	1.5
Site Adjusted Zero Period Spectral Acceleration	PGA_M	0.456g
Site Adjusted Short Period Spectral Acceleration	S_{MS}	0.999g
Site Adjusted 1-Second Period Spectral Acceleration	S_{M1}	0.584g
Short Period Design Spectral Acceleration	S_{DS}	0.666g
1-Second Period Design Spectral Acceleration	S_{D1}	0.389g
Seismic Design Category	-	D

5.4 Seismic Hazards

5.4.1 Liquefaction

Liquefaction is a phenomenon in which excess pore water pressure in loose to medium dense, saturated, nonplastic to low plasticity silts and granular soils develops during ground shaking. The increase in excess pore pressure may result in a reduction of soil shear strength and a quicksand-like condition.

Important factors in evaluating a soil's susceptibility to liquefaction include relative density, the fines content (percent of soil by weight smaller than 0.075 millimeter, passing the U.S. No. 200 sieve), and the plasticity characteristics of the fines. Relative density can be estimated from SPT N-values that were performed for this project. We performed laboratory Atterberg limits testing to evaluate the plasticity of the site soils.

The site is generally underlain by dense to very dense Residual Soil overlying Columbia River Basalt, which is not susceptible to liquefaction. Fine-grained Missoula Flood Deposits were encountered within the upper 4.5 feet bgs at boring B-5. This material was classified as very stiff, medium plasticity Lean Clay with Sand, and Atterberg Limits testing showed that this material has a Liquid Limit of 42 and a Plasticity Index of 17. This layer was also located above the highest groundwater level measured at the site during our field explorations performed in November 2020. Therefore, this layer is not expected to be liquefiable.

We judge that the potential for liquefaction and liquefaction-related hazards (such as lateral spreading) at the site is low.

5.4.2 Fault Rupture

As shown in Exhibit 2-1, the closest active mapped fault to the site is the Canby-Molalla Fault, located approximately 3.3 miles to the northeast of the project site. In our opinion, given the distance between the site and the fault, the potential for a hazard posed by ground surface fault rupture at the site is low.

5.4.3 Slope Instability

According to DOGAMI, the site is located within zones of low to moderate landslide hazard. There are no mapped active or historic landslides within the site limits documented in the DOGAMI GeoHazard database. We did not observe evidence of slope instability at the site during our November 2020 exploration, nor did we observe evidence of offsite slope instability that could pose a risk to the proposed improvements. In our opinion, the hazard potential for slope instability at the substation site is low.

6 GEOTECHNICAL DESIGN RECOMMENDATIONS

6.1 General

Geotechnical design recommendations are based on our field explorations, laboratory test results, and our understanding of the project based on current design information provided by POWER Engineers, Inc. Geotechnical design recommendations for the proposed structures are provided in the following sections. If structure or foundation types and configurations change after this report, Shannon & Wilson should be contacted to provide updated recommendations.

Based on information from POWER Engineers we understand that small and large equipment, as well as the control building, will be supported on shallow spread footings, and transformers will be constructed using mat foundations. Dead-end and transmission structures will be supported on deep foundations to resist high axial and lateral load demands. Discussions and recommendations pertaining to shallow and deep foundations are presented in the following sections.

6.2 Shallow Foundations

We understand that large and small equipment such as the circuit breakers and the control house will be placed on shallow foundations. The more heavily loaded transformers may be supported on mats. We recommend that all shallow foundations be constructed over a 1-foot thick crushed rock pad bearing on undisturbed subgrade, as discussed in Section 7.1. Portions of the site are mantled with fine grained soil (Missoula Flood deposits), while shallow residual soil (derived from Basalt). Our recommendations have been developed assuming either unit could be encountered at proposed shallow foundation locations.

6.2.1 Bearing Capacity

Spread foundations and mat foundations built over a properly constructed 12-inch-thick crushed rock pad can be designed for a gross allowable soil bearing pressure of 4,000 pounds per square foot (psf). These values apply to the total dead load and can be increased by one-third for wind or seismic loading. This bearing capacity is based on a minimum footing width of 1.5 feet and minimum embedment depth of 18 inches. A subgrade modulus of 150 pounds per cubic inch (pci) is recommended for design, regardless of foundation dimensions.

6.2.2 Settlement

For footings and mat foundations constructed as described above, we estimate a maximum total settlement of less than 1 inch under static loading conditions. Differential settlement between adjacent footings is typically 50 percent of the estimated total settlement when subgrade conditions are relatively uniform. Our settlement estimate assumes that no disturbance to the foundation soil subgrade will be permitted during excavation and that the subgrade will be properly prepared.

6.2.3 Uplift Resistance

Uplift resistance of the shallow foundations should be estimated based on the dead weight of the steel and the dead weight of the backfill material placed over the foundation. For estimating the uplift resistance, we recommend that a unit of 120 pounds per cubic foot (pcf) be used assuming that the backfill is imported crushed rock discussed in Section 7.1. If necessary, tiedown anchors can be installed to provide additional uplift resistance to shallow foundations.

6.2.4 Lateral Resistance

The soil resistance available to withstand lateral foundation loads is a function of the frictional resistance, which can develop on the base of the foundation, and the partial soil passive resistance, which is assumed to be about 50 percent of full soil passive resistance. We recommend that an allowable partial soil passive pressure, 180D psf (where D is depth of the embedment of the bottom of foundation), be used for design of sliding and overturning resistance. The allowable frictional resistance may be computed using a coefficient of friction of 0.45. The top 12 inches of soil should not be used in calculating passive resistance, as construction and post-construction activities often disturb this upper material. Typically, the lateral resistance of shallow foundations is by a combination of passive resistance along the buried portions of the foundation and sliding resistance along the base of the foundation.

6.3 Deep Foundations

The selection of an appropriate foundation system for the proposed structures dependent upon several factors. These factors include, but are not limited to foundation capacity, tolerance of total and differential settlement resulting from static loads, cost, and constructability. We determined that shallow footings are not a feasible foundation alternative to support the proposed transmission and dead-end structures due to the expected large design loads.

Deep foundations are typically evaluated when shallow footings cannot be supported by near-surface competent bearing soils. For this project, we evaluated advantages and disadvantages of using large-diameter drilled shafts and smaller diameter drilled micropiles, as summarized in Exhibit 6-1; driven piles were not considered practical due to presence of shallow basalt and the inability of driven piles to penetrate rock.

Exhibit 6-1: Advantages and Disadvantages of Deep Foundation Alternatives

Alternative	Advantages	Disadvantages
Drilled Shafts	• Can support large design loads. • Typically requires relatively less surface area compared to micro-pile foundation caps.	• Relatively long construction time per drilled shaft, especially in hard rock. High cost in drilled rock.
Micropiles	• Configurations can be easily adjusted • Potential cost savings over drilled shafts due to relatively quick installation • Can provide relatively high axial capacities • Can provide high lateral bearing resistance if installed at a batter with steel casing in upper 20 to 25 diameters	• Typically more expensive compared to drilled shaft foundations in soils. However, micropiles can be more cost effective in hard rock. • Less capable of resisting large lateral shear load unless installed at a batter with an outside casing. • Additional design effort.

Both large-diameter drilled shafts and micropiles are potentially feasible alternatives for support of these structures. However, construction of large-diameter drilled shafts in basalt bedrock can be time consuming and result in relatively high construction costs. In our past experience, PGE has opted to substitute small-diameter (6 to 10 inches) drilled micropiles for support of transmission tower structures requiring deep foundations in areas where high strength basalt bedrock is present. If micropiles are used to resist lateral loads, the structural designer should specify that a steel casing be installed along the upper portion of the micropile, for a distance equivalent to 20 to 25D (where D is the diameter of the micropile), to provide additional resistance against bending moments in this zone. The structural engineer may also consider requiring over-excavation the soil overburden such that the micropile is installed entirely in rock in order to reduce the load demand on the micropile. We have provided geotechnical design parameters for both drilled shafts and micropiles.

6.3.1 Drilled Micropiles

6.3.1.1 Micropile Axial Capacity

We performed axial capacity evaluations for 6- through 10-inch-diameter micropiles for the transmission and dead-end structures. We evaluated axial capacity for static and seismic conditions. The analyses were based on the subsurface conditions encountered in the project borings and our experience with similar soil and project conditions. We estimated unit side and tip resistance values based on the average SPT values (N-values) within each unit, laboratory tests, load tests in similar soil conditions from other projects, and our experience.

The micropiles should be designed with an unbonded length of 10 feet beginning at the base of the pile cap, and should include a minimum bonded embedment of 5 feet into intact Columbia River Basalt. According to Table C6.1 of Post-Tensioning Institute Manual DC35.1-14, "Recommendations for Prestressed Rock and Soil Anchors," average ultimate bond strengths between Granite or Basalt rock and grouted micropiles are within the range of 250 to 450 pounds per square inch (psi).

In the area near Boring B-4, we recommend assuming a rock/grout bond strength of no greater than 250 pounds per square inch for micropiles, for estimating ultimate shaft friction for portions of the micropile embedded into the Weathered Columbia River Basalt. We recommend that the bonded zone begin at a depth of at least 23 feet below ground surface in this area, where R2-R3 weathered rock was noted on the boring log for B-4.

Improvements in other areas of the site (except near B-4) may be designed using a rock/grout bond strength of up to 450 pounds per square inch; however, actual bond strengths will be dependent on construction means and methods and details of micropile design (e.g. grout placed by tremie vs pressure). The strong to very strong basalt at the site has an average Rock Quality Designation (RQD) of about 50 percent. For length estimating purposes, we recommend a rock/grout bond strength of 50 kips per square foot (approximately 350 psi) for estimating ultimate shaft friction of micropile bonded to Columbia River Basalt (except at B-4).

We recommend that allowable capacities be calculated using factors of safety of 2.0 and 1.5 for the static and seismic conditions, respectively, assuming proof-testing of micropiles will be performed during construction as discussed in Section 6.3.1.3. The above stated factors of safety are applicable for both compression and tension. We recommend against relying on end bearing for micropile design. Micropiles should be spaced at least three shaft diameters apart (3B), measured center to center, and in a single row.

6.3.1.2 Micropile Lateral Resistance

The micropile foundations will be subjected to lateral loads resulting from live and seismic loading. We understand that the laterally loaded micropile analyses will be performed with the aid of the computer program LPILE or GROUP.

Shallow rock conditions are expected to be present throughout most of the site; however, the depth to competent rock is expected to be deeper in the vicinity of Boring B-4. Based on these differing subsurface conditions across the site, we have provided two sets of recommended LPILE/GROUP parameters. Unfactored geotechnical input parameters for the LPILE computer model and shallow rock conditions are provided in Exhibit 6-2 below. More conservative deeper rock conditions may be modeled using parameters in Exhibit 6-3.

Exhibit 6-2: LPILE/GROUP Geotechnical Input Parameters for Drilled Micropile Foundations on Shallow Rock

Depth		Unit Description	LPILE Model	Effective Unit Weight (pcf)	Friction Angle (deg)	k (pci)	Uniaxial Compressive Strength (psi)
Layer Top (ft)	Layer Bottom (ft)						
0	5	Residual Soil	Sand (Reese)	130	40	225	--
5	7	Weathered Basalt	Sand (Reese)	140	45	300	--
7	30	Columbia River Basalt	Strong Rock (Vuggy Limestone)	150	--	--	4,000[1]

1 Reduced value equivalent to typical 28-day concrete strength. In areas where steel casing is present this value may be increased to 17,000 psi.

Exhibit 6-3: LPILE Geotechnical Input Parameters for Drilled Micropile Foundations on Deeper Rock

Depth		Unit Description	LPILE Model	Effective Unit Weight (pcf)	Friction Angle (deg)	k (pci)	Uniaxial Compressive Strength (psi)
Layer Top (ft)	Layer Bottom (ft)						
0	5	Residual Soil	Sand (Reese)	130	40	225	--
5	30	Weathered Basalt	Sand (Reese)	140	45	300	--
30	30+	Columbia River Basalt	Strong Rock (Vuggy Limestone)	150	--	--	4,000[1]

1 Reduced value equivalent to typical 28-day concrete strength. In areas where steel casing is present this value may be increased to 17,000 psi.

The estimated lateral capacity parameters presented in Exhibits 6-2 and 6-3 are recommended for drilled micropiles with center-to-center spacing greater than or equal to five micropile diameters (5B) and in a single row. For spacing less than 5B, or if multiple rows of micropiles are present, the lateral capacity should be reduced by the factors presented in Exhibit 6-4. Micropile spacing of less than 3B is not recommended. P-Multipliers may be linearly interpolated for spacings between 3B and 5B.

Exhibit 6-4: Lateral Capacity Reduction Factors (P Multipliers) for 3B and 5B Micropile Spacings

Spacing	Row #	P Multiplier	Spacing	Row #	P Multiplier
3B	1	0.70	5B	1	1.0
	2	0.50		2	0.85
	3	0.35		3	0.70
	3+	0.35		3+	0.70

Lateral resistance from micropile groups may also be achieved from the micropile cap. We recommend that an allowable partial soil passive pressure, 180d psf (where d is depth of the embedment of the bottom of foundation), be used for design of sliding and overturning resistance when the pile caps are embedded in crushed rock fill or residual soil. The top 12 inches of soil should not be used in calculating passive resistance. We recommend full-time observation of the micropile installation by a qualified geotechnical engineer.

6.3.2 Drilled Shafts

6.3.2.1 Drilled Shaft Axial Capacity

The drilled shafts will develop high shaft friction and adhesion to the basalt rock. Based on EPRI EL-5918 Analysis and Design of Drilled Shaft Foundations Socketed Into Rock (Equation 3-9) and an average uniaxial compressive strength of 17,000 psi based on our 6

UCS tests, the ultimate skin friction for drilled shafts is 710 psi in Columbia River Basalt (except at Boring B-4 where lower strength rock was encountered). For drilled shafts near boring B-4, we recommend using an ultimate skin friction of 240 psi in the weak to medium strong (R2-R3) rock identified at a depth of 23 feet bgs in this boring. We recommend an ultimate adhesion strength of 14 psi for Residual Soil. The EPRI Manual *"Design of Drilled Shaft Foundations Socketed Into Rock"* states that for compressible and extensible shafts it may be prudent to reduce the unit side shear resistance. A reduction in the ultimate unit side shear resistance of 30 percent may be applied to shafts loaded in uplift. We also recommend a minimum factor of safety of 2.5 for compression and uplift loads on drilled shafts.

Based on the relatively high strength of the rock and the low axial loads, we anticipate that the required factor of safety can be achieved through skin friction in the rock.

6.3.2.2 Drilled Shaft Lateral Resistance

The drilled shaft foundations will be subjected to lateral loads resulting from live and seismic loading. We understand that the laterally loaded shaft analyses will be performed with the aid of the computer program LPILE.

Shallow rock conditions are expected to be present throughout most of the site; however, the depth to competent rock is expected to be deeper in the vicinity of Boring B-4. Based on these differing subsurface conditions across the site, we have provided two sets of recommended LPILE parameters. Unfactored geotechnical input parameters for the LPILE computer model and shallow rock conditions are provided in Exhibit 6-5 below. More conservative deeper rock conditions may be modeled using parameters in Exhibit 6-6.

Exhibit 6-5: LPILE Geotechnical Input Parameters for Drilled Shaft Foundations on Shallow Rock

Depth		Unit Description	LPILE Model	Effective Unit Weight (pcf)	Friction Angle (deg)	k (pci)	Uniaxial Compressive Strength (psi)
Layer Top (ft)	Layer Bottom (ft)						
0	5	Residual Soil	Sand (Reese)	130	40	225	--
5	7	Weathered Basalt	Sand (Reese)	140	45	300	--
7	30	Columbia River Basalt	Strong Rock (Vuggy Limestone)	150	--	--	4,000[1]

1 Reduced value equivalent to typical 28-day concrete strength. In areas where steel casing is present this value may be increased to 17,000 psi.

Exhibit 6-6: LPILE Geotechnical Input Parameters for Drilled Shaft Foundations on Deeper Rock

Depth		Unit Description	LPILE Model	Effective Unit Weight (pcf)	Friction Angle (deg)	k (pci)	Uniaxial Compressive Strength (psi)
Layer Top (ft)	Layer Bottom (ft)						
0	5	Residual Soil	Sand (Reese)	130	40	225	--
5	30	Weathered Basalt	Sand (Reese)	140	45	300	--
30	30+	Columbia River Basalt	Strong Rock (Vuggy Limestone)	150	--	--	4,000[1]

1 Reduced value equivalent to typical 28-day concrete strength. In areas where steel casing is present this value may be increased to 17,000 psi.

The estimated lateral capacity parameters presented in Exhibits 6-5 and 6-6 are recommended for drilled shafts with center-to-center spacing greater than or equal to five shaft diameters (5B) and in a single row. For spacing less than 5B, or if multiple rows of shafts are present, the lateral capacity should be reduced by the factors presented in Exhibit 6-7. Shaft spacing of less than 3B is not recommended. P-Multipliers may be linearly interpolated for spacings between 3B and 5B.

Exhibit 6-7: Lateral Capacity Reduction Factors (P Multipliers) for 3B and 5B Micropile Spacings

Spacing	Row #	P Multiplier	Spacing	Row #	P Multiplier
3B	1	0.70	5B	1	1.0
	2	0.50		2	0.85
	3	0.35		3	0.70
	3+	0.35		3+	0.70

7 CONSTRUCTION CONSIDERATIONS

7.1 Site Preparation and Earthwork

7.1.1 Stripping and Grubbing

Organic material and topsoil should be stripped and removed from all proposed structure and pavement areas. Based on our explorations, we anticipate a stripping depth of approximately 3 to 6 inches. Greater depths may be necessary to remove localized zones of organic material. Stripped material should be transported off site for disposal or used as fill in landscaping areas.

We recommend that the primary root systems for trees and other vegetation be completely removed. Trees designated for preservation should be clearly marked prior site stripping. Trees and their root balls should be grubbed to the depth of the roots, which would exceed 3 feet bgs. Depending on the methods used to remove the root balls, considerable disturbance of the subgrade could occur during site clearing and grubbing. We recommend that soil disturbed during clearing and grubbing operations be improved as described in subsequent sections of this report.

7.1.2 Rock Excavation

A proposed grading plan was not available at the time of this report. Based on information from our explorations soil and residual soil were encountered from the ground surface to depths between approximately 3 and 6 feet. The residual soil may contain cobble and boulder sized strong basalt inclusions. Beneath the residual soil basalt rock ranging from weathered, extremely weak to weak (R0-R2), to fresh, very strong (R5) was encountered in all of the explorations. Very strong rock (R5) was encountered as shallow as 5 feet below ground surface in borings and B-3 and B-6. Shallow rock should be expected in areas where topographic rises are present.

While the decomposed basalt may be removed using traditional rock excavators (large size with rock teeth), the medium strong to very strong (competent rock) will require excavation methods other than traditional rock excavators, which is referred to as "rock excavation." The rock excavation techniques can consist of blasting or non-blasting techniques such as mechanical methods using pneumatic-hammer breakers or chippers will be used for finishing rock surfaces and smaller excavations.

7.1.3 Foundation Subgrade Preparation

Prior to the placement of crushed rock fill or the construction of foundations, we recommend proof rolling the subgrade with a fully loaded dump truck or similarly sized rubber-tire construction equipment to identify areas of excessive yielding. The proof rolling should be observed by a member of our geotechnical staff or a qualified geotechnical engineer who will evaluate the subgrade. If areas of excessive yielding are identified, the material should be excavated and replaced with compacted crushed rock gravel fill. During periods of extended wet weather or in areas where proof-rolling cannot be performed the subgrade should be probed by a qualified geotechnical engineer.

Imported crushed rock beneath structures should consist of ¾-inch minus well-graded crushed rock, with less than 5 percent passing the No. 200 sieve. Crushed rock beneath structures should be placed and compacted to 95 percent of the maximum dry density as determined by ASTM D 1557 (Modified Proctor) on the prepared subgrade.

All footing subgrade should be trimmed neat and carefully prepared. Any deleterious, loose, or softened material should be removed from the footing excavation prior to placing rebar and/or concrete. We recommend that the footing excavations be observed by a Geotechnical Engineer of Record or their representative prior to placing steel and concrete, to evaluate the suitability of the exposed subgrade, that the recommendations of this report have been followed, and that conditions encountered are as anticipated. All deleterious, soft or unsuitable materials observed by the Geotechnical Engineer should be removed and replaced with crushed rock. We recommend a contingency be placed in the budget for over-excavation of footing, floor slab, and mat subgrade.

7.1.4 Elimination of Subgrade Hard Spots and Fill Voids

The subgrade level could encounter surfaces of fractured, weathered, or sound (non-rippable) bedrock resulting in an excavation surface that is uneven and may possibly contain loose rock pieces. We recommend that this surface either be recompacted, loose pieces or unsatisfactory material removed, or the material be grouted in-place as described below. In addition, if cobbles, boulders, or portions of the sound rock layer extend vertically beyond a specified grade elevation into the crushed rock layer, the protruding material should be removed to eliminate any "hard" spots in the subgrade. The maximum tolerance of a particle above specific subgrade should not be more than 2 inches. If removal causes a hole or depression in the subgrade, these holes should be filled with crushed rock material to create a relatively uniform foundation support subgrade. A representative of the Geotechnical Engineer of Record should determine the depth of removal and appropriate material for filling and leveling. For recompaction, due to the likely oversized nature of the material, a procedural compaction/proof rolling method with specified and approved compaction equipment and number of passes (minimum of two vibratory coverages followed by two coverages with equipment in the static mode) is recommended.

7.2 Temporary Shoring and Dewatering of Excavations

Temporary excavations and trenches are typically the responsibility of the contractor and should comply with applicable local, state, and federal safety guidelines, including the current Occupational Safety and Health Administration (OSHA) Excavation and Trench Safety Standards. If shoring is used, we recommend that the type and design of the shoring system and dewatering be the responsibility of the contractor, who is in the best position to choose a system that fits the plan of operation. Any water that is encountered and collected during the various excavations (as well as any excavated soil) should be treated and disposed of in a manner meeting local, state, and federal environmental regulations and requirements, or as determined by the owner.

In general, based on our explorations, temporary cut slopes in on-site soils should be inclined no steeper than approximately 1H:1V (horizontal to vertical) except in competent basalt bedrock. Near vertical rock cuts may be possible in very strong basalt depending on the shape, smoothness and orientation of the rock joints. In Appendix C, Photos of Rock Cuts at SW 124 Avenue, Figures C1 and C3 show examples of cuts in the decomposed basalt. Figure C2 presents a photo that shows an example of rock cut in the basalt. This assumes that surface loads are kept away from the top of any slopes and that seepage is not on or near the slope. Flatter slopes may be necessary depending on specific site conditions.

7.3 Wet Weather Earthwork

The soil at the site contains silts and fines that may produce an unstable mixture when exposed to moisture. Such soils are susceptible to changes in water content, and they tend to become unstable and difficult or impossible to compact if their moisture content significantly exceeds the optimum. If wet conditions are encountered, we recommend the following:

- The ground surface in and surrounding the construction area should be sloped as much as possible to promote runoff of precipitation away from work areas and to prevent ponding of water.
- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough so that the removal of unsuitable soils and placement and compaction of fill materials can be accomplished on the same day.
- Any accidental overexcavation should be filled with crushed rock.
- The size of construction equipment may have to be limited to prevent soil disturbance. It may be necessary to excavate soils with a backhoe or equivalent, located so that equipment does not traffic over the excavated area. Thus, subgrade disturbance caused by equipment traffic will be minimized.
- No soil should be left uncompacted and exposed to moisture. A smooth-drum roller or equivalent should roll the surface to seal out as much water as possible.
- In-place soils or fill soils that become wet and unstable and/or too wet to suitably compact should be removed and replaced with crushed rock.
- Grading and earthwork should not be performed during periods of heavy, continuous rainfall.

We suggest that these recommendations for wet weather earthwork be included in the contract specifications.

7.4 Drilled Micropile Construction Considerations

7.4.1 General

The selection of equipment and procedures for constructing drilled micropiles should consider shaft diameter and length and subsurface conditions. The design and performance of micropiles can be significantly influenced by the equipment and construction procedures used for installation. Generally, drilled micropiles are constructed by boring to the prescribed embedment with an air rotary drill or other drilling tool. Upon completion of drilling, a steel prestressing element is placed within the borehole, and cement grout is pumped into the hole to complete the drilled micropile. Pile contractors who participate on this project should be required to demonstrate that they have suitable equipment for this project and adequate experience in the construction of drilled micropiles with similar subsurface conditions.

The contractor should anticipate that drilling in the Columbia River Basalt may be difficult and slow. The Columbia River Basalt is typically slightly to moderately weathered with rock strengths ranging from strong to very strong (R4-R5).

7.4.2 Micropile Quality Control

We recommend full-time observation of the micropiles by a qualified geotechnical engineer to observe the contractor's means, methods, and equipment as well as to assist the construction management team with an understanding of the critical issues for the micropile construction. We recommend proof-testing of all micropiles to ensure that design capacities have been achieved at all locations. In addition, the design geotechnical engineer and structural engineer should make periodic visits.

7.5 Drilled Shaft Construction Considerations

7.5.1 General

The drilled shaft installation procedures should follow the PGE Specifications for drilled shafts, combined with project-specific provisions that may apply. The selection of equipment and procedures for constructing drilled shafts should consider shaft diameter and length, as well as subsurface conditions. The design and performance of drilled shafts can be significantly influenced by the equipment and construction procedures used to install the shafts.

Generally, the drilled shafts are constructed by excavating a cylindrical bore to the prescribed embedment with a large-diameter auger or other drilling tool. Temporary or permanent casing is often used, depending on site conditions. Upon completion of drilling and inspection of the shaft, a steel rebar cage is placed, and concrete is pumped into the hole to complete the drilled shaft. There is a possibility for instability and difficult drilling in the residual soil gravels, and there may be a potential for perched groundwater flow at the interface of the soils and rock. If these conditions result in caving or instability of the drilled shafts, we recommend that the drilled shafts be constructed using fully-cased excavations down to the top of rock at the site. The drilled shafts should be constructed in the wet, and the casing should be advanced ahead of the auger.

Drilled shaft contractors who participate on this project should be required to demonstrate that they have suitable equipment for this project and adequate experience in the construction of shafts with similar subsurface conditions.

The contractor should anticipate that drilling in the Columbia River Basalt may be difficult and slow. The Columbia River Basalt is typically slightly to moderately weathered with rock strengths ranging from strong to very strong (R4-R5).

7.5.2 Drilled Shaft Quality Control

We recommend full-time observation of the drilled shafts by a qualified engineer or geologist or an engineer in order to observe the contractors' means, methods, and equipment, and to assist the drilled shaft inspector with an understanding of the critical issues for the drilled shaft construction. In addition, the design geotechnical engineer and structural engineer should make periodic visits. We recommend that crosshole sonic log (CSL) tubes be installed in every shaft and that testing be performed on the shafts in accordance with PGE specifications and any special provisions.

8 ADDITIONAL SERVICES

As previously mentioned, we recommend that Shannon & Wilson be retained to observe the geotechnical aspects of construction, particularly foundation subgrades. Observation will allow us to evaluate the subsurface conditions as they are exposed during construction and to determine that the work is accomplished in accordance with our recommendations and the intent of the project specifications.

9 LIMITATIONS

The data collection, analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist, and further assume that the explorations are representative of the subsurface conditions throughout the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the explorations. If subsurface conditions different from those encountered in the explorations are encountered or appear to be present during construction, we should be advised at once so that we can review these conditions and reconsider our recommendations, where necessary. If there is a substantial lapse of time between the submission of this report and the start of construction at the site, or if conditions have changed because of natural forces or construction operations at or adjacent to the site, we recommend that we review our report to determine the applicability of the conclusions and recommendations.

Within the limitations of scope, schedule and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice at the time this report was prepared. We make no other warranty, either express or implied. These conclusions and recommendations were based on our understanding of the project as described in this report and the site conditions as observed at the time of our explorations.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples from geotechnical borings. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

This report was prepared for the exclusive use of Power Engineers and PGE. The data and report should be provided to the contractors for their information, but our report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions included in this report.

The scope of our present work did not include environmental assessments or evaluations regarding the presence or absence of wetlands, or hazardous or toxic substances in the soil, surface water, groundwater, or air, on or below or around this site, or for the evaluation or disposal of contaminated soils or groundwater should any be encountered.

Please read the Important Information section at the back of this report to reduce your project risks.

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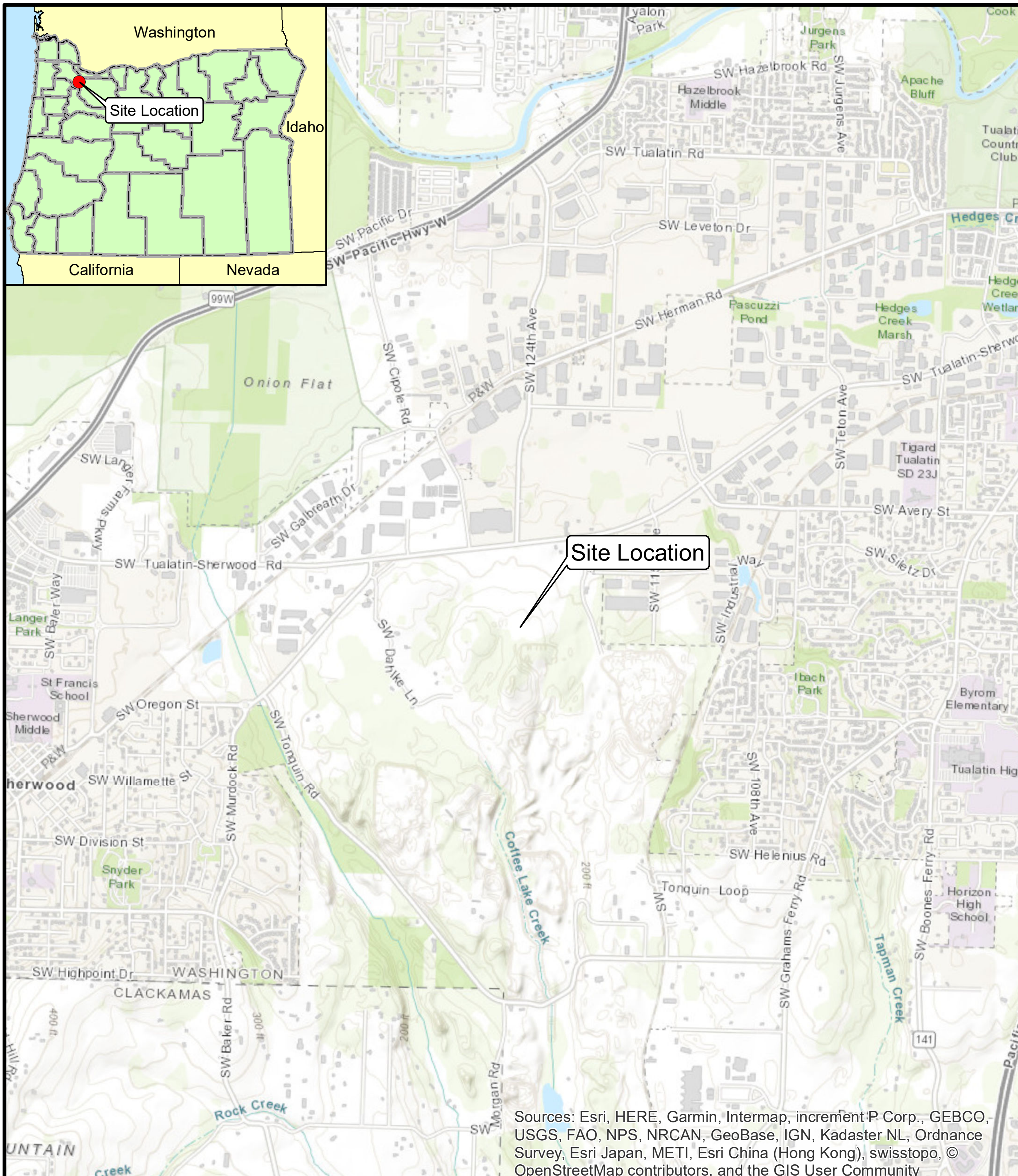

0 0.25 0.5 1

Scale in Miles

N

PGE Tonquin Substation
Tualatin, Oregon

VICINITY MAP

January 2021 106157

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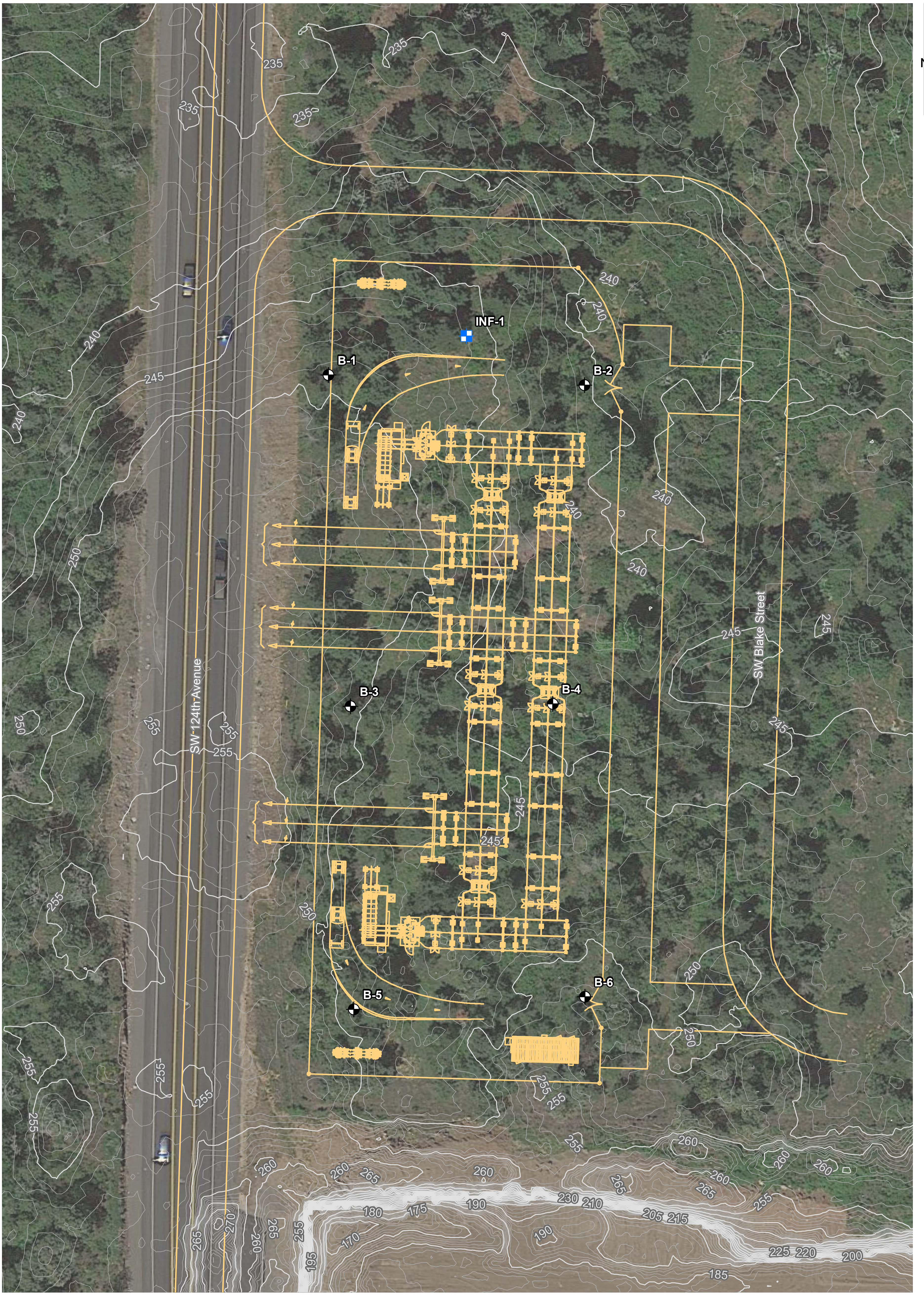
FIG. 1

Filename: T:\Projects\PDX\106000s\106157 - PGE Tonquin Substation\PGE Tonquin Substation\Figure 1 - Vicinity Map.mxd Date: 12/8/2020 Login: DSJ

Filename: T:\Projects\PDX\106000s\106157 - PGE Tonquin Substation\PGE Tonquin Substation\Figure 2 - Site and Exploration Plan_10.6.mxd Date: 12/17/2020 Login: AEH



LEGEND

- Approximate Location of Geotechnical Boring
- Approximate Location of Infiltration Test
- Approximate Location of Proposed Site Layout

0 30 60 120

Scale in Feet

NOTES

1. Aerial imagery obtained through Google Maps Satellite.
2. Contours were derived from 2014 LiDAR obtained from DOGAMI.
3. Proposed site layout based on file TNQN-1000-1-Preliminary.dwg, provided by Power Engineers, Inc., on December 15, 2020.

PGE Tonquin Substation
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SITE AND EXPLORATION PLAN

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FIG. 2

FIG. 2

APPENDIX A: FIELD EXPLORATIONS

Appendix A

Field Explorations

CONTENTS

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APPENDIX A: FIELD EXPLORATIONS

A.1 GENERAL

Shannon & Wilson, Inc., explored subsurface conditions at the proposed substation site with six geotechnical borings, designated B-1 through B-6, and one infiltration test designated IN-1. Completed boring locations were measured off existing features. Approximate boring and infiltration test locations are shown on the Site and Exploration Plan, Figure 2. Shannon & Wilson geologists were present during the explorations to observe temporary access path clearing, locate the drilling sites, check for underground utilities, observe drilling, log the materials encountered, and collect soil and rock samples for laboratory testing. Table A-1 provides a summary of borehole information, including boring designation, sampling depths, sampling types, sampling times, and results of laboratory testing.

This appendix describes the techniques used to advance and sample the borings and presents logs of the materials encountered.

A.2 DRILLING

On November 11, 2020, access paths and drill pads were cleared by a Komatsu PC 40 mini excavator provided an operated by Western States Soil Conservation Inc. of Hubbard, Oregon. After the paths and pads were cleared the geotechnical borings were performed between November 18 and November 20, 2020 by Western States using a CME-850 track mounted drill rig. The six geotechnical borings were advanced to depths ranging from 16.5 to 30.4 feet bgs. The borings were advanced using open-hole mud rotary and continuous HQ3-wireline rock coring drilling techniques.

A.2.1 Disturbed Sampling

Disturbed samples were collected in the borings, typically at 2.5- to 5-foot depth intervals in soil, using a standard 2-inch outside diameter (O.D.) split spoon sampler in conjunction with Standard Penetration Testing. In a Standard Penetration Test (SPT), ASTM D1586, 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, or N-value. The SPT N-value provides a measure of in situ relative density of cohesionless soils (silt, sand, and gravel), and the consistency of cohesive soils (silt and clay). All disturbed samples were visually identified and described in the field, sealed in plastic jars to retain moisture, and returned to our laboratory for additional examination and testing.

SPT N-values can be significantly affected by several factors, including the efficiency of the hammer used. Automatic hammers generally have higher energy transfer efficiencies than cathead driven hammers. The average measured efficiency of the automatic hammer used for this project, based on available information we received from Western States was 85 percent. For reference, cathead hammers are typically assumed to have an average energy efficiency of 60 percent. All N-values presented in this report are in blows per foot, as counted in the field. No corrections of any kind have been applied.

An SPT was considered to have met refusal where more than 50 blows were required to drive the sampler 6 inches. If refusal was encountered in the first 6-inch interval (for example, 50 for 1.5”), the count is reported as 50/1st 1.5”. If refusal was encountered in the second 6-inch interval (for example, 48, 50 for 1.5”), the count is reported as 50/1.5”. If refusal was encountered in the last 6-inch interval (for example, 39, 48, 50 for 1.5”), the count is reported as 98/7.5”.

A.2.2 Continuous HQ-Wireline Rock Coring

Continuous HQ3-wireline rock coring was used in borings B-1 through B-3, B-5 and B-6 to advance through and sample bedrock in accordance with ASTM D2113. An approximate 5-foot long HQ core barrel was used to acquire the approximate 2.5-inch diameter rock core samples. Rock core samples were measured, visually described, boxed, and photographed in the field then transported to the Shannon &Wilson laboratory for storage.

The rock core recovery (presented on the Logs of Borings) was calculated by dividing the length of core recovered in the barrel by the length of the total drilled run. This ratio is expressed as a percent.

The rock quality designation (RQD), also presented on the Logs of Borings, is a modified core recovery percentage including only the total length of the specimens of intact rock more than 4 inches in length, divided by the total length of the core run. The smaller pieces are considered to be the result of close jointing, fracturing, or weathering in the rock mass and are excluded from the determination. Difficulties such as distinguishing natural fractures in the rock core from mechanical breaks due to drilling operations restrict the use of the RQD in evaluating in situ rock properties. However, it does provide a subjective estimate of rock mass quality and a comparison of rock quality in the borings.

A.3 MATERIAL DESCRIPTIONS

In the field, soil samples were described and identified visually in general accordance with ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual

Procedure). Consistency, color, relative moisture, degree of plasticity, and other distinguishing characteristics of the samples were noted. Once returned to the laboratory, soil samples were re-examined, various standard index tests were performed, and the field descriptions and identifications were modified as necessary. We refined our visual-manual soil descriptions and identifications based on the results of the laboratory tests, using elements of the Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), ASTM D2487. However, ASTM D2487 was not followed in full, because it requires a suite of tests be performed to classify a single sample. The specific terminology used in the soil classifications is defined on the Soil Description and Log Key, Figure A1.

The rock core was classified based on International Society for Rock Mechanics methods. The specific terminology used in the rock classification is defined on the Rock Classification and Log Key, Figure B2.

A.4 LOGS OF BORINGS

Summary logs of all Shannon & Wilson geotechnical borings are presented in Figures A3 through A8. Material descriptions and interfaces on the logs are interpretive, and actual changes may be gradual. The left-hand portion of the boring logs provides description, identification, and geotechnical unit designation for the materials encountered in the boring. The right-hand portion of the boring logs shows a graphic log, sample locations and designations, borehole installation details, and a graphical representation of N-values, natural water contents, Atterberg limits, fines content, and sample recovery.

A.5 BOREHOLE ABANDONMENT

Borings were backfilled with bentonite chips in accordance with Oregon Water Resource Department regulations, up to a depth of approximately 1 foot. Native soil was used as backfill from approximately 1 foot up to the ground surface.

A.6 INFILTRATION TESTING

One in situ infiltration test was completed using the Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer in accordance with ASTM D3385. The infiltration test was performed at location IN-1, shown on the Site and Exploration Plan, Figure 2. At the infiltration test location, the area was excavated to an approximate depth of 12-inches by the excavator used to clear the access paths. The Double-ring Infiltration test was performed on November 20, 2020. The double-ring infiltrometer consisting of 24-inch diameter outer and a 12-inch diameter rings, 20-inches in height was driven into the ground of the excavated area with a sledge to a depth of 6-inches. The volume of liquid used to maintain a reference head of 6-inches of water in the inner and outer rings was measured at time intervals of 15 minutes for the first hour, 30 minutes for the second hour, and 60 minutes thereafter until a relatively constant rate is obtained or a minimum total amount of 6 hours. The ground temperature of the mid-depth of the test zone and temperature of the inner and outer water heads is also measured. The results of the last two measurements indicated a constant rate of 1.8 inches/hour. According to Washington County On-Site Stormwater Disposal System (OSDS) Design and Construction Minimum Guidelines and Requirements, Second Edition, September 26, 2007, for all infiltration facilities, a minimum infiltration rate of 0.5 inches per hour is required. Washington County further estimates percolation rates for Sandy loams/Porous silt loams/Silty clay loams at 0.2-6.0 inches per hour which is verified with the onsite measured infiltration rate of 1.8 inches per hour. Washington County provides correction factors in the OSDS to determine the design infiltration rate based on measured infiltration rate, uncertainties in the testing methods, facility geometry and reductions or infiltration rates over the long term due to plugging of soils. No corrections factors were applied to the measured infiltration rate presented. The Infiltration test results are presented in Figure B2.

Table A-1: Summary of Geotechnical Borehole Information.

Borehole Designation	Sample Designation	Sample Type	Sample Depth (ft)	Time Sampled	Notes
B-1	S-1	SPT	2.5-2.9	1504	MC-23%
B-1	S-2	SPT	5-5.2	1521	-
B-1	S-3	SPT	7.5-7.5	1533	-
B-1	C-1	HQ Rock Core	7.5-11	1544-1552	UCS-19,818psi
B-1	C-2	HQ Rock Core	11-16	1600-1613	-
B-1	C-3	HQ Rock Core	16-21	1619-1633	UCS-10,070psi
B-2	S-1	SPT	2.5-2.8	0911	-
B-2	S-2	SPT	5-5.3	0928	-
B-2	S-3	SPT	7.5-7.7	0942	-
B-2	S-4	SPT	10-10.1	0954	-

B-2	C-1	HQ Rock Core	10.1-11	1023-1027	-
B-2	C-2	HQ Rock Core	11-15	1032-1050	UCS-17,424psi
B-2	C-3	HQ Rock Core	15-18.5	1058-1119	-
B-2	C-4	HQ Rock Core	18.5-20	1151-1204	-
B-3	S-1	SPT	2.5-2.6	1138	-
B-3	S-2	SPT	5-5.1	1202	-
B-3	C-1	HQ Rock Core	5.1-6	1215-1219	-
B-3	C-2	HQ Rock Core	6-11	1225-1234	UCS-23,378psi
B-3	C-3	HQ Rock Core	11-16	1242-1255	-
B-3	C-4	HQ Rock Core	16-21	1303-1322	UCS-4,402psi
B-4	S-1	SPT	2.5-4	0903	-
B-4	S-2	SPT	5-5.8	0912	-
B-4	S-3	SPT	7.5-7.9	0922	-
B-4	S-4	SPT	10-10.4	0932	-
B-4	S-5	SPT	15-16.5	0944	-
B-4	S-6	SPT	20-21.4	0958	-
B-4	S-7	SPT	25-25.4	1012	-
B-4	S-8	SPT	30-30.4	1030	-
B-5	S-1	SPT	2.5-4	0930	LL=42, PL=25, PI=17, MC=28%
B-5	S-2	SPT	5-5.9	0938	MC-33%
B-5	S-3	SPT	7.5-8.4	0950	-

APPENDIX A: FIELD EXPLORATIONS

B-5	S-4	SPT	10-10.2	1000	-
B-5	C-1	HQ Rock Core	10.2-11	1042-1049	-
B-5	C-2	HQ Rock Core	11-15	1053-1104	-
B-5	C-3	HQ Rock Core	15-16	1110-1115	-
B-5	C-4	HQ Rock Core	16-21	1126-1142	-
B-6	S-1	SPT	2.5-2.7	1344	-
B-6	S-2	SPT	5-5.1	1412	-
B-6	C-1	HQ Rock Core	6.5-11.5	1457-1508	UCS-26,954psi Solid (no joints)
B-6	C-2	HQ Rock Core	11.5-16.5	1515-1528	-

Notes:

1 SPT-Split Spoon Sample, HQ Rock Core– HQ3 Wireline rock coring, LL-Liquid Limit, PL-Plastic Limit, PI-Plasticity Index, MC-Moisture Content, UCS-Unconfined Compressive Strength

Shannon & Wilson, Inc. (S&W), uses a soil identification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following pages. Soil descriptions are based on visual-manual procedures (ASTM D2488) and laboratory testing procedures (ASTM D2487), if performed.

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

CONSTITUENT[2]	FINE-GRAINED SOILS (50% or more fines)[1]	COARSE-GRAINED SOILS (less than 50% fines)[1]
Major	***Silt, Lean Clay, Elastic Silt,*** or ***Fat Clay***[3]	***Sand*** or ***Gravel***[4]
Modifying (Secondary) Precedes major constituent	30% or more coarse-grained: ***Sandy*** or ***Gravelly***[4]	More than 12% fine-grained: ***Silty*** or ***Clayey***[3]
Minor Follows major constituent	15% to 30% coarse-grained: ***with Sand*** or ***with Gravel***[4]	5% to 12% fine-grained: ***with Silt*** or ***with Clay***[3]
	30% or more total coarse-grained *and* lesser coarse-grained constituent is 15% or more: ***with Sand*** or ***with Gravel***[5]	15% or more of a second coarse-grained constituent: ***with Sand*** or ***with Gravel***[5]

[1]All percentages are by weight of total specimen passing a 3-inch sieve.
[2]The order of terms is: *Modifying Major with Minor*.
[3]Determined based on behavior.
[4]Determined based on which constituent comprises a larger percentage.
[5]Whichever is the lesser constituent.

MOISTURE CONTENT TERMS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

Hammer:	140 pounds with a 30-inch free fall. Rope on 6- to 10-inch-diam. cathead 2-1/4 rope turns, > 100 rpm
Sampler:	10 to 30 inches long Shoe I.D. = 1.375 inches Barrel I.D. = 1.5 inches Barrel O.D. = 2 inches
N-Value:	Sum blow counts for second and third 6-inch increments. Refusal: 50 blows for 6 inches or less; 10 blows for 0 inches.

NOTE: Penetration resistances (N-values) shown on boring logs are as recorded in the field and have not been corrected for hammer efficiency, overburden, or other factors.

PARTICLE SIZE DEFINITIONS

DESCRIPTION	SIEVE NUMBER AND/OR APPROXIMATE SIZE
FINES	< #200 (0.075 mm = 0.003 in.)
SAND	
Fine	#200 to #40 (0.075 to 0.4 mm; 0.003 to 0.02 in.)
Medium	#40 to #10 (0.4 to 2 mm; 0.02 to 0.08 in.)
Coarse	#10 to #4 (2 to 4.75 mm; 0.08 to 0.187 in.)
GRAVEL	
Fine	#4 to 3/4 in. (4.75 to 19 mm; 0.187 to 0.75 in.)
Coarse	3/4 to 3 in. (19 to 76 mm)
COBBLES	3 to 12 in. (76 to 305 mm)
BOULDERS	> 12 in. (305 mm)

RELATIVE DENSITY / CONSISTENCY

COHESIONLESS SOILS		COHESIVE SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
< 4	Very loose	< 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
> 50	Very dense	15 - 30	Very stiff
		> 30	Hard

WELL AND BACKFILL SYMBOLS

Bentonite Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Inclinometer or Non-perforated Casing
Gravel	Vibrating Wire Piezometer
Perforated or Screened Casing	

PERCENTAGES TERMS [1, 2]

Trace	< 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

[1]Gravel, sand, and fines estimated by mass. Other constituents, such as organics, cobbles, and boulders, estimated by volume.

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SOIL DESCRIPTION AND LOG KEY

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FIG. A1
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2013_BORING_CLASS1 106157.GPJ SW2013LIBRARYPDX.GLB SWNEW.GDT 12/10/20

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(Modified From USACE Tech Memo 3-357, ASTM D2487, and ASTM D2488)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL IDENTIFICATIONS
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Gravel *(less than 5% fines)*	GW	Well-Graded Gravel; Well-Graded Gravel with Sand
			GP	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Silty or Clayey Gravel *(more than 12% fines)*	GM	Silty Gravel; Silty Gravel with Sand
			GC	Clayey Gravel; Clayey Gravel with Sand
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Sand *(less than 5% fines)*	SW	Well-Graded Sand; Well-Graded Sand with Gravel
			SP	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Silty or Clayey Sand *(more than 12% fines)*	SM	Silty Sand; Silty Sand with Gravel
			SC	Clayey Sand; Clayey Sand with Gravel
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			CL	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
		Organic	OL	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
			CH	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
		Organic	OH	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat or other highly organic soils (see ASTM D4427)

FILL	Placed by humans, both engineered and nonengineered. May include various soil materials and debris.		The Fill graphic symbol is combined with the soil graphic that best represents the observed material

NOTE: No. 4 size = 4.75 mm = 0.187 in.; No. 200 size = 0.075 mm = 0.003 in.

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, Sand with Silt)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the *CL-ML* area of the plasticity chart.
2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand)* indicate that the soil properties are close to the defining boundary between two groups.
3. The soil graphics above represent the various USCS identifications (i.e., *GP*, *SM*, etc.) and may be augmented with additional symbology to represent differences within USCS designations. *Sandy Silt (ML)*, for example, may be accompanied by the *ML* soil graphic with sand grains added. Non-USCS materials may be represented by other graphic symbols; see log for descriptions.

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SOIL DESCRIPTION AND LOG KEY

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FIG. A1
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2013_BORING_CLASS2 106157.GPJ SW2013LIBRARYPDX.GLB SWNEW.GDT 12/10/20

GRADATION TERMS

Poorly Graded	Narrow range of grain sizes present or, within the range of grain sizes present, one or more sizes are missing (Gap Graded). Meets criteria in ASTM D2487, if tested.
Well-Graded	Full range and even distribution of grain sizes present. Meets criteria in ASTM D2487, if tested.

CEMENTATION TERMS[1]

Weak	Crumbles or breaks with handling or slight finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

PLASTICITY[2]

DESCRIPTION	VISUAL-MANUAL CRITERIA	APPROX. PLASITICTY INDEX RANGE
Nonplastic	A 1/8-in. thread cannot be rolled at any water content.	< 4%
Low	A thread can barely be rolled and a lump cannot be formed when drier than the plastic limit.	4 to 10%
Medium	A thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. A lump crumbles when drier than the plastic limit.	10 to 20%
High	It take considerable time rolling and kneading to reach the plastic limit. A thread can be rerolled several times after reaching the plastic limit. A lump can be formed without crumbling when drier than the plastic limit.	> 20%

ADDITIONAL TERMS

Mottled	Irregular patches of different colors.
Bioturbated	Soil disturbance or mixing by plants or animals.
Diamict	Nonsorted sediment; sand and gravel in silt and/or clay matrix.
Cuttings	Material brought to surface by drilling.
Slough	Material that caved from sides of borehole.
Sheared	Disturbed texture, mix of strengths.

PARTICLE ANGULARITY AND SHAPE TERMS[1]

Angular	Sharp edges and unpolished planar surfaces.
Subangular	Similar to angular, but with rounded edges.
Subrounded	Nearly planar sides with well-rounded edges.
Rounded	Smoothly curved sides with no edges.
Flat	Width/thickness ratio > 3.
Elongated	Length/width ratio > 3.

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ACRONYMS AND ABBREVIATIONS

ATD	At Time of Drilling
approx.	Approximate/Approximately
Diam.	Diameter
Elev.	Elevation
ft.	Feet
FeO	Iron Oxide
gal.	Gallons
Horiz.	Horizontal
HSA	Hollow Stem Auger
I.D.	Inside Diameter
in.	Inches
lbs.	Pounds
MgO	Magnesium Oxide
mm	Millimeter
MnO	Manganese Oxide
NA	Not Applicable or Not Available
NP	Nonplastic
O.D.	Outside Diameter
OW	Observation Well
pcf	Pounds per Cubic Foot
PID	Photo-Ionization Detector
PMT	Pressuremeter Test
ppm	Parts per Million
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
rpm	Rotations per Minute
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
q_u	Unconfined Compressive Strength
VWP	Vibrating Wire Piezometer
Vert.	Vertical
WOH	Weight of Hammer
WOR	Weight of Rods
Wt.	Weight

STRUCTURE TERMS[1]

Interbedded	Alternating layers of varying material or color with layers at least 1/4-inch thick; singular: bed.
Laminated	Alternating layers of varying material or color with layers less than 1/4-inch thick; singular: lamination.
Fissured	Breaks along definite planes or fractures with little resistance.
Slickensided	Fracture planes appear polished or glossy; sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps that resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay.
Homogeneous	Same color and appearance throughout.

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SOIL DESCRIPTION AND LOG KEY

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FIG. A1
Sheet 3 of 3

2013_BORING_CLASS3 106157.GPJ SW2013LIBRARYPDX.GLB SWNEW.GDT 12/10/20

BASED ON INTERNATIONAL SOCIETY FOR ROCK MECHANICS (ISRM) ROCK CLASSIFICATION METHODS[R]

FABRIC TERMS

SEDIMENTARY ROCKS

MASSIVE - Rock without significant structure

BEDDED - Regular layering from sedimentation

FISSILE - Tendency to break along laminations

METAMORPHIC ROCKS

FOLIATED - Parallel arrangement or distribution of minerals

SCHISTOSE - Parallel arrangement of tabular minerals giving a planar fissility

GNEISSOSE - Segregation of minerals into bands

CLEAVAGE - Tendency to split along secondary, planar textures or structures

STRENGTH

GRADE	DESCRIPTION	FIELD IDENTIFICATION	APPROXIMATE RANGE OF UNIAXIAL COMPRESSIVE STRENGTH (MPa)	(psi)
R0	Extremely Weak Rock	Indented by thumbnail	0.25 to 1	36 to 145
R1	Very Weak Rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1 to 5	145 to 700
R2	Weak Rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5 to 25	700 to 3,600
R3	Medium Strong Rock	Cannot be scraped or peeled by a pocket knife, specimen can be fractured with single firm blow of geological hammer	25 to 50	3,600 to 7,200
R4	Strong Rock	Specimen requires more than one blow of geological hammer to fracture it	50 to 100	7,200 to 14,500
R5	Very Strong Rock	Specimen requires many blows of geological hammer to fracture it	100 to 250	14,500 to 36,250
R6	Extremely Strong Rock	Specimen can only be chipped with geological hammer	>250	>36,250

VESCULARITY

Slightly Vesicular	1 to 10%
Moderately Vesicular	10 to 30%
Highly Vesicular	30 to 50%
Scoriaceous	>50%

JOINT ROUGHNESS

SMALL SCALE	INTERMEDIATE SCALE
Rough	Stepped
Smooth	Undulating
Slickensided	Planar

WEATHERING

TERM	DESCRIPTION
Fresh	No visible signs of rock material weathering: perhaps slight discoloration on major discontinuity surfaces.
Slightly Weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and somewhat weaker than in its fresh condition.
Moderately Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.
Highly Weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Completely Weathered	All rock is decomposed and/or disintegrated to soil. The original mass is still largely intact.
Residual Soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

DISCONTINUITY TERMS

FRACTURE - Collective term for any natural break excluding shears, shear zones, and faults

JOINT (JT) - Planar break with little or no displacement

FOLIATION JOINT (FJ) or BEDDING JOINT (BJ) - Joint along foliation or bedding

INCIPIENT JOINT (IJ) or INCIPIENT FRACTURE (IF) - Joint or fracture not evident until wetted and dried; breaks along existing surface

RANDOM FRACTURE (RF) - Natural, very irregular fracture that does not belong to a set

BEDDING PLANE SEPARATION or PARTING - A separation along bedding after extraction from stress relief or slaking

FRACTURE ZONE (FZ) - Planar zone of broken rock without gouge

MECHANICAL BREAK (MB) - Breaks due to drilling or handling; drilling break (DB), hammer break (HB)

SHEAR (SH) - Surface of differential movement evident by presence of slickensides, striations, or polishing

SHEAR ZONE (SZ) - Zone of gouge and rock fragments bounded by planar shear surfaces

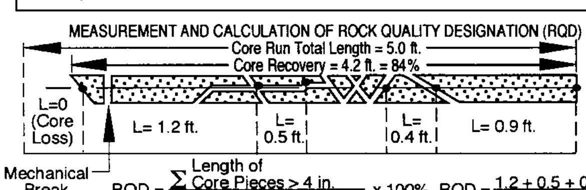
FAULT (FT) - Shear zone of significant extent; differentiation from shear zone may be site-specific

STRUCTURE SPACING TERMS

STRATIGRAPHIC	SPACING	DISCONTINUITY *
Extremely Thick	> 20 ft. (> 6 m)	Extremely Wide
Very Thick	6 to 20 ft. (2 to 6 m)	Very Wide
Thick	2 to 6 ft. (0.6 to 2 m)	Wide
Medium	8 to 24 in. (0.2 to 0.6 m)	Moderate
Thin	2.5 to 8 in. (60 to 200 mm)	Close
Very Thin	1 to 2.5 in. (20 to 60 mm)	Very Close
Laminated: Thickly	0.25 to 1 in. (6 to 20 mm)	Extremely Close
Laminated: Thinly	<0.25 in. (<6 mm)	Extremely Close

* Refers to apparent spacing along core axis unless measured orthogonal to discontinuity; should then report for each set

[R] Reference: Brown, E.T., ed., 1981, Rock Characterization Testing and Monitoring ISRM Suggested methods. New York, International Society for Rock Mechanics (ISRM).



$$RQD = \frac{\sum \text{Length of Core Pieces} > 4 \text{ in.}}{\text{Total Core Run Length}} \times 100\% \quad RQD = \frac{1.2 + 0.5 + 0.4 + 0.9}{5.0} \times 100\% \quad RQD = 60\%$$

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Tualatin, Oregon

ROCK CLASSIFICATION AND LOG KEY

January 2021 | 106157-001

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FIG. A2

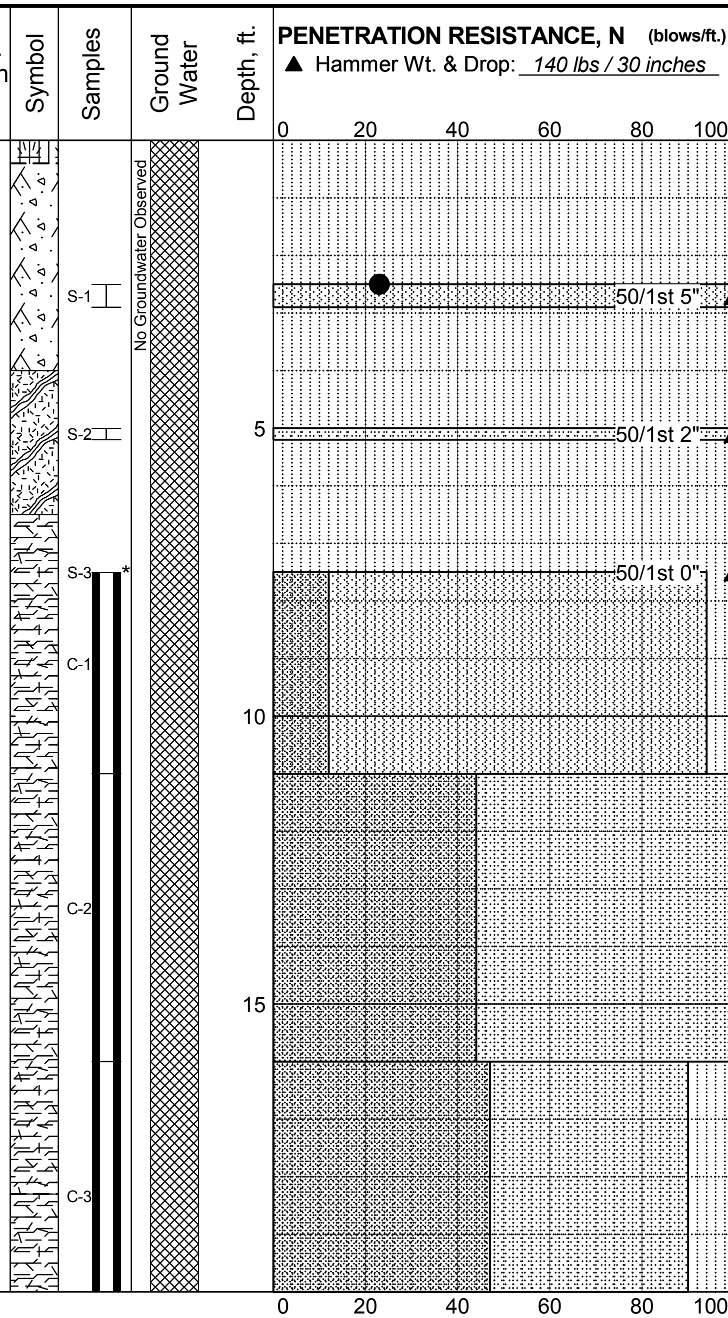
Filename: I:\EF\PDX\106000s\1016157 PGE Tonquin Substation\DRAFTING\Rock Classification- Log Key.dwg Layout: Layout1 Date: 12-10-2020 Login: CKS

Total Depth:	21 ft.	Northing:	~	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:	~	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-850 Tracked Rig	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. Depth (ft.)	Samples
Topsoil	0.4	
Very dense, brown and gray, *Silty Gravel with Sand (GM)*; moist; Fine to coarse, angular to subangular gravel; Fine to coarse sand; nonplastic fines; composed of weathered basalt fragments. **RESIDUAL SOIL**	4.0	S-1
WEATHERED BASALT: Extremely weak to weak (R0-R2); gray-brown and orange-brown; estimated extremely close to close spaced joints; highly fractured; moderate to heavy iron oxidation; highly weathered; may include zones of medium strong (R3) basalt. **WEATHERED COLUMBIA RIVER BASALT**	6.5	S-2
BASALT: Strong to very strong (R4-R5); gray to dark gray; slightly porphyritic; trace vesicles; close to moderate spaced, rough to smooth, undulating, low to moderate (0°-50°) and high (70°-90°) angle, very narrow to moderately wide (0.05-0.5 in.) aperture joints wtih iron oxide and hard clay infilling 1- to 9-mm thick; fresh to slightly weathered. *UCS 13.8-14.6 ft= 19,818 psi* **COLUMBIA RIVER BASALT** *UCS 17.5-18.1 ft= 10,770 psi*	18.3	S-3*, C-1, C-2
BASALT; Medium strong to strong (R3-R4); gray-brown; slightly porphyritic; trace vesicles.		C-3



CONTINUED NEXT SHEET

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core - HQ

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

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Tualatin, Oregon

LOG OF BORING B-1

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FIG. A3
Sheet 1 of 2

REV 3

MASTER_LOG_E 106157.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 1/4/21 Log: VCB Rev: CKS Typ: KJC

Total Depth:	21 ft.	Northing:	~	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:	~	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-850 Tracked Rig	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.) | Symbol | Samples | Ground Water | Depth, ft.

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

0 20 40 60 80 100

Continued:

BASALT; Medium strong to strong (R3-R4); gray-brown; slightly porphyritic; trace vesicles; very close to close spaced, rough to smooth, undulating and stepped, low to high (0° -90°) angle, very narrow to moderately wide (0.05-0.5 in.) aperture joints with iron oxide and hard clay infilling 1- to 5-mm thick; slightly weathered.

COLUMBIA RIVER BASALT

21.0

Completed: November 19, 2020

25

30

35

0 20 40 60 80 100

Recovery (%) RQD (%)

LEGEND

* Sample Not Recovered

Standard Penetration Test

Rock Core - HQ

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

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LOG OF BORING B-1

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FIG. A3
Sheet 2 of 2

REV 3

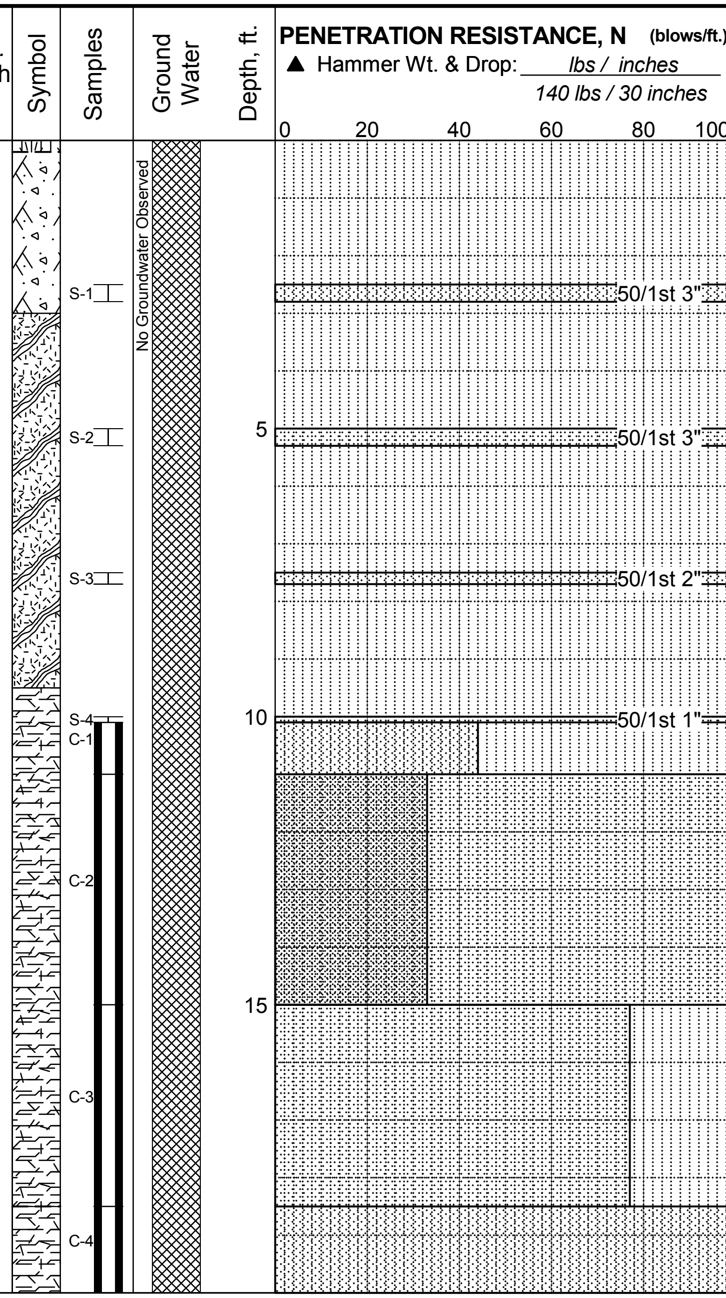
MASTER_LOG_E 106157.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 1/4/21 *Log: VCB Rev: CKS Typ: KJC*

Total Depth: 20 ft.	Northing: ~	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation: ~	Easting: ~	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	Station: ~	Drill Rig Equipment:	CME-850 Tracked Rig	Hammer Type:	Automatic
Horiz. Datum:	Offset: ~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. Depth (ft.)
Topsoil	0.2
Very dense, brown and gray, *Silty Gravel with Sand (GM)*; moist; fine to coarse, angular to subangular gravel; fine to coarse sand; low plasticity fines; moderate iron oxidation and staining; composed of weathered basalt fragments. **RESIDUAL SOIL**	3.0
WEATHERED BASALT: Extremely weak to weak (R0-R2); gray-brown and orange-brown; estimated extremely close to close jointing; highly fractured; highly weathered; may include zones of medium strong (R3) basalt. **WEATHERED COLUMBIA RIVER BASALT**	9.5
BASALT: Strong to very strong (R4-R5); gray to dark gray; slightly porphyritic; trace vesicles; close spaced, rough, undulating, low to moderate (0°-60°) and high (70°-90°) angle, close and healed to narrow (0.1 in.) aperture joints with iron oxide staining and infilling 1- to 3-mm thick; fresh to slightly weathered. *UCS 11.7-18.1 ft= 17,424 psi* **COLUMBIA RIVER BASALT**	18.5
BASALT: Medium strong (R3); brown-gray; slightly porphyritic; moderately vesicular. Continued:	



CONTINUED NEXT SHEET

LEGEND

Standard Penetration Test

Rock Core - HQ

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

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LOG OF BORING B-2

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FIG. A4
Sheet 1 of 2

REV 3

MASTER_LOG_E 106157.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 1/4/21 Log: VCB Rev: CKS Typ: KJC

Total Depth:	20 ft.	Northing:	~	Drilling Method:	Mud Rotary
Top Elevation:	~	Easting:	~	Drilling Company:	Western States
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-850 Tracked Rig
Horiz. Datum:		Offset:	~	Other Comments:	Hammer Efficiency = 85%

Hole Diam.:	6 in.
Rod Type:	NWJ
Hammer Type:	Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft.
BASALT: Medium strong (R3); brown-gray; slightly porphyritic; moderately vesicular; very close spaced, rough, undulating to stepped, low to high (0°-90°) angle, narrow to moderately wide (0.05-0.5 in.) aperture joints with iron oxide staining and hard clay infilling 1- to 9-mm thick; moderately weathered. **COLUMBIA RIVER BASALT**	20.0				
Completed: November 20, 2020					25
					30
					35

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: lbs / inches

140 lbs / 30 inches

0 20 40 60 80 100

0 20 40 60 80 100

Recovery (%) RQD (%)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

LEGEND

Standard Penetration Test

Rock Core - HQ

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

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LOG OF BORING B-2

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FIG. A4
Sheet 2 of 2

REV 3

MASTER_LOG_E 106157.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 1/4/21 *Log: VCB Rev: CKS Typ: KJC*

Total Depth: 21 ft.	Northing: ~	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation: ~	Easting: ~	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	Station: ~	Drill Rig Equipment:	CME-850 Tracked Rig	Hammer Type:	Automatic
Horiz. Datum:	Offset: ~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

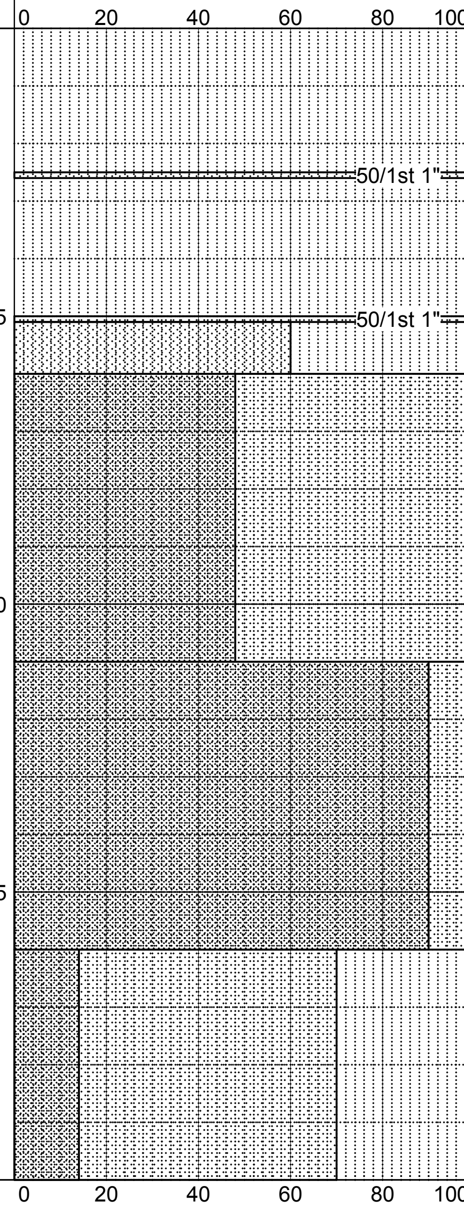
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

SOIL DESCRIPTION	Elev. Depth (ft.)	Samples	Ground Water	Depth, ft.
Topsoil	0.2		No Groundwater Observed	
Very dense, gray and brown, *Poorly Graded Gravel with Silt, Sand, Cobbles and Boulders (GP-GM)*; moist; some cobbles and boulders; fine to coarse, angular to subangular gravel; fine to coarse sand; low plasticity fines; composed of weathered basalt fragments; cobbles and boulders inferred from drill action. **RESIDUAL SOIL**	5.0	S-1		
BASALT: Strong to very strong (R4-R5); gray; slightly porphyritic; trace vesicles; close to moderate spaced, rough to smooth, undulating, low to moderate (0°-50°) and high (70°-90°) angle, narrow to moderately wide (0.05-0.5 in.) aperture joints with iron oxide staining and joint coating and infilling 1- to 4-mm thick; fresh. *UCS 9.0-9.6 ft= 23,378 psi* **COLUMBIA RIVER BASALT**	16.2	S-2, C-1, C-2, C-3		5, 10, 15
BASALT: Weak to medium strong (R2-R3); gray to black; aphanitic; moderately vesicular; close spaced, rough, undulating and stepped, low to high (0°-90°) angle, narrow to moderately wide (0.05-0.5 in.) aperture joints with iron oxide staining and joint coating; moderately weathered; top of flow at 16.4 feet.		C-4		

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: lbs / inches

140 lbs / 30 inches



CONTINUED NEXT SHEET

LEGEND

- Standard Penetration Test
- Rock Core - HQ

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

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LOG OF BORING B-3

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FIG. A5
Sheet 1 of 2

REV 3

MASTER_LOG_E 106157.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 1/4/21 Log: VCB Rev: CKS Typ: KJC

Total Depth:	21 ft.	Northing:	~	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:	~	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-850 Tracked Rig	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

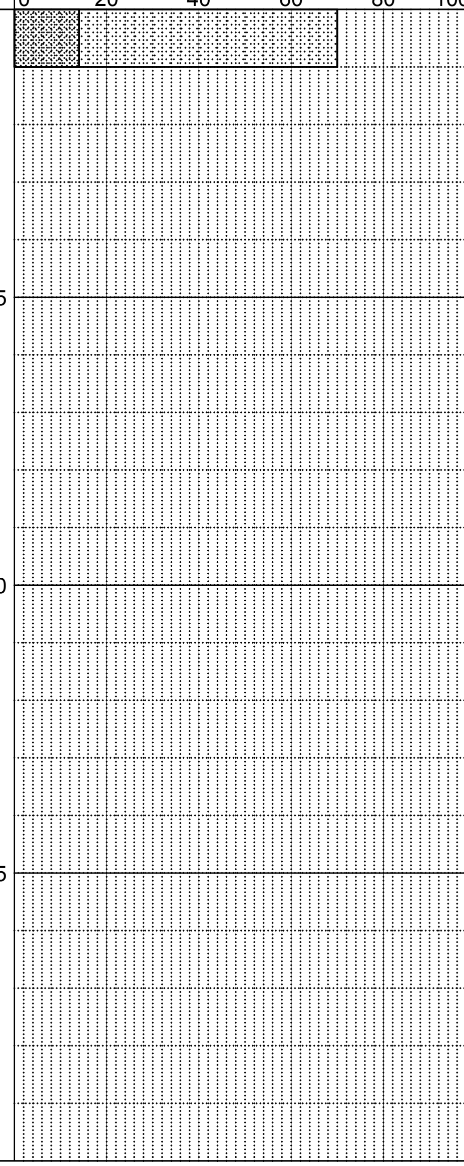
Soil Description	Elev. Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft.
Continued: BASALT: Weak to medium strong (R2-R3); gray to black; aphanitic; moderately vesicular; close spaced, rough, undulating and stepped, low to high (0°-90°) angle, narrow to moderately wide (0.05-0.5 in.) aperture joints with iron oxide staining and joint coating; moderately weathered; paleosol layer from 16.2 to 16.4 feet; top of flow at 16.4 feet. **COLUMBIA RIVER BASALT** *UCS 18-18.6 ft= 4,402 psi*	21.0				25
Completed: November 19, 2020					30
					35

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *lbs / inches*

140 lbs / 30 inches

0 20 40 60 80 100



0 20 40 60 80 100

Recovery (%) RQD (%)

● % Water Content

Plastic Limit Liquid Limit

LEGEND

Standard Penetration Test

Rock Core - HQ

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

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LOG OF BORING B-3

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FIG. A5
Sheet 2 of 2

REV 3

MASTER_LOG_E 106157.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 1/4/21 Log: VCB Rev: CKS Typ: KJC

Total Depth:	30.4 ft.	Northing:	~	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:	~	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-850 Tracked Rig	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~	Other Comments:	Hammer Efficiency = 85%		

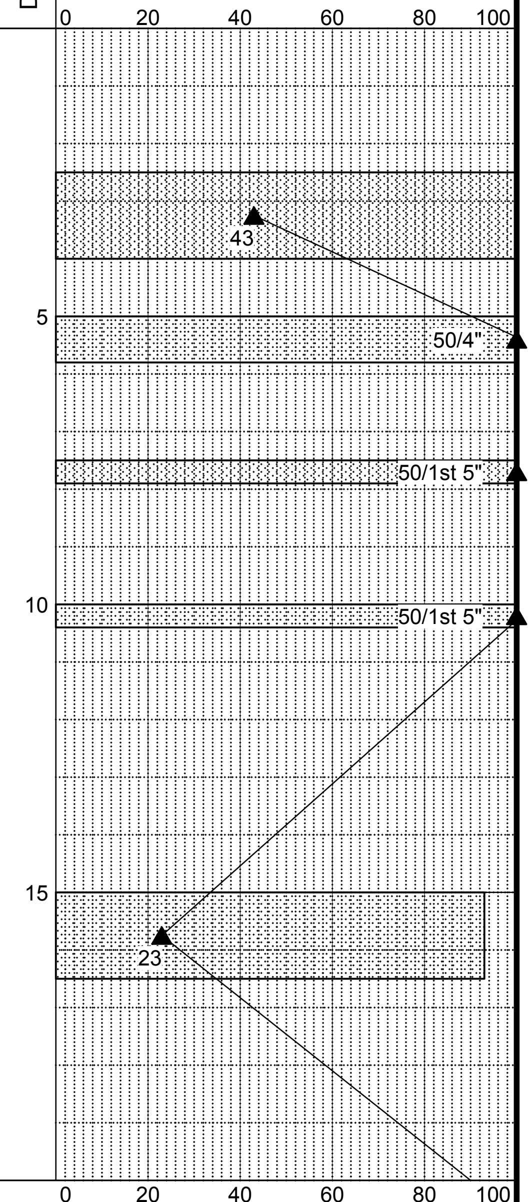
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

SOIL DESCRIPTION	Elev. Depth (ft.)	Samples	Ground Water
Dense, yellow, red, and brown, *Silty Sand with Gravel to Silty Gravel with Sand (SM/GM)*; moist; fine to coarse, angular to subangular gravel; fine to coarse sand; low plasticity fines; relict basalt texture; composed of weathered basalt fragments. **RESIDUAL SOIL**		S-1	No Groundwater Observed
WEATHERED BASALT: Very weak to medium strong (R1-R3); gray-brown, orange-brown and gray; aphanitic; estimated extremely close to close spaced joints with clay infilling; highly fractured; moderate to highly weathered. **WEATHERED COLUMBIA RIVER BASALT**	4.5	S-2, S-3, S-4	
Medium dense, brown, orange-brown and dark brown, *Clayey Gravel with Sand (GC)*; moist; fine to coarse, angular to subangular gravel; fine to coarse sand; low to medium plasticity fines; interflow rubble/paleosol layer. **RESIDUAL SOIL/ROCK FRAGMENTS**	13.0	S-5	
WEATHERED BASALT: Very weak (R1); dark gray and orange-brown; highly to completely weathered to *Silty Sand (SM)*.	18.0		

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



CONTINUED NEXT SHEET

LEGEND

Standard Penetration Test

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

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LOG OF BORING B-4

January 2021 106157

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FIG. A6
Sheet 1 of 2

REV 3

MASTER_LOG_E 106157.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 1/4/21 Log: VCB Rev: CKS Typ: KJC

Total Depth:	30.4 ft.	Northing:	~	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:	~	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-850 Tracked Rig	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~	Other Comments:	Hammer Efficiency = 85%		

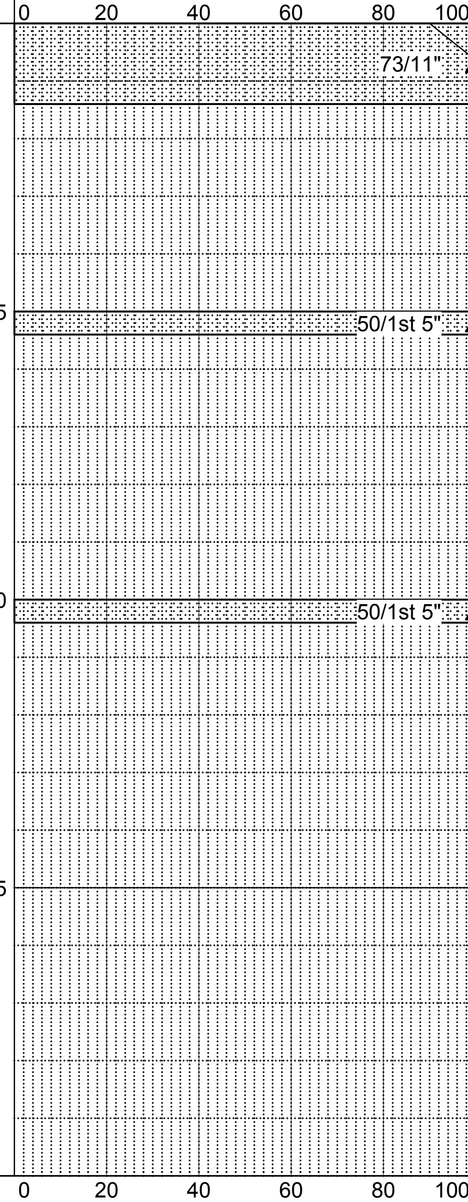
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

SOIL DESCRIPTION	Elev. Depth (ft.)	Samples	Depth, ft.
Continued: WEATHERED BASALT: Very weak (R1); dark gray and orange-brown; fine-grained; highly to completely weathered to *Silty Sand (SM)*; relict basalt texture.		S-6	
WEATHERED COLUMBIA RIVER BASALT			
WEATHERED BASALT: Weak to medium strong (R2-R3); gray-brown to gray; aphanitic; extremely close to close spaced joints; highly fractured; moderate iron oxidation and staining; moderate to highly weathered.	23.0	S-7	25
		S-8	30
Completed: November 19, 2020	30.4		35

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



Recovery (%)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

LEGEND

Standard Penetration Test

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

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LOG OF BORING B-4

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FIG. A6
Sheet 2 of 2

REV 3

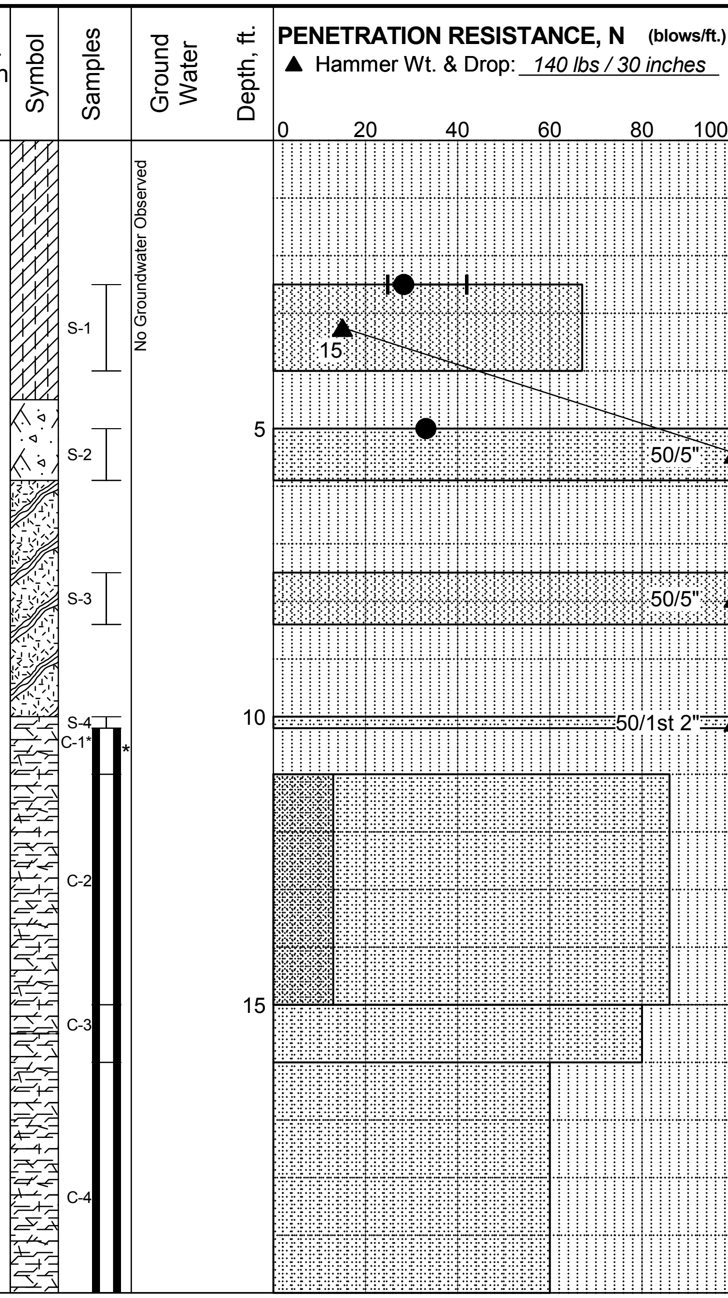
MASTER_LOG_E 106157.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 1/4/21 Log: VCB Rev: CKS Typ: KJC

Total Depth: 21 ft.	Northing: ~	Drilling Method:	Mud Rotary	Hole Diam.:	5 in.
Top Elevation: ~	Easting: ~	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	Station: ~	Drill Rig Equipment:	CME-850 Tracked Rig	Hammer Type:	Automatic
Horiz. Datum:	Offset: ~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. Depth (ft.)
Very stiff, light brown and brown, *Lean Clay with Sand (CL)*; moist; trace fine to coarse, angular to subangular gravel; fine to medium sand; medium plasticity; trace rootlets; micaceous. **FINE-GRAINED MISSOULA FLOOD DEPOSITS**	
Very hard, light gray-brown, yellow, and orange mottled, *Elastic Silt with Sand (MH)*; moist; trace to few, fine to coarse, angular to subangular gravel; fine to coarse sand; high plasticity; moderate iron oxidation and staining; relict basalt texture. **RESIDUAL SOIL**	4.5
WEATHERED BASALT: Extremely weak grading to medium strong (R0-R3); orange-brown and brown grading to gray-brown and gray; aphanitic; trace vesicles; estimated very close spaced joints; highly fractured; slight to moderate iron oxidation and staining; highly weathered grading to moderately weathered. **WEATHERED COLUMBIA RIVER BASALT**	5.9
BASALT: Strong (R4); slightly porphyritic; slight to moderately vesicular; very close to close spaced, rough to smooth, undulating, low to moderate (0°-50°) and high (70°-90°) angle, narrow to moderately wide (0.05-0.5 in.) aperture joints with iron oxide staining, and joint coating and infilling 1- to 2-mm thick; fresh. **COLUMBIA RIVER BASALT**	10.0
BASALT: Medium strong (R3); gray to black; aphanitic; moderately vesicular.	15.5



CONTINUED NEXT SHEET

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core - HQ

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

PGE Tonquin Substation
Tualatin, Oregon

LOG OF BORING B-5

January 2021 106157

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A7
Sheet 1 of 2

REV 3

MASTER_LOG_E 106157.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 1/4/21 Log: VCB Rev: CKS Typ: KJC

Total Depth: 21 ft.	Northing: ~	Drilling Method: Mud Rotary	Hole Diam.: 5 in.
Top Elevation: ~	Easting: ~	Drilling Company: Western States	Rod Type: NWJ
Vert. Datum:	Station: ~	Drill Rig Equipment: CME-850 Tracked Rig	Hammer Type: Automatic
Horiz. Datum:	Offset: ~	Other Comments: Hammer Efficiency = 85%	

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft.	PENETRATION RESISTANCE, N (blows/ft.) ▲ Hammer Wt. & Drop: 140 lbs / 30 inches
Continued: BASALT: Medium strong (R3); gray to black; aphanitic; moderately vesicular; very close to close spaced, rough, undulating and stepped, low to high (0°-90°) angle, closed and healed to moderately wide (0-0.5 in.) aperture joints with iron oxide staining; moderately weathered; paleosol layer from 15.5 to 15.7 feet; top of flow at 15.7 feet. **COLUMBIA RIVER BASALT**	21.0				25 30 35	0 20 40 60 80 100
Completed: November 18, 2020						

0 20 40 60 80 100

Recovery (%) RQD (%)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

LEGEND

* Sample Not Recovered

Standard Penetration Test

Rock Core - HQ

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

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Tualatin, Oregon

LOG OF BORING B-5

January 2021 106157

SHANNON & WILSON, INC.
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FIG. A7
Sheet 2 of 2

REV 3

MASTER_LOG_E 106157.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 1/4/21 Log: VCB Rev: CKS Typ: KJC

Total Depth:	16.5 ft.	Northing:	~	Drilling Method:	Mud Rotary
Top Elevation:	~	Easting:	~	Drilling Company:	Western States
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-850 Tracked Rig
Horiz. Datum:		Offset:	~	Other Comments:	Hammer Efficiency = 85%

Hole Diam.:	5 in.
Rod Type:	NWJ
Hammer Type:	Automatic

SOIL DESCRIPTION

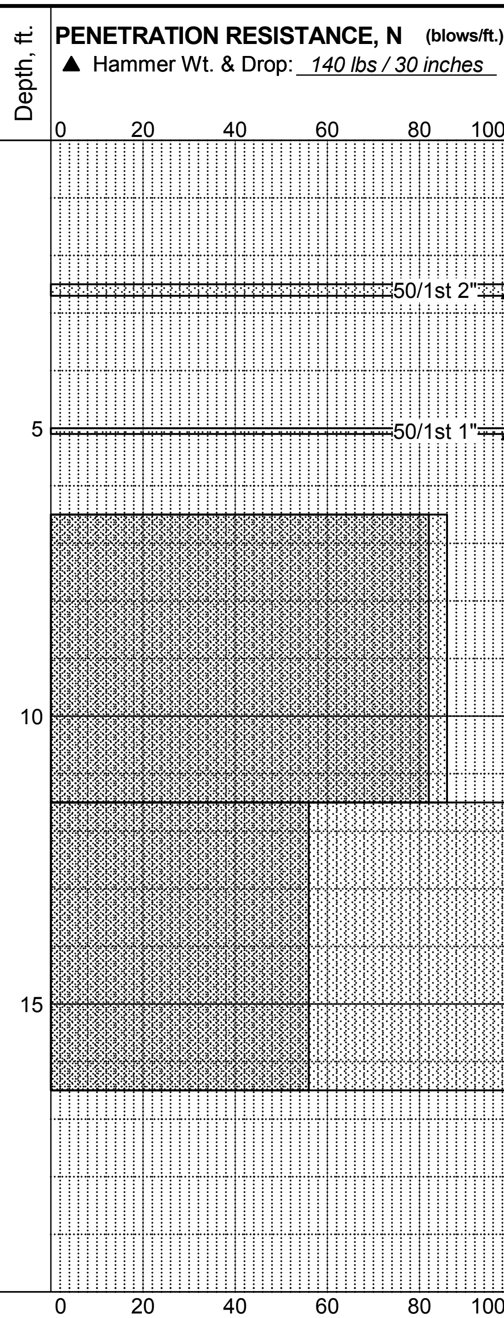
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

SOIL DESCRIPTION	Elev. Depth (ft.)	Samples	Ground Water
Very dense, gray and brown, *Poorly Graded Gravel with Silt, Sand, Cobbles and Boulders (GP-GM)*; moist; some cobbles and boulders; fine to coarse, angular to subangular gravel; fine to coarse sand; low plasticity fines; cobbles and boulders according to driller and inferred from drill action. **RESIDUAL SOIL**		S-1, S-2	No Groundwater Observed
BASALT: Very strong (R5); gray; slightly porphyritic; trace vesicles; moderate to wide spaced, rough to smooth, undulating, low to moderate (5°-45°) and high (80°-90°) angle, closed to moderately wide (0-0.5 in.) aperture joints with with iron oxide staining and infilling 1- to 2-mm thick; fresh. *UCS 6.8-7.6 ft= 26,954 psi* **COLUMBIA RIVER BASALT**	5.0	C-1, C-2	
BASALT: Medium strong to strong (R3-R4); dark gray; moderate to highly vesicular; close spaced, rough, undulating to stepped, low to moderate (10°-50°) and high (60°-90°) angle, closed to moderately wide (0-0.5 in.) aperture joints with iron oxide staining and infilling 1- to 2-mm thick; fresh; bottom of flow zone.	15.5		
	16.5		

Completed: November 18, 2020

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND

- Standard Penetration Test
- Rock Core - HQ

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

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Tualatin, Oregon

LOG OF BORING B-6

January 2021 106157

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FIG. A8

REV 3

MASTER_LOG_E 106157.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 1/4/21 Log: VCB Rev: CKS Typ: KJC

Boring B-1, C-01 (7.5-11 feet) & C-2 (11-16 feet)

Boring B-01, C-3 (16-21 feet)

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CORE BOX PHOTOGRAPHS

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FIG. A9

Boring B-02, C-01 (10-11 feet), C-2 (11-15 feet), C-3 (15-18.5 feet), & C-4 (18.5-20 feet)

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FIG. A10

Boring B-03, C-01 (5-6 feet), C-2 (6-11 feet), & C-3 (11-16 feet)

Boring B-03, C-3 (cont.), & C-4 (16-21 feet)

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CORE BOX PHOTOGRAPHS

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FIG. A11

Boring B-05, C-02 (11-15 feet), C-3 (15-16 feet), & C-4 (16-21 feet)

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FIG. A12

Boring B-06, C-01 (6.5-11.5 feet) & C-2 (11.5-16.5 feet)

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FIG. A13

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Location: ODOT Coos County Maintenance Faclity	**Date:** 11/20/20 **Job Number:** 106157	**Infiltration Test Number:** IN-1
Water level maintained using: Flow Valve	**Penetration Depth of Outer Ring:** 6-inches	**Test Method:** Double-Ring Infiltrometer

Tester's Name: Kevin Cowell, Cody Sorensen
Tester's Company: Shannon & Wilson, Inc.

Weather Conditions: Overcast, 40's

Depth (feet):	**Soil Description:**
0-1 ft	Brown, Sandy Lean Clay (CL) trace gravel.

Time	Time Interval (minutes)	Inner Ring Measurement (cm)	Outer Ring Measurement (cm)	Inner Ring Volume Change (cm^3)	Outer Ring Volume Change (cm^3)	Inner Ring Infiltration Rate (in/hr)	Outer Ring Infiltration Rate (in/hr)	Inner Ring Water Temp. (F)	Ground Temp. (F)	Remarks
10:12		15.6	15.0							
10:27	15	13.9	13.6	1240.4	3064.6	2.7	2.2	60.1	42.9	
10:49		16	14.9							
11:04	15	14.6	13.5	1021.5	3064.6	2.2	2.2	60.5	42.9	
11:06		15.8	14.9							
11:21	15	14	13.5	1313.4	3064.6	2.8	2.2	60.3	43.1	
11:22		15.7	14.9							
11:37	15	14	13.3	1021.5	3502.4	2.2	2.5	56.4	44.0	
11:39		15.6	14.9							
12:09	30	12.6	11.8	2189.0	6785.8	2.4	2.4	58.1	45.0	
12:12		16.3	15.2							
12:42	30	13.5	12.0	2043.0	7004.7	2.2	2.5	54.0	44.3	
12:48		15.5	15.0							
13:48	60	10.3	9.3	3794.2	12477.2	2.0	2.2	50.3	43.1	
13:53		15	15.6							
14:53	60	10.2	9.9	3429.4	12477.2	1.9	2.2	48.5	43.2	
14:57		15.1	14.8							
15:57	60	10.5	9.5	3356.4	11601.6	1.8	2.1	47.4	43.6	
16:01		15.2	14.9							
17:01	60	10.7	10.0	3283.5	10726.0	1.8	1.9	46.1	43.1	
						Constant Rate of Final Measurements = 1.8 in/hr				

PGE Tonquin Substation
Tualatin, Oregon

INFILTRATION TEST RESULTS IN-1

January 2021 106157-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A14

APPENDIX B: LABORATORY TEST RESULTS

Appendix B

Laboratory Test Results

CONTENTS

Figures

Attachments

Northwest Testing, Inc., Technical Report, dated December 14, 2020

APPENDIX B: LABORATORY TEST RESULTS

B.1 GENERAL

Soil samples obtained during the field explorations were described and identified in the field in general accordance with the Practice for Description and Identification of Soils (Visual-Manual Procedure), ASTM D2488. The Specific terminology used is defined in the Soil Description and Log Key, Figure A1, Appendix A. The physical characteristics of the collected samples were noted, and field descriptions and identifications were modified, as necessary, in accordance with the terminology presented in Appendix A, Figure A1.

The rock core was classified based on International Society for Rock Mechanics methods. The specific terminology used in the rock classification is defined on the Rock Classification and Log Key, Appendix A, Figure A2.

During the review, representative soil samples were selected for further testing. The material descriptions and identifications were refined/revised, as necessary, based on the results of the laboratory tests. The soil testing program included natural moisture content, fines content, Atterberg limits testing, and unconfined compressive strength of rock. All laboratory tests were performed in accordance with applicable ASTM International (ASTM) standards.

B.2 SOIL TESTING

B.2.1 Moisture (Natural Water) Content

Natural moisture content determinations were performed in accordance with ASTM D2216, on selected soil samples. The natural moisture content is a measure of the amount of moisture in the soil at the time of exploration. It is defined as the ratio of the weight of water to the dry weight of the soil, expressed as a percentage. The results of moisture content determinations are presented on the Drill Logs in Appendix A and in Table A-1.

B.2.2 Atterberg Limits

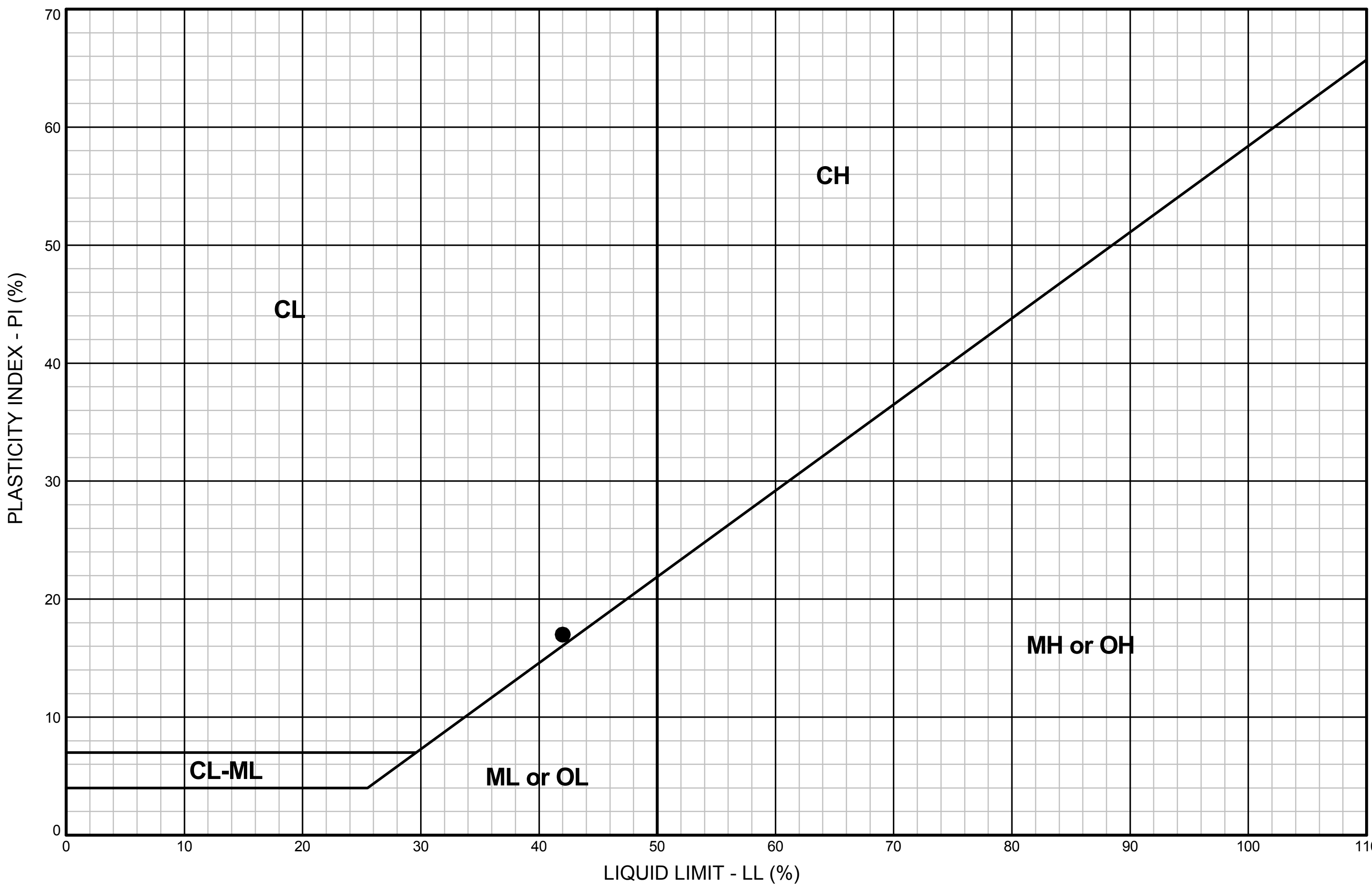
Atterberg limits were determined for select samples in accordance with ASTM D4318. This analysis yields index parameters of the soil that are useful in soil identification, as well as in a number of analyses, including liquefaction analysis. An Atterberg limits test determines a soil's liquid limit (LL) and plastic limit (PL). These are the maximum and minimum moisture contents at which the soil exhibits plastic behavior. A soil's plasticity index (PI) can be determined by subtracting PL from LL. The results are shown in Figure B1 and shown graphically on the Logs of Borings in Appendix A. For the purposes of soil description, Shannon & Wilson uses the term nonplastic to refer to soils with a PI less than 4, low plasticity for soils with a PI range of 4 to 10, medium plasticity for soils with a PI range of 10 to 20, and high plasticity for soils with a PI greater than 20.

B.3 ROCK TESTING

B.3.1 Compressive Strength and Elastic Moduli of Intact Rock Core

The compressive strength and elastic moduli of selected rock core specimens were tested using ASTM D7012 – Method C, Compressive Strength and Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression. This testing was performed by NTI. Prior to testing, each core specimen was trimmed so that the ends were flat and the length to diameter ratio was between 2.0:1 and 2.5:1. ASTM D7012 generally consists of placing a prepared core specimen between two bearing plates and applying an axial load. The load is increased at a constant rate and measured continuously until failure. During application of the increasing axial load, axial and lateral strain of the core sample are continuously measured. The highest load achieved and the length of the rock core at failure are recorded. Measurements made during the test are used to calculate uniaxial compressive strength, Young's modulus, and Poisson's ratio. The bulk density of each specimen was also measured prior to testing.

The unconfined uniaxial compressive strength values of all tested samples are presented on the Logs of borings in Appendix A. Detailed results of the tests are presented in the report prepared by NTI, which is attached to the end of this appendix.



NOTES

1) Atterberg limits tests were performed in general accordance with ASTM D4318 unless otherwise noted in the report.

2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.

3) Plasticity adjectives used in sample descriptions correspond to plasticity index as follows:
- Nonplastic (NP) (< 4%)
- Low Plasticity (4 to 10%)
- Medium Plasticity (10 to 20%)
- High Plasticity (> 20%)

ATT MAIN 106157.GPJ SHAN WIL.GDT 1/4/21

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	LL %	PL %	PI %[3]	NAT. W.C. %	FINES %
● B-5, S-1	2.5	*CL*	*Lean Clay with Sand*	42	25	17	28	

PGE Tonquin Substation
Tualatin, Oregon

ATTERBERG LIMITS RESULTS

January 2021 106157

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B1

FIG. B1

Northwest Testing, Inc.
A Division of Northwest Geotech, Inc.
9120 SW Pioneer Court, Suite B, Wilsonville, Oregon 97070 | ph: 503.682.1880 fax: 503.682.2753 | www.nwgeotech.com

TECHNICAL REPORT

Report To:	Ms. Veronica Biesiada Shannon & Wilson, Inc. 3990 S.W. Collins Way, Suite 203 Lake Oswego, Oregon 97035	**Date:** **Lab No.:**	12/14/2020 20-338
Project:	PGE Tonquin Substation - 106157	**Project No.:**	2966.1.1

Report of: Unconfined compression testing of rock.

Sample Identification

NTI completed unconfined compression testing of rock on samples delivered to our laboratory by a Shannon & Wilson representative on November 23, 2020. Testing was performed in accordance with the standards indicated. Our laboratory test results are summarized on the following table and attached pages.

Laboratory Testing

Compressive Strength of Intact Rock Core Specimens (ASTM D 7012 Method C)				
Sample ID	**Diameter (inches)**	**Height (inches)**	**Rate of Loading (lbs/s)**	**Uniaxial Compressive Strength (psi)**
B-1 C-3 @ 17.5 – 18.1 Ft.	2.40	4.84	198	10,070
B-2 C-2 @ 11.7 – 13.0 Ft.	2.40	4.84	219	17,424
B-3 C-4 @ 18.0 – 18.6 Ft.	2.40	4.89	24	4,402

Attachments: Compressive Strength Test Results

Copies: (1) Addressee

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REVIEWED BY: Michael Ginsbach

TECHNICAL REPORT

Report To: Ms. Veronica Biesiada
Shannon & Wilson, Inc.
3990 S.W. Collins Way, Suite 203
Lake Oswego, Oregon 97035

Date: 12/14/2020

Lab No.: 20-338

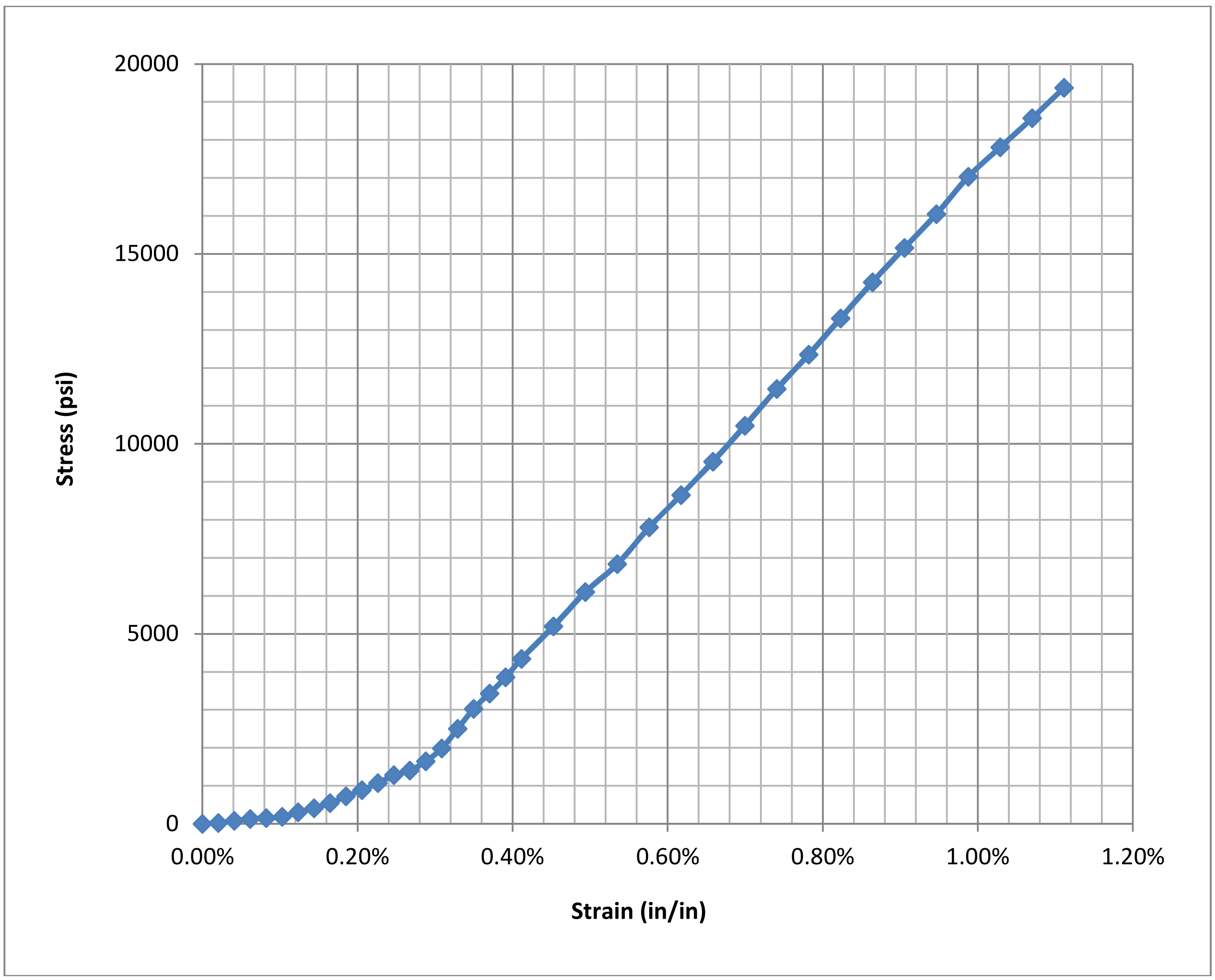
Project: PGE Tonquin Substation - 106157

Project No.: 2966.1.1

Laboratory Testing

Compressive Strength of Intact Rock Core Specimens (ASTM D 7012 Method C)

Sample ID	Diameter (inches)	Height (inches)	Rate of Loading (lbs/s)	Uniaxial Compressive Strength (psi)
B-1 C-1 @ 13.8 – 14.6 Ft.	2.40	4.86	155	19,818



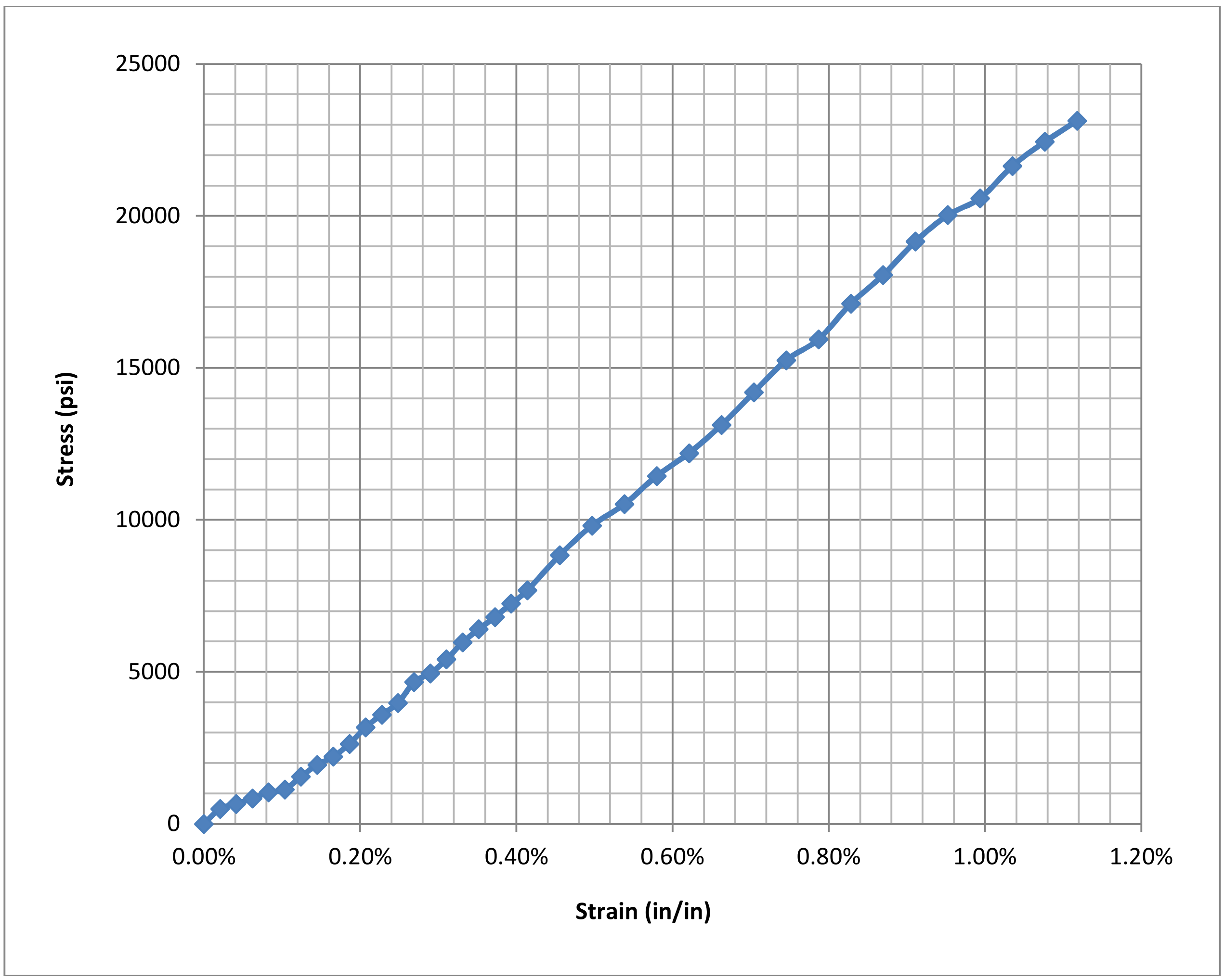
TECHNICAL REPORT

Report To:	Ms. Veronica Biesiada Shannon & Wilson, Inc. 3990 S.W. Collins Way, Suite 203 Lake Oswego, Oregon 97035	**Date:** **Lab No.:**	12/14/2020 20-338
Project:	PGE Tonquin Substation - 106157	**Project No.:**	2966.1.1

Laboratory Testing

Compressive Strength of Intact Rock Core Specimens (ASTM D 7012 Method C)

Sample ID	Diameter (inches)	Height (inches)	Rate of Loading (lbs/s)	Uniaxial Compressive Strength (psi)
B-3 C-2 @ 9.0 – 9.6 Ft.	2.40	4.83	297	23,378



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Northwest Testing, Inc.
A Division of Northwest Geotech, Inc.
9120 SW Pioneer Court, Suite B, Wilsonville, Oregon 97070 | ph: 503.682.1880 fax: 503.682.2753 | www.nwgeotech.com

TECHNICAL REPORT

Report To: Ms. Veronica Biesiada
Shannon & Wilson, Inc.
3990 S.W. Collins Way, Suite 203
Lake Oswego, Oregon 97035

Date: 12/14/2020

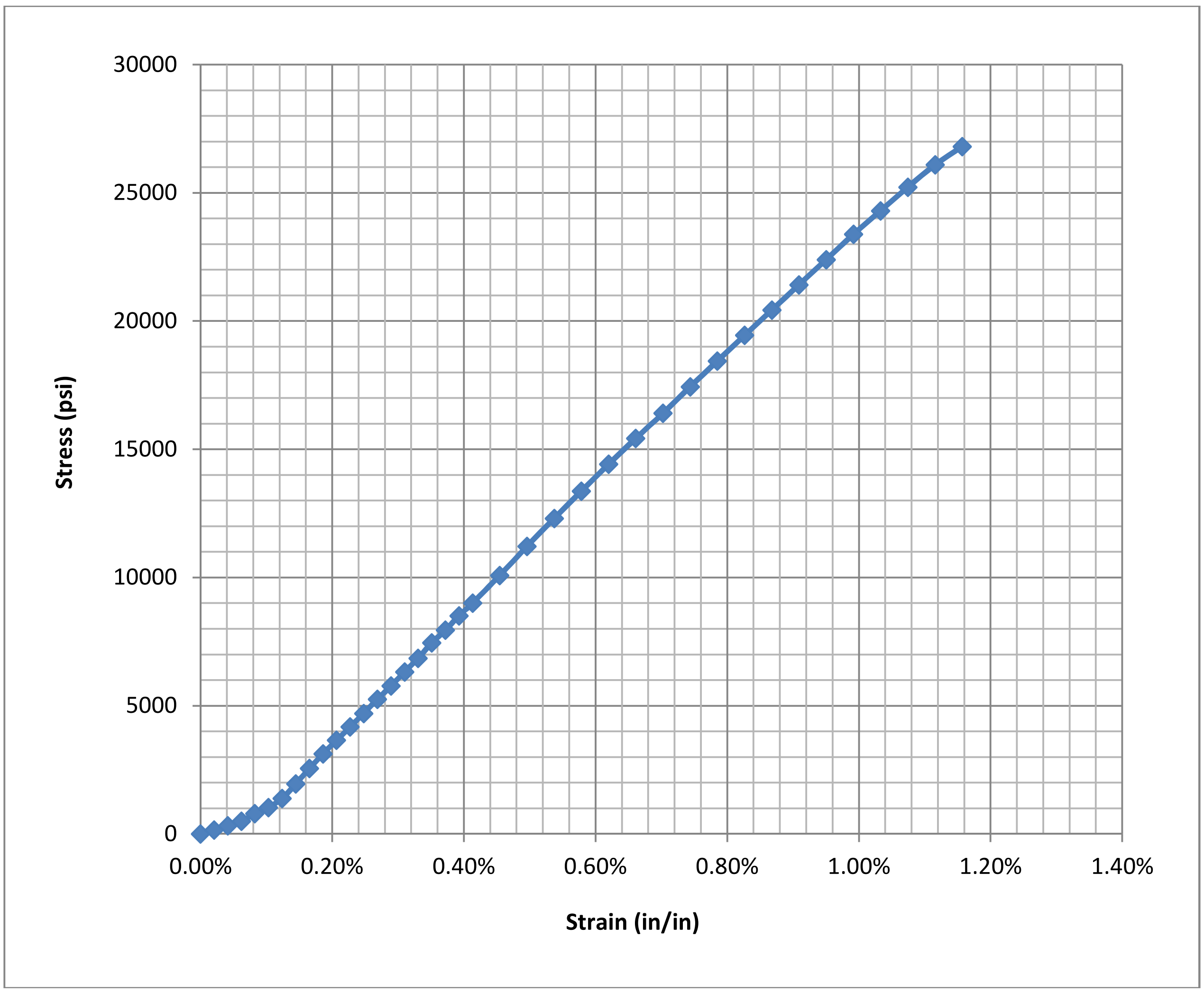
Lab No.: 20-338

Project: PGE Tonquin Substation - 106157

Project No.: 2966.1.1

Laboratory Testing

Compressive Strength of Intact Rock Core Specimens (ASTM D 7012 Method C)				
Sample ID	**Diameter (inches)**	**Height (inches)**	**Rate of Loading (lbs/s)**	**Uniaxial Compressive Strength (psi)**
B-6 C-1 @ 6.8 – 7.6 Ft.	2.40	4.84	176	26,954



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 REVIEWED BY: Michael Ginsbach

Appendix C

Example Rock Cuts on SW 124th Ave

APPENDIX C: EXAMPLE ROCK CUTS ON SW 124TH AVE

PGE Tonquin Substation
Tualatin, Oregon

PHOTOGRAPHS OF ROCKCUT AT SW 124TH AVENUE

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FIG. C1

FIG. C1

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PHOTOGRAPHS OF ROCKCUT AT SW 124TH AVENUE

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FIG. C2

FIG. C2

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Tualatin, Oregon

PHOTOGRAPHS OF ROCKCUT AT SW 124TH AVENUE

January 2020

106157

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FIG. C3

FIG. C3

Important Information

About Your Geotechnical/Environmental Report

IMPORTANT INFORMATION

IMPORTANT INFORMATION

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining

IMPORTANT INFORMATION

your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims

IMPORTANT INFORMATION

being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland