SUPPLEMENTARY INFORMATION

EXHIBIT A

Geotechnical Exploration & Evaluation, Washington Ave Bore Crossing
by McMillen Jacobs dated March 26, 2021
(72 pages)

THE CITY OF ROSEBURG
TIMBER CAPITAL OF THE NATION

Washington Ave Bore Crossing Project

Geotechnical Exploration & Evaluation Report

Final Submittal
Revision No. 1

McMILLEN JACOBS ASSOCIATES

March 2021

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Table of Contents

List of Tables

List of Figures

Appendices

Acronyms and Abbreviations

ASCE	American Society of Civil Engineers
ASTM	American Society of Testing and Materials
bgs	below ground surface
bpf	blows per foot
City	City of Roseburg
CLSM	controlled low strength material
CSZ	Cascadia Subduction Zone
DIP	ductile iron pipe
E’	modulus of soil reaction
FEMA	Federal Emergency Management Agency
HDD	Horizontal Directional Drilling
HDPE	high-density polyethylene
ID	inside diameter
McMillen Jacobs	McMillen Jacobs Associates
M_s	constrained soil modulus
M_W	mean moment magnitude
NAVD88	North American Vertical Datum, 1988
OD	outside diameter
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction
OWRD	Oregon Water Resources Department
pcf	pounds per cubic foot
PGA	peak ground acceleration
PGV	peak ground velocity
PGD	permanent ground displacement
Project	Washington Avenue Bore Crossing Project
psi	pounds per square inch
SA	spectral acceleration
S_{M1}	SA 1.0-sec period spectral acceleration
S_{MS}	SA 0.2-sec period spectral acceleration

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Revision Log

Revision No.	Date	Revision Description
0	January 15, 2021	DRAFT Submittal – 30% Design
1	March 26, 2021	Final Submittal

1.0 Introduction

1.1 General

The City of Roseburg (City) retained McMillen Jacobs Associates (McMillen Jacobs) to provide engineering consultation services for the Washington Avenue Bore Crossing project (Project), including alternatives analysis; design; preparation of construction documents; services during bidding; and construction management services. The Project is in Roseburg, Oregon, between the Washington Avenue and Oak Avenue bridges, as shown on the Vicinity Map, Figure 1.

This report presents the results of our geotechnical investigation and provides recommendations for design and construction of the Project. A more specific technical memorandum summarizing our analyses and preliminary design recommendations for the HDD pipeline are provided under separate cover in the Preliminary Design Technical Memorandum (McMillen Jacobs, 2021).

1.2 Project Description

The Project includes replacing the existing 12-inch diameter steel water main, installed in 1976 on the Washington Avenue bridge. The new water main will be a 14-inch outside diameter (OD)/11.3-inch inside diameter (ID) HDPE pipe and will be installed below the South Umpqua River using Horizontal Directional Drilling (HDD) methods. The HDD alignment is approximately 740 feet long in plan view and runs between the Washington Avenue and Oak Avenue bridges. To facilitate the connection of the new HDD watermain to the City's existing water main, the Project also includes approximately 380 feet of new 12-inch ID, ductile iron (DI) connection piping. The Project alignment is shown on the Site Plan, Figure 2.

1.3 Purpose and Scope of Work

The purpose of our work is to evaluate the subsurface conditions along the HDD alignment and provide geotechnical design and construction recommendations to develop construction documents. Specifically, our scope of work included the following:

- Characterization of subsurface conditions along the proposed pipeline alignment based on geotechnical explorations and laboratory testing performed by McMillen Jacobs;
- Discussion of regional and local geology;
- Geologic and seismic hazard evaluation and design recommendations for the trenchless crossings and pipeline;
- Geotechnical engineering assessments, design, and construction recommendations for the HDD pipeline and connection piping;
- Preparation of this Geotechnical Exploration and Evaluation Report; and
- Preparation of the construction plans and specifications.

1.4 Authorization

McMillen Jacobs was retained by the City of Roseburg to provide geotechnical and pipeline design services for, and in connection with the Washington Ave Bore Crossing Project per the agreement dated September 8, 2020.

2.0 Site Investigation

2.1 General

The field investigation and laboratory testing are summarized in the following sections. Appendix A provides the boring logs along with a key to terms and symbols. Laboratory testing results are presented in Appendix B and on the boring logs. Rock core photographs are provided in Appendix C.

2.2 Field Explorations

Three geotechnical borings (B-1 through B-3) were advanced between November 3 and November 5, 2020, using a truck-mounted CME-75 drill rig provided and operated by Western States Soil Conservation, Inc of Hubbard, Oregon. The boring depths ranged from 30.1 feet to 65 feet below ground surface (bgs). Borings B-1 and B-2 penetrated approximately 55 feet and 45.5 feet, respectively, into basalt bedrock. Boring B-3 was advanced primarily for the purpose of identifying the depth to basalt bedrock; penetrating approximately 2.5 feet into these materials. The borings were advanced using hollow-stem auger and mud-rotary drilling methods within soil materials and HQ wireline, triple-tube rock coring techniques in rock materials in accordance with ASTM D2113. The drilling was performed under the direction of a geotechnical engineer from McMillen Jacobs, who examined and described the soil and rock materials encountered. The locations of the exploratory borings are shown on Figure 2.

Upon completion, the borings were backfilled with 3/8-inch chipped bentonite, in accordance with Oregon Water Resources Department (OWRD) abandonment requirements (OAR 690-240 Construction, Maintenance, Alteration Conversion and Abandonment of Monitoring Wells, Geotechnical Holes and Other Holes in Oregon). At boring B-1, the asphalt surface of the parking lot was restored with compacted cold asphalt.

2.2.1 Soil Sampling

Standard Penetration Tests (SPT) were completed in accordance with ASTM Test Method D1586, “Standard Method for Penetration Test and Split-Barrel Sampling of Soils”. The sampler was driven with a 140-pound auto-trip hammer falling 30 inches. Recorded blows for each 6 inches of sample penetration are shown on the boring logs. The N-value, or number of blows required to drive the sampler the final 12 inches was used in our analyses. Disturbed samples were obtained from the split barrel for subsequent classification and index testing. (see Section 2.3).

2.2.2 Rock Coring

Upon encountering intact rock in borings B-1 and B-2, the borings were further advanced using an HQ wireline triple-barrel (HQ3) rock coring assembly, consisting of a 96 millimeter (3-3/8 inch) outer diameter and a 61.1-millimeter (2-3/8 inch) inner diameter core barrel. The wireline retrieval process consists of the following steps: 1) a retriever is lowered through the drill rod, via a wireline, to release the locking mechanism at the head of the inner core barrel; 2) the inner core barrel is removed; 3) the rock core is removed from the barrel; and 4) the inner core barrel is returned and locked into the triple-tube assembly. Water was used to cool the drill bit, to flush cuttings from the boring, and to provide lateral support to the boring wall. The maximum core run length was 5 feet.

Core recovery and Rock Quality Designation (RQD) were measured in the field on each core run while the core was still in the sleeve. Core recovery is the ratio of the sample length collected to the core run length, expressed as a percentage. RQD is the sum of the length of sound core pieces in excess of 4 inches and dividing by the total length of the core run (also expressed as a percentage). RQD provides a general indication of the degree of jointing or fracturing in a rock mass. Photographs of the rock core are presented in Appendix C.

2.3 Laboratory Testing

Field samples obtained during explorations were transported to McMillen Jacobs' office for further examination and subsequently transported to the testing laboratories. The testing program included the following:

- Moisture content (ASTM D2216);
- Atterberg limits (ASTM Test Method D4318)
- Percent Fines (passing No. 200 sieve) (ASTM D1140)
- Uniaxial Compressive Strength of Intact Rock Cores Specimens (ASTM D7012, Method C)

Laboratory testing on selected soil was completed by Breccia Geotechnical Testing, LLC. of Tigard, Oregon. Laboratory testing on selected rock core samples was completed by Northwest Testing, Inc. of Wilsonville, Oregon. All tests were completed in accordance with the above-referenced ASTM standards. Test results are provided in the boring logs in Appendix A and individually presented in Appendix B.

3.0 Site Description

3.1 Pipeline Alignment

At the east end of the Project alignment, the tie-in connection to the City's existing water main is located at the south end of the City-owned Riverside Park parking lot, on the north side of SE Oak Avenue. Approximately 150 feet of 12-inch ID, DI connection piping will be installed between the tie-in connection and the HDD entry pit; generally running north through the parking lot. From the entry pit, the approximately 740-foot long HDD alignment runs west under Riverside Park and the multi-use concrete path along the east riverbank before crossing the South Umpqua River between the Washington Avenue and Oak Avenue bridges. The HDD alignment emerges on the west side of the river in the receiving pit, located on private property at the base of the access road between the Washington Avenue and Oak Avenue bridges adjacent to the river. Approximately 230 feet of 12-inch diameter DI connection piping will be installed between the HDD entry pit and the tie-in connection; generally running northwest along the gravel access road between the Washington Avenue and Oak Avenue bridges before turning north to cross SE Washington Avenue. The tie-in connection is located approximately 20 feet east of the existing bus stop on the south side of the Adapt facility.

3.2 Surface Conditions

On the east side of the river, the HDD alignment is entirely within public/City-owned right-of-way. Surface conditions include an asphalt-paved parking lot at the HDD launch pit, with grass, landscaped areas, and occasional trees within Riverside Park. At the west end of Riverside Park, a multi-use concrete path runs along the top of the riverbank slope. Surface topography in Riverside Park slopes gently to the west from the HDD launch pit towards the top of the east riverbank slope; dropping 8 vertical feet over 175 horizontal feet. The east riverbank slope has an overall vertical relief of about 20 feet and descends at approximate gradients of 2.5H:1V (horizontal:vertical) to 2H:1V, with some locally steeper areas descending at up to 1.5H:1V. The east riverbank slope is vegetated with ivy and blackberry vine and occasional trees. The South Umpqua River channel bottom is relatively flat; undulating between elevation 417 and 418.5 feet along the HDD alignment.

The west riverbank slope ascends approximately 7 to 8 feet from the shoreline to the crest of the riverbank slope at gradients up to 2.5H:1V. The west riverbank is sparsely vegetated with occasional small trees and grass. The crest of the west riverbank slope (approximately elevation 434 feet) then gently descends west approximately 2 to 3 vertical feet to a broad, relatively flat, and non-vegetated area between the Washington Avenue and Oak Avenue bridges, where the HDD receiving pit is located within private property, at approximately elevation 432 feet. The slope inclinations beyond (e.g., to the west) increase locally due to the artificial fill placed to develop the properties along the riverside (e.g., hotel properties and the bridge approach embankments).

3.3 Geology

3.3.1 Regional Geology

The Coast physiographic province of Oregon and Washington, stretching from the Strait of Juan de Fuca in the north to the Coquille River in the south, encapsulates a diverse range of topography, but this varied landscape is unified into a single geologic province by its underlying submarine basaltic basement – the Siletz terrane.

In the late Paleocene to early Eocene, 63 to 46 million years ago, the Siletz terrane began forming when submarine volcanic vents off the North American continent became active (Orr and Orr, 2002). Over time, lava extruded by these submarine vents accumulated to form a miles-thick pile of basalt that was too massive to move into the subduction zone. Instead, it stuck on the edge of the continent where it remains accreted to the North America plate forming the underpinnings of the Coast Range.

The southeast corner of the coast province abuts the Mesozoic rocks of the Klamath mountains at the Wildlife Safari thrust fault (Wells et al, 2000). At this margin, the basaltic basement of the Siletz terrane has buckled under compressive stress, to form the Roseburg anticlinorium, a larger uplift arranged into a complex series of smaller folds.

3.3.2 Local Geology

Based on a review of regional geologic mapping, the project site is located in the Roseburg anticlinorium of early Eocene to late Paleocene age basalt flows of the Siletz River Volcanics – the geologic unit composing the southern extent of the Siletz terrane (Wells et al, 2000). Here, steeply dipping, tightly folded submarine basalt flows reach a thickness of at least 3.7 miles. The bedrock at the project location is composed of submarine pillow basalt with closely packed pillow lava, columnar jointed sheet flows, and pillow breccia. The basalt is fractured and contains vesicle and fracture fillings of clay, calcite, quartz, and zeolite.

Holocene and Pleistocene alluvium unconformably overlie the basaltic bedrock along the margins of the Umpqua River. A step down in elevation toward the river indicates two ages of terraced alluvial sediments on the east and west bank of the river. The higher, older Pleistocene-age fluvial terrace deposits on the east side of the river are described as semi-consolidated and consist of poorly graded, stratified gravel, sand, and silt. The lower, younger Holocene-age fluvial deposits on the west side of the river are described as being unconsolidated to poorly consolidated, composed of poorly graded, sub-angular to well-rounded boulders, gravel, sand, and silt (Wells et al, 2000).

3.4 Subsurface Conditions

The subsurface materials along the project alignment have been grouped into three geotechnical units based on their engineering properties, geologic origins, and distribution. These units are: Fill, Fluvial Deposits (e.g. Alluvium), and Siletz River Volcanics (SRV) and are discussed in the following sections. Further details regarding the subsurface conditions are provided in the boring logs in Appendix A and laboratory test results in Appendix B. Figure 3 shows the anticipated geologic profile along the HDD alignment. Our findings are generally consistent with the geologic cross section shown on the plan and elevation drawing for the Washington Ave. Bridge (Oregon State Highway Department, 1961), included as Appendix D.

3.4.1 Fill

We encountered fill materials at the ground surface in borings B-1 and B-2. Boring B-1 was advanced through an asphalt pavement parking lot. Thus, the artificial fill materials are associated with the asphalt pavement section, consisting of 3 inches of asphalt concrete underlain by approximately 2.5 feet of silty gravel base aggregate (well-rounded gravel up to approximately 2 inches in diameter). In boring B-2, thin/surficial fill was encountered at the ground surface and extended to a depth of approximately 1 foot below ground surface (bgs) and consisted of silt with gravel and cobbles associated with the surfacing of the access road between the Washington Avenue and Oak Avenue bridges. We did not collect any soil samples within the artificial fill materials. However, we did observe difficult drilling within the artificial fill (e.g., augers scraping on cobbles and slow drilling rates) in boring B-2.

We anticipate fill materials will become more prominent to the west of the anticipated HDD receiving pit; mainly due to the proximity of the Washington Avenue and Oak Avenue bridge approach embankments. During our site reconnaissance, we observed various fill materials (e.g., boulders, gravel, and debris) at the surface in the vicinity of the bridge approach embankments. Therefore, we anticipate subsurface conditions in these areas may contain variable amounts of these observed materials.

3.4.2 Fluvial Deposits (Alluvium)

Fluvial deposits (referred to hereafter as alluvium) were encountered beneath the artificial fill in borings B-1 and B-2 and approximately at the ground surface in boring B-3 and extend to depths ranging from 8.5 feet to 27.5 feet bgs. In general, the alluvium includes an upper, predominately fine-grained zone (i.e., silt, clay, and sand) underlain by a predominately gravel zone before transitioning to the Siletz River Volcanics unit. The following sections describe the alluvium at each boring location.

3.4.2.1 Boring B-1 (2.5 to 10 feet bgs)

Alluvium was encountered below the pavement section and generally consists of stiff, lean clay with trace fine sand. The alluvium grades to gravelly clay from approximately 7.5 feet to 8.5 feet bgs, based on our observation of the drill cuttings. From 8.5 to 10 feet bgs, we observed the clay soil cuttings sticking to the drill bit; indicative of high plasticity clay (i.e., fat clay). One SPT was performed in the alluvium at 5 feet bgs, resulting in an SPT N-value of 12 blows per foot (bpf), interpreted as stiff. The moisture content of this sample was 38.3 percent. Basalt was encountered at 10 feet bgs, based on drilling action and SPT refusal (e.g., 50 or more blow counts for 6 inches or less penetration).

3.4.2.2 Boring B-2 (1 to 9.5 feet bgs)

Alluvium was encountered below the artificial fill, consisting of sandy silt. Two SPTs were performed in the sandy silt deposits; with SPT N-values of 12 bpf at 2.5 feet bgs and 1 bpf at 5 feet bgs, indicating stiff and very soft conditions. Respective moisture contents of the samples at 2.5 feet bgs and 5 feet bgs were 25.5 and 41.6 percent and respective fines contents of 57.6 and 62.5 percent. The alluvium grades to silty gravel at approximately 6.5 feet bgs, based on an abrupt change to more difficult and choppy drilling action. It is important to note that drilling action at 6.5 feet bgs was indicative of cobbles and potentially boulders. One SPT was performed in the silty gravel at 7.5 feet bgs, resulting in an SPT N-value of 73 blows for 9 inches, indicating very dense conditions. We observed difficult and slow drilling action (e.g., auger bit scraping on rock) at approximately 9.5 feet bgs, at which point an SPT was performed; resulting in 50 blows for 2 inches of penetration. We interpret 9.5 feet bgs as the transition depth to basalt.

3.4.2.3 Boring B-3 (0 to 27.5 feet bgs)

In boring B-3, alluvium was encountered at approximately the ground surface, below grass and minor topsoil, and extends to approximately 27.5 feet bgs. The following points summarize the alluvium encountered in boring B-3:

- **0 to 10 feet bgs:** Medium stiff, moist, sandy silt and lean clay with sand.
    - *Sample at 5 feet bgs:* SPT N-value of 7 bpf; moisture content of 30.3 percent.
- **10 to 20 feet bgs:** Very loose, moist, medium plasticity, clayey sand.
    - *Sample at 10 feet bgs:* SPT N-values of 4 bpf; moisture content of 30.7 percent; fines content of 48.7 percent; Atterberg Limits Test (LL = 38, PL = 20, PI = 18).
    - *Sample at 15 feet bgs:* SPT N-values of 1 bpf; moisture content of 35.6 percent.
- **20 to 23 feet bgs:** Very soft, wet, medium plasticity, lean clay.
    - *Sample at 20 feet bgs:* SPT N-value of 0 bpf; moisture content of 44.8 percent; Atterberg Limits Test (LL = 42, PL = 25, PI = 17).
- **23 to 27.5 feet bgs:** Very dense, wet, poorly graded gravel with clay and sand, inferred from an abrupt change to more difficult and choppy drilling action at 23 feet bgs. Based on drilling action, there are likely cobbles present in this interval.
    - *Sample at 25 feet bgs:* SPT N-value of 70 bpf.

3.4.3 Siletz River Volcanics Basalt

We encountered Siletz River Volcanics (basalt) in all borings below the alluvium. The depth-to-rock in each borehole is as follows: 10 feet bgs in boring B-1; 9.5 feet bgs in boring B-2; and 27.5 feet bgs in boring B-3. We interpreted the transition depth between the alluvium and the basalt unit based on drilling observations and SPT refusal.

Basalt strength varies from weak (R2) to strong (R4), but was typically medium strong (R3). The results of eight unconfined compressive strength (UCS) tests ranged from 3,174 to 8,024 psi, with an average compressive strength of 4,838 psi. A unit weight was also measured for each of the UCS test specimens; ranging from 161.7 pounds per cubic foot (pcf) to 179.2 pcf. Rock strength descriptors are presented in Table 3-1 and a summary of compressive strength test results and unit weights are presented in Table 3-2.

The basalt is typically slightly weathered to fresh, with close to moderately spaced joints (joint spacing ranging from 2 inches to 2 feet), gray, and with an overall fine-grained groundmass/texture (i.e., aphyric) with very few relatively large crystals (i.e., phenocrysts) present. The basalt is also very frequently veined with calcite. Joints are typically rough and planar at low to high angles. Open joints faces are typically coated with calcite and occasionally iron oxide staining. Closed joints are typically healed with calcite and to a lesser degree with fines (e.g., clay).

Core recovery was typically excellent (e.g., 100 percent); the only exceptions being 88 percent (B-1, Core Run No. 2 from 16 to 21 feet bgs) and 58 percent (B-1, Core Run No. 4 from 26 to 31 feet bgs). The Rock Quality Designation (RQD) within the basalt unit ranged from 0 to 100, with typical values above 80 percent. The average RQD values in borings B-1 and B-2 were 81 and 91 percent, respectively.

An HQ3 coring assembly with a diamond-carbide button drill bit was used to advance the boring within the intact basalt. Based on our observations, the time required to advance a single 5-foot core run ranged from 12 to 22 minutes; averaging about 15 minutes. Drilling fluid (e.g., water) circulation was typically positive and maintained during rock coring operations, with no major loss of fluid circulation observed.

Other notable observations within the basalt unit during our field exploration include the following:

- An approximately 2-foot zone between 16 and 18 feet bgs in boring B-1, in which the basalt was intensely fractured (extremely closely spaced joints) and typically consisted of 3-inch minus gravel-sized clasts. Recovery in this region was 88 percent and the RQD was 27 percent.
- Relatively poor recovery (58 percent) and 0 percent RQD from 26 to 31 feet bgs in boring B-1. Basalt within this interval exhibited very close to extremely close spaced joints (e.g., less than 1 inch to 2 inches) with frequent clay infilling.

Table 3-1. Definition of Rock Strength Descriptions

Grade[1]	Approximate Uniaxial Compressive Strength (psi)	Term
R0	35 – 150	Extremely Weak
R1	150 – 700	Very Weak
R2	700 – 3,600	Weak
R3	3,600 – 7,200	Medium Strong
R4	7,200 – 14,500	Strong
R5	14,500 – 36,000	Very Strong
R6	>36,000	Extremely Strong

Note: Rock strength grades from Brown (1981).

Table 3-2. Unconfined Compressive Strength (ASTM D7012, Method C) Summary

Exploration ID	Depth Interval (feet bgs)	Unit Weight (pcf)	Uniaxial Compressive Strength (psi)	Strength Term, Grade
B-1	21.7 to 23.0	175.2	6,972	Medium Strong, R3
	36.5 to 37.5	173.6	3,683	Medium Strong, R3
	57.3 to 58.2	161.7	4,257	Medium Strong, R3
	62.1 to 62.9	166.8	3,479	Weak, R2
B-2	16.0 to 16.7	179.2	8,024	Strong, R4
	30.0 to 31.2	170.9	3,174	Weak, R2
	49.1 to 50.0	162.7	4,460	Medium Strong, R3
	53.7 to 55.0	168.8	4,653	Medium Strong, R3

Note: Depth interval rounded to the nearest tenth.

3.5 Groundwater

The use of mud-rotary drilling methods and HQ rock coring methods precluded the observation of groundwater during the drilling operations because these methods introduce drilling mud and water into the borehole. However, we did observe changes in sample moisture content which can be an indicator of possible saturation. Based on these field observations and laboratory moisture contents, we conclude the following at each exploration location:

- Boring B-1: No apparent groundwater observations were made, and shallow bedrock was encountered (e.g., approximately 10 feet bgs).
- Boring B-2: Observation of saturated soils in sample at approximately 8 feet bgs. This depth corresponds to the approximate water level in the adjacent South Umpqua River.
- Boring B-3: Observation of saturated soils in sample at approximately 20 feet bgs. This likely corresponds to seasonally perched groundwater conditions over the relatively impervious basalt bedrock surface.

Groundwater levels may vary with precipitation, the time of year, and other factors. In general, static groundwater levels along the HDD alignment are controlled by the South Umpqua River levels. Groundwater highs occur near the end of the wet season in late spring or early summer and groundwater lows occur near the end of the dry season in the early fall. In addition, the basalt bedrock surface is relatively impervious and conducive to perched groundwater conditions during the wetter winter months.

It should be noted that ordinary high water was marked at an elevation of 431.4 feet along the west riverbank by Land and Water Environmental Services, Inc.

4.0 Seismic and Geologic Hazards

4.1 Regional Seismicity

The Pacific Northwest is a seismically active region that has three principle types of seismic sources. These sources include (1) the Cascadia Subduction Zone (CSZ) megathrust, which represents the boundary (interface) between the down going Juan de Fuca plate and the overriding North American plate; (2) faults located within the Juan de Fuca plate (referred to as CSZ intraplate or intraslab sources); and (3) crustal faults principally in the North American plate (Wong and Silva, 1998).

4.2 Site Classification

The project site was assigned a seismic site class following code-based procedures in ASCE's Minimum Design Loads and Associates Criteria for Buildings and Other Structures, ASCE 7-16 (ASCE, 2017). Site class is used to categorize common subsurface conditions into broad classes to which ground motion attenuation and amplification effects are assigned and accounts for conditions encountered in the upper 100 feet of the subsurface profile. The ASCE 7-16 procedures define Site Class based on an average standard penetration resistance value ($\overline{N}$). Using SPT blowcount data, we calculated $\overline{N}$ values associated with Site Class B, D, and E based on the varying depth to bedrock in the borings (e.g., ranging from 9.5 to 27.5 feet bgs). For the open-cut segments of the pipeline, we recommend a Site Class D for design. However, because the HDD pipe will be installed almost entirely in basalt bedrock, we conclude Site Class B (i.e., rock) is appropriate for design.

4.3 Seismic Design Parameters

Recommended seismic parameters for the design of the pipeline are provided in Table 4-1 and are based on Maximum Considered Earthquake (MCE_R) with an expected peak bedrock acceleration having a 2-percent probability of exceedance in 50 years (2,475-year return period). We developed the MCE spectral response accelerations using the online ASCE 7 Hazard Tool, which references ground motion procedures in accordance with ASCE 7-16 (ASCE, 2017).

It is important to note that Section 11.4.8 of ASCE 7-16 requires a site-specific ground motion hazard analysis be performed on structures on Site Class D sites with a 1-second spectral response acceleration parameter (S_1) greater than 0.2g. However, exception No. 2 in Section 11.4.8 states that a site-specific ground motion hazard analysis is not required if the structure's fundamental period of vibration (T) is less than 1.5Ts. When this condition is met, the seismic response coefficient C_s shall be calculated using equation 12.8-2 in ASCE 7-16. The following provides a summary of these parameters:

- The pipeline is buried and therefore, T will be less than 0.5 second;
- T_s equals the design 0.2-second spectral response parameter S_{DS} divided by the design 1-second spectral response parameter S_{D1}. Using this equation and the S_{DS} and S_{D1} values in Table 4-1, T_s equals 1.122 and $1.5T_s$ equals 1.683; and
- T is less than $1.5T_s$ and therefore, a site-specific ground motion hazard analysis is not required, and the seismic response coefficient C_S shall be calculated using equation 12.8-2 in ASCE 7-16.

Table 4-1. MCE Spectral Acceleration Parameters for Pipeline Design

Parameter	Site Class B for HDD Pipeline Design		Site Class D for Open-Cut Pipeline Design	
	0.2 Second	1 Second	0.2 Second	1 Second
Mapped MCE_R (Rock site)	$S_S = 0.813g$	$S_1 = 0.463g$	$S_S = 0.813g$	$S_1 = 0.463g$
Site Coefficients	$F_a = 1.0$	$F_v = 1.0$	$F_a = 1.175$	$F_v = 1.837$
Site-Adjusted MCE_R	$S_{MS} = 0.813g$	$S_{M1} = 0.463g$	$S_{MS} = 0.954g$	$S_{M1} = 0.851$
Design MCE_R	$S_{DS} = 0.542g$	$S_{D1} = 0.308g$	$S_{DS} = 0.636g$	$S_{D1} = 0.567$
Mapped MCE_G PGA (Rock Site)	0.395g		0.395g	
Site Coefficient F_{PGA}	1.0		1.205	
Site-adjusted MCE_G PGA	0.395g		0.476g	
Site-Adjusted Peak Ground Velocity (PGV) (cm/sec)	44		80	

4.4 Liquefaction

Liquefaction is a phenomenon whereby saturated cohesionless soils, generally sands and silts, undergo significant loss of strength and stiffness when they are subjected to vibration or large cyclic ground motions produced by earthquakes. During earthquake shaking (e.g., undrained conditions), loads are transferred from the soil skeleton to the pore-water with consequent reduction in the soil shear strength. The shear strength of a cohesionless soil is directly proportional to the effective stress; equal to the difference between the overburden pressure and the pore water pressure. The susceptibility of sands, gravels, and sand-gravel mixtures to liquefaction is typically assessed based on in-situ penetration resistance tests (e.g., SPT, CPT, etc.).

For fine-grained soils, however, susceptibility to liquefaction can be characterized into three categories (Boulanger and Idriss, 2014):

1. **Sand-like behavior:** Fine-grained soils with a plasticity index less than 7 are considered to exhibit classical soil liquefaction behaviors like clean sand.
2. **Clay-like behavior:** Fine-grained soils with a plasticity index greater than 12 are not considered to be liquefiable; however, cyclic softening and strain accumulation from seismic shaking should be considered.
3. **Transitional behavior:** Fine-grained soils with a plasticity index between 7 and 12 should be considered as “transitional” soils, and their seismic behaviors are expected to include cyclic softening, strength loss, and post liquefaction settlement.

We conclude the impacts of liquefaction-induced settlement on the HDD pipe and associated connection piping are negligible for the following reasons:

- The HDD pipe will be installed almost entirely in basalt bedrock, which is not susceptible to liquefaction.
- Subsurface conditions encountered in boring B-3 (e.g., thickest soil profile encountered) within the potentially saturated zone (based on OHW level) consist of clayey sand and lean clay; both of which exhibit clay-like behavior based on the results of Atterberg Limits tests.

Our conclusion is further supported by our review of available online liquefaction susceptibility mapping on the Oregon Department of Geology and Mineral Industries (DOGAMI) Statewide Geohazards Viewer (HazVu), which does not indicate a liquefaction hazard at the Project site (Oregon DOGAMI, 2020a).

4.5 Lateral Spreading

Lateral spreading is a liquefaction related phenomenon that results in ground displacement during an earthquake and occurs in sloping ground or flat ground with a free face. Lateral spreading occurs where continuous layer of liquefiable soil is present across the site. Because liquefaction is not considered a hazard, consequently, lateral spreading is not considered a hazard for the Project.

4.6 Fault Rupture

We reviewed the USGS online *U.S. Quaternary Faults Interactive Map* (USGS, 2020) which includes all active and potentially active known faults. No known faults pass through the pipeline alignment or within the City of Roseburg. The closest faults are located approximately 13 miles north of the Project site, referred to as the Unnamed Faults Near Sutherlin (USGS Fault ID No. 862). Therefore, the risk of fault rupture along the HDD alignment is negligible.

4.7 Slope Stability

On the east side of the river, the steepest slopes are the east riverbank slopes; generally, 20 to 25 feet high, with gradients sloping downward to the west at 2.5H:1V, with some locally steeper areas up to 1.5H:1V. We did not observe any indication of significant past slope movement such as bowed trees, hummocky ground, tension cracks, or slide scarps along the east riverbank slope. In addition, the HDD pipeline will be embedded in basalt in the vicinity of the east riverbank slope. Therefore, we conclude the risk of damage to the HDD pipeline due to failure of the east riverbank slope is negligible.

On the west side of the river, the steepest slopes are the Washington Avenue and Oak Avenue bridge approach embankment slopes, which are up to approximately 10 feet high and descend at gradients up to approximately 1.5H:1V. We observed these embankment slopes to be in satisfactory condition, with only minor surficial raveling of cobble-sized material on the north-facing Oak Avenue embankment slope. If the connection pipeline from the HDD to the existing watermain runs between the Oak Avenue and Washington Avenue bridges, we conclude the risk of damage to the pipeline is negligible due to slope instability of the embankments. However, the pipeline trench should be properly shored with a positive shoring system (e.g., maintaining pressure against the trench sidewalls) to minimize embankment instability and/or movement during construction.

Our conclusions are supported by available landslide susceptibility maps on the Oregon Department of Geology and Mineral Industries (DOGAMI) Statewide Geohazards Viewer (HazVu), which indicates a “Low – Landsliding Unlikely” hazard level at the Project site (Oregon DOGAMI, 2020a).

4.8 Abrupt Offsets

Abrupt offsets typically occur due to liquefaction or where structures founded on transitions between soil and rock experience different magnitudes of settlement. The HDD pipeline will be embedded almost entirely in basalt bedrock and transitioning into soil near the entry and exit points. Liquefaction-induced settlement is not a concern for this Project, and we anticipate settlement at the rock-soil transition zones to be negligible. In addition, changes in subgrade stress conditions will be negligible due to the installation of the connection piping between the new HDD pipeline and the existing watermain and consequently, we anticipate the static settlement of the connection piping will be negligible. Therefore, we do not consider abrupt offset to be a hazard to this Project.

4.9 Debris Flows

Debris flows are particularly dangerous to life and property because they move rapidly, destroy/consume objects in their path, and can strike without warning. They commonly originate as water-laden landslides on steep slopes and transform into liquefied masses of fragmented rock, muddy water, and other debris that discharge from canyons onto valley floors. For a debris flow to initiate, the following conditions must be present: 1) an over-steepened slope; 2) an abundant supply of unconsolidated material; 3) abundant moisture/water; and 4) sparse vegetation. The most susceptible areas to debris flows include: bases of steep hillsides; canyon and incised drainage outlets; narrow stream channels; road cuts or over-steepened slopes; and places where debris flows have occurred in the past.

DOGAMI’s online Statewide Landslide Information Layer for Oregon (SLIDO) (DOGAMI, 2020b) shows no mapped debris flows within the Project area. The Project is located within the broad South Umpqua River valley with no nearby canyon or incised drainage outlets. Furthermore, there are no over-steepened, sparsely vegetated slopes in the immediate project vicinity. Based on these assessments, we do not consider debris flows to be a hazard to this Project.

4.10 Flood Hazard

We assessed the flood hazard potential at the Project site using the online ASCE 7 Hazard Tool, which references the Federal Emergency Management Agency (FEMA) flood maps showing inundation limits in the project area for the 100-and 500-year floods (ASCE, 2020). The flood mapping indicates the 100- and 500-year floods have an inundation elevation of 452 feet (NAVD88). Recommendations for flood hazard mitigation are provided in Section 5.1.4.

4.11 Other Hazards

Other geologic and seismic hazards including tsunami and seiche, are not considered risks for this project.

5.0 Design Recommendations

5.1 Pipeline Design

Geotechnical design recommendations for the connection piping using open-cut installation methods are provided in the following sections. Further details of our HDD analyses and preliminary design recommendations are provided in the Preliminary Design Technical Memorandum (McMillen Jacobs, 2021). All specifications referenced in this section refer to the most current Oregon Standard Specifications for Construction (OSSC) (ODOT, 2021).

Approximately 150 feet of 12-inch ID DI connection piping will be installed between the HDD entry pit and the tie-in connection; generally running south through the parking lot. At the west end of the HDD alignment, the tie-in connection is located approximately 20 feet east of the existing bus stop on the south side of the Adapt facility. Approximately 230 feet of 12-inch diameter DI connection piping will be installed between the HDD exit pit and the tie-in connection; generally running northwest along the gravel access road between the Washington Avenue and Oak Avenue bridges before turning north to cross SE Washington Avenue. Installation depths of the DI connection piping have not been finalized to date and will depend on potential conflicts with existing utilities and other existing infrastructure.

5.1.1 Pipeline Subgrade Support

Based on existing watermain invert depths and subsurface conditions encountered in boring B-1 (near tie-in on SE Oak Avenue) and boring B-2 near the exit pit), we anticipate connection piping subgrade conditions will consist of native alluvial deposits. The undisturbed alluvium should provide adequate subgrade support of the proposed pipeline without modification. However, foundation stabilization may be needed in areas of very soft and wet subgrade conditions. Recommendations for foundation stabilization are provided in Section 5.2.3.

5.1.2 Soil Design Parameters

Flexible pipes, such as DIP, derive their load carrying capacity from their interaction with the pipe zone backfill. Load carrying capacity depends on the type and depth of the pipe, surrounding soil conditions, type and density of the backfill, and thickness of compacted pipe zone backfill between the pipe and the native soil in the trench wall. Based on the anticipated subsurface soil types and relative densities, we have developed geotechnical parameters to be used for pipeline design. These are provided in Table 5-1.

Table 5-1. Pipeline Geotechnical Design Parameters

Property	Native Alluvium	Granular Backfill	CLSM
Moist Unit Weight, γ_m (pcf)	120	130	125
Saturated Unit Weight, γ_{sat} (pcf)	125	135	125
Friction Angle, ϕ (degrees)	30	36	36
Modulus of Soil Reaction, E' (psi)[1]	1,000	2,500	3,000
Soil/DI Pipe Friction Coefficient, μ	0.25	0.4	0.4

Notes:
1. Modulus of soil reaction values are unfactored.
2. Modulus of soil reaction was estimated assuming 2 to 6 feet of cover.

The design parameters presented in Table 5-1 are appropriate for use in the Iowa deflection formula (Spangler, 1941) and are consistent with American Water Works Association Manual M11 (2004). Note that the Modulus of soil reaction, E', is approximately equivalent to the constrained soil modulus, M_s.

The pipe should also be designed considering traffic loads. These loads will vary depending on final depth of cover over the pipeline.

5.1.3 Buoyancy and Flotation

Groundwater data indicate that groundwater depth is below the pipe invert. However, we evaluated flotation of the pipeline using the FEMA-mapped 100-year flood elevation of 452 feet (NAVD88), which is appropriate for long-term design. In addition, we evaluated for a condition assuming an empty pipe and a minimum Factor of Safety of 1.5. The results of our evaluation indicate that a 3-foot depth of cover is sufficient to prevent flotation.

5.1.4 Flood Hazard Mitigation

Flooding will result in buoyancy and water inundation hazards at vaults and other pipeline appurtenances. Submerged structures (e.g., vaults) with sealed lids will experience buoyant forces if water cannot rapidly enter the structure to equilibrate the buoyancy effect. Conversely, water inundation into structures may prevent the proper functioning of appurtenances such as air release valves or damage electronics. Flooding impacts include erosion and scour. Increased flow volume and velocities of surface water during flooding have the potential to erode existing soil from above and adjacent to the pipeline.

As discussed in Section 4.10, the FEMA 100-year flood water surface elevation is 452 feet, which inundates nearly the entire Project area. Installation of appurtenances below the 100-year flood water surface elevation is not recommended. Blow offs should be located out of the floodway where possible. Discharge piping should extend to the floodway channel with energy dissipation if needed.

5.2 Backfill Materials

5.2.1 Bedding and Pipe Zone

The pipe bedding and pipe zones should be constructed with imported, well-graded crushed rock material, such as ¾-inch minus crushed aggregate as specified in OSSC Section 02630.10, Dense-Graded Aggregate with less than 5 percent by weight passing the No. 200 sieve. The material must be suitable for compaction and able to be worked under the haunch of the pipe. We recommend a minimum 6-inch pipe bedding thickness. Where weak, disturbed subgrade is encountered, a foundation stabilization layer should be placed below the bedding is discussed in Section 5.2.3.

Above pipe bedding, imported crushed rock aggregate should be used for the pipe zone, which typically extends at least 12 inches above the top of the pipe. Bedding and pipe zone materials should be compacted to at least 95 percent of the maximum dry density (MDD) as determined by ASTM D698 (Standard Proctor).

The onsite excavation materials are not suitable for use as bedding or pipe zone backfill.

5.2.2 Controlled Low Strength Material (CLSM)

Controlled Low Strength Material (CLSM) is commonly used as an alternative to granular fill in portions of pipelines. CLSM fill mixtures are typically composed of a combination of cement, water, fine aggregate, and fly ash. The material is flowable and self-leveling, which greatly simplifies placement around pipelines. Where CLSM is used, we recommend specifying an unconfined compressive strength of 50 to 200 psi, such that it will protect the pipe, but also have the ability to be removed, if required.

5.2.3 Foundation Stabilization

Based on the subsurface explorations along the HDD alignment, and available well logs (close to the tie-in on W Harvard Avenue), subgrade conditions will consist of native alluvium soils and should generally provide competent subgrade support. However, these soils are moisture-sensitive and easily disturbed if left exposed to water or from general construction activities. If the subgrade becomes weakened due to construction activities or soft/wet subgrade is encountered in localized areas due to perched groundwater, a foundation stabilization layer may be required.

To construct the foundation stabilization layer, the trench should be overexcavated a minimum 12 inches below the bottom of the bedding and replaced with the foundation stabilization layer. The foundation stabilization layer should consist of compacted, free-draining aggregate consisting of 1-½ to ¾- inch conforming with the requirements of OSSC Section 00430.11. Vibratory compaction equipment is not recommended due to risk of additional disturbance to the subgrade. A geotextile should be used below the aggregate as described in Section 5.3.2. The foundation stabilization backfill may also be used as the drainage layer for in-trench dewatering, as described in Section 6.5.

5.2.4 Trench Zone Backfill

Beneath pavements, gravel roads, sidewalks, and other structures, trench zone backfill should consist of imported, well-graded, ¾-inch minus crushed aggregate as specified in OSSC Section 02630.10, Dense-Graded Aggregate. Trench zone backfill should be compacted to at least 95 percent of its maximum dry density as determined by ASTM D698 and placed in 12-inch maximum loose lifts. Trench zone backfill should be placed and compacted up to the base of any fill materials associated with the final pavement section. Native alluvium soils excavated from trenches are not suitable for re-use as trench backfill beneath paved areas or structures. Alternatively, CLSM may be used as trench backfill in areas beneath pavements, gravel roads, sidewalks, and other structures. We understand the City prefers to use CLSM as trench zone for the connection piping within the private property on the north-northwest side of the HDD exit pit; for both pipeline protection and to identify the presence of the pipeline should excavation ever occur over it.

In areas not below pavements or structures, excavated native soils may be suitable for use as trench zone backfill, but must be free of debris, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches, and other deleterious materials. The suitability of use as trench zone backfill also depends on the gradation and moisture content of the soil. As its fines content increases, the soil becomes more sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible. The Contractor should be responsible for determining whether site soils can be cost-effectively moisture conditioned for re-use and compacted. If not, Class B or Class C materials as specified in OSSC Section 00405.14 would be appropriate for use as trench zone backfill in these areas. Compaction of all backfill soils in City ROW should be to minimum 95 percent of MDD.

5.3 Geotextiles

5.3.1 Separation Geotextiles

In general, the widespread use of separation geotextiles is not anticipated for the Project. However, they may be required in localized areas of trench seepage or for protection of subgrade, or in other areas identified during construction. If used, separation geotextiles should consist of a “needle-punched”, non-woven separation fabric meeting the requirements shown in Table 02320-4 in OSSC Section 02320.

5.3.2 Reinforcement Geotextiles

A reinforcement geotextile should be installed beneath the foundation stabilization layer. We recommend using a geotextile that provides both separation/filtration and reinforcement. The geotextile should be installed over the base of the trench and extend up to the top of the foundation stabilization layer (below bedding) at a minimum. Reinforcement geotextiles should meet the requirements for Type 2, woven riprap geotextiles, as shown in Table 02320-2 in OSSC Section 02320.

5.4 Pipeline Seismic Design

The primary considerations for pipeline seismic design are transient loading/movement and permanent ground displacement (PGD). Transient loading is the shaking hazard produced by seismic wave propagation and the amplifications due to surface soil conditions and topography. PGD is the ground movement resulting from surface fault rupture, landslide or slope failure, soil liquefaction, lateral spreading, and differential settlement. Recommended seismic design parameters including PGA, PGV, S_{MS}, and S_{M1} are presented in Table 4-1. These seismic design parameters may be used for seismic pipeline design. Typically, transient strains from ground shaking are not a governing design criterion. Rather, PGD governs design.

Based on our seismic and geologic hazards evaluation presented in Section 4.0, the hazards contributing to potential PGD (e.g., liquefaction, lateral spreading, fault rupture, and slope stability) are negligible at the Project site. Therefore, PGD is not a consideration for pipeline design.

5.5 Thrust Restraint

Thrust forces are anticipated to develop at bends, reducers, offsets, tees, wyes, dead ends and valves along the pipeline (Moser, 2001). Thrust forces are usually counterbalanced by frictional resistance along the backfill-pipe interface, bearing or gravity thrust blocks, restrained joints, or a combination of these resistances. For thrust block design, we recommend the following allowable bearing capacities, at various depths of cover to the center of the thrust block (in parentheses): 750 psf (up to 3 feet); 1,000 psf (between 3 and 5 feet); and 1,200 psf (greater than 5 feet). A friction coefficient of 0.4 along the pipe-to-backfill interface, and 0.3 between the thrust block and subgrade may be used for calculating frictional resistance.

6.0 Construction Recommendations

6.1 Trench Excavation

In general, soils within the anticipated trench excavation depths of 4 to 9 feet bgs (not finalized to date) should consist of native alluvium including silt, clay, sand and gravel. Scattered zones of soft, fine-grained soils may be encountered within the trench excavations. In general, we do not anticipate groundwater will be encountered during construction. However, perched groundwater may be encountered during the winter and spring months. The soils within the trench can be excavated using conventional excavation equipment.

The trench width should extend a minimum of 12 inches beyond the width of the pipe. Where trench shielding or shoring is used, the 12 inches should be measured between the pipe and inside face of the shielding or shoring. This will allow for the use of mechanical compaction equipment on the sides of the pipe. The final trench excavation should be performed with a straight-edged excavator bucket to minimize disturbance to the base of the trench. Following excavation, the trench base should be thoroughly cleaned of loosened or disturbed soils.

The predominately fine-grained and saturated subgrade soils are susceptible to strength loss due to disturbance from construction activities or weather, particularly if excavated during the wet season. As discussed in Section 5.2.3, a foundation stabilization layer may need to be installed below the pipe bedding. The final excavation of the trench should be observed by a representative of the geotechnical engineer to verify suitability of the subgrade and determine if additional excavation is necessary.

6.2 Pipeline Backfill

Recommended material types and compaction requirements for pipeline backfill are included in Section 5.2. Procedures to achieve proper density of a compacted fill depends on the size and type of compaction equipment, the number of passes, loose lift thickness, and subgrade properties. Compaction equipment may be limited by the trench dimensions and lift size may need to be reduced to achieve the required compaction. The methods of compaction should be left to the Contractor’s discretion.

6.3 Surface Restoration

Pavement above the pipe trench should be replaced to match the either the existing pavement section or to current City or ODOT standards and specifications. The existing pavement sections should be saw cut to full depth and a minimum width of 6 inches beyond the edge of the trench and removed (e.g., “T” cut). The gravel access road between the Washington Avenue and Oak Avenue bridges should be replaced to match existing conditions or to current City standards and specifications.

6.4 Temporary Shoring

6.4.1 Trench Shoring

Maintenance of safe working conditions, including temporary excavation stability, should be the responsibility of the Contractor. All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. Site soils are generally

OSHA Type C. For excavations up to 20 feet, the maximum allowable temporary slope for Type C soils is 1.5H:1V if fully dewatered. Excavations deeper than 4 ft will require shoring.

Precautions should be taken during removal of the shoring materials to minimize disturbance of the pipe, underlying bedding materials, and trench sidewalls. The open excavation behind a trench box should be backfilled immediately after the trench box has been placed. Heavy construction equipment, construction materials, excavated soil, and vehicular traffic should not be allowed within a distance from the edge of the excavation equal to the depth of the excavation, unless the shoring system has been designed for the additional lateral pressure.

Open-cut trenches up to 15 feet deep can typically be performed with conventional trench shield methods. However, trench shields do not provide positive pressure against the sidewalls of the trench. This is likely to allow the trench sidewall to move or slough against the shield. The trench sidewall movement or sloughing may potentially, in turn, affect the adjacent roadways, bridge embankments, and other nearby utilities causing short-term or long-term settlements. This will be an important consideration to minimize potential movement of the approach embankment slopes for open-cut pipeline installation along the gravel access road between the Washington Avenue and Oak Avenue bridges. Within this segment, the pipeline trench shoring should consist of a positive shoring system, such as slide rail, closed (full face) sheeting with hydraulic struts, or trench boxes with immediately backfill to fill any voids behind the shoring. We anticipate that ODOT will have specific shoring requirements to protect the bridge approach embankments slopes during construction.

6.4.2 Shoring Plans

The Contractor is responsible for reviewing the boring logs, selecting and designing the specific methods, monitoring the excavations for safety, and providing shoring required to protect personnel and adjacent structural elements and utilities. Shoring deeper than 6 feet should be designed by a registered engineer who should be provided with a copy of this report. Shoring should be designed and constructed to support the earth pressures presented on Figure 4, plus surcharge loads from construction equipment, construction materials, excavated soils, vehicular traffic, as well as the adjacent bridge approach embankments. Groundwater control should be incorporated in accordance with Section 6.5. If soils behind impermeable shoring systems are not dewatered, the hydrostatic pressure should be added as an additional surcharge.

6.5 Groundwater Control

If work is completed during the wet season, areas along the open-cut trenches may encounter perched groundwater conditions. Based on their predominately fine-grained characteristics and overall low hydraulic conductivity, the native alluvium soils within the anticipated trenching depths are not anticipated to produce large volumes of groundwater. Therefore, we anticipate that groundwater inflow can be controlled with a well-constructed, sump pumping dewatering system. Sump pumps should be installed with close spacing to maintain water levels below the subgrade surface. In the case that large volumes of water seepage are encountered, perforated drainpipes installed in drainage layers (i.e., crushed rock) may be necessary to convey water to the sump pump systems and could be incorporated with the foundation stabilization layer.

6.6 Temporary Cuts

Depending on the proposed excavation and shoring plan for different project elements, temporary cut slopes may be required during construction. If open cuts are utilized, maximum slope inclinations must be made in accordance with the OSHA regulations referenced in Section 6.4.1.

Temporary slope recommendations do not consider site constraints such as groundwater, surcharge, or nearby structures. Temporary slopes should be evaluated on a case-by-case basis and incorporate groundwater conditions, soil classification, and site constraints. Slopes should be inspected and maintained as required by OSHA.

With time and the presence of seepage and precipitation, the stability of temporary unsupported cut slopes can be significantly reduced. Therefore, temporary slopes kept open for construction activities should be protected from erosion by installing a surface water diversion ditch or berm at the top of the slope and covering the cut face with well-anchored plastic sheets. In addition, the Contractor should monitor the stability of the temporary cut slopes and adjust the construction schedule and slope inclination accordingly. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the Contractor and all excavations must comply with current federal, state, and local requirements.

6.7 Wet Weather Earthwork

The work areas at the HDD entry and exit pits, as well as along the DI connection piping trenches, are either asphalt-paved or gravel-surfaced. We don’t anticipate these surfaces degrading due to wet weather or heavy construction equipment traffic. However, we recommend the following practices if earthwork is performed during extended periods of wet weather or in wet conditions:

- Cover the base of trenches with trench foundation materials as described in Section 5.2.3.
- Trench areas should be protected from surface water runoff by placing sandbags or by other means to promote runoff of precipitation away from work areas and to prevent ponding of water in trenches.
- Plastic covers, sloping, ditching, sumps, dewatering, and other measures should be employed in work areas and slopes as necessary to permit timely completion of work. Bales of straw and/or geotextile silt fences should be used to control surface soil movement and erosion.
- Trench excavation should be completed in small sections and backfilled at the end of each day to reduce exposure to wet conditions.
- Excavation or the removal of unsuitable soil should be followed promptly by placement and compaction of trench stabilization fill.
- The size and type of construction equipment used may have to be limited to prevent soil disturbance.

7.0 Closure

This Geotechnical Exploration and Evaluation Report has been prepared for Washington Ave Bore Crossing Project in Roseburg, Oregon. The data, analyses, conclusions and recommendations presented in this report are based on the subsurface conditions at the time that the geotechnical investigation for the project was completed. This report also contains information and data collected from other relevant studies conducted, as well our site reconnaissance and our professional experience and judgement.

In the performance of geotechnical work, specific information is obtained at specific locations at specific times, and geologic conditions can change over time. It should be acknowledged that variations in soil conditions may exist between exploration and exposed locations and this report does not necessarily reflect variations between different explorations. The nature and extent of variation may not become evident until construction. McMillen Jacobs Associates is not responsible for the interpretation of the data contained in this report by anyone; as such interpretations are dependent on each person’s subjectivity. If, during construction, conditions different from those disclosed by this report are observed or encountered, McMillen Jacobs Associates should be notified at so we can observe and review these conditions and reconsider our recommendations where necessary.

The site investigation and this report were completed within the limitations of the McMillen Jacobs Associates approved scope of work, schedule, and budget. The services rendered have been performed in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions in the same area. McMillen Jacobs Associates is not responsible for the use of this report in connection with anything other than the project at the location described above.

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REGISTERED PROFESSIONAL ENGINEER
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OREGON
June 7, 2012
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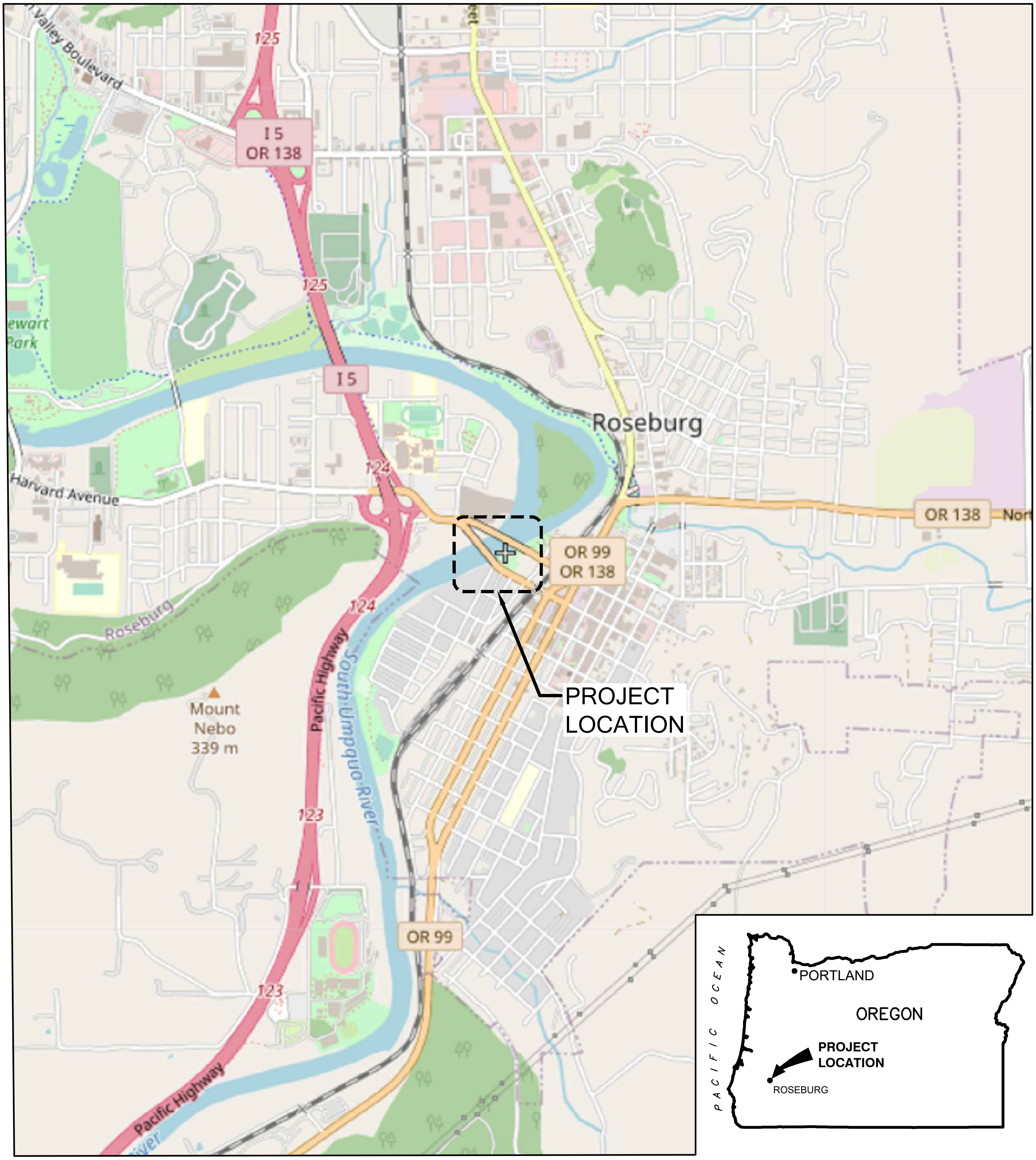
Figures



PROJECT VICINITY MAP

SCALE: NTS

N

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CITY OF ROSEBURG

WASHINGTON AVE BORE CROSSING

GEOTECHNICAL EXLORATION & EVALUATION REPORT
PROJECT VICINITY MAP

FIG.1

MARCH 2021

Path: C:\Users\Ruston\Box\Jobs\6194.0 Washington Ave Bore Crossing City of Roseburg OR\CADD\03 Drawings\FIGURES\FIG 1.dwg Plot date: Jan 14, 2021, CAD User: Ruston

SITE PLAN

SCALE: 1"=80'   0'   80'   160'

LEGEND:

B-01 ⌖	BOREHOLE LOCATION
— — —	PROPERTY LINE
— R/W —	RIGHT OF WAY

NOTES:

1. AERIAL PHOTO PROVIDED BY MURRAYSMITH IN JANUARY 2021.
2. EXPLORATION LOCATIONS ARE APPROXIMATE

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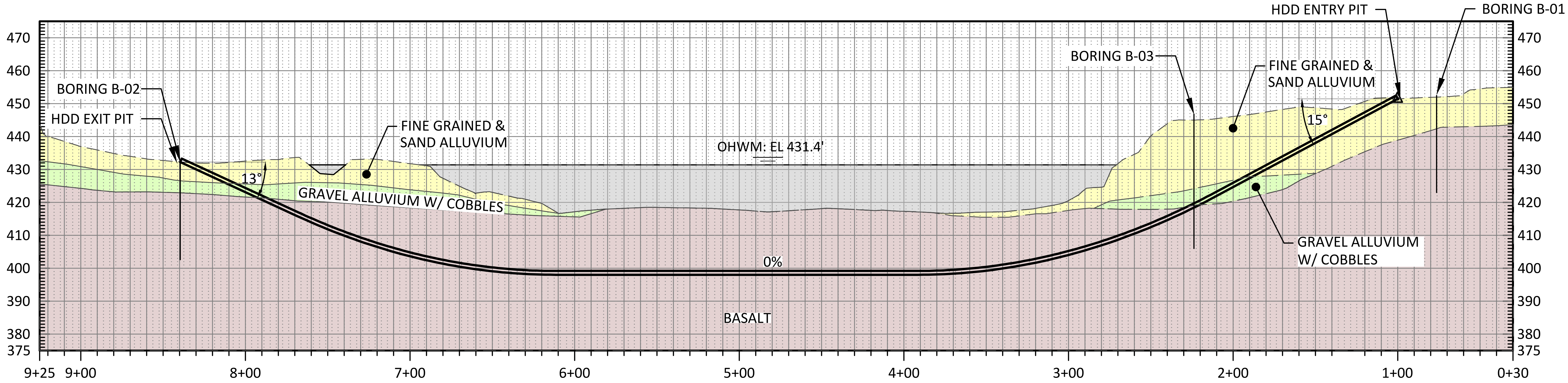
CITY OF ROSEBURG
WASHINGTON AVE BORE CROSSING
GEOTECHNICAL EXPLORATION & EVALUATION REPORT
SITE PLAN

FIG.2

March 2021



PROFILE

SCALE: 1"=60' H

0' 60' 120'

1"=30' V

0' 30' 60'

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CITY OF ROSEBURG

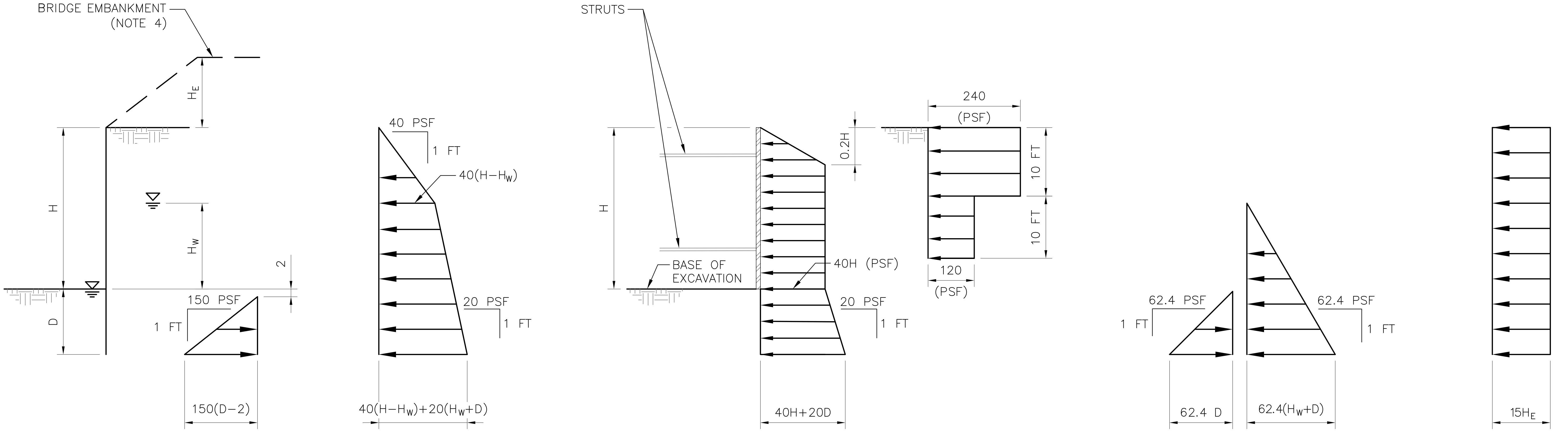
WASHINGTON AVE BORE CROSSING

GEOTECHNICAL EXLORATION & EVALUATION REPORT
GEOLOGICAL PROFILE

FIG.3

MARCH 2021

Path: C:\Users\Ruston\Box\Jobs\6194.0 Washington Ave Bore Crossing City of Roseburg OR\CADD\03 Drawings\FIGURES\FIG 3.dwg Plot date: Jan 15, 2021, CAD User: Ruston



PASSIVE PRESSURE

ACTIVE EARTH PRESSURE FOR CANTILEVER WALLS (NOTE 2)

EARTH PRESSURE FOR BRACED EXCAVATIONS (NOTE 2)

TEMPORARY SURCHARGE PRESSURE (NOTE 3)

HYDROSTATIC PRESSURE

ROADWAY EMBANKMENT SURCHARGE PRESSURE (NOTE 4)

LATERAL EARTH PRESSURE DIAGRAMS FOR TEMPORARY EXCAVATION SHORING

SCALE: NTS

NOTES:

1. THE EARTH PRESSURE FOR CANTILEVER AND BRACED EXCAVATION ARE THE MINIMUM TO BE USED FOR DESIGN OF THE TEMPORARY EXCAVATION SUPPORT SYSTEM. THE LOADS DO NOT INCLUDE ANY FACTORS OF SAFETY THAT MUST BE APPLIED IN THE SHORING SYSTEM DESIGN. CONTRACTOR IS RESPONSIBLE FOR EVALUATING ACTUAL LOADS, BUT IN NO CASE SHALL BE LESS THAN THOSE SHOWN.
2. DEPENDING ON SHORING SYSTEM DESIGN, USE EITHER EARTH PRESSURE FOR CANTILEVER SHORING OR BRACED EXCAVATIONS USING BOTH SIMULTANEOUSLY IS NOT REQUIRED.
3. THE TEMPORARY SURCHARGE PRESSURE SHOWN IS THE MINIMUM REQUIRED AND BASED ON A VERTICAL SURCHARGE OF 600 PSF AT THE TOP OF THE SHORING. CONTRACTOR SHALL DEVELOP SPECIFIC SURCHARGE PRESSURES BASED ON ACTUAL EQUIPMENT USED.
4. SHORING SIDE IMMEDIATELY ADJACENT TO THE EMBANKMENT SHALL BE DESIGNED TO CONSIDER EMBANKMENT SURCHARGE LOAD.

LEGEND:

H	HEIGHT OF TEMPORARY EXCAVATION IN FEET
H_W	HEIGHT OF WATER ABOVE THE BASE OF EXCAVATION
D	DEPTH OF EMBEDMENT
H_E	HEIGHT OF EMBANKMENT

McMILLEN JACOBS ASSOCIATES

CITY OF ROSEBURG
WASHINGTON AVE BORE CROSSING
GEOTECHNICAL EXLORATION & EVALUATION REPORT
EARTH PRESSURE DIAGRAM
TEMPORARY SHORING

FIG.4

MARCH 2021

Appendix A

Field Explorations

Key to Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS based on ASTM D2487 & D2488)

MAJOR DIVISIONS			GROUP/SYMBOL		TYPICAL DESCRIPTION
COARSE-GRAINED SOILS (50% or more retained by No. 200 sieve)	GRAVELS (more than 50% retained on No. 4 sieve)	CLEAN GRAVELS (less than 5% fines)	GW	▨	WELL-GRADED GRAVEL
			GP	▨	POORLY GRADED GRAVEL
		GRAVELS (with 5 to 12% fines)	GW-GM	▨	WELL-GRADED GRAVEL WITH SILT
			GW-GC	▨	WELL-GRADED GRAVEL WITH CLAY
			GP-GM	▨	POORLY GRADED GRAVEL WITH SILT
			GP-GC	▨	POORLY GRADED GRAVEL WITH CLAY
		GRAVELS WITH FINES (more than 12% fines)	GM	▨	SILTY GRAVEL
			GC	▨	CLAYEY GRAVEL
	SANDS (less than 50% retained on No. 4 sieve)	CLEAN SANDS (less than 5% fines)	SW	░	WELL-GRADED SAND
			SP	░	POORLY GRADED SAND
		SANDS (with 5 to 12% fines)	SW-SM	░	WELL-GRADED SAND WITH SILT
			SW-SC	░	WELL-GRADED SAND WITH CLAY
			SP-SM	░	POORLY GRADED SAND WITH SILT
			SP-SC	░	POORLY GRADED SAND WITH CLAY
		SANDS WITH FINES (more than 12% fines)	SM	░	SILTY SAND
			SC	░	CLAYEY SAND
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS & CLAYS (liquid limit less than 50)	INORGANIC	ML	▥	SILT
			CL	▧	LEAN CLAY
		ORGANIC	OL	▧	LOW PLASTICITY ORGANIC CLAY
	SILTS AND CLAYS (liquid limit greater than 50)	INORGANIC	MH	▥	ELASTIC SILT
			CH	▧	FAT CLAY
		ORGANIC	OH	▧	HIGH PLASTICITY ORGANIC CLAY
	SILT/CLAY (liquid limit 12 -25; PI 4-7)	INORGANIC	CL-ML	▧	CLAYEY SILT/SILTY CLAY
HIGHLY ORGANIC SOILS	PRIMARILY ORGANIC MATTER		PT	▨	PEAT

Note:
Dual symbols (symbols separated by a hyphen, e.g. SP-SM) are used for soils between 5% and 15% fines or when liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

Abbreviations

AL	Atterberg Limit
MC	Moisture Content
LL	Liquid Limit
PL	Plastic Limit

Sample and Test Symbols

⧗	SPT Sample 2" OD
■	Shelby Tube Sample
✋	Grab Sample
▮▮	Rock Core Run
N	Blows/ft
○	Moisture Content
⊢⊣	Liquid Limit/Plastic Limit

Well and Backfill Symbols

▦	Bentonite Chips
▨	Concrete
░	Sand
■	Asphalt
⁘	Gravel
▧	Grout
◫	Observation Well Solid Interval
▤	Observation Well Screened Interval
▮	Vibrating Wire Piezometer
▼	Measured groundwater level

Modifiers and Percentages

Trace	Component is present up to 5%
Slightly	Component is present between 5% - 15%
With (Sand or Gravel)	Component is present between 15% - 30%
Sandy or Gravelly	Component is present between 30% - 50%

Modifiers and Percentages

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

Fine-Grained Soils

Relative Consistency	N, SPT Blows/Foot	Field Test
Very Soft	0 to 1	Easily penetrated several inches by thumb. Extrudes between thumb and fingers when squeezed.
Soft	2 to 4	Easily penetrated one inch by thumb. Molded by light finger pressure.
Medium Stiff	5 to 8	Can be penetrated over $\frac{1}{4}$ inch with moderate pressure. Molded by strong finger pressure.
Stiff	9 to 15	Indented about 1/4 inch by thumb, but penetrated only with great effort.
Very Stiff	16 to 30	Readily indented by thumbnail.
Hard	> 30	Indented with difficulty by thumbnail.

Coarse-Grained Soils

Relative Density	N, SPT Blows/Foot	Field Test (Penetration with 1/2 in. hand probe)
Very Loose	0 to 4	3 ft.
Loose	5 to 10	1 to 2 ft.
Medium Dense	11 to 30	3 to 12 in.
Dense	30 to 50	1 to 3 in.
Very Dense	> 50	

Key to Boring Logs - Rock

Rock Strength

Description	Recognition	Uniaxial Compressive Strength (psi)
Extremely Weak Rock	Indented by thumbnail	30 to 150
Very Weak Rock	Peeled by pocket knife	150 to 700
Weak Rock	Peeled with difficulty by pocket knife	700 to 3,600
Moderately Strong Rock	Indented 5 mm with sharp end of pick	3,600 to 7,200
Strong Rock	One hammer blow to fracture	7,200 to 14,500
Very Strong Rock	Many hammer blows to fracture	14,500 to 36,000
Extremely Strong Rock	Only chipped by hammer blows	> 36,000

Core Recovery Calculation (%)

$$\frac{\Sigma \text{ Length of recovered core}}{\text{Total Length of core run}} \text{ x100}$$

RQD Calculation (%)

$$\frac{\Sigma \text{ Length of core pieces} > 4 \text{ in.}}{\text{Total Length of core run}} \text{ x100}$$

Rock Weathering

Residual Soil	Entirely decomposed to secondary minerals; material can be easily broken by hand
Completely Weathered	Almost entirely decomposed to secondary minerals; material can be granulated by hand
Highly Weathered	More than half of the rock is decomposed
Moderately Weathered	Rock is discolored and noticeably weakened, but less than half is decomposed
Slightly Weathered	Rock is slightly discolored, but not noticeably lower in strength than fresh rock
Fresh	Rock shows no discoloration, loss of strength, or other effect of weathering or alteration

Discontinuity Type

J	Joint
FJ	Joint along foliation
S	Shear
F	Fault
HJ	Healed joint
MB	Mechanical break
B	Joint along bedding

Discontinuity Spacing

Extremely Close Spacing	Less than 1 inch apart
Very Close Spacing	1 to 2.5 inches apart
Close Spacing	2.5 to 8 inches apart
Moderate Spacing	8 inches to 2 feet apart
Wide Spacing	2 to 6.5 feet apart
Very Wide Spacing	6.5 to 20 feet apart
Extremely Wide Spacing	Greater than 20 feet apart

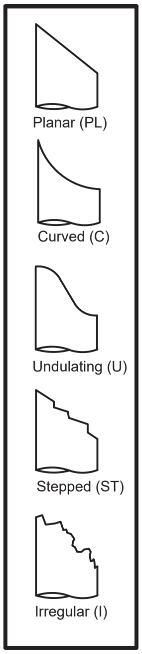
Sample Symbols

▮▮	HQ3 Rock Coring

Lithology Graphics

Basalt	▨	Core Loss/ No Recovery	☐

Fracture Shape



Surface Roughness

Slickensided	Surface has smooth, glassy finish with visual evidence of striations
Smooth	Surface appears smooth and feels so to the touch
Slightly Rough	Asperities on discontinuity surfaces are distinguishable and can be felt
Rough	Ridges and side-angle steps are evident, surface feels very abrasive
Very Rough	Near vertical steps and ridges occur on discontinuity surface

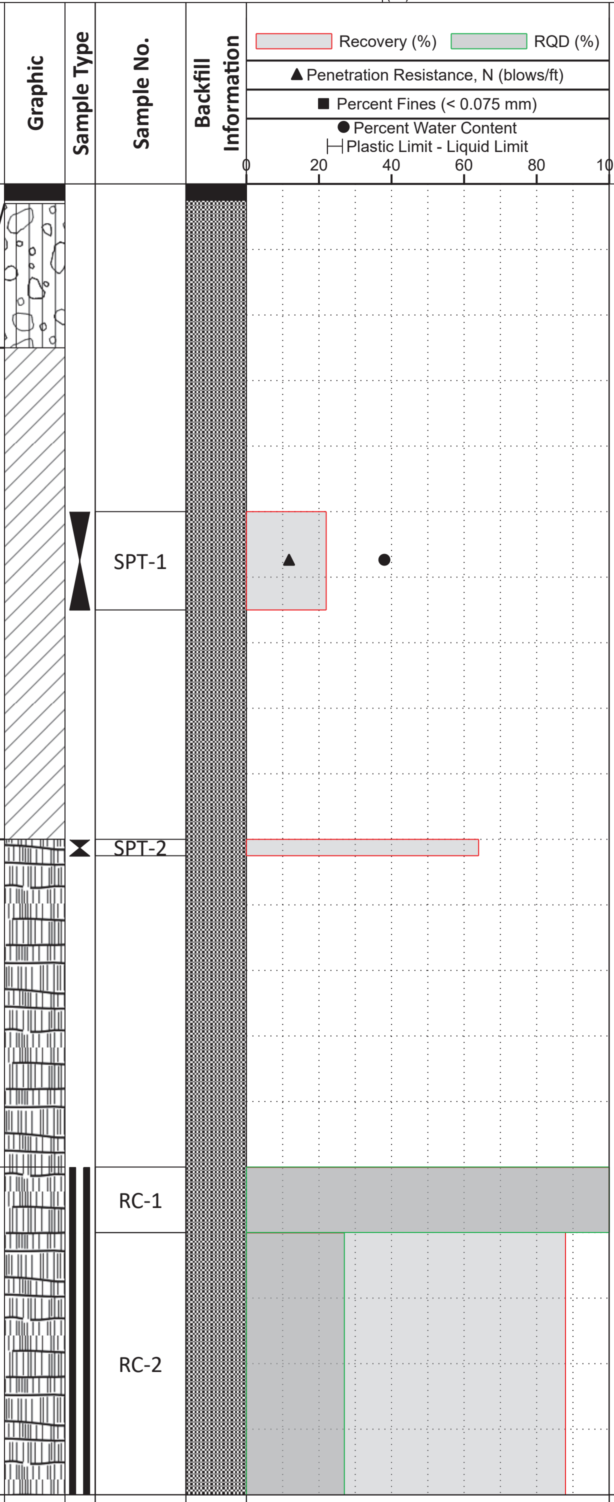
Project: Washington Ave Bore Crossing City of Roseburg OR
Project Location: Roseburg, OR
Project Number: 6194.0

Log of Boring B-01

Date(s) Drilled	**Nov 05 2020**	Client	**City of Roseburg**	Logged By	**J. Quinn**	Checked By	**F. Sariosseiri**
Drilling Method/ Rig Type	**Mud Rotary and HQ Wireline/CME 75**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**65.0 ft.**		
Hole Diameter	**3.75 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**452.4 ft./NAVD88**		
Location	**East end of HDD alignment**	Coordinates	**159949.30E,137626.60N**	Hammer Efficiency (%)			

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Remarks and Tests
		0	Asphalt Concrete (3 inches thick). [Pavement]	
		1	Silty GRAVEL (GM); typically 1- to2-inch diameter, rounded gravel. [Fill]	
		2.5	Stiff, moist, brown and gray, Lean CLAY (CL); trace fine sand, medium plasticity. [Alluvium]	
448				
		7.5	*7.5 - 8.5 ft.: fine, angular gravel present.*	Difficult drilling at 7.5 feet.
		8.5	*8.5 ft.: Grades to Fat CLAY (CH) at 8.5 feet bgs.*	
443				
		10	BASALT, gray, slightly weathered to fresh, medium strong. [Siletz River Volcanics]	Basalt description from 10 to 15 feet bgs based on drill cuttings. 44 minutes to advance from 10 to 15 feet bgs with tricone drill bit.
438				
		15	BASALT, gray, slightly weathered to fresh, medium strong, very closely to widely spaced joints, joints typically rough and planar with calcite infilling. [Siletz River Volcanics]	Begin coring at 15 feet bgs.
		16	*16.0 ft.: becomes moderately weathered, with extremely closely spaced joints.*	
		18	*18.0 ft.: becomes slightly weathered to fresh, closely spaced joints, calcite and iron oxide staining on joint surfaces.*	
433				
		19.3	*19.3 - 21.0 ft.: very closely spaced joints, joints infilled with gravel and clay.*	
		20		

Sample No.	N Value
SPT-1	12
SPT-2	100
RC-1	
RC-2	



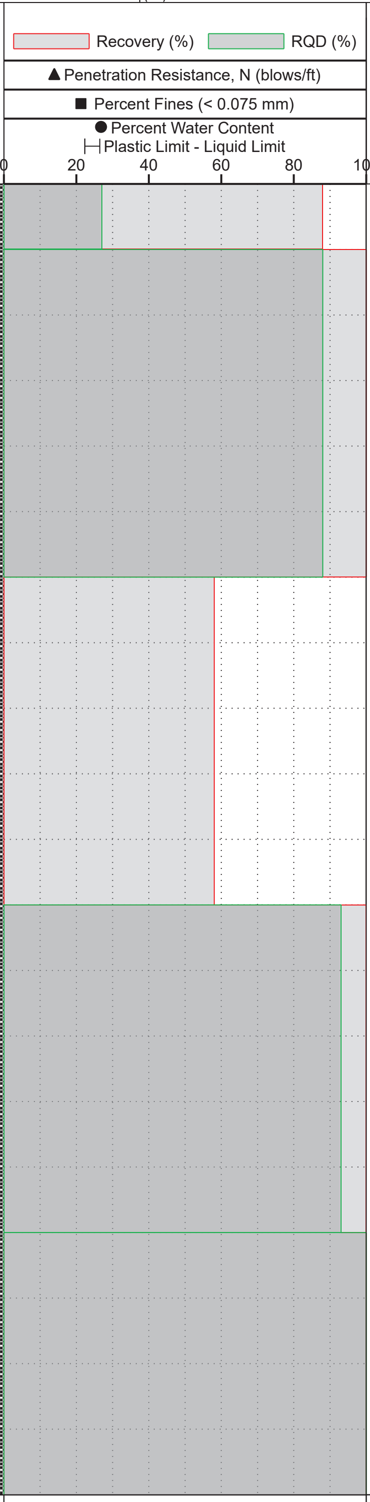
Project: Washington Ave Bore Crossing City of Roseburg OR
Project Location: Roseburg, OR
Project Number: 6194.0

Log of Boring B-01

Date(s) Drilled	**Nov 05 2020**	Client	**City of Roseburg**	Logged By	**J. Quinn**
Checked By	**F. Sariosseiri**				
Drilling Method/ Rig Type	**Mud Rotary and HQ Wireline/CME 75**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**65.0 ft.**
Hole Diameter	**3.75 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**452.4 ft./NAVD88**
Location	**East end of HDD alignment**	Coordinates	**159949.30E,137626.60N**	Hammer Efficiency (%)	

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Sample No.	Remarks and Tests
		21	BASALT, gray, slightly weathered to fresh, medium strong, very closely to widely spaced joints, joints typically rough and planar with calcite infilling. [Siletz River Volcanics]		
		22	*21.0 ft.: becomes fresh, joints become closely to moderately spaced, joints healed with calcite, calcite mineralization on open joints.*		21.7 to 23.0 ft.: UCS = 6,972 psi Unit Weight = 175.2 pcf
		23		RC-3	
428		24			
		25			
		26	*26.0 - 31.0 ft.: slightly weathered to fresh, very closely to extremely closely spaced joints with frequent clay infilling between joints.*		Frequent soft/ clay within Core Run No. 4. Clay likely washed away by drilling fluid which could explain low core recovery
		27			
		28		RC-4	
423		29			
		30			
		31	*31.0 ft.: becomes fresh, moderately spaced joints, calcite mineralization on joint surfaces, calcite healed joints, frequent calcite veins.*		
		32			
		33			
418		34		RC-5	
		35			
		36			
		37			36.5 to 37.5 ft.: UCS = 3,683 psi Unit Weight = 173.6 pcf
		38		RC-6	
413		39			
		40			

Graphic | Sample Type | Backfill Information | N Value



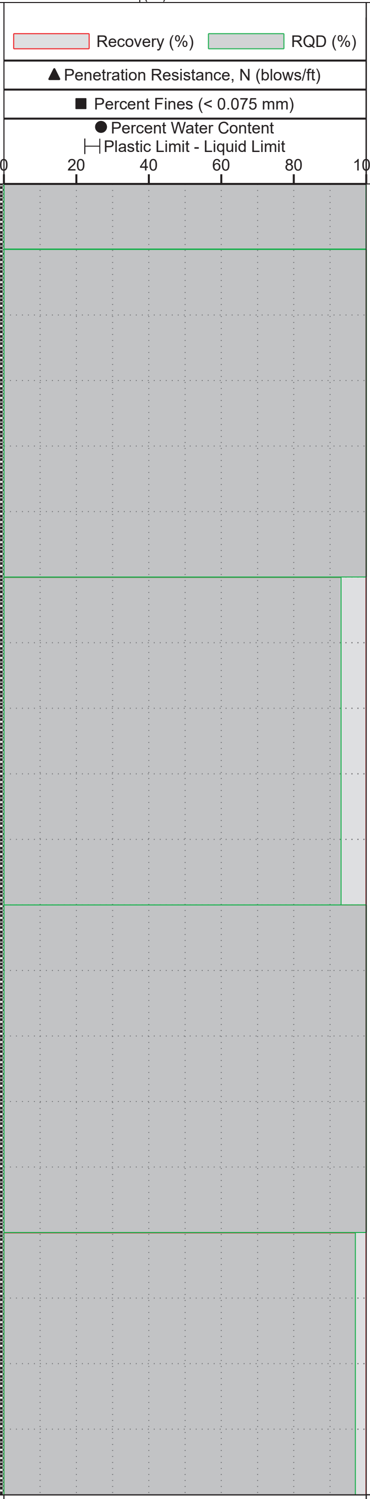
Boring B-01

Project: Washington Ave Bore Crossing City of Roseburg OR
Project Location: Roseburg, OR
Project Number: 6194.0

Log of Boring B-01

Date(s) Drilled	**Nov 05 2020**	Client	**City of Roseburg**	Logged By	**J. Quinn**	Checked By	**F. Sariosseiri**
Drilling Method/ Rig Type	**Mud Rotary and HQ Wireline/CME 75**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**65.0 ft.**		
Hole Diameter	**3.75 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**452.4 ft./NAVD88**		
Location	**East end of HDD alignment**	Coordinates	**159949.30E,137626.60N**	Hammer Efficiency (%)			

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Graphic	Sample Type	Sample No.	Backfill Information	N Value	Remarks and Tests
		41	BASALT, gray, slightly weathered to fresh, medium strong, very closely to widely spaced joints, joints typically rough and planar with calcite infilling. [Siletz River Volcanics]						
		42							
		43				RC-7			
408		44							
		45							
		46	*46.0 ft.: becomes moderately to widely spaced joints.*						
		47							
		48				RC-8			
403		49							
		50							
		51							
		52							
		53				RC-9			
398		54							
		55							
		56	*56.0 - 58.7 ft.: increased calcite veining.*						
		57							
		58	*58.0 ft.: becomes darker gray to green-gray.*			RC-10			57.3 to 58.2 ft.: UCS = 4,257 psi Unit Weight = 161.7 pcf
393		59							
		60							



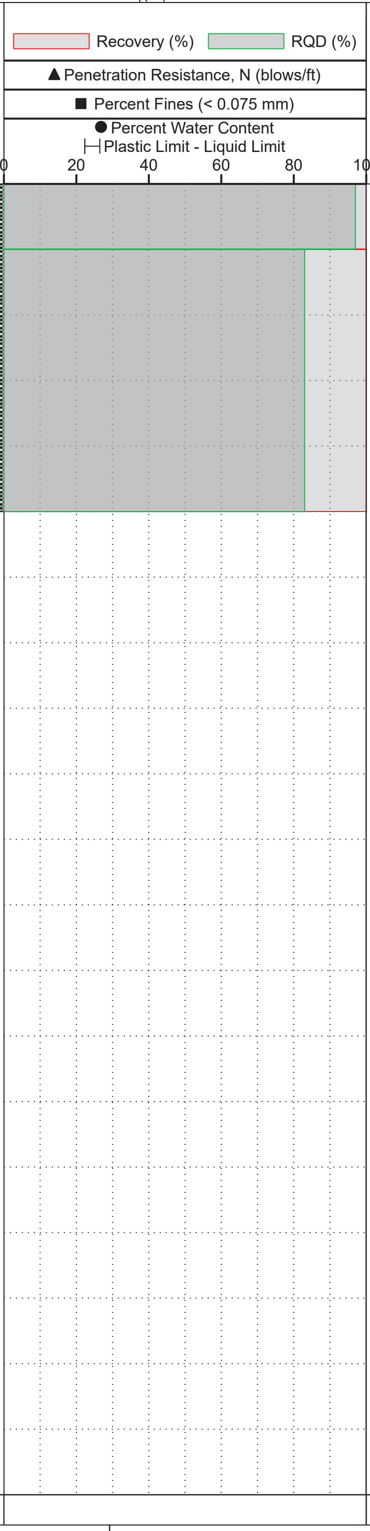
McMILLEN JACOBS ASSOCIATES

Project: Washington Ave Bore Crossing City of Roseburg OR
Project Location: Roseburg, OR
Project Number: 6194.0

Log of Boring B-01

Date(s) Drilled	**Nov 05 2020**	Client	**City of Roseburg**	Logged By	**J. Quinn**	Checked By	**F. Sariosseiri**
Drilling Method/ Rig Type	**Mud Rotary and HQ Wireline/CME 75**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**65.0 ft.**		
Hole Diameter	**3.75 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**452.4 ft./NAVD88**		
Location	**East end of HDD alignment**	Coordinates	**159949.30E,137626.60N**	Hammer Efficiency (%)			

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Graphic	Sample Type	Sample No.	Backfill Information	N Value	Remarks and Tests
		61 62 63 64	BASALT, gray, slightly weathered to fresh, medium strong, closely to moderately spaced joint, joints typically rough and planar with calcite infilling. *62.0 ft.: Becomes weak at 62 feet.*			RC-11			62.1 to 62.9 ft.: UCS = 3,479 psi Unit Weight = 166.8 pcf
388		65							Borehole completed at 65ft. below ground surface (bgs).
383		66 67 68 69 70							
378		71 72 73 74 75							
373		76 77 78 79 80							



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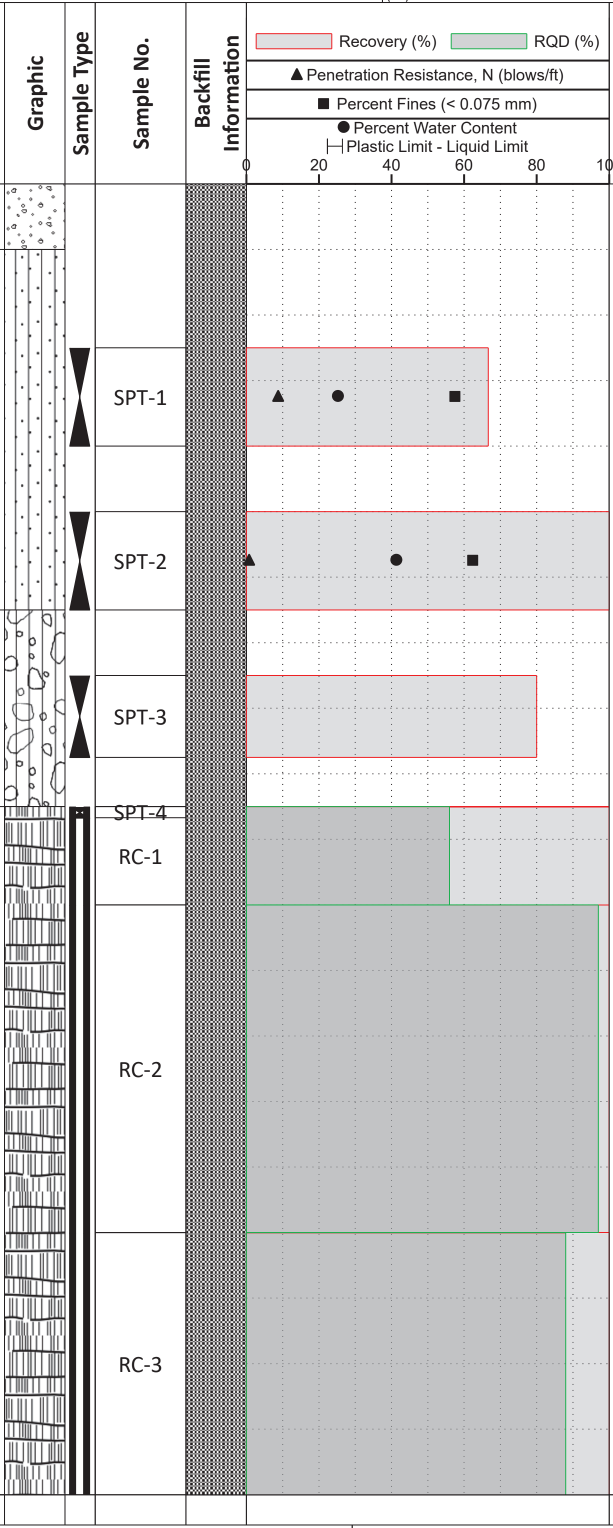
Project: Washington Ave Bore Crossing City of Roseburg OR
Project Location: Roseburg, OR
Project Number: 6194.0

Log of Boring B-02

Date(s) Drilled	**Nov 03 2020 - Nov 04 2020**	Client	**City of Roseburg**	Logged By	**J. Quinn**
				Checked By	**F. Sariosseiri**
Drilling Method/ Rig Type	**Mud Rotary and HQ Wireline/CME 75**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**55.0 ft.**
Hole Diameter	**3.75 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**432.4 ft./NAVD88**
Location	**West end of HDD alignment**	Coordinates	**159384.30E,138142.70N**	Hammer Efficiency (%)	

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Sample No.	N Value	Remarks and Tests
		0 - 1	GRAVEL with cobbles [Fill]			Difficult drilling in upper foot.
		1 - 6.5	Loose, moist, gray to blue-gray, Silty SAND (SM); fine sand, trace fine rootlets, non-plastic fines. [Alluvium]	SPT-1	9	S-1: 57.6% fines by ASTM D1140
428			*5.0 ft.: Becomes very loose, dark gray.*	SPT-2	1	S-2: 62.5% fines by ASTM D1140
	▽ (8)	6.5 - 9.5	Very dense, moist to wet, gray and brown, Silty GRAVEL (GM); fine to coarse gravel, trace sand, non-plastic fines. [Alluvium] *8.5 ft.: decomposed rock structure apparent.*	SPT-3	100	Slow difficult drilling at 6.5 ft. bgs, indicates cobbles and potentially boulders.
423		9.5 - 20	BASALT, dark gray, slightly weathered to fresh, strong, closely to moderately spaced joints, joints typically rough and planar. [Siletz River Volcanics] *9.5 - 9.9 ft.: extremely closely spaced joints.* *9.5 - 25.0 ft.: Occasional iron oxide staining on joint surfaces.* *11.0 ft.: becomes fresh.*	SPT-4 RC-1	100	Begin coring at 9.5 feet bgs.
418			*13.9 - 15.1 ft.: calcite healed joint.*	RC-2		
413				RC-3		16.0 to 16.7 ft.: UCS = 8,024 psi Unit Weight = 179.2 pcf



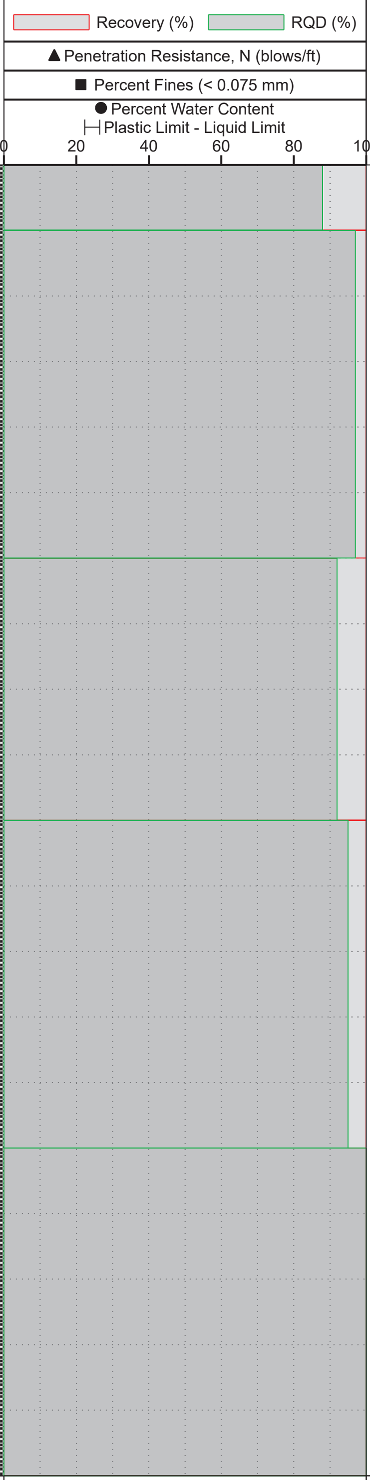
Boring B-02

Project: Washington Ave Bore Crossing City of Roseburg OR
Project Location: Roseburg, OR
Project Number: 6194.0

Log of Boring B-02

Date(s) Drilled	**Nov 03 2020 - Nov 04 2020**	Client	**City of Roseburg**	Logged By	**J. Quinn**	Checked By	**F. Sariosseiri**
Drilling Method/ Rig Type	**Mud Rotary and HQ Wireline/CME 75**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**55.0 ft.**		
Hole Diameter	**3.75 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**432.4 ft./NAVD88**		
Location	**West end of HDD alignment**	Coordinates	**159384.30E,138142.70N**	Hammer Efficiency (%)			

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Sample No.	Remarks and Tests
		20.0	BASALT, dark gray, slightly weathered to fresh, strong, closely to widely spaced joints, joints typically rough and planar, frequent calcite veining. [Siletz River Volcanics] *20.0 - 20.5 ft.: extremely closely spaced joints.*		
		21			
		22			
		23		RC-4	
408		24			
		25			
		26	*26.5 ft.: gravel infilled joint.*		
		27			
		28		RC-5	
403		29			
		30	*30.0 ft.: gravel infilled joint.* *30.0 - 32.0 ft.: becomes weak, based on UCS test results.*		30.0 to 31.2 ft.: UCS = 3,174 psi Unit Weight = 170.9 pcf
		31			
		32			
		33		RC-6	
398		34			
		35			
		36			
		37		RC-7	
		38			
393		39			
		40			

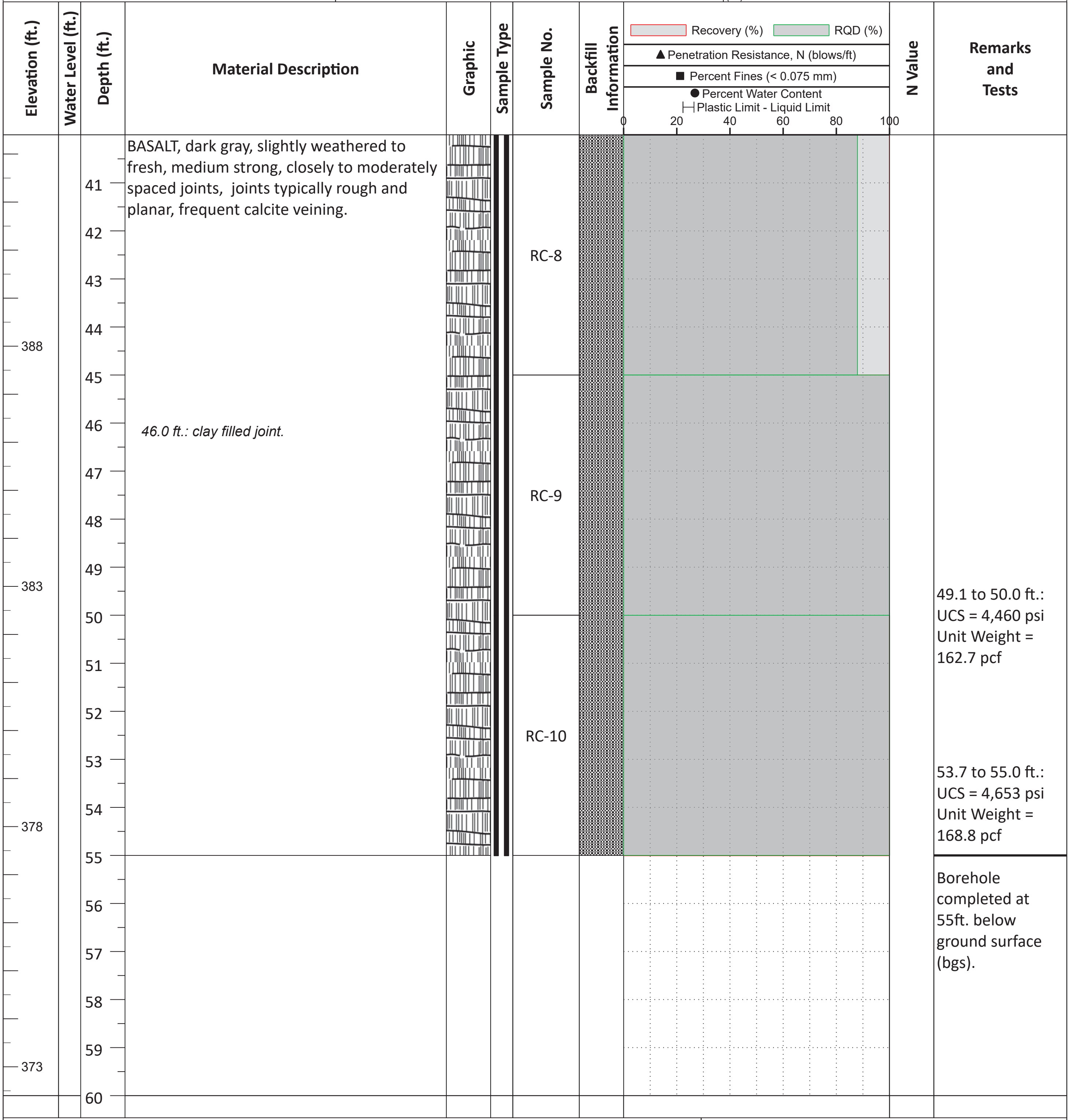


Graphic | Sample Type | Backfill Information | N Value

Project: Washington Ave Bore Crossing City of Roseburg OR
Project Location: Roseburg, OR
Project Number: 6194.0

Log of Boring B-02

Date(s) Drilled	**Nov 03 2020 - Nov 04 2020**	Client	**City of Roseburg**	Logged By	**J. Quinn**	Checked By	**F. Sariosseiri**
Drilling Method/ Rig Type	**Mud Rotary and HQ Wireline/CME 75**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**55.0 ft.**		
Hole Diameter	**3.75 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**432.4 ft./NAVD88**		
Location	**West end of HDD alignment**	Coordinates	**159384.30E,138142.70N**	Hammer Efficiency (%)			



BASALT, dark gray, slightly weathered to fresh, medium strong, closely to moderately spaced joints, joints typically rough and planar, frequent calcite veining.

46.0 ft.: clay filled joint.

49.1 to 50.0 ft.:
UCS = 4,460 psi
Unit Weight = 162.7 pcf

53.7 to 55.0 ft.:
UCS = 4,653 psi
Unit Weight = 168.8 pcf

Borehole completed at 55ft. below ground surface (bgs).

McMILLEN JACOBS ASSOCIATES

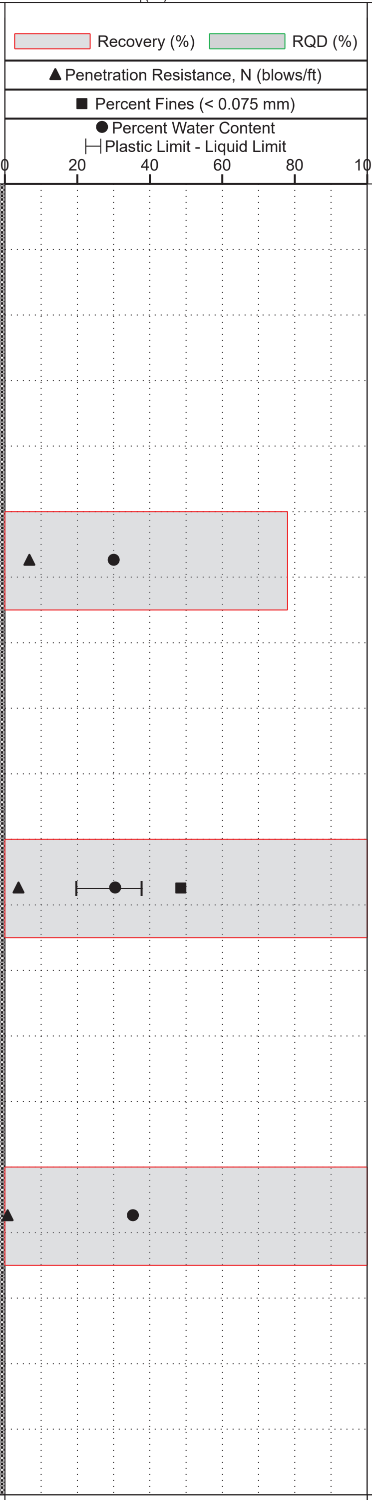
Boring B-02

Project: Washington Ave Bore Crossing City of Roseburg OR
Project Location: Roseburg, OR
Project Number: 6194.0

Log of Boring B-03

Date(s) Drilled	**Nov 04 2020**	Client	**City of Roseburg**	Logged By	**J. Quinn**	Checked By	**F. Sariosseiri**
Drilling Method/ Rig Type	**Mud Rotary/CME 75**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**30.1 ft.**		
Hole Diameter	**4.00 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**446.8 ft./NAVD88**		
Location	**Riverside Park**	Coordinates	**159822.90E,137704.30N**	Hammer Efficiency (%)			

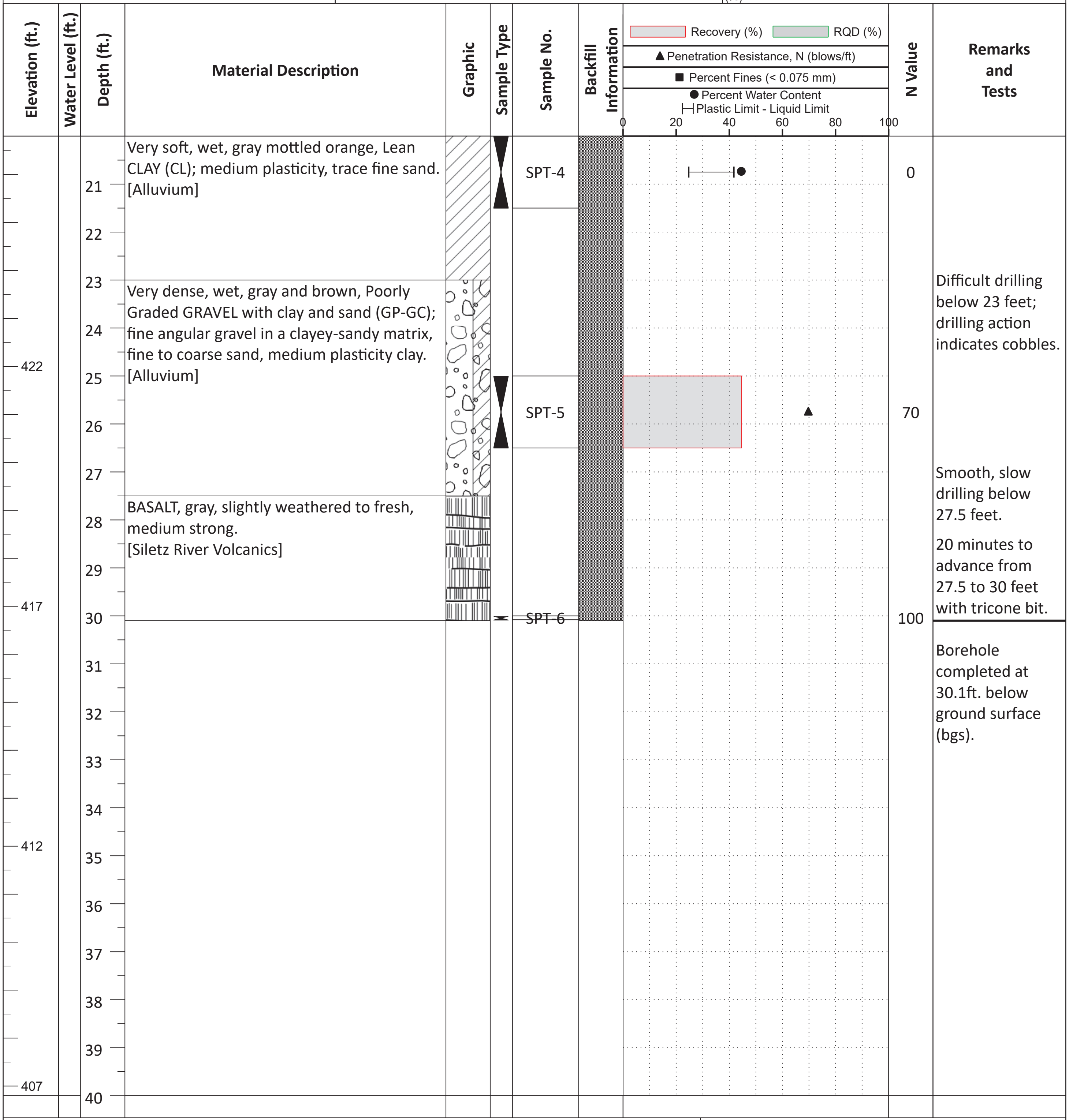
Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Graphic	Sample Type	Sample No.	Backfill Information	N Value	Remarks and Tests
		0	Moist, brown, Sandy SILT (ML); fine grained sand, low plasticity. [Alluvium]						
442		5	Medium stiff, moist, brown, Lean CLAY (CL) with sand; fine grained sand, medium plasticity. [Alluvium] *5.0 - 10.0 ft.: sand content increases with depth.*			SPT-1		7	
437		10	Very loose, moist, brown with occasional grey lenses, Clayey SAND (SC); fine grained sand, medium plasticity, occasional gray clay seams (<0.25" thick). [Alluvium]			SPT-2		4	S-2: 48.7% fines by ASTM D1140 Smooth drilling from ground surface to 23 feet.
432		15	*15.0 ft.: Becomes very soft, brown and gray mottled.*			SPT-3		1	
		18	Very soft, wet, gray mottled orange, Lean CLAY (CL); medium plasticity, trace fine sand. [Alluvium]						
427		20							



Project: Washington Ave Bore Crossing City of Roseburg OR
Project Location: Roseburg, OR
Project Number: 6194.0

Log of Boring B-03

Date(s) Drilled	**Nov 04 2020**	Client	**City of Roseburg**	Logged By	**J. Quinn**	Checked By	**F. Sariosseiri**
Drilling Method/ Rig Type	**Mud Rotary/CME 75**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**30.1 ft.**		
Hole Diameter	**4.00 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**446.8 ft./NAVD88**		
Location	**Riverside Park**	Coordinates	**159822.90E,137704.30N**	Hammer Efficiency (%)			



Depth (ft.)	Material Description	Sample No.	N Value	Remarks and Tests
20–23	Very soft, wet, gray mottled orange, Lean CLAY (CL); medium plasticity, trace fine sand. [Alluvium]	SPT-4	0	
23–27.5	Very dense, wet, gray and brown, Poorly Graded GRAVEL with clay and sand (GP-GC); fine angular gravel in a clayey-sandy matrix, fine to coarse sand, medium plasticity clay. [Alluvium]	SPT-5	70	Difficult drilling below 23 feet; drilling action indicates cobbles.
27.5–30.1	BASALT, gray, slightly weathered to fresh, medium strong. [Siletz River Volcanics]	SPT-6	100	Smooth, slow drilling below 27.5 feet. 20 minutes to advance from 27.5 to 30 feet with tricone bit.
				Borehole completed at 30.1ft. below ground surface (bgs).

McMILLEN JACOBS ASSOCIATES

Appendix B

Laboratory Testing

Breccia Geotechnical Testing, LLC.	**Natural Moisture Content (ASTM D2216)**

Client: McMillen Jacobs Associates
By: FS
Project Name: Washington Avenue HDD
Date: 11/16/2020
Project Number: 6194.0

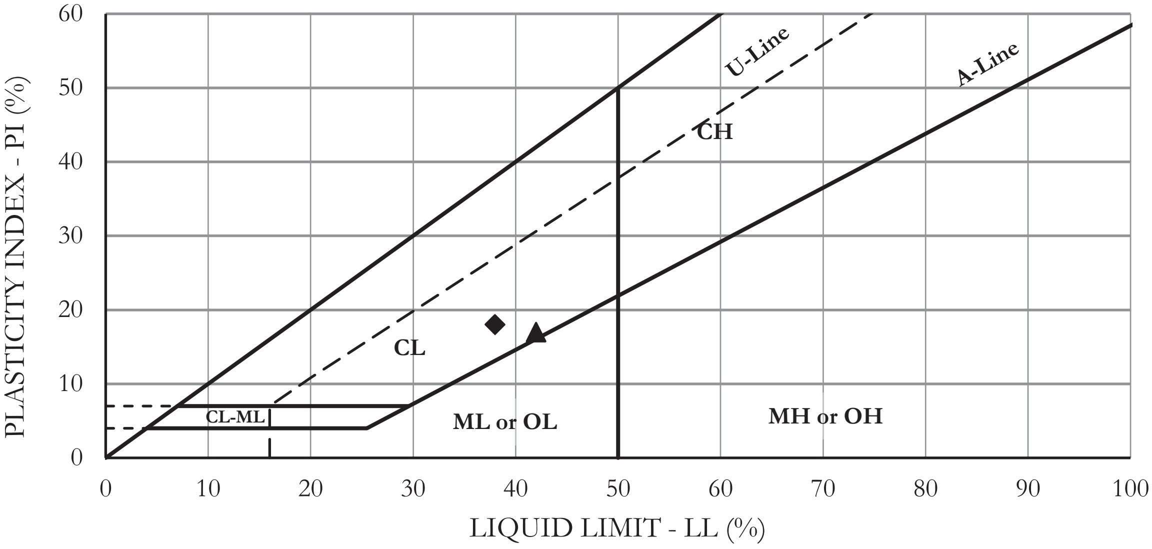
Exploration ID	B-1	B-3	B-3			
Samples ID	S-1	S-1	S-3			
Samples Depth (ft.)	5	5	15			
Moisture Content (%)	38.3	30.3	35.6			

Breccia Geotechnical Testing, LLC.	**Percent Fines (ASTM D1140)**

Client:	McMillen Jacobs Associates	By:	FS
Project Name:	Washington Avenue HDD	Date:	11/16/2020
Project Number:	6194.0		

Exploration ID	B-2	B-2	B-3			
Samples ID	S-1	S-2	S-2			
Samples Depth (ft.)	2.5	5	10			
Moisture Content (%)	25.5	41.6	30.7			
Percent Fines (%)	57.6	62.5	48.7			

ATTERBERG LIMITS TEST RESULTS (ASTM D4318)



	Boring	Sample ID	Depth (feet)	Moisture Content (%)	Atterberg Limits			%Pass #200	USCS
					LL	PL	PI		
◆	B-3	S-2	10	30.7	38	20	18	48.7	SC
▲	B-3	S-4	20	44.8	42	25	17	--	CL

Remarks

Project: Washington Avenue HDD
Project No.: 6194.0
Location: Roseburg, OR

Breccia Geotechnical Testing, LLC.
Brecciageolab@gmail.com
Tel: 971-246-1324

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:	Jeff Quinn, P.E. McMillen Jacobs Associates 1500 SW First Avenue, Suite 750 Portland, Oregon 97201	**Date:** **Lab No.:**	11/25/2020 20-334
Project:	Laboratory Services	**Project No.:**	3594.1.1

Report of: Unconfined compression, unit weight, and photos of rock

Sample Identification

As requested, NTI provided an unconfined compression testing, unit weight testing, and photos of rock cores on samples delivered to our laboratory by a McMillen Jacobs representative on November 11, 2020. Testing was performed in accordance with the standards indicated. Our laboratory test results are summarized on the following pages.

Attachments: Laboratory Test Results
Laboratory Photo Logs

Copies: (1) Addressee

This report shall not be reproduced except in full, without written approval of Northwest Testing, Inc.

REVIEWED BY: Michael A. Ginsbach

TECHNICAL REPORT

Report To: Jeff Quinn, P.E.
McMillen Jacobs Associates
1500 SW First Avenue, Suite 750
Portland, Oregon 97201

Date: 11/25/2020

Lab No.: 20-334

Project: Laboratory Services

Project No.: 3594.1.1

Laboratory Testing

Compressive Strength and Unit Weight of Intact Rock Core Specimens (ASTM D7012 Method C, ASTM D4543)

Sample ID	Diameter (inches)	Height (inches)	Approximate Rate of Loading (lbs/s)	Uniaxial Compressive Strength (psi)	Unit Weight (pcf)
B-1 @ 21.7 – 23.0 Ft.	2.38	4.79	190	6972	175.2
B-1 @ 36.5 – 37.5 Ft.	2.38	4.74	200	3683	173.6
B-1 @ 57.3 – 58.2 Ft.	2.38	4.71	190	4257	161.7
B-1 @ 62.1 – 62.9 Ft.	2.38	4.79	190	3479	166.8

Compressive Strength and Unit Weight of Intact Rock Core Specimens (ASTM D7012 Method C, ASTM D4543)

Sample ID	Diameter (inches)	Height (inches)	Approximate Rate of Loading (lbs/s)	Uniaxial Compressive Strength (psi)	Unit Weight (pcf)
B-2 @ 16.0 – 16.7 Ft.	2.38	4.77	200	8024	179.2
B-2 @ 30.0 – 31.2 Ft.	2.38	4.78	200	3174	170.9
B-2 @ 49.1 – 50.0 Ft.	2.38	4.69	180	4460	162.7
B-2 @ 53.7 – 55.0 Ft.	2.38	4.72	180	4653	168.0

TECHNICAL REPORT

Report To: Jeff Quinn, P.E.
McMillen Jacobs Associates
1500 SW First Avenue, Suite 750
Portland, Oregon 97201

Date: 11/25/2020

Lab No.: 20-334

Project: Laboratory Services

Project No.: 3594.1.1

Laboratory Photo Log

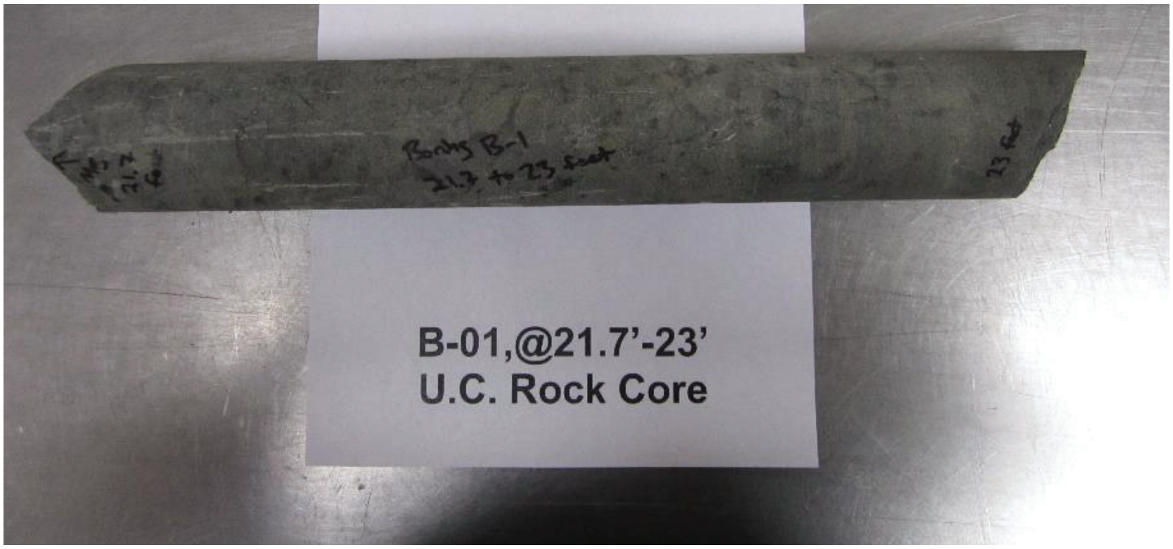


Figure 1. B-1 @ 21.7 – 23.0 Ft. core as received.

Figure 2. B-1 @ 21.7 – 23.0 Ft. core after unconfined strength testing.

TECHNICAL REPORT

Report To:	Jeff Quinn, P.E. McMillen Jacobs Associates 1500 SW First Avenue, Suite 750 Portland, Oregon 97201	**Date:** **Lab No.:**	11/25/2020 20-334
Project:	Laboratory Services	**Project No.:**	3594.1.1

Laboratory Photo Log

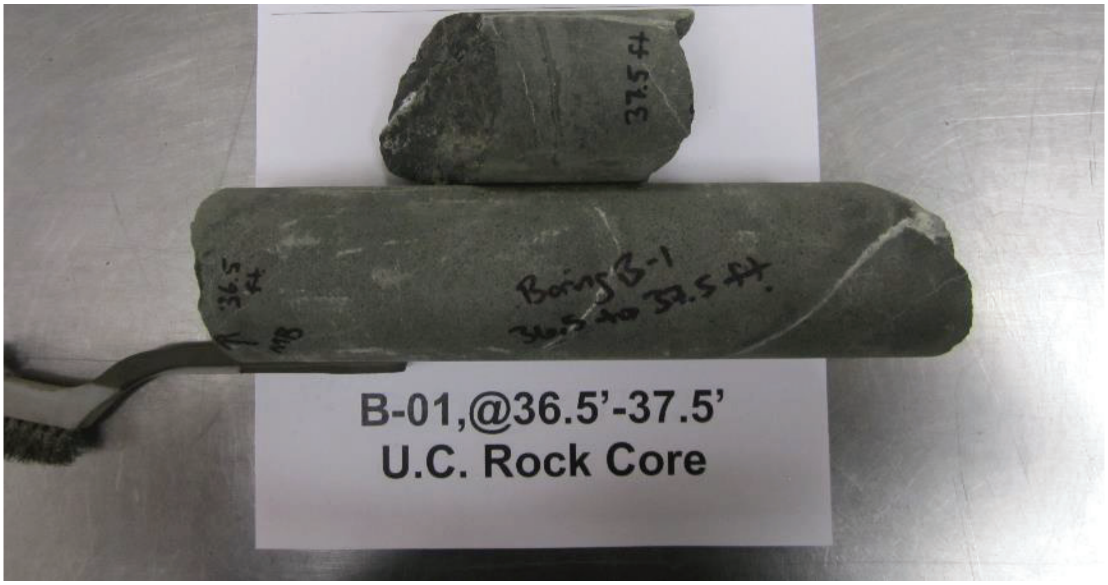


Figure 3. B-1 @ 36.5 – 37.5 Ft. core as received.

Figure 4. B-1 @ 36.5 – 37.5 Ft. core after unconfined strength testing.

TECHNICAL REPORT

Report To: Jeff Quinn, P.E.
McMillen Jacobs Associates
1500 SW First Avenue, Suite 750
Portland, Oregon 97201

Date: 11/25/2020

Lab No.: 20-334

Project: Laboratory Services

Project No.: 3594.1.1

Laboratory Photo Log

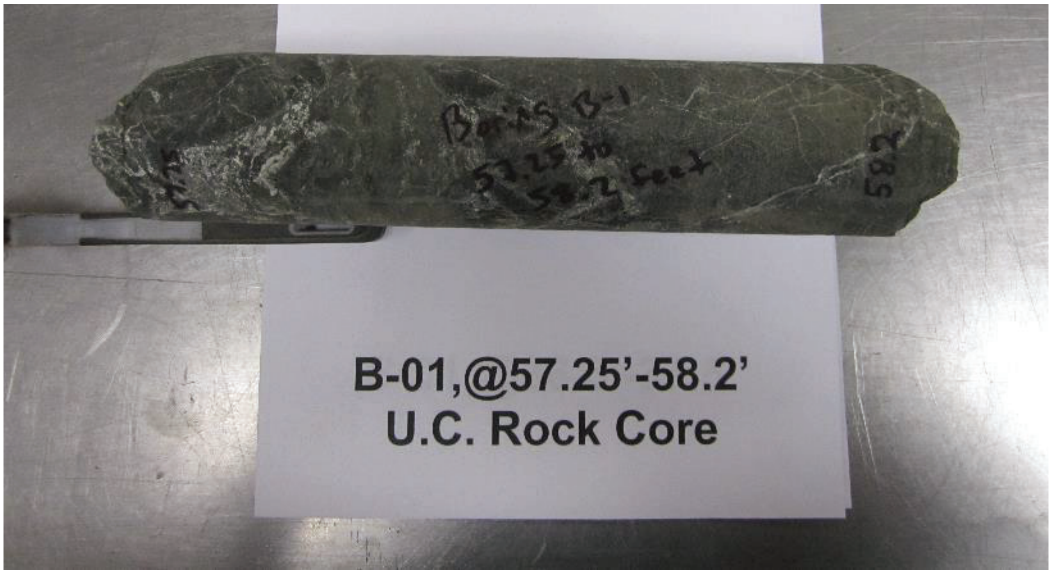


Figure 5. B-1 @ 57.3 – 58.2 Ft. core as received.

Figure 6. B-1 @ 57.3 – 58.2 Ft. core after unconfined strength testing.

TECHNICAL REPORT

Report To:	Jeff Quinn, P.E. McMillen Jacobs Associates 1500 SW First Avenue, Suite 750 Portland, Oregon 97201	**Date:** **Lab No.:**	11/25/2020 20-334
Project:	Laboratory Services	**Project No.:**	3594.1.1

Laboratory Photo Log

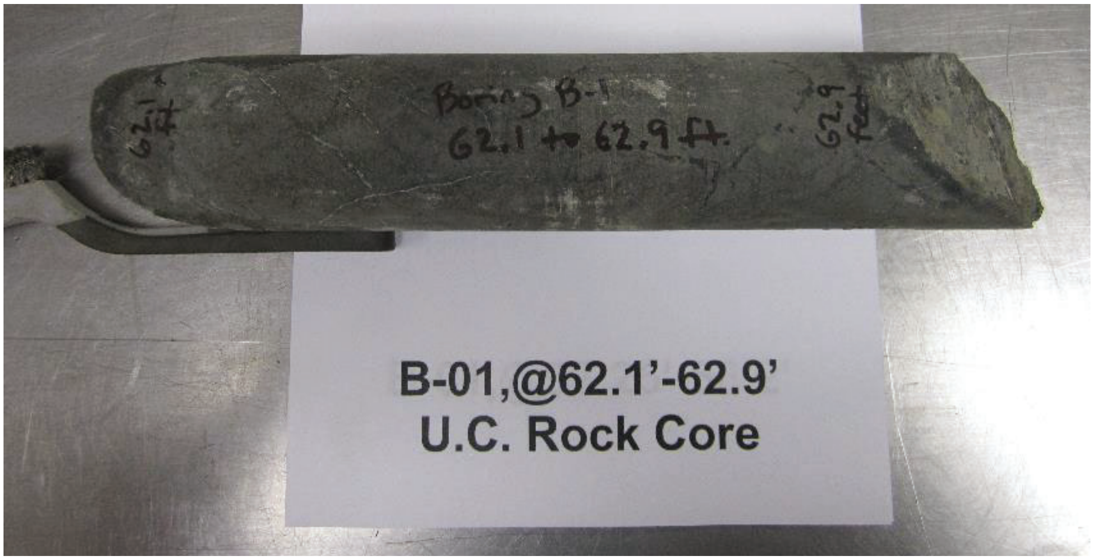


Figure 7. B-1 @ 62.1 – 62.9 Ft. core as received.

Figure 8. B-1 @ 62.1 – 62.9 Ft. core after unconfined strength testing.

TECHNICAL REPORT

Report To:	Jeff Quinn, P.E. McMillen Jacobs Associates 1500 SW First Avenue, Suite 750 Portland, Oregon 97201	**Date:** **Lab No.:**	11/25/2020 20-334
Project:	Laboratory Services	**Project No.:**	3594.1.1

Laboratory Photo Log

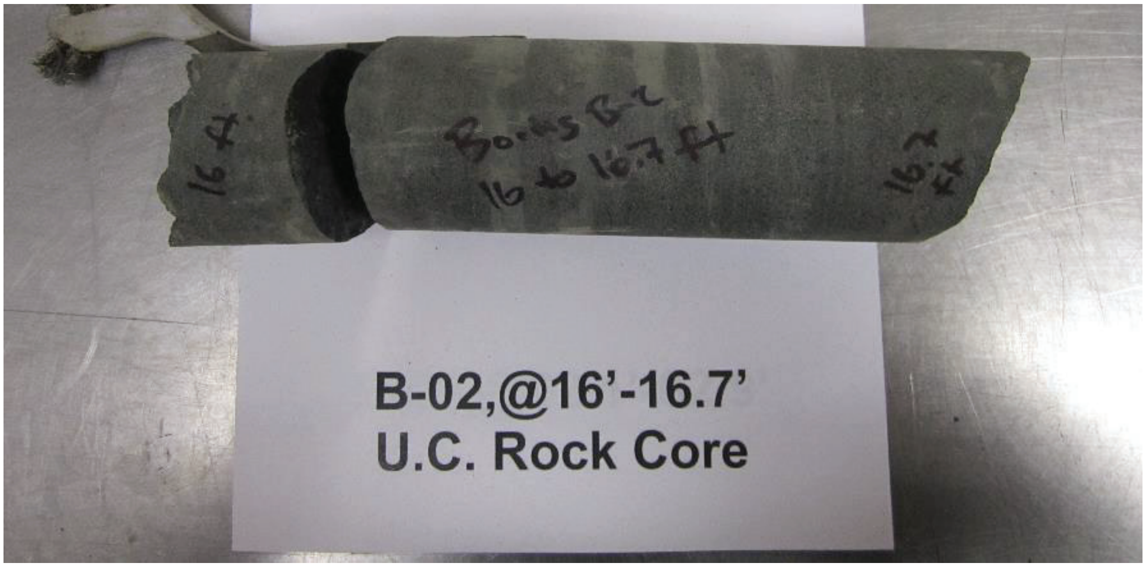


Figure 9. B-2 @ 16.0 – 16.7 Ft. core as received.

Figure 10. B-2 @ 16.0 – 16.7 Ft. core after unconfined strength testing.

TECHNICAL REPORT

Report To:	Jeff Quinn, P.E. McMillen Jacobs Associates 1500 SW First Avenue, Suite 750 Portland, Oregon 97201	**Date:** **Lab No.:**	11/25/2020 20-334
Project:	Laboratory Services	**Project No.:**	3594.1.1

Laboratory Photo Log

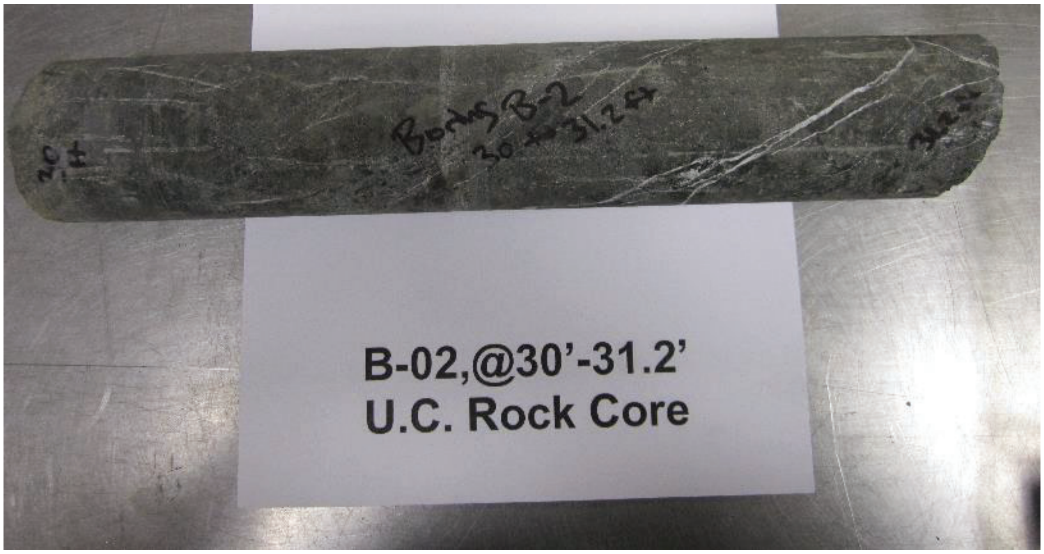


Figure 11. B-2 @ 30.0 – 31.2 Ft. core as received.

Figure 12. B-2 @ 30.0 – 31.2 Ft. core after unconfined strength testing.

TECHNICAL REPORT

Report To:	Jeff Quinn, P.E. McMillen Jacobs Associates 1500 SW First Avenue, Suite 750 Portland, Oregon 97201	**Date:** **Lab No.:**	11/25/2020 20-334
Project:	Laboratory Services	**Project No.:**	3594.1.1

Laboratory Photo Log

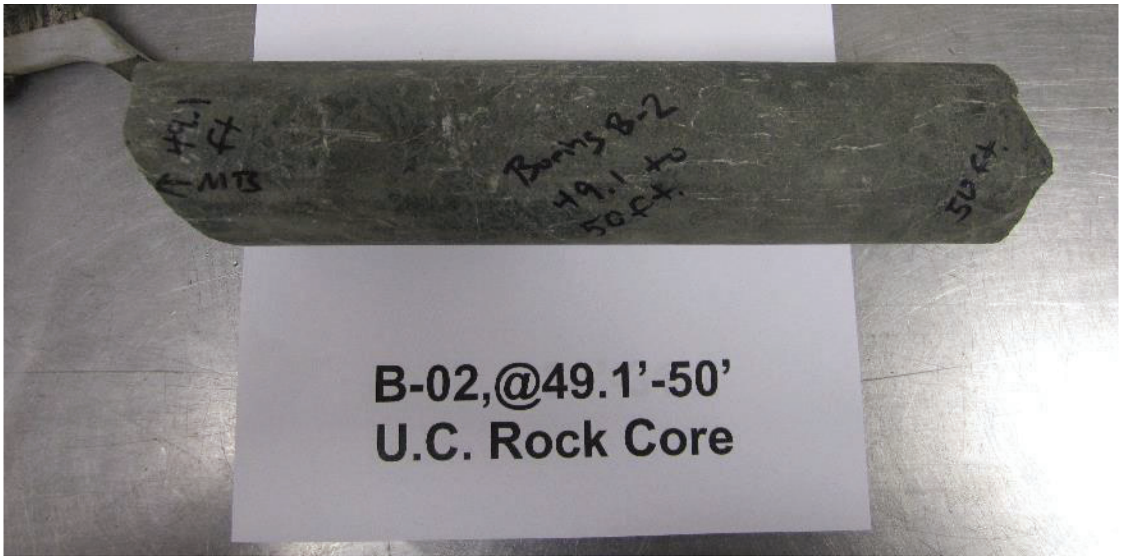


Figure 13. B-2 @ 49.1 – 50.0 Ft. core as received.

Figure 14. B-2 @ 49.1 – 50.0 Ft. core after unconfined strength testing.

TECHNICAL REPORT

Report To:	Jeff Quinn, P.E. McMillen Jacobs Associates 1500 SW First Avenue, Suite 750 Portland, Oregon 97201	**Date:** **Lab No.:**	11/25/2020 20-334
Project:	Laboratory Services	**Project No.:**	3594.1.1

Laboratory Photo Log

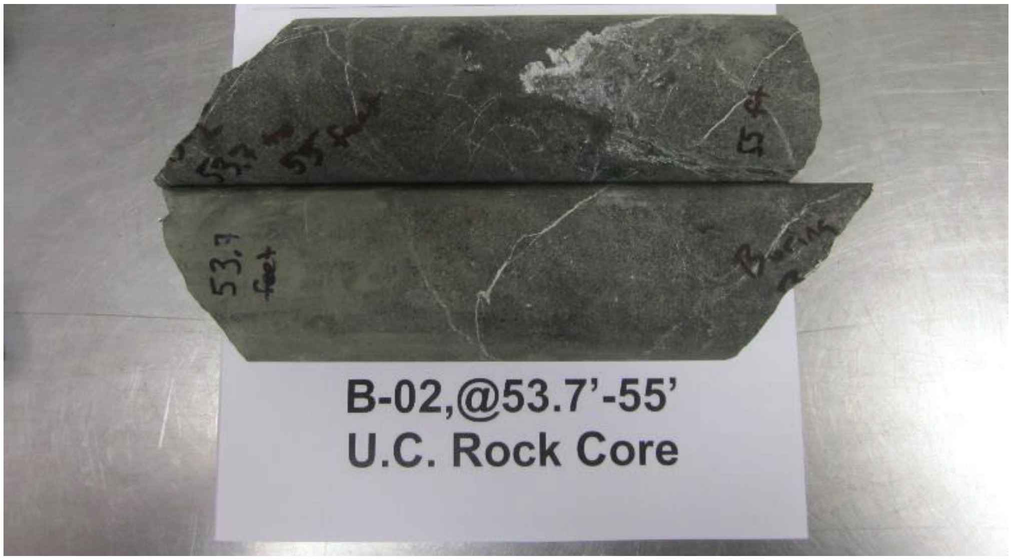
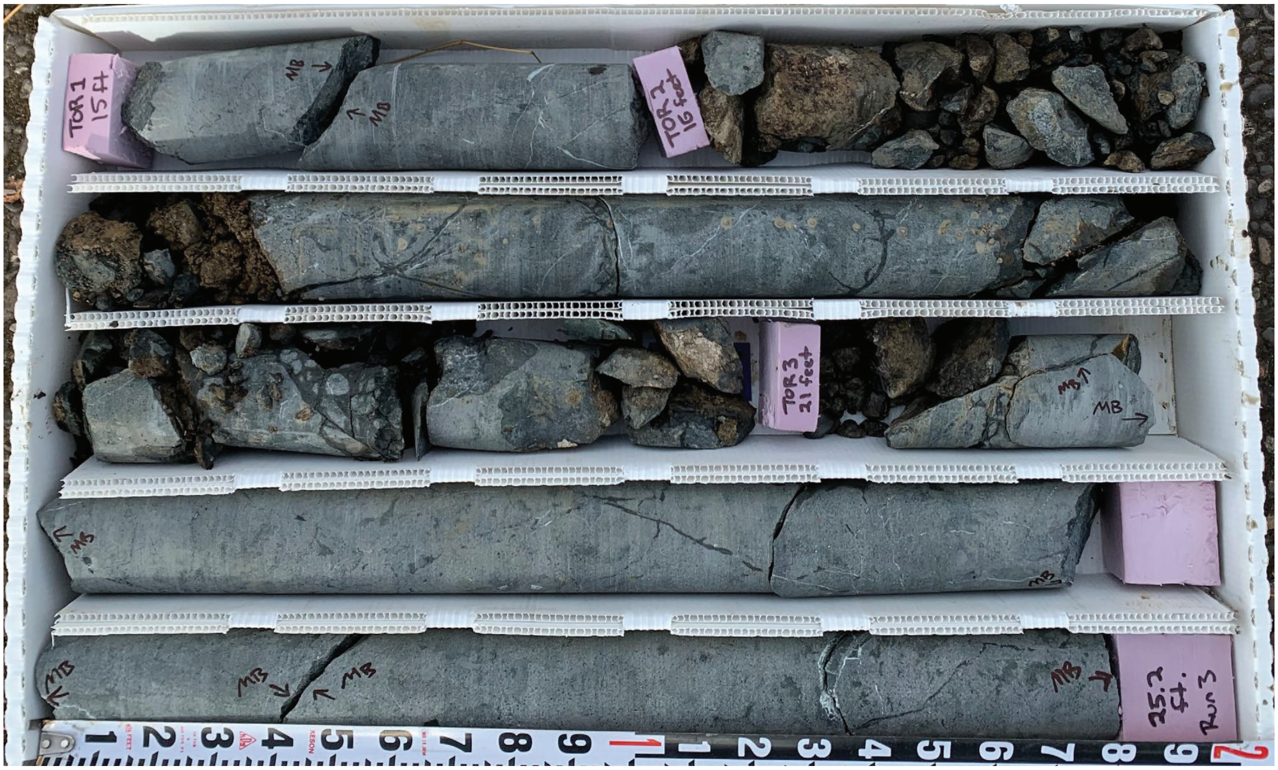


Figure 15. B-2 @ 53.7 – 55.0 Ft. core as received.

Figure 16. B-2 @ 53.7 – 55.0 Ft. core after unconfined strength testing.

Appendix C

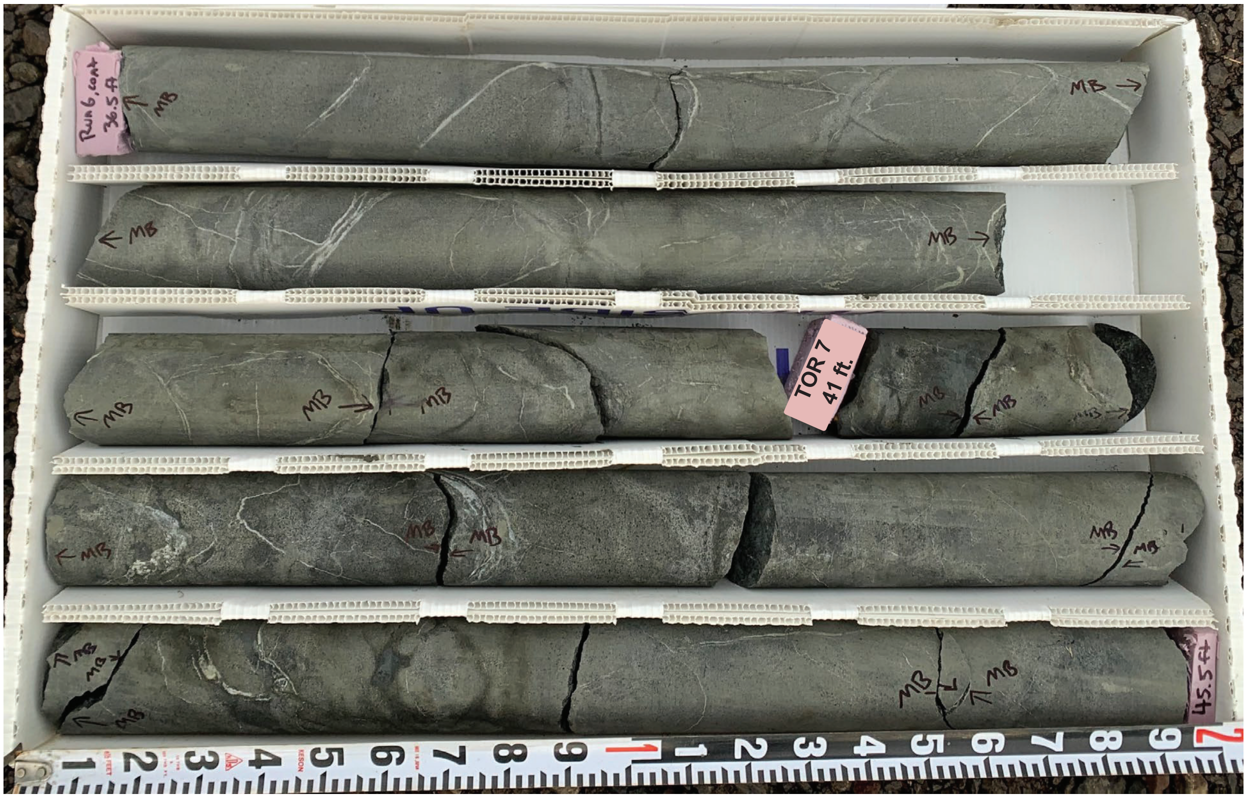
Rock Core Photographs



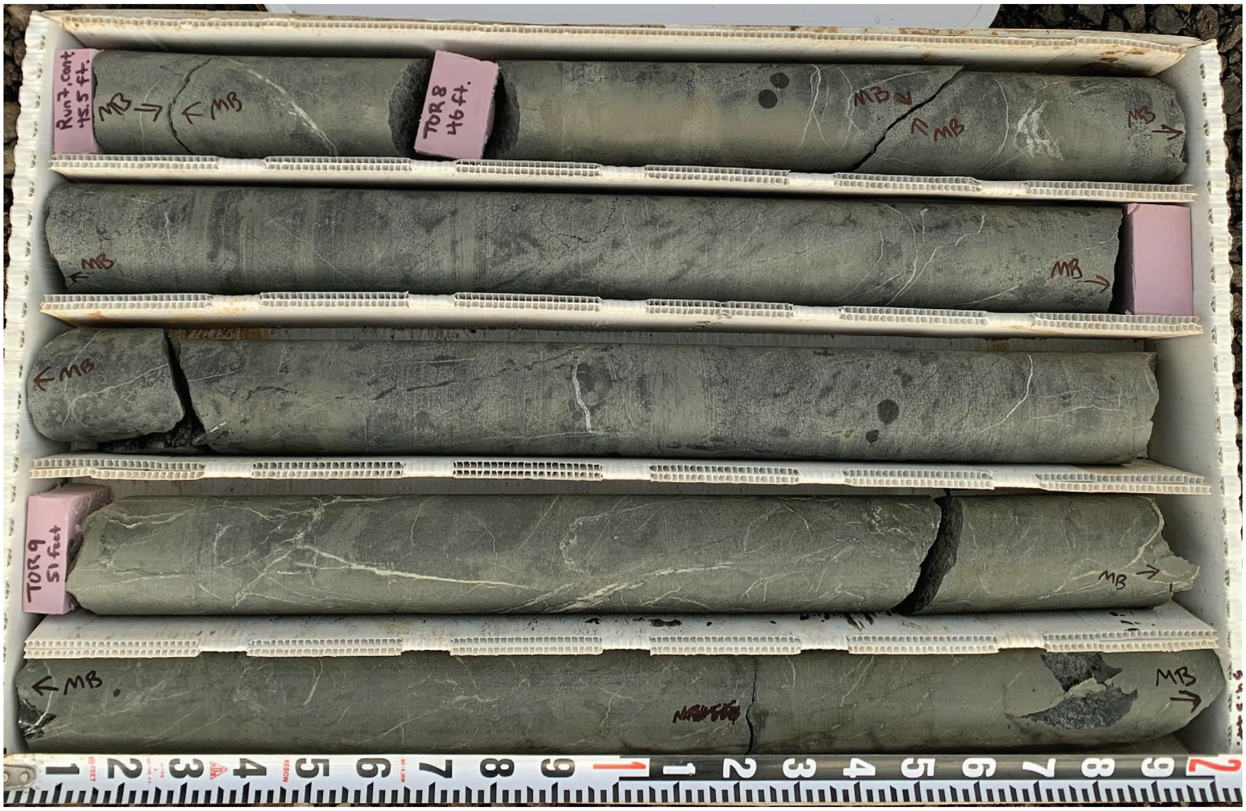
BOREHOLE: B-1, 15 TO 25.2 FEET

BOREHOLE: B-1, 25.2 TO 36.5 FEET

McMILLEN JACOBS ASSOCIATES	WASHINGTON AVE BORE CROSSING PROJECT	DECEMBER 2020
	ROCK CORE PHOTOGRAPHS	FIGURE C-1

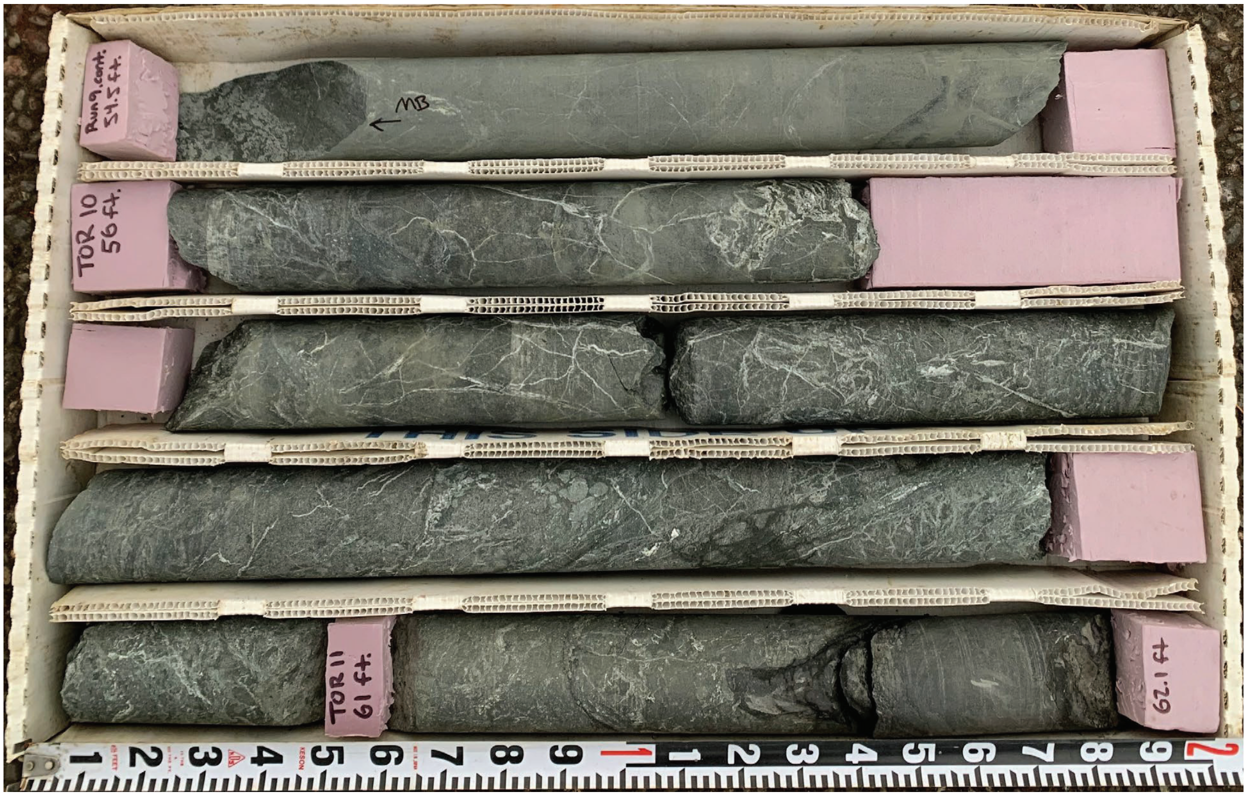


BOREHOLE: B-1, 36.5 TO 45.5 FEET

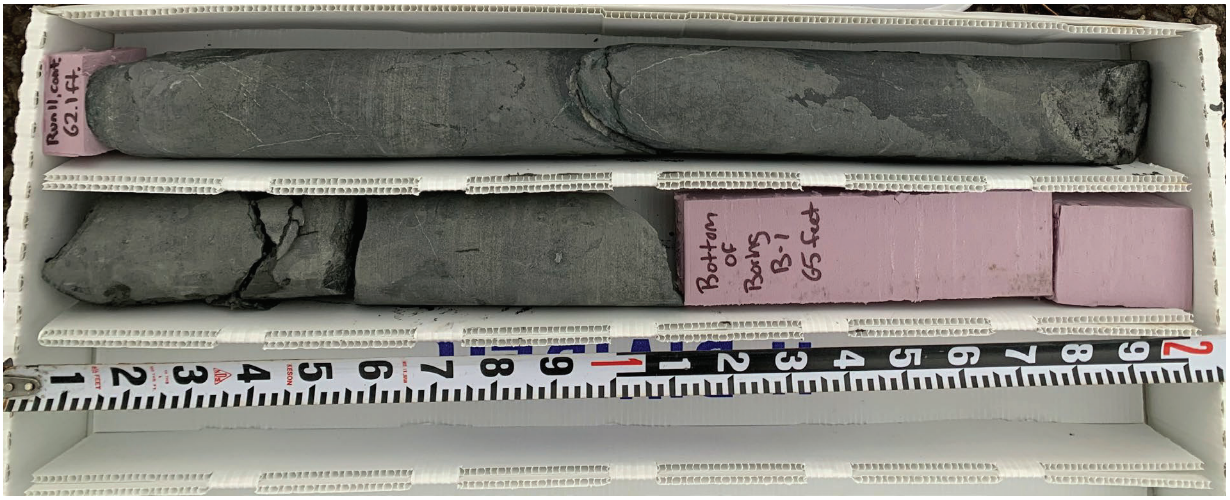


BOREHOLE: B-1, 45.5 TO 54.5 FEET

McMILLEN JACOBS ASSOCIATES	WASHINGTON AVE BORE CROSSING PROJECT	DECEMBER 2020
	ROCK CORE PHOTOGRAPHS	FIGURE C-2



BOREHOLE: B-1, 54.5 TO 62.1 FEET



BOREHOLE: B-1, 62.1 TO 65 FEET

McMILLEN JACOBS ASSOCIATES

WASHINGTON AVE BORE CROSSING PROJECT	DECEMBER 2020
ROCK CORE PHOTOGRAPHS	FIGURE C-3

BOREHOLE: B-2, 9.5 TO 18.2 FEET

BOREHOLE: B-2, 18.2 TO 27.8 FEET

McMILLEN JACOBS ASSOCIATES

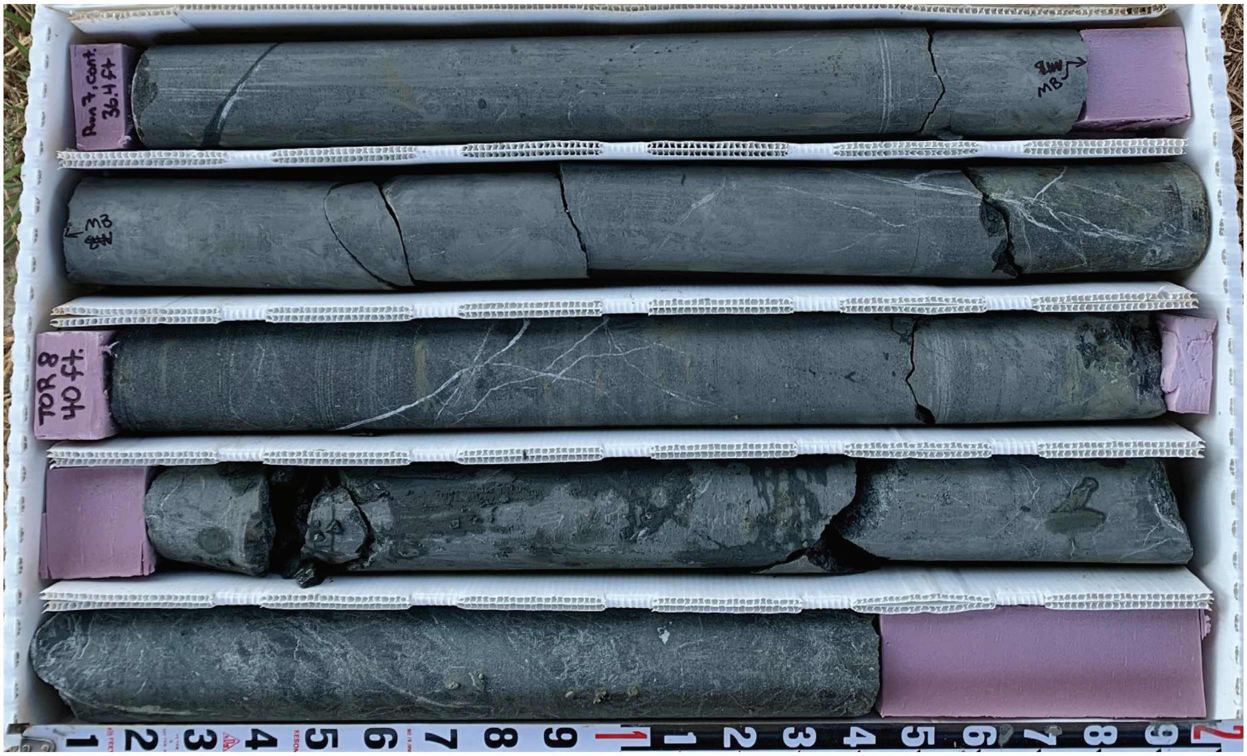
WASHINGTON AVE BORE CROSSING PROJECT

DECEMBER 2020

ROCK CORE PHOTOGRAPHS

FIGURE C-4

BOREHOLE: B-2, 27.8 TO 36.4 FEET



BOREHOLE: B-2, 36.4 TO 45.0 FEET

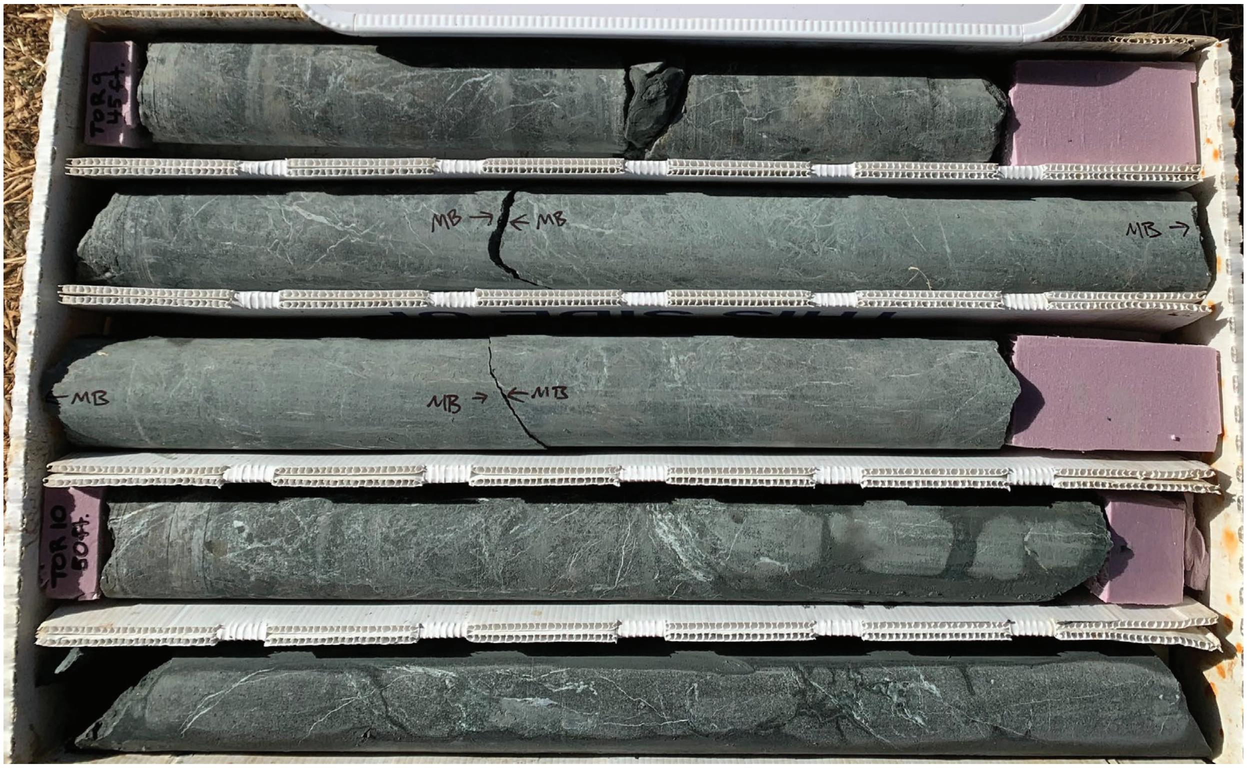
McMILLEN JACOBS ASSOCIATES

WASHINGTON AVE BORE CROSSING PROJECT

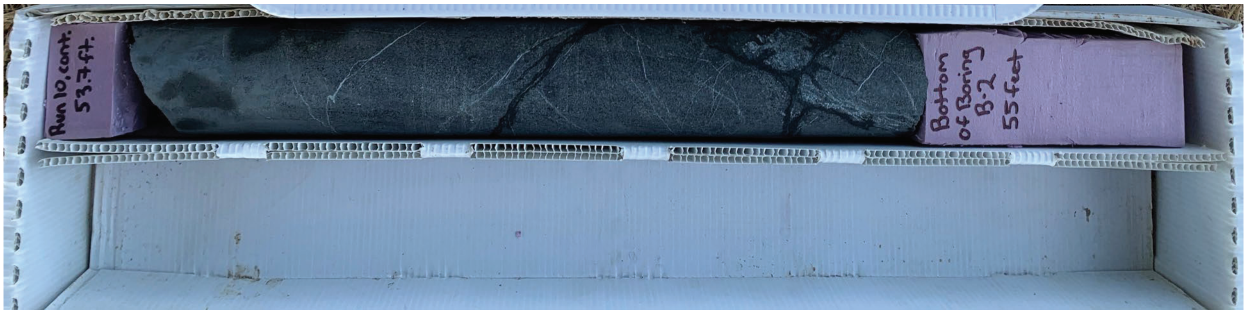
DECEMBER 2020

ROCK CORE PHOTOGRAPHS

FIGURE C-5



BOREHOLE: B-2, 45.0 TO 53.7 FEET



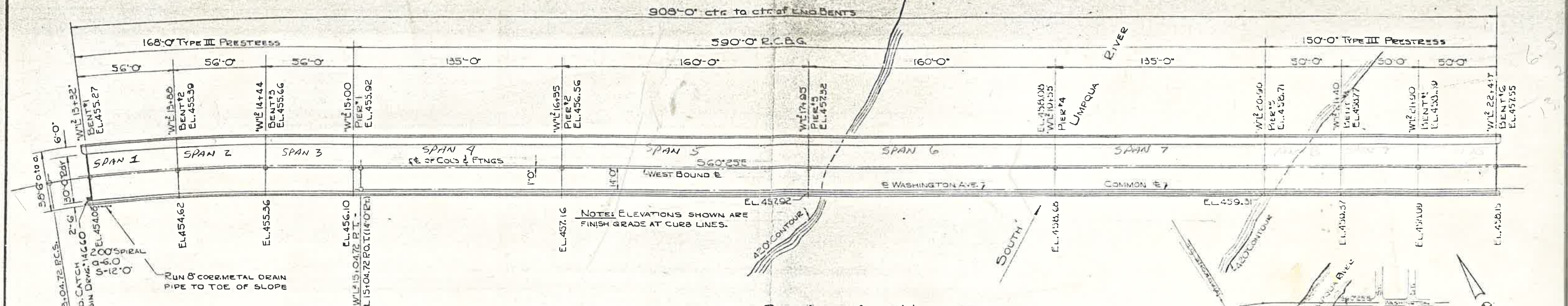
BOREHOLE: B-2, 53.7 TO 55.0 FEET

McMILLEN JACOBS ASSOCIATES	WASHINGTON AVE BORE CROSSING PROJECT	DECEMBER 2020
	ROCK CORE PHOTOGRAPHS	FIGURE C-6

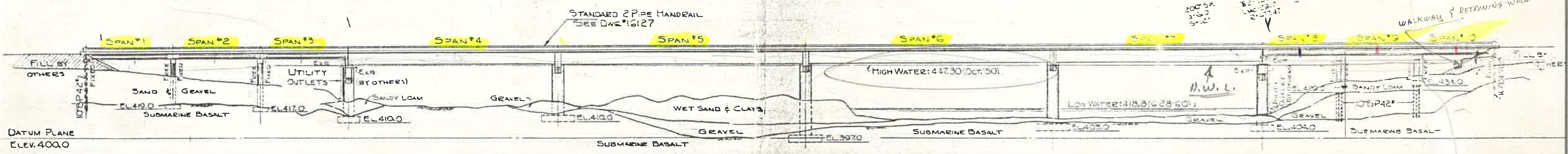
Appendix D

Geologic Profile for Washington Ave. Bridge

(Oregon State Highway Department, 1961)



P L A N



E L E V A T I O N

Scale: 1" = 30'-0"

GENERAL NOTES

Bridge designed for H20-S16-44

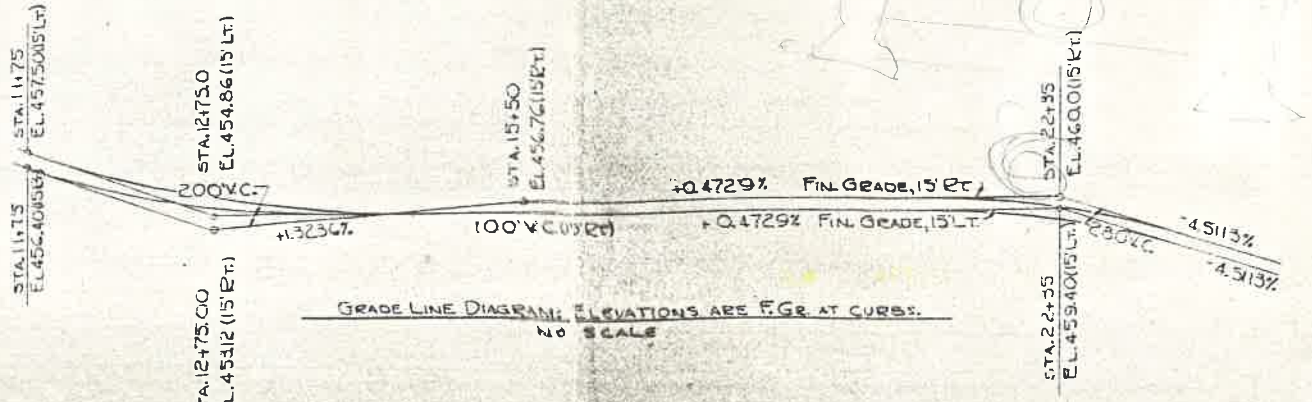
All concrete shall be class "A" and have a breaking strength of 3300 p.s.i. in 28 days (fc 1320 p.s.i.)

All reinforcing steel shall be intermediate grade deformed bars. Bars from #3 thru #11, inclusive, shall conform to A.S.T.M. specification A-305 and shall be lapped 20 diameters at all splices unless noted or shown otherwise. Bars #14 & #18 shall conform to A.S.T.M. specification A-408 and shall be lapped 35 diameters at all splices unless noted or shown otherwise. All bars shall be placed 2" clear of nearest face of concrete unless noted or shown otherwise. (fs-20,000 p.s.i.)

See Dwg. #16742 & 43 for concrete and reinforcing steel in Pre-Stressed Beams.

Footing elevations subject to change depending on foundation material encountered. Reinforcing steel for columns shall not be fabricated untill final footing elevations are determined in the field.

All workmanship and materials shall conform to the Specifications for Bridges of the Oregon State Highway Commission.



Grade Line Diagram: Elevations are F.Gr. at curbs.
No Scale

G. S. Paxson
Mar. 9, 1961

File

NOTE
The scale of this print is ½ that of the original drwg.
For example
Indicated scale ¾" = 1'-0"
should be read ⅜" = 1'-0"
Indicated scale 1" = 10'-0"
should be read 1" = 20'-0"

APPROVED:
Ivan D. Merchant
BRIDGE ENGINEER
G S Paxson
ASST. STATE HIGHWAY ENGINEER
STATE HIGHWAY ENGINEER
DESIGNED L.R. CHECKED
DRAWN J.B.L. CALC. BOOK 581

OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION
WASHINGTON AVE. BRIDGE
OVER SOUTH UMPQUA RIVER
CITY OF ROSEBURG
~~DOUGLAS COUNTY~~
PLAN & ELEVATION

DATE 4-11-61 BRIDGE NO. 8839
ACCOMPANIED BY DWGS. 16736 THRU 16748, 14660 & 1627
SHEET 1 OF 14
DRAWING NO. 16735