Exhibit L: Geotechnical Report

FOR LAND USE PERMITTING (EXHIBIT B)

Willamette Water Supply
Our Reliable Water

Willamette Water Supply Program

WTP_1.0

Geotechnical Engineering Report

FINAL

Prepared For:

CDM Smith

May 12, 2020

McMILLEN JACOBS ASSOCIATES

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Acronyms and Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASTM	American Society of Testing and Materials
bgs	below ground surface
CLSM	controlled low strength material
CM/GC	Construction Manager/General Contractor
cm	centimeter
CRB	Columbia River Basalt
CSZ	Cascadia subduction zone
E’	modulus of soil reaction
FEMA	Federal Emergency Management Agency
Ft./sec	feet per second
GDR	Geotechnical Data Report
GER	Geotechnical Engineering Report
GHR	Geologic Hazards Report
IBC	International Building Code
kg	kilogram
Ma	millions of years before present
mg	milligram
mgd	million gallons per day
McMillen Jacobs	McMillen Jacobs Associates
mV	millivolts
ND	non detect
NGVD29	National Geodetic Vertical Datum of 1929
OAR	Oregon Administrative Rules
ODOT	Oregon Department of Transportation
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction
pcf	pounds per cubic foot
pci	pounds per cubic inch
psf	pounds per square feet
PGA	Peak Ground Acceleration
PGE	Portland General Electric
PGV	peak ground velocity
Project	WTP_1.0 Project
PSHA	probabilistic seismic hazard analysis
psf	pounds per square foot
psi	pounds per square inch
PVC	polyvinylchloride
RMR	Rock Mass Rating
RQD	Rock Quality Designation
SA	spectral acceleration
SEI	Structural Engineering Institute
S_{M1}	SA 1.0-sec period spectral acceleration
S_{MS}	SA 0.2-sec period spectral acceleration

Sta.	station
TVWD	Tualatin Valley Water District
UCS	unconfined compressive strength
WWSP	Willamette Water Supply Program
Ω	ohm

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Revision Log

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Final Submittal	May 12, 2020	Incorporated reviewers' comments

1.0 Introduction

1.1 General

McMillen Jacobs Associates (McMillen Jacobs) has been retained by CDM Smith to provide geotechnical services for the Willamette Water Supply Program WTP_1.0 Project. Tualatin Valley Water District (TVWD) and the Cities of Hillsboro and Beaverton are the project owners. The project is located in Sherwood, Washington County, Oregon. The project location is shown in Figure 1 and the site layout is shown in Figure 2. This Geotechnical Engineering Report (GER) summarizes the results of the explorations, geotechnical analyses, and construction recommendations for the WTP_1.0 Project.

1.2 Project Description

The Willamette Water Supply Program (WWSP) is a drinking water infrastructure program being implemented by the TVWD and the Cities of Hillsboro and Beaverton to provide a seismically resilient water supply for their service area. The WWSP includes more than 30 miles of transmission pipelines, ranging from 24 to 66 inches in diameter, extending from the Willamette River in Wilsonville to the TVWD and Hillsboro service areas in Washington County, which includes the cities of Hillsboro and Beaverton. In addition to the new WTP_1.0, the WWSP also includes two 15-million-gallon water storage tanks, and a raw water pumping station. The new system elements are being designed to meet future demand and to provide redundancy in the event of an emergency.

The WWSP has been divided into multiple design packages and work is proceeding with a phased approach. The WTP_1.0 is a new water treatment plant with an initial treated water design capacity of 60 million gallons per day (mgd) and future build-out treated water design capacity of 120 mgd. The project is located within the City of Sherwood, Oregon. The primary structures of WTP_1.0 include facilities for filtration, ozone contactors, UV reactors, ballasted flocculation, solids dewatering and transfer, a clearwell, gravity thickeners, and equalization/overflow basins. The WTP_1.0 elements are shown in Figure 2.

The project includes construction of several access roads within the water treatment plant as well as construction of a portion of SW Blake Street that extends from SW 124th Avenue to the plant's western property line. It is noted that preliminary plans show SW Blake Street to dead-end at the property's west border. Future plans to extend the roadway are not available at this time.

WTP_1.0 will need to be connected to the existing infrastructure including the raw water pipeline, treated water pipeline, sewer, and storm drain in SW Tualatin-Sherwood Road and SW 124th Avenue. Connection to the existing infrastructure will require coordination with PLM_4.0, the City of Sherwood, Washington County, and the developer of the parcel north of WTP_1.0 site.

The owners have selected a Construction Manager/General Contractor (CM/GC) project delivery method. The CM/GC contractor will be involved throughout the design from the preliminary design to the final detailed design phase. The construction is anticipated to begin in early 2022.

Construction of SW Blake Street, access roads, and most of the plant structures will require excavations on the order of 5 to 10 feet. Construction of the clearwell structure will require an excavation of up to 30 feet in depth.

1.3 Purpose and Scope of Work

The purpose of our work is to evaluate the subsurface conditions within the footprint of the proposed development and to provide geotechnical engineering design and construction recommendations for subsequent use by the design team to develop construction documents. Specifically, the scope of our work includes the following:

- Characterization of subsurface conditions within the proposed treatment plant based on geotechnical explorations and laboratory testing;
- Geotechnical engineering assessments and design recommendations for foundations;
- Provide lateral earth pressure for retaining walls and embedded structures;
- Recommendations for bedding, backfill, and compaction criteria for foundations, piping, and retaining walls;
- Seismic hazard evaluation results and seismic geotechnical recommendations for the design of WTP_1.0 structures and piping;
- Identification of feasible excavation methods, temporary cut supports, and applicable lateral earth pressures;
- Cut-and-fill slope requirements;
- Recommendations for design and construction of cut slopes for SW Blake Street;
- Recommendations for design and construction of asphalt pavement for SW Blake Street;
- Recommendations for groundwater control methods; and
- Preparation of this Geotechnical Engineering Report.

1.4 Project Geotechnical Reports

This Geotechnical Engineering Report has been developed for the use of the design team and summarizes geotechnical analyses and recommendations in support of the Project design. Other related geotechnical documents have been developed for this project and are referred to in this report. These documents are as follows:

- *Willamette Water Supply Program – WTP_1.0 Geotechnical Data Report (McMillen Jacobs, 2019a)*
- *Willamette Water Supply Program – WTP_1.0 Fault Location Study Technical Memorandum (McMillen Jacobs, 2019b)*
- *Willamette Water Supply Program –WTP_1.0 Geologic Hazards Report* (*McMillen Jacobs, 2020a)*
- *Willamette Water Supply Program – WTP_1.0 Site-Specific Seismic Response Spectrum for WTP_1.0 (McMillen Jacobs, 2020b)*
- *Geotechnical Report – Probabilistic Seismic Hazard Analysis, Willamette Water Supply Program, Clackamas and Washington County, Oregon. (Shannon & Wilson, 2017)*

The Fault Location Study Technical Memorandum, Geologic Hazards Report (GHR), and Site-Specific Seismic Response Spectrum for WTP_1.0 are included in Appendices F, G, and H, respectively.

1.5 Authorization

The Tualatin Valley Water District authorized the WTP_1.0 project work on behalf of the Owners, under the terms and conditions of an agreement between the Owners and CDM Smith dated July 24, 2018. McMillen Jacobs has been retained by CDM Smith to provide geotechnical design services for, or in connection with, the Willamette Water Supply Program WTP_1.0 Project (Project) per their Subconsultant Agreement dated August 17, 2018.

2.0 Site Explorations

2.1 General

The subsurface exploration program in support of the WTP_1.0 project was performed between December 3, 2018 and February 15, 2019. A total of 12 exploratory borings, 15 air-track probe holes, 6 test pits, and a geophysical study consisting of three seismic refraction survey lines were completed within the limits of the proposed treatment plant. Details of the field investigation and laboratory testing program are included in the Geotechnical Data Report (GDR) for WTP_1.0 (McMillen Jacobs, 2019a). The approximate locations of the explorations are shown in Figures 2 and 3. The geotechnical exploration logs, geophysical survey results, and laboratory testing results are included in Appendices A, B, and C, respectively. The field exploration and laboratory testing programs are summarized in the following sections.

2.2 Field Explorations

2.2.1 Exploratory Borings

A total of 12 borings designated WTP_1.0-B-01 through WTP_1.0-B-12 were advanced for the project. Western States Soil Conservation, Inc. of Hubbard, Oregon, completed the borings using a track-mounted CME-850 drill rig. The depths of the borings ranged between 15 feet and 45 feet below ground surface (bgs). Borings were advanced using mud-rotary drilling techniques within soil material, and HQ-triple-tube wireline coring in rock material.

2.2.2 Air-Track Probe Holes

Fifteen air-track probe holes were advanced to approximately 35 feet bgs across the site. The probe holes were advanced using a track-mounted Furukawa HCR 900 ES drill, owned and operated by McCallum Rock Drilling of Chehalis, Washington. Probe holes are designated as WTP_1.0-P-01 through WTP_1.0-P-15 in this report.

The drill is powered by hydraulics which rotate the drill rods and provide down-pressure on the impact bit. Cuttings are blown from the borehole by compressed air injected down-hole through the drill rods. Depth to bedrock can be estimated by the rate of down-hole advancement of the bit, drill reaction, and by observation of the drill cuttings. Subsurface sampling is not a part of this exploratory method.

2.2.3 Test Pits

Six test pits were excavated using a Hitachi 210LC excavator owned and operated by Richter Logging Company of Forest Grove, Oregon. The test pits were excavated to identify the depth to bedrock and to evaluate the effort needed to excavate the rock. The test pits were terminated at depths where practical equipment refusal was encountered. The test pits were advanced to a maximum depth of 4.5 feet bgs. Test pit excavations are designated as WTP_1.0-TP-01 through WTP_1.0-TP-06 in this report.

2.2.4 Geophysical Explorations

Geophysical explorations were completed on February 15, 2019, by Siemens and Associates of Bend, Oregon. The testing included three seismic refraction tests and three shear wave micrometer (ReMi) profiles. Location of the geophysical lines are shown in Figure 3. Details of the geophysical exploration procedures and results are included in Appendix B.

2.2.5 Piezometers

One-inch diameter “open-tube” polyvinylchloride (PVC) piezometers were installed in 5 borings to allow long-term measurements of groundwater levels. Key piezometer construction details are provided in Table 2-1. The results of groundwater level measurements are presented in Section 3.5 of this report.

Table 2-1. Piezometer Construction Detail

Boring ID WTP_1.0-	Well Tip (feet)	Approximate Screen Interval (feet)
B-01	45.0	44.8 – 29.8
B-03	19.5	20 – 10
B-08	30.0	20 – 10
B-10	39.3	39.1 – 24.8
B-11	34.7	35 – 20

Piezometers were constructed in accordance with requirements of the Oregon Water Resources Department (OAR 690-240 Construction, Maintenance, Alteration Conversion and Abandonment of Monitoring Wells, Geotechnical Holes and Other Holes in Oregon).

2.3 Laboratory Testing

Soil and rock samples obtained from the exploratory borings were re-examined in the McMillen Jacobs office and classified independently of the field boring log description to provide a quality control check of the field classifications. Representative samples were selected for laboratory testing, which included the following:

- Unconfined Compressive Strength of Intact Rock Core Specimens (ASTM D7012, Method C)
- Point Load Strength Index (ASTM D5731)
- Natural Moisture Content (ASTM D2216)
- Atterberg Limits (ASTM D4318)
- Corrosivity:
  - pH (ASTM G51/T289)
  - Resistivity (ASTM G57) Redox (ASTM G200)
  - Sulfides (Qualitative by Lead Acetate Paper)
  - Chlorides (ASTM D4327)
  - Sulfates (ASTM D4327)

Moisture content, Atterberg limits, and corrosivity tests were completed by Benchmark Geolabs of McMinnville, Oregon. Unconfined compressive strength tests on rock cores were completed by Northwest Geotech, Inc. of Wilsonville, Oregon, and point load tests were completed by McMillen Jacobs. The tests were completed in accordance with the above-referenced ASTM standards. The laboratory test results are presented Appendix C.

2.4 Previous Geotechnical Investigations

Two previous projects that included geotechnical explorations have been completed in the immediate vicinity of the WTP_1.0 site. These include explorations completed by Jacobs Associates (now McMillen Jacobs Associates) for the SW 124th Transmission Line (PLM_3.0) project (Jacobs Associates, 2015) and the explorations completed by Geotechnical Resources Inc. (GRI) for pre-purchase assessment of a site located north of the WTP_1.0 site (GRI, 2016).

2.4.1 SW 124th Avenue Transmission Line (PLM_3.0)

In 2014, Jacobs Associates prepared the SW 124th Avenue Water Transmission Line Geotechnical Engineering Report, which included data from five soil borings with rock coring and two test pits along the segment of SW 124th Avenue that borders the eastern edge of the WTP_1.0 site. These explorations provide characterizations of the soil and basalt encountered directly east of the WTP_1.0 site. These borings and test pits (B-3 through B-7, TP-1, and TP-2) are shown in Figure 3. The explorations logs are provided in Appendix D.

2.4.2 Pre-purchase Due Diligence 90-Acre Site Investigation

In 2016, GRI conducted a preliminary subsurface investigation as part of a pre-purchase due diligence evaluation for the 90-acre property located at 12900 SW Tualatin-Sherwood Road. The investigation included 48 borings drilled using open-hole air-rotary impact drilling methods to estimate depth to rock, rock weathering, and rock hardness. Basalt was encountered in those explorations within the WTP_1.0 footprint at depths ranging from 0 to 18 feet bgs. Basalt hardness was observed as very soft (R1) to medium hard (R3) with zones of hard (R4). Weathering ranged from completely decomposed to fresh. Groundwater depth measurements varied and indicated perched groundwater conditions. A summary of exploration data along with a map showing these exploration locations are provided in Appendix D.

3.0 Site Description

The WTP_1.0 is located on an approximately 35-acre site at 12900 SW Tualatin-Sherwood Road in Sherwood, OR. The project site is located south of Tualatin-Sherwood Road and west of SW 124th Avenue on a vacant and largely wooded parcel. The project's north property line is approximately 1,100 feet south of SW Tualatin-Sherwood Road; the intervening land, currently vacant, was formerly in agricultural use. SW 124th Avenue borders the project's eastern property boundary. The western boundary is largely bordered by vacant woodland and a construction contractor's equipment yard. The southern boundary is a woodland which is crossed diagonally, northwest to southeast, by a Portland General Electric (PGE) powerline easement. Figure 1 shows the location of the site relative to surrounding features.

A woodland and active agricultural land lie east of SW 124th Avenue opposite the northern portion of the project site. An active rock quarry operation is present east of SW 124th Avenue opposite the southeastern portion of the project site. A former, and now reclaimed, rock quarry site is present south beyond the southern property line. Currently, access to the site is via a narrow gravel driveway which formerly served the now removed farmhouse at the above address. The surface conditions and geology of the site are described below.

The layout of the WTP_1.0 facilities and adjacent site features are shown in Figures 2 and 3. Elevations in this report are based on National Geodetic Vertical Datum 1929 (NGVD29).

3.1 Surface Conditions

The existing ground surface elevation varies across the proposed WTP_1.0 site footprint with a maximum elevation of approximately 285 feet near the center of the site and a minimum elevation of approximately 210 feet. Most of the proposed WTP_1.0 site and immediate adjacent areas are wooded with thick underbrush, including large thick patches of Himalayan blackberry and pervasive poison oak. The over story incudes numerous large Douglas fir, Oregon white oak, madrone, and other tree species. Several decomposing saw-cut stumps suggest that the site was logged in the past. An existing farmhouse and several out-buildings were located near the northern edge of the property during the field explorations but have since been removed.

Bare rock outcrops were noted at several locations near the WTP_1.0 site and a prominent rock face with approximately a 10-foot step down was observed north of the WTP_1.0 footprint. Although difficult to see through the thick underbrush, this rock face appears to be continuous from the northeast corner of the site westward and around to the southwest corner. Seven wetland areas have been identified by others within the site. Two small wetland areas, one northeast of the WTP_1.0 footprint and another near the midpoint of the WTP footprint were saturated at the time of our explorations in December 2018 and February 2019.

3.2 Geology

Regionally, the site lies within the Willamette lowland, a structural lowland between uplifted marine rocks of the Coast Range and volcanic rocks of the Cascade Range. The Coast Range, to the west of the lowland, consists of several thousand feet of Tertiary marine sandstone, siltstone, shale, and associated volcanic and intrusive rocks. The Cascade Range, to the east of the lowland, consists of volcanic lava flows, ash-flow tuffs, and pyroclastic and epiclastic debris. Marine and continental strata interfinger beneath and adjacent to the Willamette Lowland.

Four major depositional basins are present within the Willamette Lowland. These depositional basins include: the southern Willamette basin, northern Willamette basin, Portland basin, and the Tualatin basin. These basins, separated in most places by folded or faulted uplands of Columbia River Basalt bedrock, have locally accumulated more than 1,600 feet of fluvial sediment derived from adjacent uplifted blocks of Columbia River Basalt, the Cascade and Coast Ranges, and transported into the region by the Columbia River.

Locally, WTP_1.0 lies on the broad summit of the Parrett Mountain uplift which forms the boundary between the Tualatin Valley and the northern Willamette Valley. Parrett Mountain is an uplifted block of Columbia River Basalt. During the period of slow upward movement, exposed surficial basalt gradually developed a deep profile of weathering that graded from decomposed silt and clay soils at the surface through iron-stained and open-jointed weathered rock to relatively fresh dark gray basalt at depth. In the late Pleistocene, about 15,000 to 12,000 years ago, a series of catastrophic floods occurred as a result of periodic flooding released from Glacial Lake Missoula. These flood waters inundated the Columbia River system and back flooded up the Willamette River. The flood waters also surged through the Tualatin Mountains ("Portland Hills") gap at Lake Oswego, dumping boulders and coarse gravel at the mouth of the gap. This coarse flood debris grades westward to sand and then to micaceous, clayey to fine sandy silt across the Tualatin Valley. Many of the floods that entered the Tualatin Valley were sufficiently large enough to overtop the Parrett Mountain ridge crest. The flood waters then cascaded south into the Willamette Valley, scouring and eroding the soil and weathered basalt surface of the ridge crest in the process. This area of flood-scoured over-flow channels is now referred to as the Tonquin Scabland. The proposed WTP_1.0 is located on the Parrett Mountain ridge crest and within the area overtopped and scoured by the catastrophic floods.

3.3 Subsurface Conditions

We identified several geological units at the site consisting of Topsoil, Missoula Flood Deposits, Residual Soil, and Columbia River Basalt. These units were identified based on their geologic origin, stratigraphic position, engineering properties, and their distribution in the subsurface. Variations in subsurface conditions may exist between the locations of the borings.

Brief descriptions of the identified geologic units are provided below. Detailed descriptions of the units and accompanying laboratory test data are included in the GDR for WTP_1.0 (McMillen Jacobs, 2019a).

- **Topsoil**: Present on the ground surface and consists predominantly of 3 to 12-inches of very soft, dark brown to black, low plasticity organic silt.

- **Missoula Flood Deposits**: Consists of two facies: (1) Valley Fill deposits; which consist of stiff, moist, slightly yellow to orange-brown mottled silt and occur at lower elevations in the former agricultural field at the northeast corner of the project site, and (2) Channel Fill deposits; which consist of soft to stiff, slightly yellow-brown silt with scattered subangular cobbles and boulders and occur in the flood-scoured uplands in and adjacent to the areas now occupied by wetlands.

- **Residual Soil**: Generally, consists of very dense or stiff to hard mixtures of silt with trace sand and scattered to numerous angular, iron-stained gravel- to cobble-sized rock fragments.

- **Columbia River Basalt (CRB)**: Basalt was typically within 6 feet of the ground surface, except near the northern limit of the site (proposed SW Blake Street), where basalt was encountered between 6 and 16 feet bgs. The Columbia River Basalt (CRB) Unit includes basalt that is highly weathered to fresh. The basalt ranges from weak to very strong and moderately to intensely fractured with iron-stained joint surfaces. Unconfined compressive strengths (UCS) ranged from approximately 12,000 psi to 34,000 psi, with an average of 22,000 psi. Corrected Point Load Strength Index ($I_{S(50)}$) ranged from approximately 450 psi to 1,860 psi, with an average value of 1,440 psi. This correlates to an approximate average UCS of 25,000 psi using a site-specific correlation factor of 17.5. Calculation of the site-specific correlation factor is provided in Appendix C. The results of UCS and point load tests indicate 90 percent of tested samples have UCS of greater than 16,000 psi suggesting very strong rock (R5). A histogram of UCS test data is included in Appendix E.

A summary of the depth to rock in each exploration location is provided in Tables Table 3-1 through Table 3-3 below.

Table 3-1. Summary of Test Pit Explorations

Exploration ID, WTP_1.0-	Approximate Ground Surface Elevation (feet)^1	Exploration Depth (feet)	Depth to Rock (feet)	Rock Surface Elevation (feet)^2
TP-1	239	4.5	2.0	237
TP-2	265	4.0	4.0	261
TP-3	234	4.0	2.0	232
TP-4	251	4.0	0.8	250
TP-5	271	3.0	0.8	270
TP-6	276	0.5	0.2	275

Notes:
1. Elevations are based on survey data.
2. Elevations rounded to the nearest foot.

Table 3-2. Summary of Geotechnical Borehole Explorations

Exploration ID, WTP_1.0-	Approximate Ground Surface Elevation (feet)[1]	Exploration Depth (feet)	Depth to Rock (feet)	Rock Surface Elevation (feet)
B-01	236	45	4	232
B-02	238	40	10	228
B-03	244	20	1	243
B-04	269	15	3	266
B-05	256	15	1	255
B-06	262	15	4	258
B-07	253	20	2.5	250.5
B-08	276	30	0.5	275.5
B-09	272	31.5	1	271
B-10	265	40	1.5	263.5
B-11	263	35	3	260
B-12	257	30	5	252

Note:

1. Elevations are based on survey data rounded to the nearest foot.

Table 3-3. Summary of Air-Track Probe Hole Explorations

Exploration ID, WTP_1.0-	Approximate Ground Surface Elevation (feet)[1]	Exploration Depth (feet)	Depth to Bedrock (feet)	Rock Surface Elevation (feet)
P-01	237	35	6	231
P-02	254	35	5	249
P-03	268	35	2	266
P-04	260	35	5	255
P-05	243	35	6	237
P-06	264	35	6	258
P-07	233	35	16	217
P-08	232	32	9	223
P-09	234	35	7	227
P-10	232	35	6	226
P-11	238	35	0	238
P-12	230	35	3	227
P-13	238	35	2	236
P-14	247	35	6	241
P-15	253	35	7	246

Note:

1. Elevations are based on survey data rounded to the nearest foot.

Six cross sections depicting the subsurface geology at the site have been provided in Figures 4A through 4F.

3.4 Rock Mass Classification

Two rock mass classification systems were used to evaluate and characterize rock mass conditions for evaluating ground behavior, excavation methods, and design of retaining structures and rock cuts. The classification methods included Rock Quality Designation (RQD) and Rock Mass Rating (RMR). These classification systems were originally developed for tunneling but are useful in characterizing slopes and rock excavatability as well.

3.4.1 Rock Quality Designation (RQD)

RQD is a relationship between the total length of recovered core pieces equal to or longer than 4 inches to the total length of the individual core run (Bieniawaski, 1989). RQD is presented as a percent with higher percentages representing more intact and higher quality rock. Values of RQD are graphically shown on the boring logs in Appendix A. The RQD for the borings ranged from 0 to 100 percent, indicating “very poor” to “excellent” rock quality. The average RQD is 58 percent, indicating a “fair” rock quality. RQD of rock cores varied through depth of the boreholes. Table 3-4 presents the correlation of RQD to rock quality and percentage of core runs that fall within each of these ranges. Graphs of percentage of core run versus RQD range, and percentage of total core footage versus RQD for each borehole are provided in Appendix E.

Table 3-4. Rock Quality Description Based on RQD

RQD Value (percent)	Description of Rock Quality	Percentages of Core Runs (percent)
<25	Very Poor	25
25 – 50	Poor	21
51 – 75	Fair	17
76 – 90	Good	18
>90	Excellent	19

3.4.2 Rock Mass Rating (RMR) System

The RMR is a rating system that considers various rock mass parameters including the strength of intact rock, RQD, joint spacing, condition of joints, and groundwater conditions (Bieniawaski, 1989). Each of these parameters is given a numerical rating based on relative importance of the specific parameter on the behavior of the rock mass. This rating is adjusted to account for joint orientation depending on the favorability of the joint orientation for the specific project. The overall rating of rock mass, termed RMR, is calculated as the sum of the individual rating of the parameters minus the adjustment for joint orientation (if applicable). Based on the final RMR, the rock mass is classified. Rock mass classification determined from RMR is presented in Table 3-5. Based on the conditions encountered in the borings, the RMR ranged between 55 and 65 indicating “fair” to “good” rock conditions.

Table 3-5. Rock Mass Classification Based on RMR System

RMR Rating	Class No.	Rock Mass Description
<20	I	Very Poor
21 – 40	II	Poor
41 – 61	III	Fair
61 – 80	IV	Good
81 - 100	V	Very Good

3.5 Groundwater

Groundwater measurements were made between December 2018 and November 2019 at locations where piezometers were installed. Results of the groundwater measurements are provided in Table 3-6.

Table 3-6. Groundwater Measurements Summary

Boring ID WTP_1.0-	Depth to Groundwater(feet)									Groundwater Elevation Range (feet)
	12/27/2018	12/28/2018	01/03/2019	02/07/2019	02/08/2019	02/14/2019	05/06/2019	07/01/2019	11/05/2019	
B-01	-	2.0	1.7	3.0	-	0[1]	4.5	7.1	8.3	227.7 – 236
B-03					6.7	1.8	9.9	12.7	15.2	228.8 – 242.2
B-08						14.4	17.1	See Note 2		<256 – 261.6
B-10	33.0	-	34.0	35.7	-	34.9	35.5	36.1	37.6	227.4 – 232
B-11						9.7	13.9	16.2	25.8	237.2 – 253.3

Notes:
1. The monument was underwater, groundwater assumed at the ground surface.
2. Groundwater was below piezometer screen interval.

Evaluation of nearby water well logs indicate static groundwater levels to be approximately 100 feet below the ground surface. However, water levels measured in the borings (see Table 3-6) range from near zero to a maximum of 37.6 feet below the ground surface. In our opinion, groundwater levels measured in the piezometers represents perched groundwater surfaces.

Groundwater permeability in basaltic lava flows is often greater parallel to the flow units than perpendicular to them. The upper and lower surfaces of individual flow units are usually more vesicular, fractured, and brecciated than the internal portions of the units which tend to be more massive or blocky. Although vertical jointing is usually present through the flow interiors, the joint apertures are often narrow or gradually become in-filled with fine-grained surface soils that migrate downward with the infiltration of surface water and with the products of weathering and decomposition of the rock. In our opinion, precipitation ponds on the rock surface or infiltrates to shallow rock layers with low permeability during the wet season and then evaporates or slowly seeps downward to deeper low-permeability layers during the dry season.

Groundwater level across the site will likely coincide with water level in the wetlands during wet season and varies between elevations EL. 227 and EL. 237 feet during dry season. Groundwater levels can vary with precipitation, the time of year, and/or other factors. Generally, 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.

4.0 Seismic and Geologic Hazard Evaluation

4.1 General

Seismic and geologic hazards are discussed in detail in the Geologic Hazards Report (GHR, McMillen Jacobs, 2020a) and are summarized in this section. The GHR is included in Appendix G. Detailed discussion of local faults and seismic sources are presented in the WTP_1.0 Fault Location Study Memorandum (McMillen Jacobs, 2019b), included in Appendix F.

The seismic hazards evaluation has been performed in general accordance with the IBC 2018 and ASCE's Minimum Design Loads for Buildings and Other Structures, 2016 Edition (ASCE/SEI 7-16). Design ground motions presented herein are based on the system-wide Probabilistic Seismic Hazard Analysis (PSHA) performed for the Willamette Water Supply Program (Shannon & Wilson, 2017). Project design criteria (WWSP, 2017) require designing the new facilities (i.e. structures and pipelines) for the 2,475-year return period seismic event.

4.2 Regional Seismicity

The Pacific Northwest is a seismically active region that has three principle seismic sources: (1) the Cascadia Subduction Zone (CSZ) megathrust, which represents the interface between the subducting 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). Faulting and seismicity associated with Cascade volcanoes are also potential sources of seismicity, though they generally don't impact sites in the Willamette Valley. Seismic sources are further described in the WTP_1.0 Fault Location Study Memorandum (McMillen Jacobs, 2019c). These sources are all considered in the system-wide PSHA (Shannon & Wilson, 2017)

4.3 Site Classification

The project site was assigned a seismic site class following code-based procedures in ASCE/SEI 7-16, Chapter 20 (2017). Site class is used to categorize common subsurface conditions into broad classes to which ground motion attenuation and amplification effects are assigned. Site class accounts for the conditions encountered at the upper 100 feet of subsurface profile. Shallow bedrock was encountered during the subsurface investigation. The proposed structures are supported on bedrock, fill placed over bedrock, or fill placed on a relatively thin layer of soil over bedrock. Considering the measured shear wave velocity of bedrock and assuming a shear wave velocity of 1,500 feet per second for crushed aggregate fill, a Site Class B is appropriate for design purposes.

However, since the project PSHA does not include seismic parameters for Site Class B, we recommend using seismic parameter values for Site Class B/C included in PSHA. Note that some of the structures will be supported on fill placed on top of the rock. Assuming fill will be consisted of compacted crushed aggregate and considering the shear wave velocities measured during geophysical testing, using Site Class B/C is appropriate for structures within the fill area (and therefore all structures required for the project).

4.4 Seismic Design Parameters

A system-wide PSHA has been performed for the WWSP (Shannon & Wilson, 2017). The Uniform Hazard Spectra for a 2,475-year return period event for WTP_1.0 assuming a 5-percent damping ratio is presented in Table 4-1. The full Mean Uniform Hazard Response Spectrum for a 2,475-year return period event for both 5 and 0.5-percent damping ratios and vertical response spectrum are provided in the Site-Specific Site Response Spectrum (McMillen Jacobs, 2019d), included in Appendix H.

Table 4-1. Seismic Design Parameters

Parameter	Value
Soil Profile Class	B/C
Peak Bedrock Acceleration (g)	0.463
SA Peak Ground Acceleration (g)	0.463
SA 0.2-sec Period Spectral Acceleration (S_{MS})	1.062
SA 1.0-sec Period Spectral Acceleration (S_{M1})	0.400
SA 2.0-sec Period Spectral Acceleration	0.175
SA Peak Ground Velocity (PGV) (cm/sec)	40

Note: All spectral accelerations are adjusted for site class.

4.5 Seismic Sources and Hazard Deaggregation

The PSHA produces a mean source event that generates the spectral accelerations included in Table 4-1. For the 2,475-year seismic event and over the range periods between 0 and 5 seconds, the mean magnitude ranges from 7.8 to 8.7 and mean source-to-site distance ranges from 51 to 86 km.

As a part of the system-wide PSHA for the WWSP, seismic hazard deaggregation was performed (Shannon & Wilson, 2017). The deaggregation data identify the earthquake sources, magnitudes, and distances that contribute to the ground motion hazard for a particular return interval and spectral period. The deaggregation results indicate that multiple earthquake sources are significant contributors to the ground motion hazards. The seismic sources and the percentage of relative contribution of the three primary sources are listed below:

- Large megathrust events (between magnitudes 8.6 and 9.4) at distances of 50 to 150 km: 60-65% contribution to hazard;
- Shallow crustal events (up to magnitude 7.5) at distances of less than 25 km: 30-35% contribution to hazard; and
- Deep intraslab CSZ events (up to magnitude 7.5) at distances of about 45 to 100 km: <5% contribution to hazard.

The relative contribution of the different earthquake sources may be used in the development of time histories for detailed seismic analyses, including site specific response analyses, if required.

4.6 Liquefaction

Liquefaction is a phenomenon affecting saturated, loose sandy and non-plastic silty soils in which cyclic rapid shearing from an earthquake results in a drastic loss of shear strength and a transformation from a solid mass to a viscous, heavy fluid mass. The results of soil liquefaction potentially include loss of shear strength, loss of soil through sand boils, flotation of buried chambers/pipes, and post liquefaction settlement.

The site is underlain by shallow bedrock basalt, which is not susceptible to liquefaction. Therefore, liquefaction is not considered a hazard.

4.7 Slope Stability

The site is underlain by shallow bedrock and the majority of structures are founded below grade on the rock. Most areas surrounding the site are relatively flat. Therefore, slope stability is not considered a hazard.

4.8 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 free face. Liquefaction is not anticipated at the site. Therefore, lateral spreading is not considered a hazard.

4.9 Fault Rupture

There are no known active faults that cross the site. The nearest faults, the Sherwood-Lake Oswego and Canby-Mollala faults, are located about 2 kilometers north and 8 kilometers east of the site, respectively. We consider the risk of fault rupture across the WTP_1.0 to be negligible.

4.10 Buoyancy and Flotation

When pipes or hollow structures are installed under the groundwater table it is possible that they can float if the upward buoyant forces on the pipe exceed the downward gravitational forces from the soil cover or weight of the structures. Recommendations regarding buoyancy and floatation are included in Section 5.3 and 5.4.6 for structures and piping, respectively.

4.11 Flood Hazard

The Federal Emergency Management Agency (FEMA) has published maps with estimated flood inundation limits in the project area for 100-year and 500-year floods. These maps indicate 100-year and 500-year floodwater elevations between 140 and 150 feet. The lowest floor slab elevation (clearwell) in the treatment plant will be an elevation of approximately 229 feet. Therefore, the risk of precipitation induced flooding is negligible and is not considered a hazard.

A breach of Scoggins Dam would result in a large outflow of water and flooding along the Tualatin River due to the rapid draining of Henry Hagg Lake. The flood elevation resulted from the breach of Scoggins dam is anticipated to be near 500-year flood elevation, which is lower than the lowest point in the treatment plant. Therefore, the risk of flooding due to a dam break is negligible and is not considered a hazard.

4.12 Abrupt Settlement

Abrupt settlement generally occurs due to liquefaction or where structures (i.e. buildings and pipelines) are founded on the transition between soil and rock. Liquefaction is not anticipated at the site. Recommendations for subgrade preparation of structures bearing within the transition between soil and rock are provided in Section 5.1.1. Abrupt settlement is not considered a hazard.

4.13 Other Hazards

No significant geologic hazards such as landslides, slope instabilities, tsunamis, seiches, debris flows, expansive soils, or collapsible soils were identified within the proposed WTP_1.0 area.

5.0 Design Recommendations

The WTP_1.0 includes a 60 mgd treatment facility with future expansion to 120 mgd. The project also includes construction of SW Blake Street that connects SW 124th Avenue near the northeast corner of the site to a point at the southwest corner of the site. Construction of SW Blake Street and most of the WTP_1.0 structures will require excavations on the order of 5 to 10 feet and fill on the order of 2 to 8 feet. Up to 30 feet of excavation may be required for construction of the clearwell structure. The site finish grade will be between elevations of 255 and 265 feet.

The existing ground surface elevation varies across the site with a high point at approximately 285 feet near the plant and the ground descending in all directions away from the plant to a low point at an approximate elevation of 210 feet. Geotechnical explorations encountered basalt bedrock at various depths across the site. Basalt was typically within 6 feet of the ground surface, except near the northern limit of the site (proposed SW Blake Street), where basalt was encountered between 6 and 16 feet bgs. According to 60-percent project plans, excavations for most of the planned structures will extend into the basalt bedrock.

There are more than 24 new structures associated with the project, as shown in Figure 2. Geotechnical design recommendations for the structures, associated piping system, and planned roadway are presented in the following sections. All specifications referenced in this section refer to the Oregon Standard Specifications for Construction (ODOT, 2018). These specifications are referred to as OSSC hereafter.

Note that the site layout and buildings configuration are based on the 60-percent design and some details were incomplete at the time this report was prepared. If site configuration is modified or additional details become available, McMillen Jacobs should be contacted to update recommendations, as appropriate.

5.1 Foundation Design Recommendations

Based on the conditions encountered in the explorations and anticipated loading conditions, the proposed structures can be supported on shallow foundations (i.e. continuous and spread footings or mat foundations) bearing on rock or prepared subgrade. Based on discussions with the design team underdrains may be used under several structures. A summary of the proposed structures with the bottom of foundation elevation and anticipated subgrade condition is presented in Table 5-1.

Table 5-1. Anticipated Subgrade Conditions and Recommendations for Subgrade Preparation

Area Description	Structure Description	Bottom of Footing Elevation (feet)	Finished Grade Elevation (feet)	Foundation Type	Anticipated Bearing Pressure[1] (psf)	Anticipated Subgrade Condition (Fill/Soil/Rock)	Recommendation for Subgrade Preparation[2,3]
Administration	Steel Building	259.0	260.7	Spread	2,000	Fill over Soil	Excavate the surficial soil to firm subgrade condition, place compacted structural fill
Standby Generators	Equipment	257.0	259.0	Mat	1,000	Soil	Excavate the surficial soil to firm subgrade condition, place compacted structural fill
Equipment Storage Shed	Steel Building	256.0	258.0	Spread	2,000	Fill over soil	Excavate the surficial soil to firm subgrade condition, place compacted structural fill
Flash Mix	Steel Building	259.0	260.7	Spread	2,000	Rock	Place a min. 6 inches of compacted structural fill or leveling concrete
Ballasted Flocculation	Concrete Tank	257.0	260.7	Mat	2,800	Rock	Place a min. 6 inches of compacted structural fill or leveling concrete
Ozone Generation	Interior Building	258.0	260.7	Strip	2,000	Rock	Place a min. 6 inches of compacted structural fill or leveling concrete
Ozone Contactor	Concrete Tank	257.0	260.7	Mat	2,800	Rock	Place a min. 6 inches of compacted structural fill or leveling concrete
Filtration	Concrete Tank	252.0	260.7	Mat	2,800	Rock	Place a min. 6 inches of compacted structural fill or leveling concrete
Maintenance & Primary Switchgear	Steel Building	259.0	260.7	Spread	2,000	Rock and Fill over soil (Mainly on rock)	Excavate rock to 2 feet below the bottom of foundation and excavate the soil to firm subgrade. Backfill the area with compacted structural fill
Ultraviolet Disinfection	Concrete Tank	242.0	260.7	Mat	550	Rock	Place a min. 6 inches of compacted structural fill or leveling concrete
Clearwell Overflow	Concrete Tank	236.0	257.0	Mat	2,800	Rock	Place a min. 6 inches of compacted structural fill or leveling concrete
Exterior Chemicals	Vertical and Horizontal Steel Storage Tanks	259.0	260.7	Spread	2,000	Rock	Place a min. 6 inches of compacted structural fill or leveling concrete
Chemical Building	Steel Building	257.0	260.7	Spread/Mat	2,000	Rock	Place a min. 6 inches of compacted structural fill or leveling concrete
Clearwell	Concrete Tank (Prestressed & CIP)	221.0	256.0 to 252.0	Mat/Membrane	3,500	Rock	Place a min. 6 inches of compacted structural fill or leveling concrete
Equalization/Overflow Basins	Concrete Tank	230.0	256.7 to 255.0	Mat	1,750	Rock	Place a min. 6 inches of compacted structural fill or leveling concrete
Recycle Pump Station Electrical Building	Steel Building	253.0	255.0	Spread	2,000	Soil or Fill over soil	Excavate the surficial soil to firm subgrade condition, place compacted structural fill

Area Description	Structure Description	Bottom of Footing Elevation (feet)	Finished Grade Elevation (feet)	Foundation Type	Anticipated Bearing Pressure[1] (psf)	Anticipated Subgrade Condition (Fill/Soil/Rock)	Recommendation for Subgrade Preparation[2,3]
Gravity Thickeners	Concrete Tank	246.2	259.0	Mat	1,650	Tank No. 1 is likely on rock, Tank No. 2 is likely on Fill over soil	Assuming Tank No. 1 is on rock and Tank No. 2 is on Soil: Place a min. 6 inches of compacted structural fill or leveling concrete under Tank No.1. For Tank No. 2: Excavate the surficial soil to firm subgrade condition, place compacted structural fill. If one of the tank foundations supported on rock and soil: Excavate the rock 2 feet below the bottom of foundation and excavate the soil to firm subgrade. Backfill the area with compacted structural fill
Gravity Thickeners	Stairs/Splitter Box	257.0	259.0	Spread	2,000	Fill over soil	Excavate the surficial soil to firm subgrade condition, place compacted structural fill
Dewatering Building	Concrete & Steel Building	257.0	259.0	Spread/Strip	2,000	Soil	Excavate the surficial soil to firm subgrade condition, place compacted structural fill
Thicken Sludge Storage	Concrete Tank	255.2	259.0	Mat	1,250	Rock or thin soil over rock	Excavate the surficial soil to rock, backfill with compacted structural fill. A min. 6 inches of structural fill or leveling concrete is required under the foundation
Thicken Sludge Pump Station	Concrete Tank	243.7	259.0	Mat	1,250	Rock	Place a min. 6 inches of compacted structural fill or leveling concrete
Finish Water Pump Station	Steel Building	256.0	258.0	Spread	2,000	Soil and Fill over soil	Excavate the surficial soil to firm subgrade condition, place compacted structural fill
Finish Water Wetwell	Concrete Tank	220.0	258.0	Mat	3,500	Rock	Place a min. 6 inches of compacted structural fill or leveling concrete
Surge Tanks	Horizontal Steel Tanks	254.0	258.0 to 252.0	Spread/Mat	2,000	Soil and Fill over soil	Excavate the surficial soil to firm subgrade condition, place compacted structural fill

Notes:

1. Anticipated bearing pressure provided by CDM Smith.
2. Structural fill should consist of ¾-inch minus Dense-Graded Aggregates conforming to OSSC Section 02630.10.
3. The structural fill should be compacted within +/-2 percent of the optimum moisture content value and to a minimum of 100 percent of the maximum dry density determined by ASTM D698 (Standard Proctor).

5.1.1 Subgrade Preparation

Satisfactory subgrade support for spread footings or mat foundations associated with the proposed structures can be achieved on basalt bedrock, a well-compacted and well-constructed base drainage layer, or on imported structural fill that is properly placed and compacted on the bedrock or firm subgrade (i.e. undisturbed soil). Structural fill beneath foundations, if used, should extend a minimum of 12 inches beyond the edge of the foundation. For foundations supported on compacted structural fill with a thickness greater than 12 inches, the structural fill should extend beyond the edge of the foundation a distance equal to the height of the fill or 3 feet, whichever is less. Recommendations for structural fill are provided in Section 6.6.2 of this report.

For foundations directly supported on rock, we recommend a minimum of 6 inches, but not more than 12 inches of leveling course be placed on rock to minimize stress concentrations under the foundation. Alternately, rock surface beneath foundations may be leveled with a leveling concrete or grout layer. If an underdrain system is used, we recommend 18 inches of underdrain aggregate under 6 inches of structural fill leveling course.

For foundations supported on soil (i.e. no rock), we recommend excavating surficial soil to a firm subgrade condition. A minimum 12 inches of compacted structural fill should be placed below the foundation. The subgrade should be evaluated by proof rolling using a loaded haul vehicle (i.e. water truck or dump truck) in accordance with ODOT Test Method 158. If the area is not accessible by proof roll equipment, the soil subgrade should be evaluated by the Geotechnical Engineer prior to placement of steel reinforcement or concrete forms.

For foundations located within the transition between soil and rock, we recommend excavating rock to 2 feet below the bottom of foundation and excavate the soil to firm subgrade. The excavated areas should be backfilled with compacted structural fill.

5.1.2 Minimum Footing Width and Embedment

For at-grade structures, the minimum spread footing widths should be in conformance with 2019 Oregon Structural Specialty Code. As a guideline, we recommend individual spread footings have a minimum width of 24 inches, and continuous wall footings have a minimum width of 18 inches. Bottom of footings should be founded at least 18 inches below the lowest permanent adjacent grade to develop lateral capacity and for frost protection.

5.1.3 Bearing Capacity and Settlement

Bearing capacity for structures will depend on depth of foundations and locations on site. Based on the condition of rock observed in the geotechnical explorations, footings founded directly on rock or on rock with leveling course should be proportioned for a net allowable bearing pressure of 20,000 pounds per square foot (psf) (FHWA, 2002). This bearing pressure is a net bearing pressure and applies to the total of dead and long-term live loads. Settlement is anticipated to be negligible.

Footings supported on soil and prepared subgrade should be proportionated for a net allowable bearing pressure of 2,000 psf. Total settlement for the bearing pressure of 2,000 psf will be less than 1 inch assuming the subgrade is prepared based on the recommendations provided in this report. The differential settlement is anticipated to be on the order of one-half of the total settlement between adjacent footings or across a mat foundation.

The allowable bearing pressure may be increased by one-third for transient loading such as wind or seismic loads.

5.1.4 Mat Foundations and Floor Slabs

For mat foundations and floor slabs supported on rock or on prepared subgrade overlaying rock, we recommend using a modulus of subgrade reaction of 450 pounds per cubic inch (pci). For foundations supporting on prepared subgrade overlaying undisturbed soil, we recommend using a modulus of subgrade reaction of 250 pci. The subgrade modulus values represent anticipated values, which would be obtained in a standard in situ plate test with a 1-foot square plate. Use of this subgrade modulus for floor slab design should include appropriate modifications based on dimensions as necessary.

5.1.5 Lateral Resistance

Lateral resistances for the foundations can be provided by frictional resistance between the subgrade (i.e. soil or rock) and the bottom of the foundations, and by passive resistance around the footings. For the base frictional resistance, we recommend using an ultimate friction coefficient of 0.7 for cast-in-place concrete on rock and a coefficient of 0.6 for cast-in-place concrete on crushed aggregate. A coefficient of 0.4 may be used for pre-cast concrete foundations (i.e. vaults and manholes) on crushed aggregate.

Lateral resistance can also be provided by passive resistance of the foundations, especially for below ground structures. Utilizing full passive resistance for a foundation backfilled with compacted aggregate will require lateral displacement on the order of 2 percent of foundation depth. For example, a foundation embedded 20 feet bgs, will require approximately 5 inches of displacement to utilize full passive resistance. Large displacements may not be tolerable by structures. Therefore, at-rest conditions may be assumed on the resisting side (USACE, 1989). We recommend using an equivalent fluid density of 55 pcf for foundations above groundwater level, and an equivalent fluid density of 30 pcf for foundations below groundwater. Note, that these values are not factored. The designer should include an appropriate factor of safety.

For passive resistance of the foundations placed directly against rock, we recommend using a uniform passive pressure of 10,000 psf. This value includes a factor of safety of 3.0.

5.2 Embedded Walls and Lateral Earth Pressures

Backfill material placed behind foundations, walls, and retaining structures should consist of free-draining crushed aggregate conforming to OSSC 00510.12. Backfill placed within 5 feet of structure walls should be compacted to not more than 95% of the dry density determined by the Standard Proctor test (ASTM D698). This is to limit compaction pressure on the wall that would be greater than the at-rest pressure and potentially damage the wall. Large and heavy equipment, particularly compaction equipment, should not be allowed to operate near the walls during construction. The compaction equipment used within 5 feet of the wall should be hand compaction equipment, walk-behind, or self-propelled rollers not weighing more than 1,000 pounds. Loose lift thickness may need to be reduced where hand compaction equipment is used when placing and compacting fill.

The retaining structures and walls should be designed based on the lateral earth pressure diagrams provided in Figure 5. Active and at-rest lateral earth pressures were developed assuming a soil friction angle of 36 degrees and unit weight of 135 pcf. Seismic lateral earth pressure was calculated based on recommendations provided by Wagner et al., (2017) assuming a PGA 0.463g. WWSP (2018) requires designing below grade walls based on the worse of the two conditions in stiff soil:

- At-rest earth pressures (i.e. non-yielding condition)
- Active earth pressures combined with the seismic increment of earth pressures

The structural designer will select appropriate earth pressure based on wall stiffness.

As discussed previously, using full passive earth pressure for lateral resistance will require large displacements that may not be tolerable by the structure. Therefore, we recommend using lateral earth pressure equal to at-rest conditions the resisting side of the walls. We recommend using an equivalent fluid density of 55 pcf for portion of the walls above groundwater level, and an equivalent fluid density of 30 pcf for portions of the walls below groundwater.

5.3 Uplift and Flotation Considerations

Below-grade structures should be designed to resist uplift forces during periods of high groundwater. Based on groundwater levels measured during our field investigation, we recommend using a design groundwater level equal to the ground surface. This is due to potential for the collection of groundwater in wall backfill and subgrade. Although runoff will likely dissipate into the bedrock, the dissipation rate may be slower than collection rate leading to temporary hydrostatic and uplift pressure below structure foundations.

Uplift force is resisted by the weight of the structure itself, the weight of the backfill projected vertically above the outside edge of foundations extending beyond the vertical walls, and the friction force within backfill or between backfill and the excavation interface. However, for structures with a large footprint relying on structure and backfill weight to counteract uplift may not be sufficient or may not be structurally favorable for the base slab design. In these cases, an underdrain system or tie-down anchors may need to be considered.

Based on the foundation elevations and anticipated groundwater elevations, in most cases the weight of the structure and the weight of backfill on the outside edge of the footings is sufficient to resist the anticipated uplift force. The exception to this is the clearwell structure and the overflow basin, where the uplift force is larger than the weight of the structure and backfill. We anticipated uplift mitigation will be required for these structures. Uplift mitigation may include construction of an underdrain system or tie-down anchors.

If an underdrain or perimeter drain system is to be considered during design, we recommend that the system consist of an 18-inch thick layer of drain rock underlain by 6 inches of leveling course below the foundations and slabs, with 4-inch diameter perforated pipes located at the mid-height of the drainage layer. The pipes should be connected to a manhole with a pump system. The pump system can be turned on prior to maintenance for hydrostatic pressure relief and dewatering. The underdrain system can also be used as leak detection system under structures. A typical underdrain system is shown in Figure 6. More details about the underdrain system will be provided in the project plans and specifications, if an underdrain system is to be utilized.

Tie-down anchors include small diameter (typically 1 to 1.5-inch diameter bars) grouted anchors that are post-tensioned. The tie-down anchors may be used to reduce concrete slab thickness and excavation volume. However, tie-down anchors may cause stress variations in slabs and it can be difficult to construct a watertight connection at the anchor-structure interface. If tie-down anchors are utilized, we recommend the following for preliminary design:

- A minimum unbonded anchor length of 10 feet and a minimum bonded anchor length of 5 feet;
- An allowable grout to ground bond strength of 200 psi.

Tie-downs will be anchored in bedrock. More details about tie-down anchors (i.e. spacing, length, and construction details) will be incorporated in the project plans and specifications, if tie-down anchors are to be utilized.

5.4 Underground Pipelines

5.4.1 Pipeline Subgrade Support

Materials within the pipe zone are anticipated to consist of basalt bedrock, residual soil, or Missoula Flood Deposits consisting of medium stiff to stiff silt. These materials can adequately support the pipelines without modification.

5.4.2 Pipe Zone Geotechnical Design Parameters

Flexible pipes derive their load carrying capacity from their interaction with the pipe zone backfill as the pipe deflects under load and pushes laterally against the soil. Load carrying capacity depends on the depth of the pipe, the surrounding soil conditions, the type and density of the backfill, and the thickness of compacted pipe zone backfill between the pipe and the native soil/rock in the trench wall. Based on the anticipated subsurface soil types and relative densities, we have developed geotechnical design parameters to be used for pipeline design. These are provided in Table 5-2.

Table 5-2. Pipeline Design Parameters

Property	Native Soil	Basalt Bedrock	Granular Backfill	CLSM
Moist Unit Weight, γ_m (pcf)	120	165	135	125
Saturated Unit Weight, γ_{sat} (pcf)	125	165	140	125
Friction Angle, ϕ (degrees)	30	45	38	34
Modulus of Soil Reaction, E' (psi)	700	>10,000	2400	3,000

The design parameters presented in Table 5-2 are appropriate for use in the Iowa Deflection formula (Spangler, 1941) and are consistent with American Water Works Association Manual M11 (2004).

5.4.3 Backfill Materials

We recommend that the pipe bedding and pipe zone in the trench be constructed with imported, well-graded crushed rock, such as ¾- inch minus crushed aggregate conforming to OSSC Section 02630.10, Dense Graded Aggregate. We recommend a minimum 6 inches of bedding below pipe invert. Pipe bedding and pipe zone backfill should be compacted to 95 percent of the Standard Proctor maximum dry density, except the portion directly below the pipe that should be leveled without compaction. This will allow for uniform pressure distribution under the pipe invert.

In zones where pipes are installed within soil and below groundwater, fines from the trench sidewall will migrate into the bedding and pipe. This may lead to loss of lateral support and surface settlement. We recommend wrapping the pipe bedding and pipe zone in a separation geotextile to minimize fines migration. The separation geotextile should consist of a "needle-punch", non-woven separation fabric meeting the requirements for non-woven subgrade (separation) geotextile, as shown in Table 02320-4 Section 02320."

The trench width should extend a minimum of 18 inches beyond the side of the pipe. This will allow for the use of mechanical compaction equipment and placing backfill behind the pipe haunch.

Where the pipeline is located below paved areas or structures, trench backfill above the pipe zone should consist of imported granular aggregates, meeting the requirements of Class B or Class C materials per OSSC Section 0405.14.

5.4.4 CLSM Backfill

CLSM may be used as an alternative to granular fill in pipeline trenches. 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. The material typically is specified to have unconfined compressive strength of 50 to 200 psi. CLSM materials should meet the requirements of OSSC Section 00442.00, Controlled Low Strength Materials. Per WWSP (2017), the trench width should extend at least 6 inches beyond each side of the pipe (i.e. pipe outside diameter + 12 inches + trench protection).

We recommend using CLSM as pipe bedding and pipe zone backfill in pipe connection areas (i.e. pipe connecting to manholes and vaults, pipe junctions) and areas where the subgrade transitions from rock to soil. This will alleviate difficulties associated with backfill and compaction of aggregate in tight areas. In addition, it reduces the risk of differential settlement and problems resulting from poor compaction.

5.4.5 Soil Corrosion Characteristics

Corrosion index testing was performed on soil samples collected from borings and test pits. The average and range of the results of the laboratory testing for chlorides, sulfates, sulfides, pH, and soil resistivity are presented in Table 5-3.

Table 5-3. Summary of Soil Corrosivity Testing Results

Test	Average Value	Range
Oxidation-Reduction Potential (mV)	512	502 – 520
Water Soluble Chloride (mg/kg)	9	7 – 14
Water Soluble Sulfate (mg/kg)	14	2 – 36
Sulfide (mg/kg)	ND[1]	--
pH (pH units)	6.2	5.9 – 6.3
Soil Resistivity (Ω–cm)[2]	5,100	2,300 – 7,900

Notes:
1. ND: Non-detect.
2. Three soil resistivity tests were performed. Resistivity of one of the samples reported as 79,679 Ω-cm. Although the integrity of test was confirmed by the lab, we did not include this value assuming it is not a typical value for soils present at the site.

Considering the resistivity of soil ranging between 2,300 and 7,900 Ω-cm, the soil across the site is classified as moderately corrosive, based on the criteria provided by WWSP (2017).

Note that the values provided in Table 5-3 are for soil samples only and not representative of rock. If desired, further evaluation would need to be provided by a corrosion specialist.

5.4.6 Pipe Uplift and Floatation

Groundwater levels vary across the WTP_1.0 site. During dry season, groundwater is anticipated to be between elevation 227 feet and 237 feet across most of the site (i.e. deeper than 20 feet below the finished grade). During wet season, runoff may accumulate in trenches before dissipating into ground. For evaluation of pipe uplift and flotation, we conservatively assumed the groundwater level at the ground surface.

Section 2.2 of WWSP (2017) requires a minimum factor of safety of 1.5 against pipe flotation and requires a minimum cover of 4 feet in open country, 6 feet in collector streets, 7 feet in arterial streets, and 6 feet in developed areas. We evaluated flotation for pipes up to 84 inches in diameter. The results indicated factor of safety of 1.5 or greater for the minimum 4-foot cover.

For pipe diameter smaller than 48-inches, the minimum factor of safety of 1.5 against flotation can be achieved by depth of cover equal to pipe diameter. However, the construction live load and traffic load during the project life should be considered in the design.

5.5 Retaining Walls

The site development for the treatment plant requires construction of several retaining walls on the order of 10 to 15 feet. The retaining wall details were not available at the time this report was prepared. Recommendations for design and construction of the retaining walls will be provided in a supplemental report.

5.6 SW Blake Street Design Recommendations

SW Blake Street extends west from SW 124th Avenue. The road turns south near the northwest corner of the site and continues to the southwest corner of the site. The road is slightly over ¼-mile long and consists of a 45-foot wide paved area with approximately 15-foot wide shoulders on either side. Construction of SW Blake Street will require cut and fill heights on the order of 6 and 8 feet, respectively. 60-percent Design Drawings of SW Blake Street is provided in Appendix I.

Based on our explorations, fine grained soils are anticipated along the east end of the SW Blake Street alignment between Sta. 18+00 and 25+00 and along the south end of the alignment between Sta. 10+00 and 15+00. Basalt bedrock is expected to be excavated along the south shoulder of the road where the alignment transitions between north-south and east-west from approximately Sta. 15+00 to 17+00 where a cut of about 6 feet is shown (note: stationing is based on preliminary drawings).

5.6.1 Cut Slope Recommendations

Cut slopes are planned for the construction of SW Blake Street. Preliminary drawings show up to a 14-foot cut near Sta. 17+00, north of the WTP_1.0 structures. Based on our explorations, this excavation is expected to be through basalt bedrock.

A 1H:1V (horizontal:vertical) permanent slope is shown on preliminary plans along the south and east side of SW Blake Street and has its greatest vertical relief, approximately 14 feet, near Sta. 11+00. In this region we encountered basalt bedrock at a depth of approximately 6 feet below the existing ground surface. Bedrock is overlain by fine-grained soils. The resulting cut slope face would include the exposed fine-grained soils overlying basalt bedrock. We consider the rock slopes to be globally stable, however the overlying fine-grained soils could become unstable over time.

For preliminary purposes, at permanent cut slopes we recommend cutting exposed fine-grained soils of the cut to a slope of no steeper than 2H:1V. Rock may be cut at ½H:1V. We recommend constructing a 5-foot wide (minimum) ditch at the base of the cut slopes to catch any sloughing debris.

5.6.2 Fill

Construction of the westbound lane of SW Blake Street will require fill placement on the existing slopes. Most of the alignment will require fill heights on the order of 2 to 3 feet. Fill height on the order of 4 to 8 feet is anticipated from Sta. 12+50 to 15+50, and from Sta. 17+50 to 19+50.

It is anticipated that the project will generate an excessive amount of excavated material (soil and rock) associated with treatment plant facilities. Much of this material is anticipated to be acceptable for use as embankment material. Embankment material should be free of organics and deleterious material. Since topsoil (generally upper 3”-12” of the soil profile) consists of predominantly organic material, the topsoil is poorly suited for use as fill and should be removed from the site or used in a nonstructural fashion (such as landscaping). The maximum particle size in excavated material used for fill should be limited to 6 inches. The slope of embankments constructed using this material should not be steeper than 2H:1V. The top of fill elevation should allow for a minimum of 12 inches of base aggregate for pavement construction.

Alternatively, the rock generated from the excavation of the treatment plant facilities can be processed and used in construction of the embankments. The rock should meet the requirement of stone embankment material provided in OSSC 00330.16. If stone embankment material is used, embankment slopes can be constructed as steep as 1.5H:1V.

Recommendations for embankment construction are provided in Section 6.7.

5.6.3 Pavement

An estimated traffic count was not available at the time this report was prepared. Washington County requires a minimum of 3-inch thick asphaltic concrete for the new roads within the county. Considering SW Blake Street is located within an industrial area, heavy truck traffic is anticipated in future. Therefore, we recommend flexible pavement section for the proposed SW Blake Street to consist of a minimum of 7 inches of asphaltic concrete underlain by 12 inches of base rock consistent with SW 124th Avenue pavement section.

Most of the access roads within the treatment plant will be paved with asphaltic concrete. We recommend using a minimum 6-inch thick asphaltic concrete underlain by 12 inches of base rock.

Base rock should meet the requirements of OSSC Section 2630.10 for ¾-inch minus. Per Oregon Department of Transportation’s Pavement Design Guide (ODOT, 2019), we recommend a Level 3, ½-inch dense graded hot mix asphalt with PG 64-22 asphalt binder.

An area to east of the chemical building will be paved with Portland Cement Concrete Pavement. Areas immediately outside of dewatering building also may be paved with concrete pavement. Following the procedures provided in AASHTO (1993) for low volume roads, we recommend using a minimum 6-inch thick concrete pavement underlain by 12 inches of base rock. The concrete should be Class 4000 – 1½ paving concrete. Longitudinal and transvers bars should be selected following the recommendations of

ODOT Standard Drawings DET1600 and DET1602 for dowelled and undowelled plain concrete pavement, respectively.

Recommendations for pavement construction are provided in Section 6.8.

5.7 Reuse of Excavated Rock

Excavation for the proposed structures and roads (including SW Blake Street and access roads within the plants) will likely generate large volumes of rock that potentially can be used as structural fill aggregate if processed. The aggregates should meet the requirements of OSSC Section 02630.

There is an active quarry near the site east of SW 124th Avenue and we understand the rock excavated during construction of PLM_3.0 was processed and used as aggregate for backfill and construction of SW 124th Avenue. We anticipate the rock excavated at the treatment plant can be used as source of aggregate if processed.

6.0 Construction Recommendations

6.1 Rock Excavation

Construction of the WTP_1.0 elements will generally require 5 to 10 feet of excavation. The exception are the equalization/overflow basins, where up to 20 feet of excavation is anticipated and the clearwell and finished water pump station and wetwell structures, where up to 30 feet of excavation is anticipated. Most of the explorations within the treatment plant encountered rock within 5 feet of the ground surface. Therefore, most excavations will encounter rock. Based on the conditions encountered in the explorations, the rock is anticipated to consist of weak (R2) to very strong (R5), fresh to highly weathered, moderately to intensely fractured basalt. RQD of the rock ranged between 0 and 100 percent, with an average value of 58 percent. The results of laboratory testing indicate unconfined compressive strength of rock ranged between 12,000 and 34,000 psi, with an average value of 22,000 psi. The results of the geophysical explorations indicate the upper 10 to 15 feet of rock typically exhibit compression wave velocities of 6,000 to 8,000 feet per second (ft./sec). However, compression wave velocities up to 12,000 ft./sec were measured near the rock surface at several locations.

Deep excavation of the basalt will likely require controlled blasting in conjunction with the use of pneumatic breakers. Expansive grout may be used in sensitive areas.

The contractor should be responsible for selecting appropriate rock excavation techniques to prevent damage to the existing structures (i.e. PGE transmission lines and the exiting utilities in SW 124th Avenue) and minimize over-break or over-cut beyond the structure footprint. PGE will need to be notified with the proposed construction plan, including blasting plan, for a field supervisor to review. In addition, the selection of excavation methods and procedures should consider the impact to the foundation subgrade preparation and backfill placement.

Protruding rock of more than 3 inches above the specified subgrade elevation or 3 inches into the designated pipe zone should not be allowed unless approved by the engineer. Any large protrusions should be removed.

6.1.1 Trenching

Trench excavations to install subsurface piping are anticipated to be up to 15 feet deep. Trench excavations will be advanced into rock. Excavations may be completed using specialized equipment such as a hydraulic hammer attachment mounted on an excavator, nonexplosive methods such as hydraulic and chemical splitting, line drilling and/or drilling and blasting.

6.2 Temporary Cuts

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 Occupational Safety and Health Administration (OSHA). 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.

We understand in general, construction of the proposed structures will require 5 to 8 feet of horizontal clearance between the edge of cut and structure.

6.2.1 Soil Cuts

In general, near surface soil within the project area consists of soft to medium stiff silt and stiff residual soil. In accordance with OSHA, the soils throughout the project are classified as Type C. For excavations up to 20 feet, the maximum allowable temporary slope for Type C soils is 1.5H:1V (horizontal: vertical), if fully dewatered.

The stability of temporary unsupported cut slopes can be significantly reduced with time, the presence of shallow groundwater, and precipitation. 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.

6.2.2 Rock Cuts

In general, the basalt present across the project area is considered to be “stable rock” per OSHA and will stand at a vertical orientation for excavations less than 20 feet deep. However, since anticipated depths of excavations for some of the WTP_1.0 structures are greater than 20 feet and we anticipate contractor personnel at the bottom of the excavations during construction, we recommend the following for planning temporary excavations through basalt:

- For rock cuts 20 feet in depth or less, cuts may be sloped at 1/4H:1V;
- For rock cuts greater than 20 feet in depth, cuts may be sloped 1/4H:1V which includes a 4-foot wide bench at the mid-height. Benching reduces the overall slope angle and helps to protect the work area at the base of the slope from rockfall debris.

Considering global stability, rock cuts of up to 30 feet will be stable. However, minor rockfall from open cuts should be anticipated. Rockfall can be minimized by using controlled blasting along the perimeter of the excavation, and scaling cuts after excavation. Within the basalt bedrock, occasional and randomly-occurring weak zones or highly fractured zones are anticipated. These zones may locally destabilize the excavation and may require support. Potential support methods may include rock bolts or a shotcrete facing. Alternatively, wire mesh pinned to the face of the slope using short dowels can be used to reduce rockfall debris into the excavation.

Ultimately, design of rock cuts and temporary support measures are the responsibility of the contractor.

6.3 Temporary Shoring

6.3.1 Trench Shoring

Trench excavation for pipes and utilities will encounter near surface soil consisting of silty and clayey soil with cobbles and boulders to a maximum depth of 10 feet, underlain by basalt bedrock.

It is anticipated that the near-surface soils will be sloped down to the rock line (rather than vertically shored) before rock excavation. Cut slopes are discussed in Section 6.2.1. If vertical shoring is used, earth pressures shown in Figure 7 should be used for shoring design. A lateral earth pressure of 100 psf is recommended for rock material in order to account for localized weak and fractured zones and possible rockfall.

Within continuous rock material, it is anticipated that near-vertical slopes will stand unsupported. However, minor rockfall from trench sidewalls should be excepted. We recommend using a trench box or other measures in all trench excavations to protect workers and materials from sloughing and rockfall.

6.3.2 Shoring Plans

Temporary shoring systems in soil should be designed considering the earth pressures shown in Figure 7. Any equipment or traffic surcharge should be additive to these earth pressures. Additional safety systems will be required for deep trenches to protect workers from rockfall.

Selection of the shoring system and the safety of temporary excavation and cut slopes is solely the responsibility of the contractor. The contractor must submit an excavation and shoring plan to the Engineer prior to construction. The plan should show the design of the shoring, bracing, sloping, or other provisions to be made for worker protection from the hazard of caving ground for any trench or excavation over 5 feet in depth in soil. The contractor should be aware of, and familiar with, applicable local, state, and federal safety regulations including the current OSHA Excavation and Trench Safety Standards. The shoring plan must be prepared and stamped by a registered Civil or Structural Engineer in the State of Oregon.

6.4 Groundwater Control

We recommend completing excavations during summer and early fall when groundwater is typically at its lowest, and while minimal ponded water is present at the wetlands.

The groundwater measured in the piezometers and observed in our test pit explorations represent a perched groundwater condition at the site. Considering the depth of excavations and the measured groundwater elevations within piezometers, excavation for the majority of the proposed structures may not encounter groundwater. The excavation for the clearwell, finished water pump station wetwell and equalization/overflow basins structures will likely encounter groundwater.

If encountered, groundwater in excavations can be adequately controlled by a sump pump system. Large excavations (i.e. the Clearwell structure) may require installing several sump pump systems. A sump pump system consists of a groundwater control system in which the groundwater is allowed to enter the excavation. The water is then diverted to a sump area where it is collected and pumped out. Groundwater control should be continued until the structures are completed. The pumped water should be collected in tanks and properly disposed into a drainage system that would not recharge into the excavation.

We recommend constructing ditches or berms to divert any surface runoff from entering the excavation. Ponding water on the site and near the excavations should be prevented. Ponded water may infiltrate into the ground and act as a recharge source for groundwater seepage into excavations.

6.5 Blasting Plan

Based on the conditions encountered in our explorations, the contractor is likely to select drilling and blasting methods for excavation for some of the planned structures. The drilling and blasting should conform to the requirements of Section 00335.00 of the OSSC at a minimum. The contractor must submit a blasting plan prepared by a person qualified and experienced in blasting work at least 28 days before beginning of drilling and blasting work. The blasting plan must provide details of drilling and blasting pattern, vibration, flyrock, noise reduction methods, blast area security measures, and traffic control.

Drilling and blasting excavation methods generate vibrations. Blast designs must be developed to limit vibrations to levels that do not adversely affect existing nearby structures. Blast designs involve interrelated parameters including round length, blast hole size, spacing, location, explosive strength, and the delay and firing sequence. Delays are used to detonate fractions of seconds after blast initiation to make sure each charge will fire into a cavity created by an earlier charge.

If blasting is used, nearby structures, such as houses and commercial buildings within a 300 feet radius of the blast, should be pre-surveyed for documenting the existing conditions. Seismographs that are specifically designed to monitor construction blasting should be used during construction to monitor blast vibrations to verify that actual vibration levels are within an acceptable range at critical structures. If a blast results in unacceptable vibrations, special modifications to the blasting procedures should be made, such as using different delay patterns, reduction in size of individual blasts, shorter and/or smaller diameter blast holes, closer spacing of blast holes, reduction of explosives, or a combination thereof as necessary to improve results.

Temporary closure of SW 124th Avenue may be required during blasting. The contractor should coordinate with the Washington County and provide the county with plans for review and approval prior to any blasting.

As discussed previously, blasting will require coordination and approval of plans by PGE prior to any blasting near PGE structure. The Contractor should schedule the work accordingly to allow sufficient time for PGE review.

6.6 Fill Materials and Compaction Criteria

We anticipate that various fill materials will be used for the construction of this project and that their specific locations and placement criteria will be described in the construction plans and specifications. The following sections include fill criteria that are subject to modification under specific design recommendations and the development of construction plans and specifications.

The specified compaction criteria and optimum moisture in the following sections is in accordance with ASTM D698 (Standard Proctor), unless otherwise specified.

6.6.1 On-Site Soils

The surficial soils excavated for the proposed structures are expected to consist of predominantly fine-grained soils and include rock fragments in various sizes ranging from gravel to boulders. Some organic matter, such as root zones, are also expected within the excavation of surficial soils.

When processed, the on-site soils can be used as general purpose, non-structural fill. This fill is intended in landscaping areas or general embankment not subjected to surface loading. General purpose fill soils should not include particle sizes larger than 6 inches, should be free of organic matter, and should be moisture conditioned to allow for mechanical compaction.

It is possible that the basalt bedrock excavated can be processed for use as structural fill. This will likely involve crushing the larger rock fragments, processing them into various particle sizes, stockpiling the processed material on site, and a mixing and blending operation to create the recommended grading for structural fill.

6.6.2 Structural Fill

Structural Fill should consist of ¾-inch minus Dense-Graded Aggregates conforming to OSSC Section 02630.10 in settlement-sensitive areas, beneath structures, and pavement (not requiring high permeability).

The structural fill should be compacted within +/-2% of the optimum moisture content value and to a minimum 98 percent of the maximum dry density. Loose lift thickness should be 8 inches or less, and each lift of compacted structural fill should be tested by a qualified representative of a testing agency prior to placement of subsequent lifts. Ultimately, minimum lift thicknesses are dependent on the type of compaction equipment available and can be revised accordingly.

6.6.3 Embedded Walls and Retaining Wall Backfill

Backfill placed within 5 feet of retaining walls or embedded foundations should be compacted to not more than 95 percent of the maximum dry density to limit compaction pressure on the wall that could exceed the recommended at-rest pressure. Large and heavy equipment, particularly compaction equipment, should not be allowed to operate near the walls during construction. The compaction equipment used within 5 feet of the wall should be hand compaction equipment or walk-behind or self-propelled rollers with a static weight of less than 1,000 pounds. Loose lift thickness may need to be reduced where hand compaction equipment is used.

6.6.4 Bedding and Pipe Zone Backfill

We recommend that the pipe bedding and pipe zone in the trench 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. The material must be suitable for compaction and be able to be worked under the curvature of the pipe.

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, except the portion of bedding directly underneath the pipe that should be leveled without compaction effort.

6.6.5 Trench Backfill

Where the piping system is located below paved areas or structures, trench backfill above the pipe zone should consist of structural fill in accordance with Section 6.6.2 of this report. Similarly, trench backfill in these areas should be compacted to at least 95% of the maximum dry density.

For areas outside of the paved areas or below structures, trench backfill may consist of imported granular aggregates, meeting the requirements of Class B or Class materials as defined by OSSC 00405.14. Trench backfill in these areas should be compacted to 90 percent of maximum dry density per ASTM D698. The upper 18 inches of the trench should be backfilled with topsoil to allow for vegetation growth.

Trench backfill should be tested for compaction every 2 vertical feet and up to 50 linear feet of trench. All sampling and testing should be performed by an independent testing laboratory. Where trench depths or conditions preclude density testing because of worker safety concerns, trench backfill placement and compaction should be observed and documented on a full-time basis by the contractor’s approved testing agency until the backfill reaches an elevation at which density testing can commence.

6.6.6 CLSM

CLSM is a self-compacting, cementitious material that is typically considered when backfilling localized areas. Due to its flowable characteristics, CLSM typically can be placed in restricted-access excavations where placing and compacting fill is difficult. If chosen for use at this site, we recommend the CLSM conform with Section 00442.00 of OSSC.

6.7 Engineered Fill and Embankment Construction

Construction of fill and embankments should be completed according to OSSC Section 00330.00. We recommend that the embankment footprint be cleaned and grubbed to an extent of 10 feet beyond the toe of the fill. Roots, tree stumps and other vegetation should be removed to a depth of 6 inches. The holes resulted from grubbing should be backfilled with clean fill. Clearing and grubbing should be performed in accordance with OSSC Section 00320.00. For slopes with 5H:1V or less, which is the case for SW Blake Street, we recommend roughening or scarifying the surface to positively bond embankment material with the existing ground. For slopes steeper than 5H:1V, existing ground should be benched in accordance with OSSC Section 00330.42.

Fill should be placed in horizonal layers. Fill material that is moisture-density testable should be placed in layers not exceeding 8 inches and compact to 95 percent of relative density. Moisture content of the fill at the time of compaction should be within minus 4 percent to plus 2 percent of optimum moisture content.

For fill materials that are non-moisture density testable due to rock fragments, the material should be placed in near horizontal layers with thickness not exceeding 12 inches. Each layer should be compacted with a minimum of four full coverages using a 20,000 lbs. or larger smooth drum vibratory roller. At a minimum, one deflection test for each layer should be performed in accordance with ODOT TM 158.

6.8 Pavements

Asphaltic concrete should be placed in lifts 3-inch thick or less. Aggregate base course should be compacted to 95 percent of the maximum dry density determined by ASTM D1557 (Modified Proctor). Hot mix asphalt should be compacted to 92 percent of the theoretical maximum density as determined by ASTM D2041 (Rice Specific Gravity). A minimum of one field density test should be performed for each compacted lift of pavement material every 50 linear feet. Asphaltic concrete pavement should be constructed according with OSSC 00744.00 Asphalt Concrete Pavement.

Portland Cement concrete pavement within the area east of the chemical building should be constructed according with OSSC Section 00756.00 Plain Concrete Pavement.

6.9 Wet Weather Earthwork

Soil conditions should be evaluated in the field by the geotechnical engineer or his representative at the initial stage of site preparation to determine whether the recommendations within this section should be incorporated into construction. If earthwork is performed during extended periods of wet weather or in wet conditions, we recommend the following:

- 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; and
- The size and type of construction equipment used may have to be limited to prevent soil disturbance.

FOR LAND USE PERMITTING (EXHIBIT B)

7.0 Closure

This Geotechnical Engineering Report has been prepared for the Willamette Water Supply Program WTP_1.0 Project located in Sherwood, 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, as well as 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 once 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.

MCMILLEN JACOBS ASSOCIATES

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OREGON
Mar. 12, 2013
FARID SARIOSSEIRI
EXPIRES: 6/30/2020

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Associate Engineer

Jeremy Fissel, PE
Project Engineer

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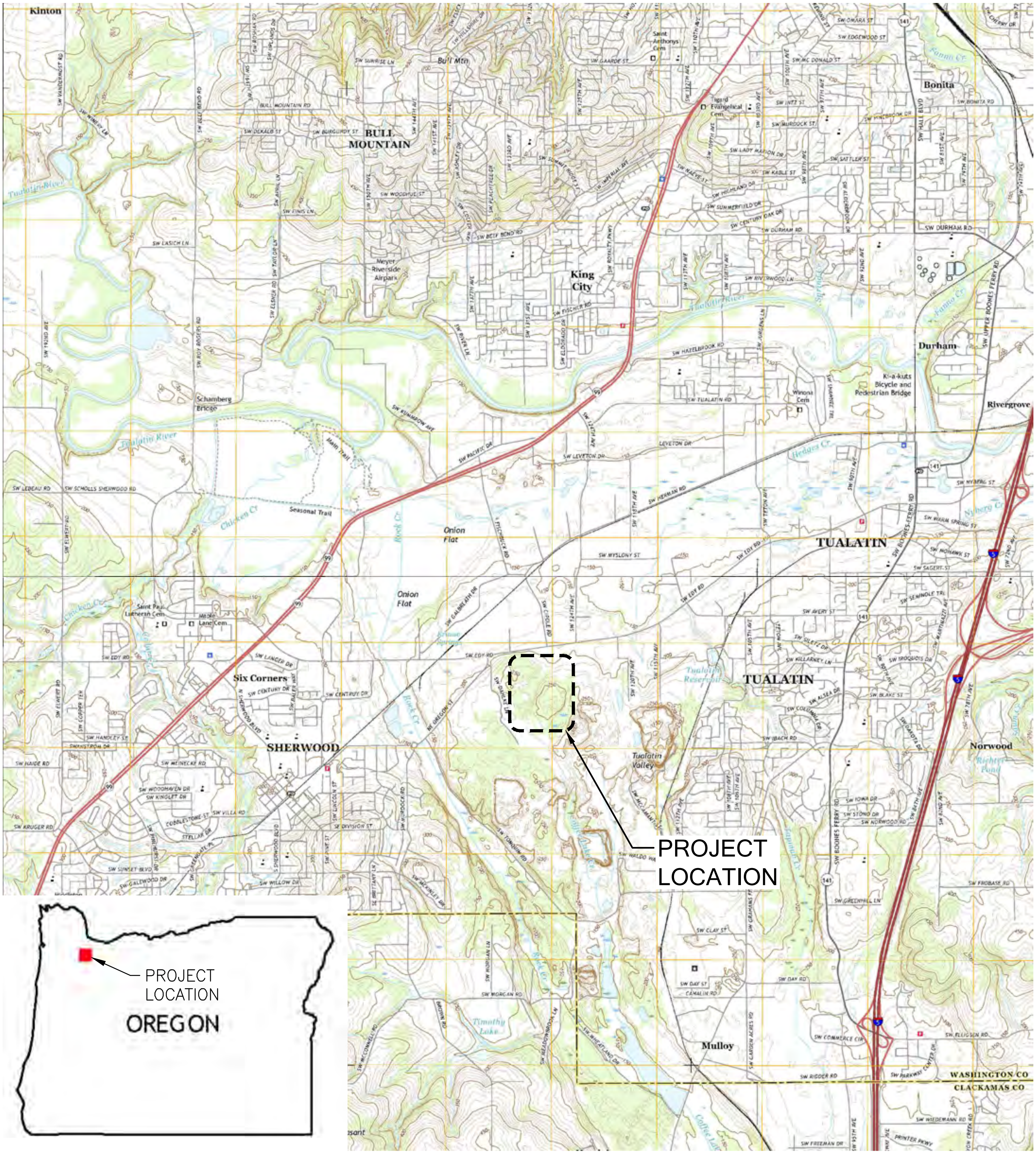
Figures

FOR LAND USE PERMITTING (EXHIBIT B)



PROJECT VICINITY MAP

SCALE: NTS

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WILLAMETTE WATER SUPPLY PROGRAM

WTP_1.0

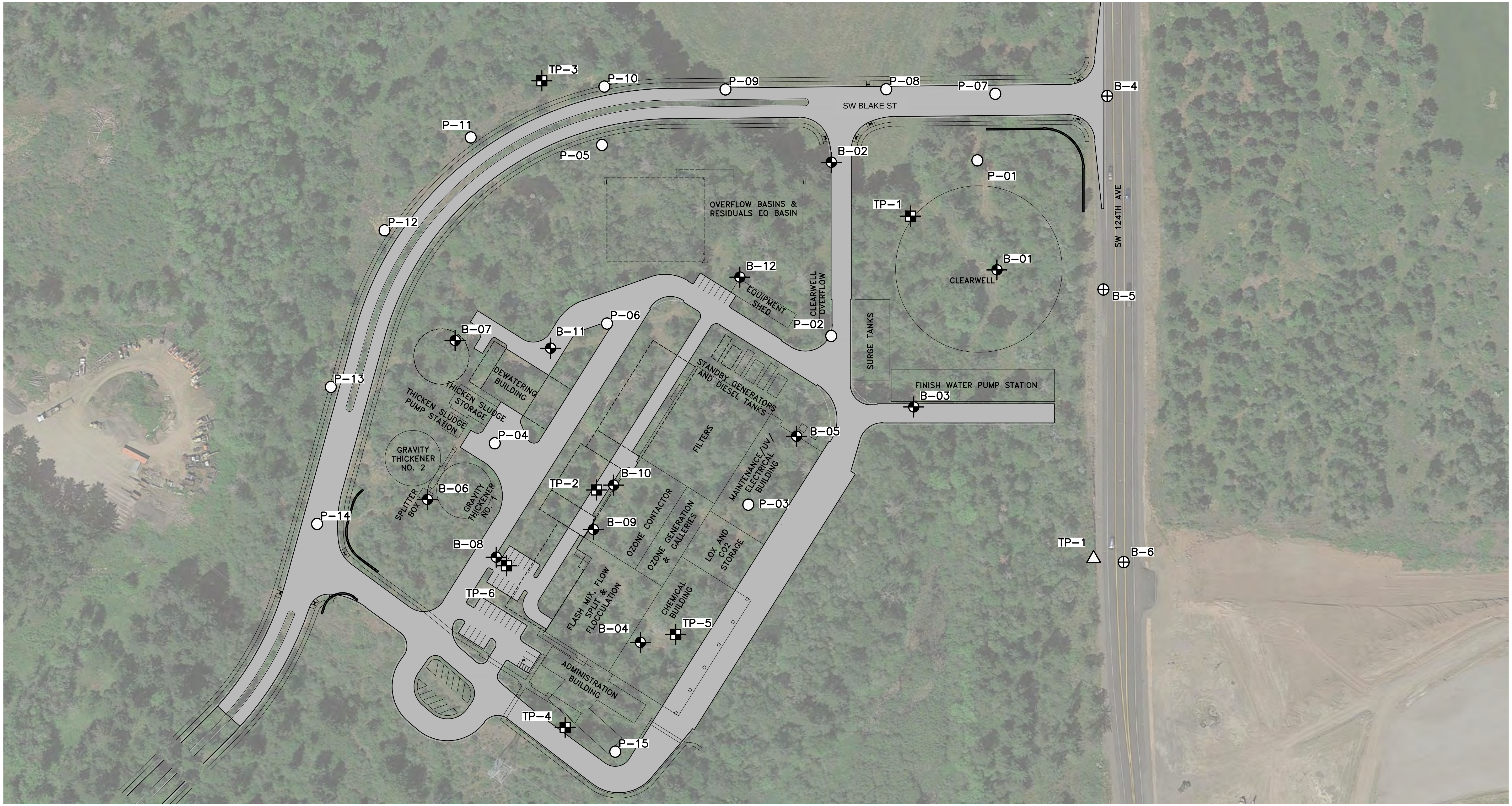
GEOTECHNICAL ENGINEERING REPORT
PROJECT VICINITY MAP

FIG.1

MAY 2020

Path: C:\Users\cburke\Box\Jobs\5887.0 WWSP WTP 1.0\CADD\GER_Fig.1.dwg Plot date: Dec 06, 2019, CAD User: cburke

FOR LAND USE PERMITTING (EXHIBIT B)



LEGEND:

WTP_1.0–P–1	○	AIR–TRACK DRILLING PROBE LOCATION
WTP_1.0–B–1	⊕	BOREHOLE LOCATION
WTP_1.0–TP–1	⊞	TEST PIT LOCATION
B–1	⊕	BOREHOLE COMPLETED AS PART OF PLM_3.0
TP–1	△	TEST PIT COMPLETED AS PART OF PLM_3.0
	⬚	SPACE RESERVED FOR FUTURE EXPANSION

NOTES:

1. BASE MAP PROVIDED BY CDM SMITH IN DEC 2019.
2. EXPLORATION LOCATIONS ARE APPROXIMATE.

N

0" 150' 300'

McMILLEN JACOBS ASSOCIATES

WILLAMETTE WATER SUPPLY PROGRAM
WTP_1.0
GEOTECHNICAL ENGINEERING REPORT
SITE LAYOUT

FIG.2

MAY 2020

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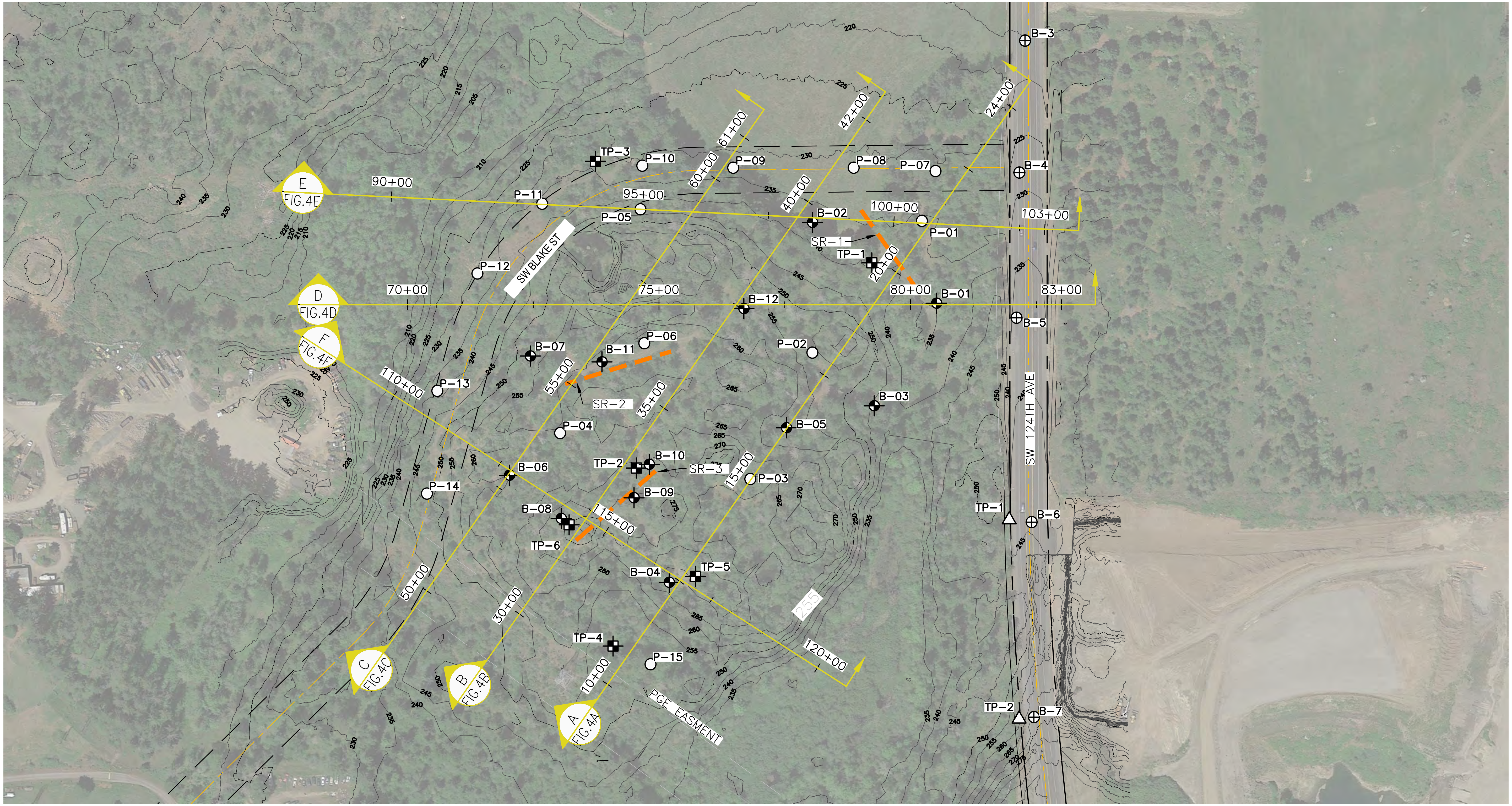
FOR LAND USE PERMITTING (EXHIBIT B)

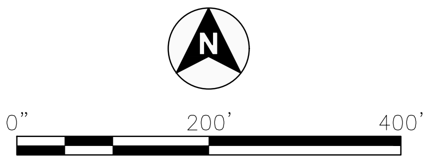


LEGEND:

WTP_1.0–P–1	○	AIR–TRACK DRILLING PROBE LOCATION
WTP_1.0–B–1	⊕	BOREHOLE LOCATION
WTP_1.0–TP–1	⊞	TEST PIT LOCATION
B–1	⊕	BOREHOLE COMPLETED AS PART OF PLM_3.0
TP–1	△	TEST PIT COMPLETED AS PART OF PLM_3.0
SR–1	— — —	GEOPHYSICAL EXPLORATIONS

NOTES:

1. BASE MAP PROVIDED BY CDM SMITH IN NOV 2019.
2. EXPLORATION LOCATIONS ARE APPROXIMATE.



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WILLAMETTE WATER SUPPLY PROGRAM
WTP_1.0
GEOTECHNICAL ENGINEERING REPORT
EXPLORATION PLAN

FIG.3

MAY 2020

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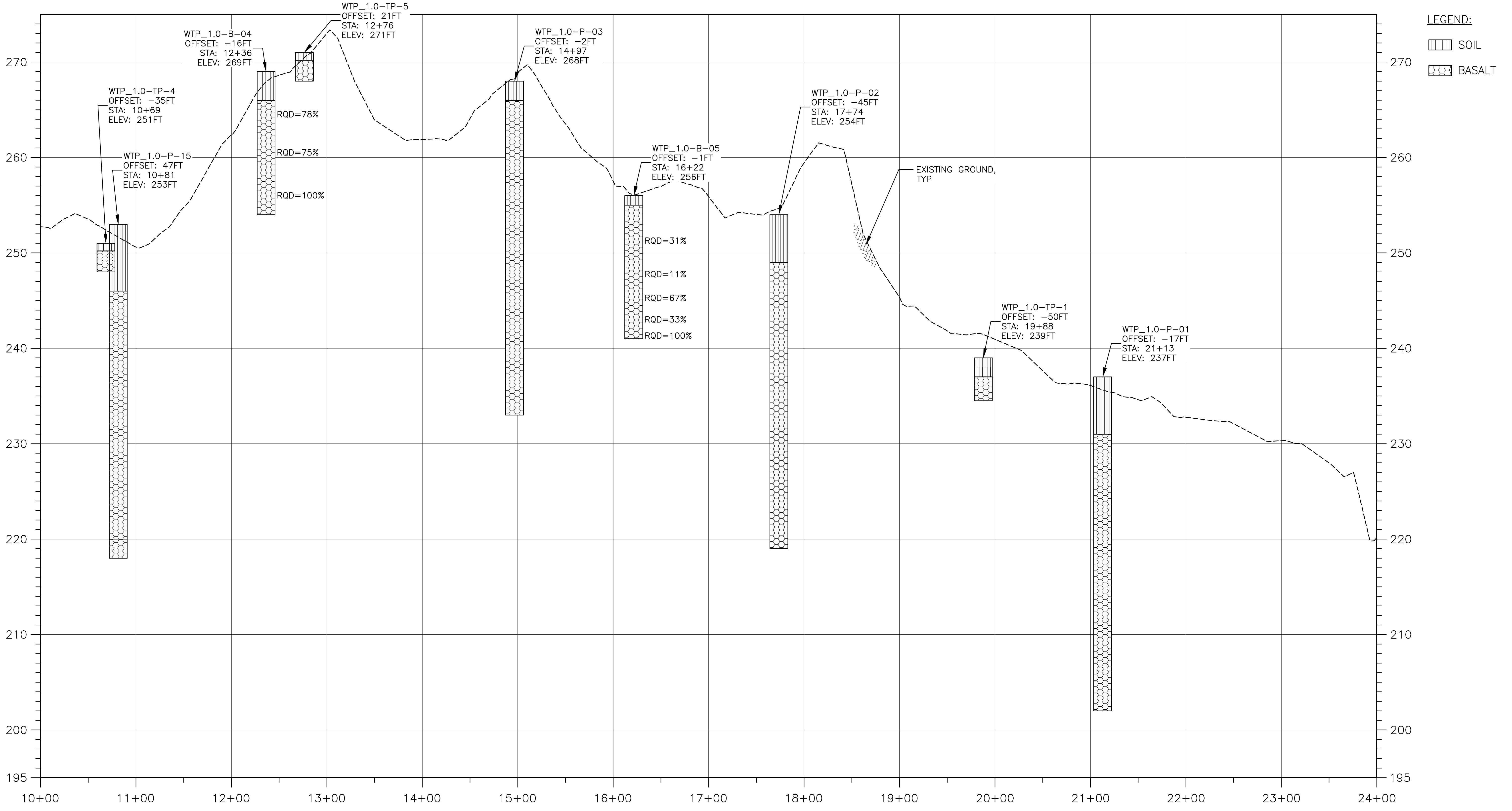
FOR LAND USE PERMITTING (EXHIBIT B)



NOTES:

1. PROFILE IS BASED ON TOPOGRAPHIC SURVEY PROVIDED BY CDM SMITH IN NOV. 2019.
2. BOREHOLE LOCATIONS ARE APPROXIMATE.
3. NEGATIVE OFFSET IS LEFT OF PROFILE AND POSITIVE OFFSET IS RIGHT OF PROFILE.

GEOLOGICAL PROFILE STA 10+00 TO 24+00

A
–

SCALE: 1"=100' HORIZ
1"=10' VERT
VERTICAL EXAGGERATION =10X

0" 100' 200'
0" 10' 20'

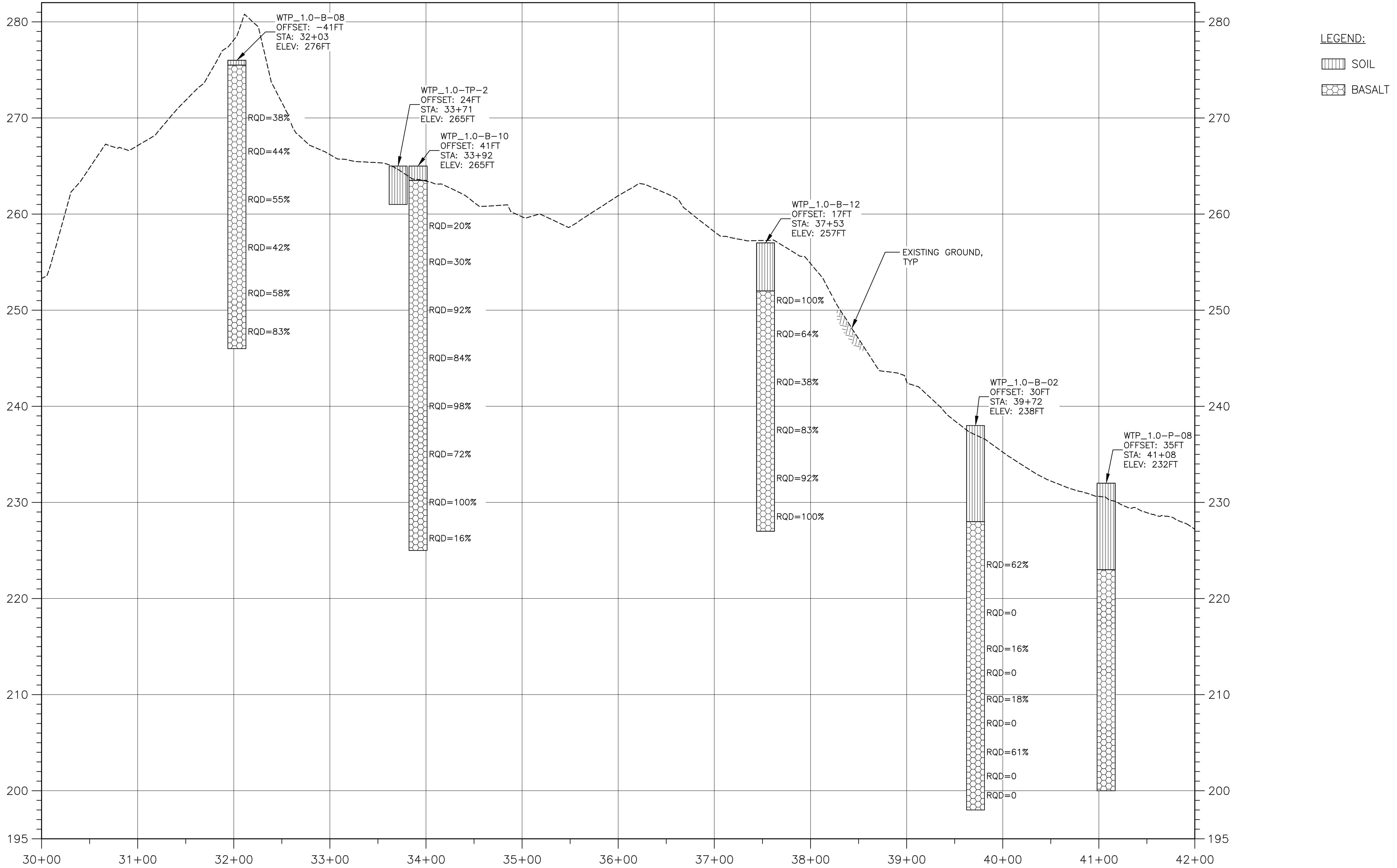
McMILLEN JACOBS ASSOCIATES

WILLAMETTE WATER SUPPLY PROGRAM
WTP_1.0
GEOTECHNICAL ENGINEERING REPORT
GEOLOGICAL PROFILE STA 10+00 TO 24+00

FIG.4A

MAY 2020

FOR LAND USE PERMITTING (EXHIBIT B)



NOTES:

1. PROFILE IS BASED ON TOPOGRAPHIC SURVEY PROVIDED BY CDM SMITH IN NOV. 2019.
2. BOREHOLE LOCATIONS ARE APPROXIMATE.
3. NEGATIVE OFFSET IS LEFT OF PROFILE AND POSITIVE OFFSET IS RIGHT OF PROFILE.

GEOLOGICAL PROFILE STA 30+00 TO 42+00 (B/–)

SCALE: 1"=100' HORIZ
1"=10' VERT
VERTICAL EXAGGERATION =10X

0" 100' 200'

0" 10' 20'

McMILLEN JACOBS ASSOCIATES

WILLAMETTE WATER SUPPLY PROGRAM

WTP_1.0

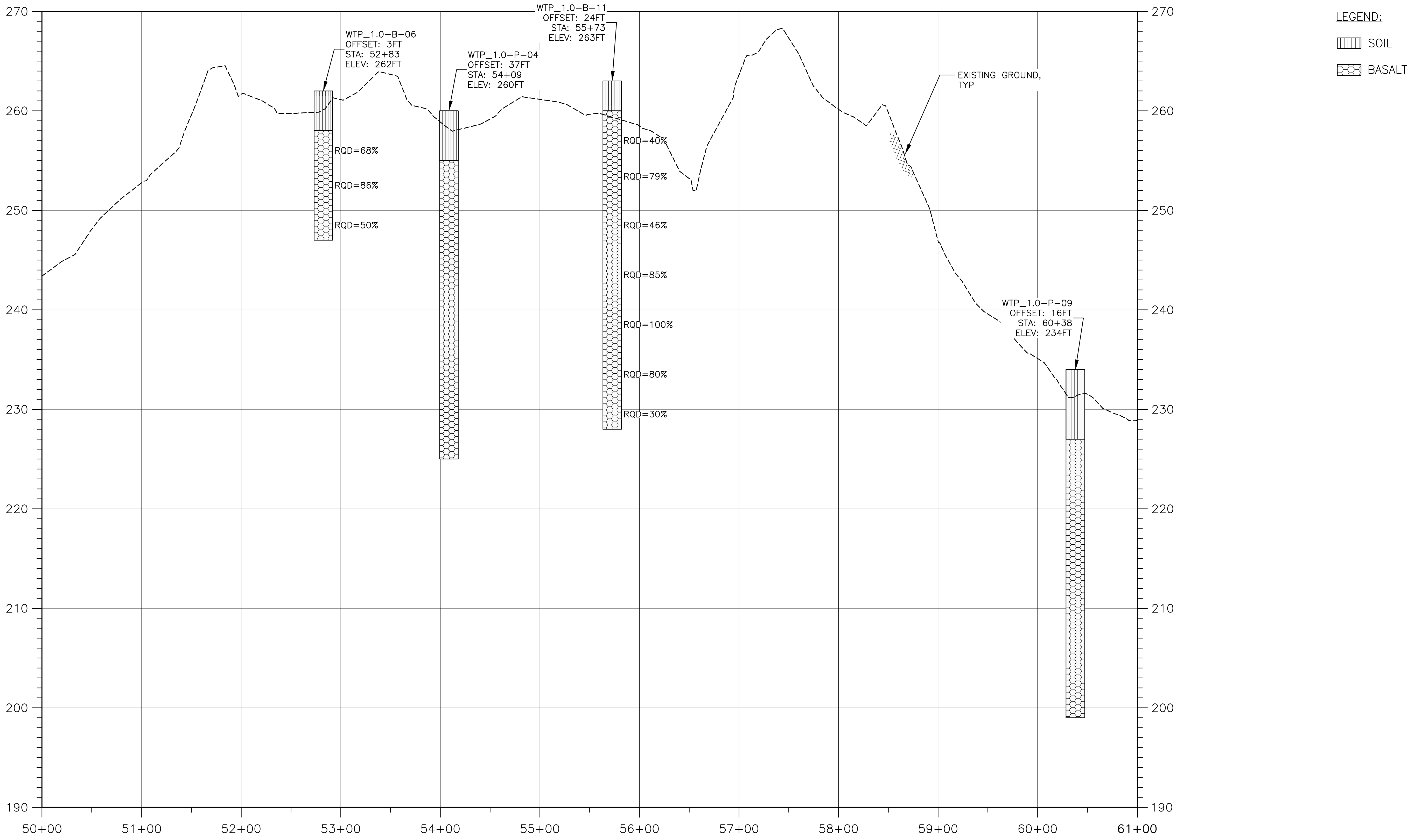
GEOTECHNICAL ENGINEERING REPORT
GEOLOGICAL PROFILE STA 30+00 TO 42+00

FIG.4B

MAY 2020

Path: C:\Users\cburke\Box\Jobs\5887.0 WWSP WTP 1.0\CADD\GER\Fig.4B.dwg Plot date: Dec 19, 2019, CAD User: cburke

FOR LAND USE PERMITTING (EXHIBIT B)



NOTES:

1. PROFILE IS BASED ON TOPOGRAPHIC SURVEY PROVIDED BY CDM SMITH IN NOV. 2019.
2. BOREHOLE LOCATIONS ARE APPROXIMATE.
3. NEGATIVE OFFSET IS LEFT OF PROFILE AND POSITIVE OFFSET IS RIGHT OF PROFILE.

GEOLOGICAL PROFILE STA 50+00 TO 61+00

C
–

SCALE: 1"=100' HORIZ
1"=10' VERT
VERTICAL EXAGGERATION =10X

0" 100' 200'
0" 10' 20'

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JACOBS
ASSOCIATES

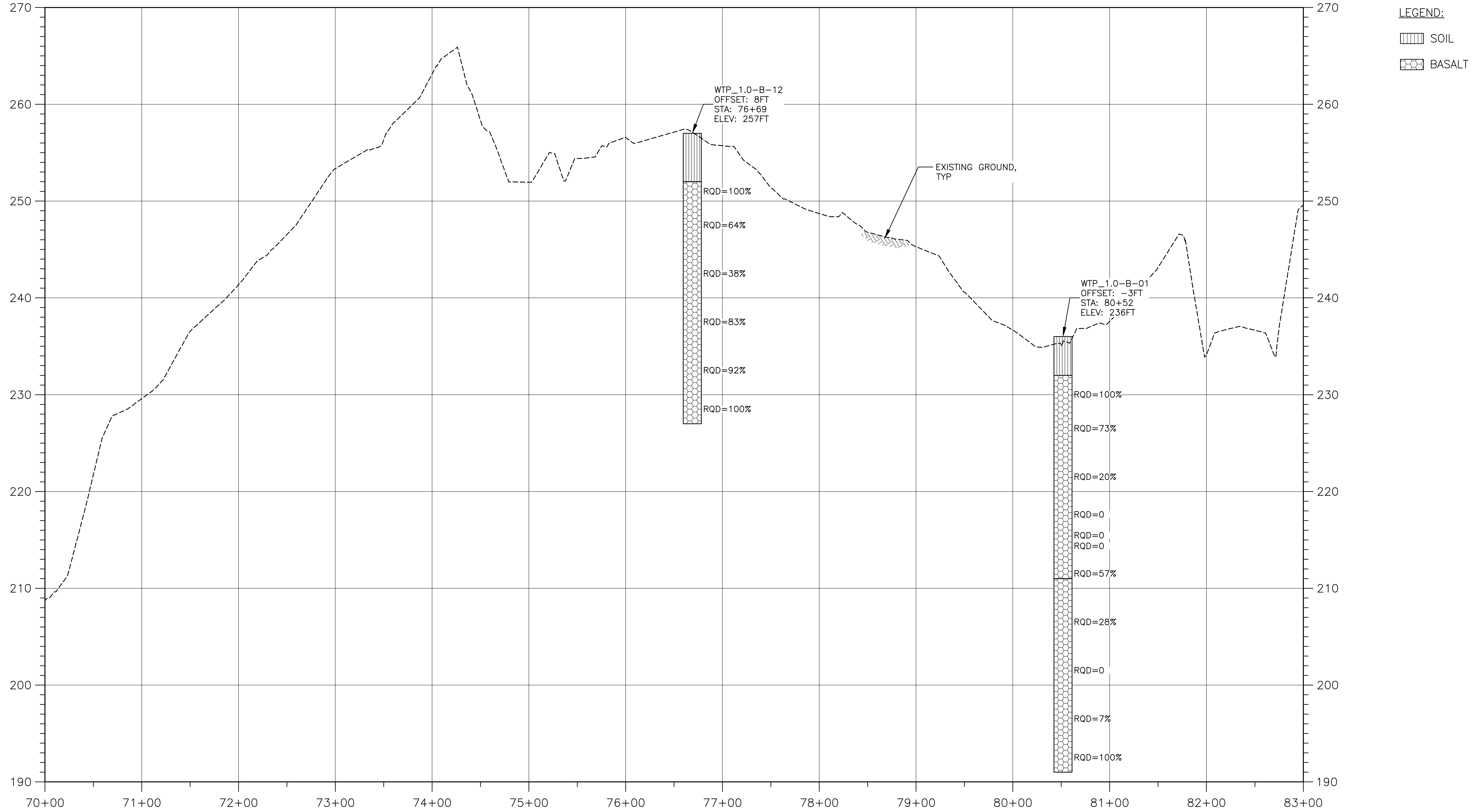
WILLAMETTE WATER SUPPLY PROGRAM
WTP_1.0
GEOTECHNICAL ENGINEERING REPORT
GEOLOGICAL PROFILE STA 50+00 TO 61+00

FIG.4C

MAY 2020

Path: C:\Users\cburke\Box\Jobs\5887.0 WWSP WTP 1.0\CADD\GER\Fig.4C.dwg Plot date: Dec 19, 2019, CAD User: cburke

FOR LAND USE PERMITTING (EXHIBIT B)



NOTES:

1. PROFILE IS BASED ON TOPOGRAPHIC SURVEY PROVIDED BY CDM SMITH IN NOV. 2019.
2. BOREHOLE LOCATIONS ARE APPROXIMATE.
3. NEGATIVE OFFSET IS LEFT OF PROFILE AND POSITIVE OFFSET IS RIGHT OF PROFILE.

GEOLOGICAL PROFILE STA 70+00 TO 83+00

D
–

SCALE: 1"=100' HORIZ
1"=10' VERT
VERTICAL EXAGGERATION =10X

0" 100' 200'

0" 10' 20'

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WILLAMETTE WATER SUPPLY PROGRAM

WTP_1.0

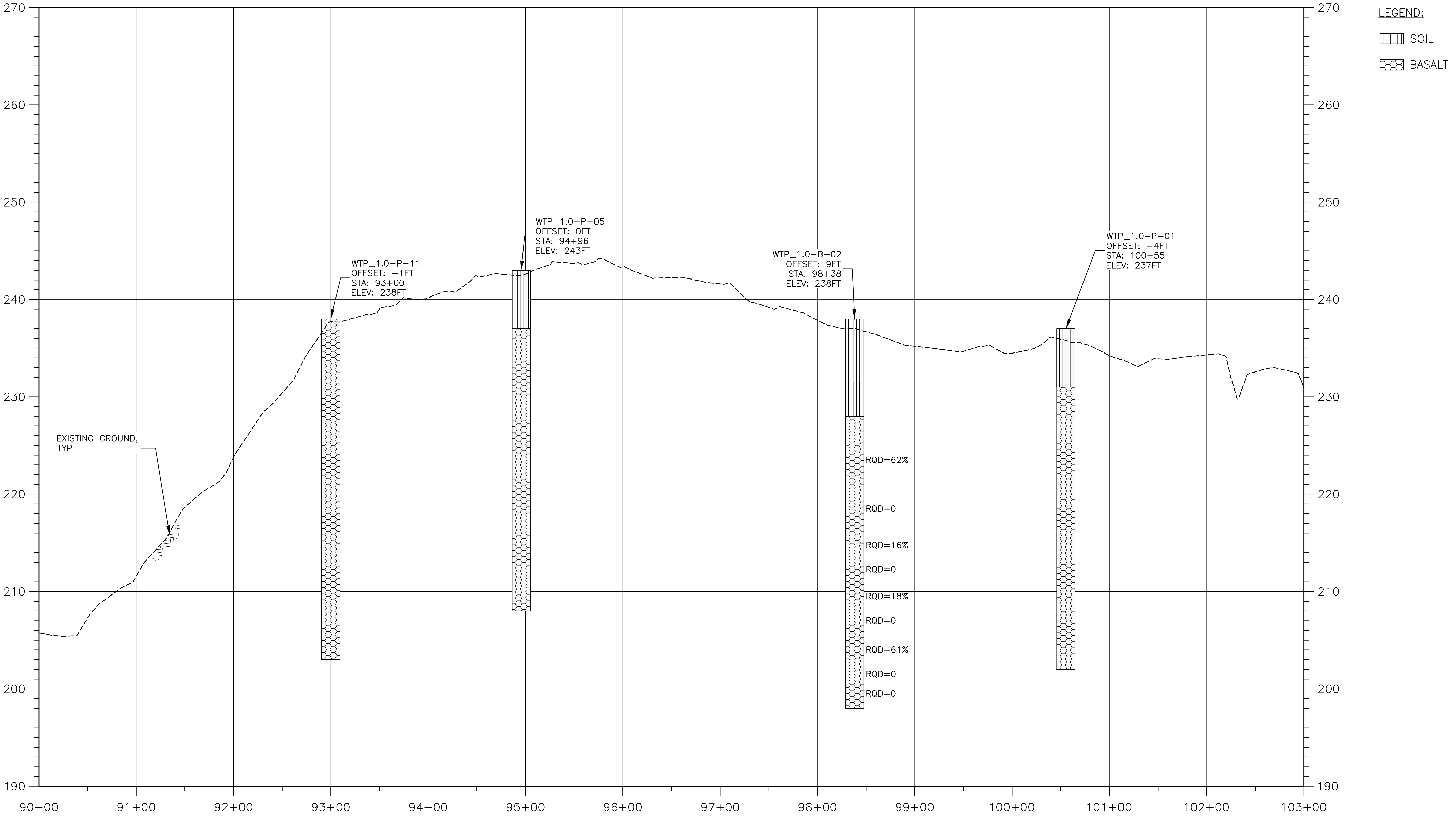
GEOTECHNICAL ENGINEERING REPORT
GEOLOGICAL PROFILE STA 70+00 TO 83+00

FIG.4D

MAY 2020

Path: C:\Users\cburke\Box\Jobs\5887.0 WWSP WTP 1.0\CADD\GER\Fig.4D.dwg Plot date: Dec 19, 2019, CAD User: cburke

FOR LAND USE PERMITTING (EXHIBIT B)



NOTES:

1. PROFILE IS BASED ON TOPOGRAPHIC SURVEY PROVIDED BY CDM SMITH IN NOV. 2019.
2. BOREHOLE LOCATIONS ARE APPROXIMATE.
3. NEGATIVE OFFSET IS LEFT OF PROFILE AND POSITIVE OFFSET IS RIGHT OF PROFILE.

GEOLOGICAL PROFILE STA 90+00 TO 103+00

E
—

SCALE: 1"=100' HORIZ
1"=10' VERT
VERTICAL EXAGGERATION =10X

0" 100' 200'

0" 10' 20'

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WILLAMETTE WATER SUPPLY PROGRAM

WTP_1.0

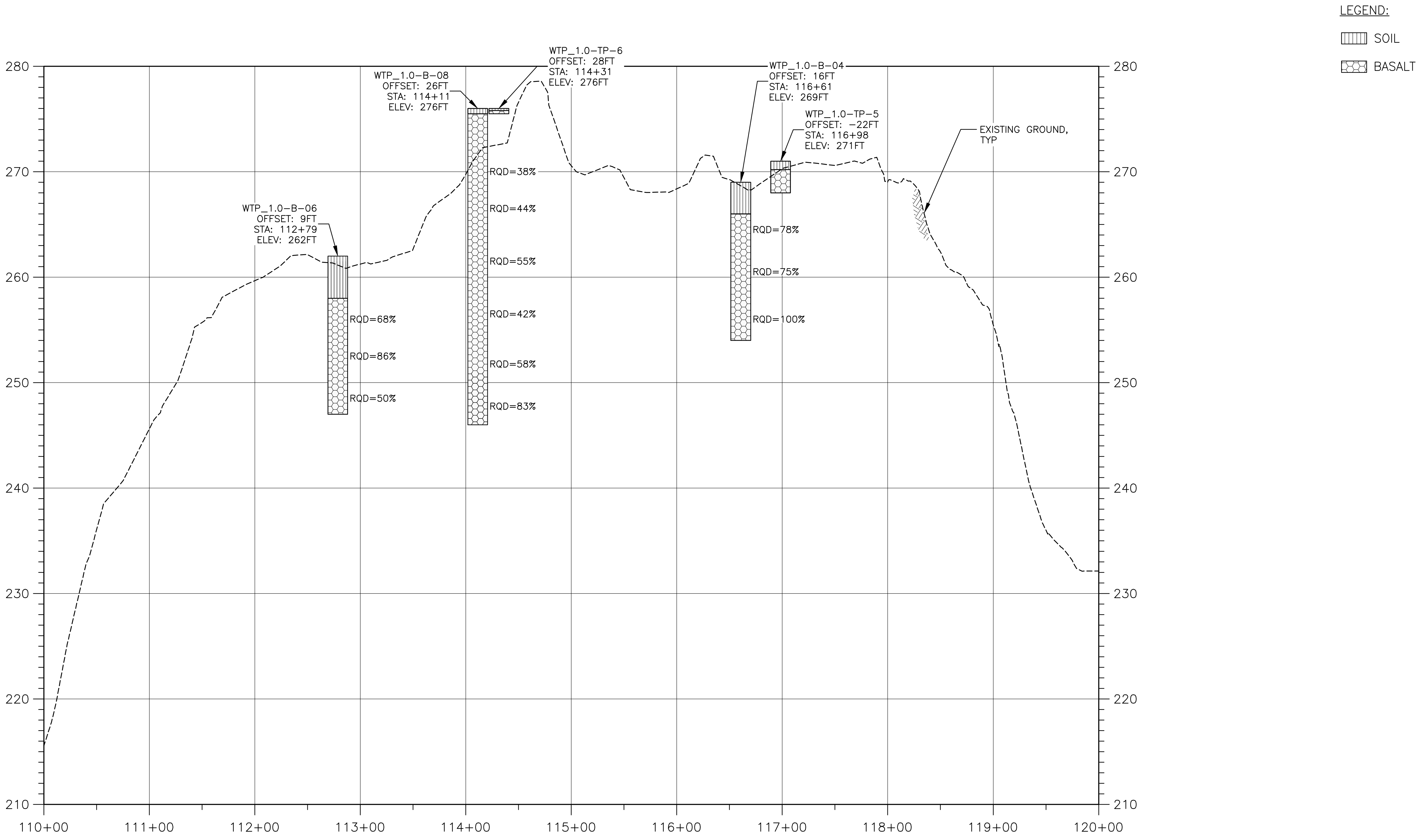
GEOTECHNICAL ENGINEERING REPORT
GEOLOGICAL PROFILE STA 90+00 TO 103+00

FIG.4E

MAY 2020

Path: C:\Users\cburke\Box\Jobs\5887.0 WWSP WTP 1.0\CADD\GER\Fig.4E.dwg Plot date: Dec 19, 2019, CAD User: cburke

FOR LAND USE PERMITTING (EXHIBIT B)



GEOLOGICAL PROFILE STA 110+00 TO 120+00 (F/–)

SCALE: 1"=100' HORIZ
1"=10' VERT
VERTICAL EXAGGERATION =10X

0" 100' 200'

0" 10' 20'

NOTES:

1. PROFILE IS BASED ON TOPOGRAPHIC SURVEY PROVIDED BY CDM SMITH IN NOV. 2019.
2. BOREHOLE LOCATIONS ARE APPROXIMATE.
3. NEGATIVE OFFSET IS LEFT OF PROFILE AND POSITIVE OFFSET IS RIGHT OF PROFILE.

McMILLEN JACOBS ASSOCIATES

WILLAMETTE WATER SUPPLY PROGRAM
WTP_1.0
GEOTECHNICAL ENGINEERING REPORT
GEOLOGICAL PROFILE STA 110+00 TO 120+00

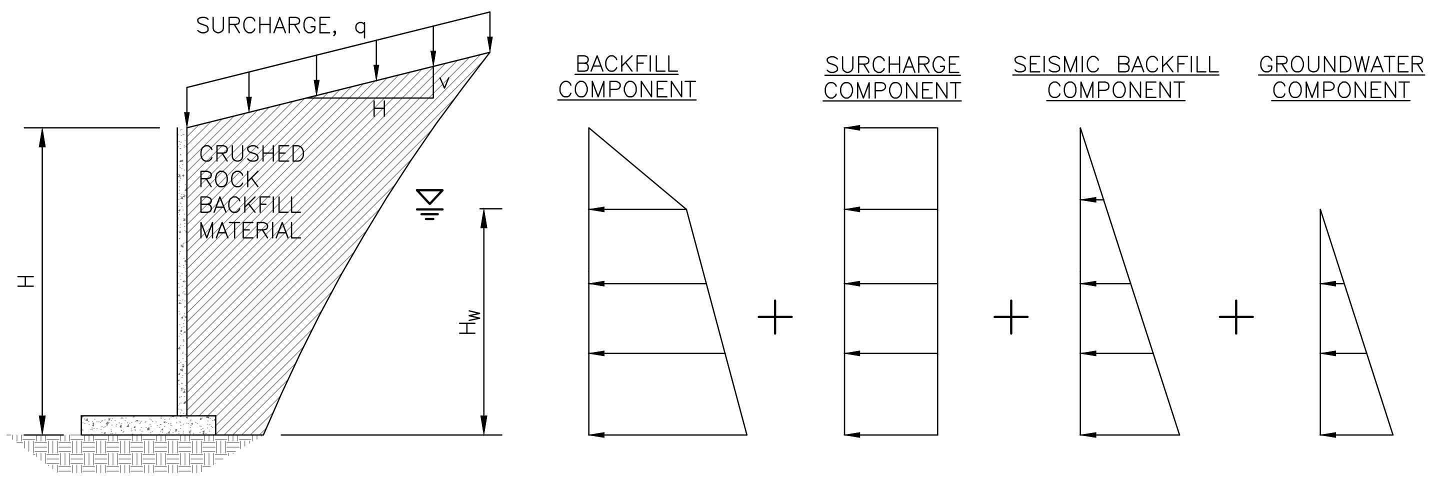
FIG.4F

MAY 2020

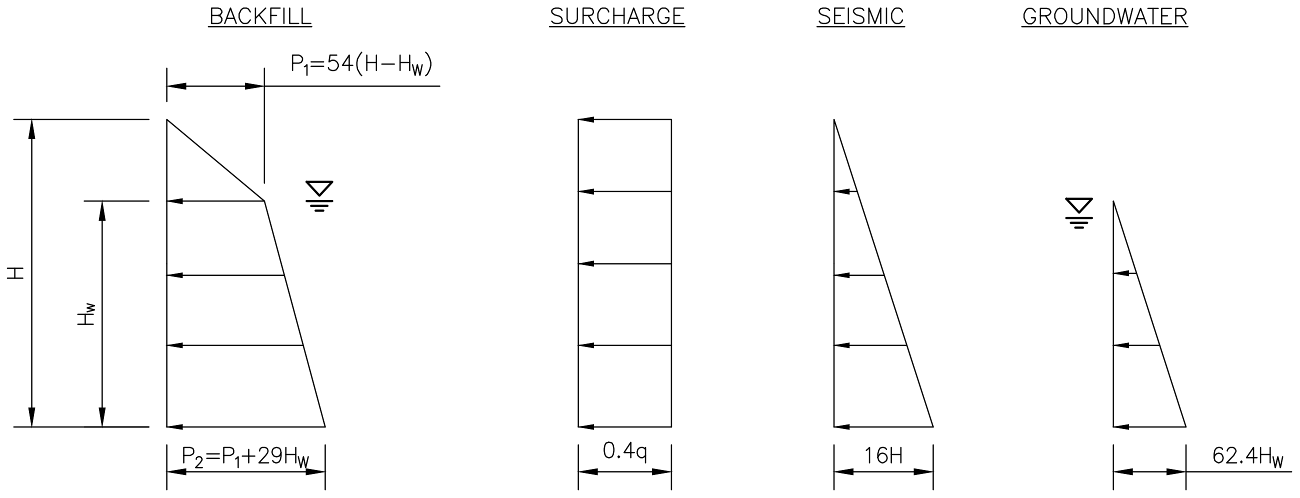
Path: C:\Users\cburke\Box\Jobs\5887.0 WWSP WTP 1.0\CADD\GER\Fig.4F.dwg Plot date: Dec 19, 2019, CAD User: cburke

FOR LAND USE PERMITTING (EXHIBIT B)

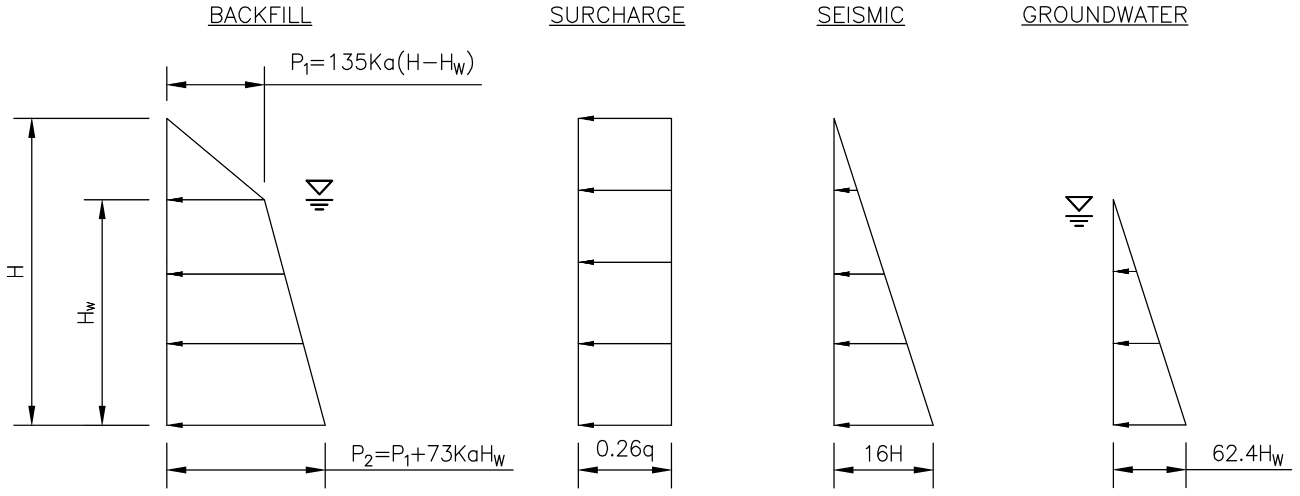
LATERAL EARTH PRESSURES ON EMBEDDED WALLS & STRUCTURES



NON–YIELDING EMBEDDED WALLS & STRUCTURES



YIELDING EMBEDDED WALLS & STRUCTURES



NOTES:

1. LATERAL EARTH PRESSURES WERE DEVELOPED ASSUMING UNIT WEIGHT OF 135 PCF AND FRICTION ANGLE OF 36 DEGREES FOR BACKFILL MATERIAL.
2. ACTIVE EARTH PRESSURE COEFFICIENT SHOULD BE SELECTED BASED ON BACKFILL SLOPE FROM TABLE PROVIDED IN THIS DRAWING
3. SEISMIC EARTH PRESSURE BASED ON 2,475–YEAR RETURN INTERVAL EVENT.

LEGEND:

H	HEIGHT OF BACKFILL ABOVE THE BASE OF EXCAVATION
Hw	HEIGHT OF WATER ABOVE BASE OF THE WALL
▽	ASSUMED GROUNDWATER LEVEL
q	UNIFORM SURCHARGE LOAD IN PSF

TABLE. ACTIVE EARTH PRESSURE COEFFICIENT FOR VARIOUS BACKFILL SLOPE

BACKFILL SLOPE (H:V)	Ka
FLAT	0.26
4:1	0.30
3:1	0.32
2:1	0.38

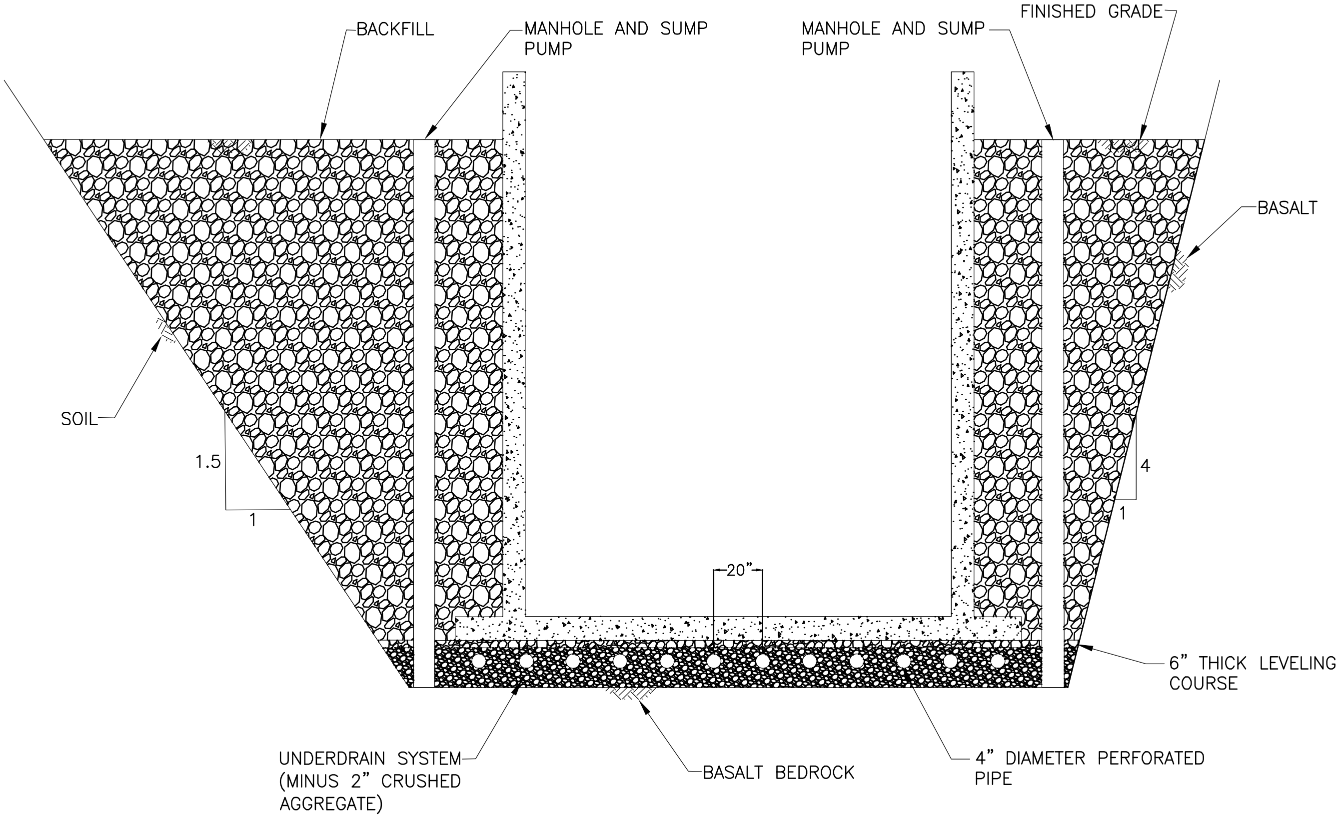
McMILLEN JACOBS ASSOCIATES

WILLAMETTE WATER SUPPLY PROGRAM
WTP_1.0
GEOTECHNICAL ENGINEERING REPORT
LATERAL EARTH PRESSURE DIAGRAM

FIG.5

MAY 2020

Path: C:\Users\cburke\Box\Jobs\5887.0 WWSP WTP 1.0\CADD\GER\Fig.5.dwg Plot date: Dec 19, 2019, CAD User: cburke



UNDERDRAIN SYSTEM
SCALE: NTS

McMILLEN JACOBS ASSOCIATES

WILLAMETTE WATER SUPPLY PROGRAM

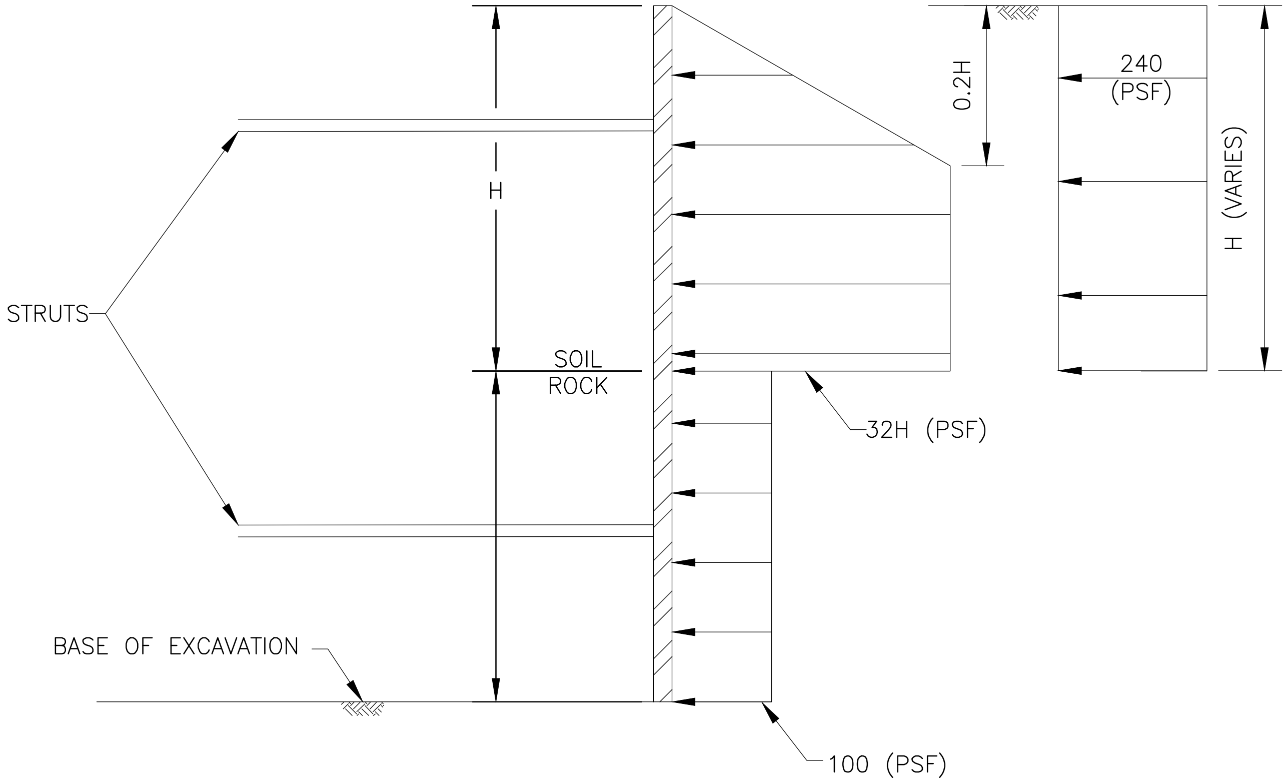
WTP_1.0

GEOTECHNICAL ENGINEERING REPORT
CONCEPTUAL UNDERDRAIN SYSTEM

FIG.6

MAY 2020

Path: C:\Users\cburke\Box\Jobs\5887.0 WWSP WTP 1.0\CADD\GER\Fig.6.dwg Plot date: Dec 11, 2019, CAD User: cburke



APPARENT LATERAL EARTH PRESSURE DIAGRAM – TEMPORARY TRENCH SHORING
SCALE: NTS

NOTES:

1. EARTH PRESSURE DIAGRAM ASSUMES BRACED EXCAVATION SUPPORT.
2. SURCHARGE LOAD MAY VARY BASED ON CONTRACTOR EQUIPMENT LOADS.
3. EARTH PRESSURE DIAGRAM ASSUMES LEVEL GROUND BEHIND SHORING.
4. EARTH PRESSURE DIAGRAM DOES NOT CONSIDER GROUNDWATER – GROUNDWATER PRESSURE SHOULD BE ADDED IF GROUNDWATER IS PRESENT.

LEGEND:

H HEIGHT OF SOIL

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WILLAMETTE WATER SUPPLY PROGRAM
WTP_1.0
GEOTECHNICAL ENGINEERING REPORT
LATERAL EARTH PRESSURE DIAGRAM
TEMPORARY TRENCH SHORING

FIG.7

MAY 2020

Path: C:\Users\cburke\Box\Jobs\5887.0 WWSP WTP 1.0\CADD\GER\Fig.7.dwg Plot date: Dec 11, 2019, CAD User: cburke

Appendix A

Geotechnical Explorations

FOR LAND USE PERMITTING (EXHIBIT B)

McMILLEN JACOBS ASSOCIATES

Key to Boring Logs
WWSP WTP_1.0

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 12% fines or when liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

Coare-Grained Soils

Relative Density	N, SPT Blows/Foot
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-Grained Soils

Relative Consistency	N, SPT Blows/Foot
Very Soft	< 2
Soft	2 - 4
Medium Stiff	5 - 8
Stiff	9 - 15
Very Stiff	16 - 30
Hard	> 30

Abbreviations

AL	Atterberg Limit
MC	Moisture Content
SA	Sieve Analysis
LL	Liquid Limit
PL	Plastic Limit

Sample Symbols

⧗	SPT Sample 2" OD
■	Shelby Tube Sample
✋	Grab Sample
N	Blows/ft

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

Test Symbols

□	Blows/Ft
○	Moisture Content
├┤	Liquid Limit/Plastic Limit

Modifiers & Percentages

Trace	Component is present at less than 5% of the less than 3-inch portion.
With (Sand or Gravel)	Coarse particles present at levels estimated at 12-30%.
Sandy or Gravelly	Coarse particles present at levels estimated at 30-50%.

Moisture Content

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

FOR LAND USE PERMITTING (EXHIBIT B)

McMILLEN JACOBS ASSOCIATES

Key to Boring Logs - Rock

WWSP WTP_1.0

Rock Strength

Grade	Approximate Uniaxial Compressive Strength (psi)	Term
R0	<100	Extremely Weak
R1	100 - 1,000	Very Weak
R2	1,000 - 4,000	Weak
R3	4,000 - 8,000	Medium Strong
R4	8,000 - 16,000	Strong
R5	>16,000	Very Strong

Core Recovery Calculation (%)

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

RQD Calculation (%)

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

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

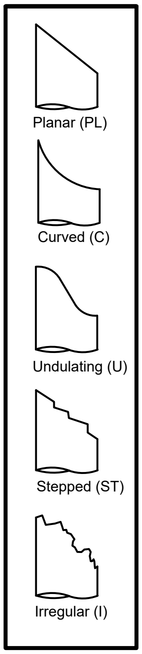
Rock Fracturing

Intensely Fractured	Fractures spaced less than 2 inches apart
Highly Fractured	Fractures spaced 2 inches to 1 foot apart
Moderately Fractured	Fractures spaced 1 foot to 3 feet apart
Slightly Fractured	Fractures spaced 3 feet to 10 feet apart
Massive	Fractures spaced greater than 10 feet apart

Sample Symbols

▮▮	HQ3 Rock Coring

Fracture Shape



Lithology Graphics

Basalt	▥

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

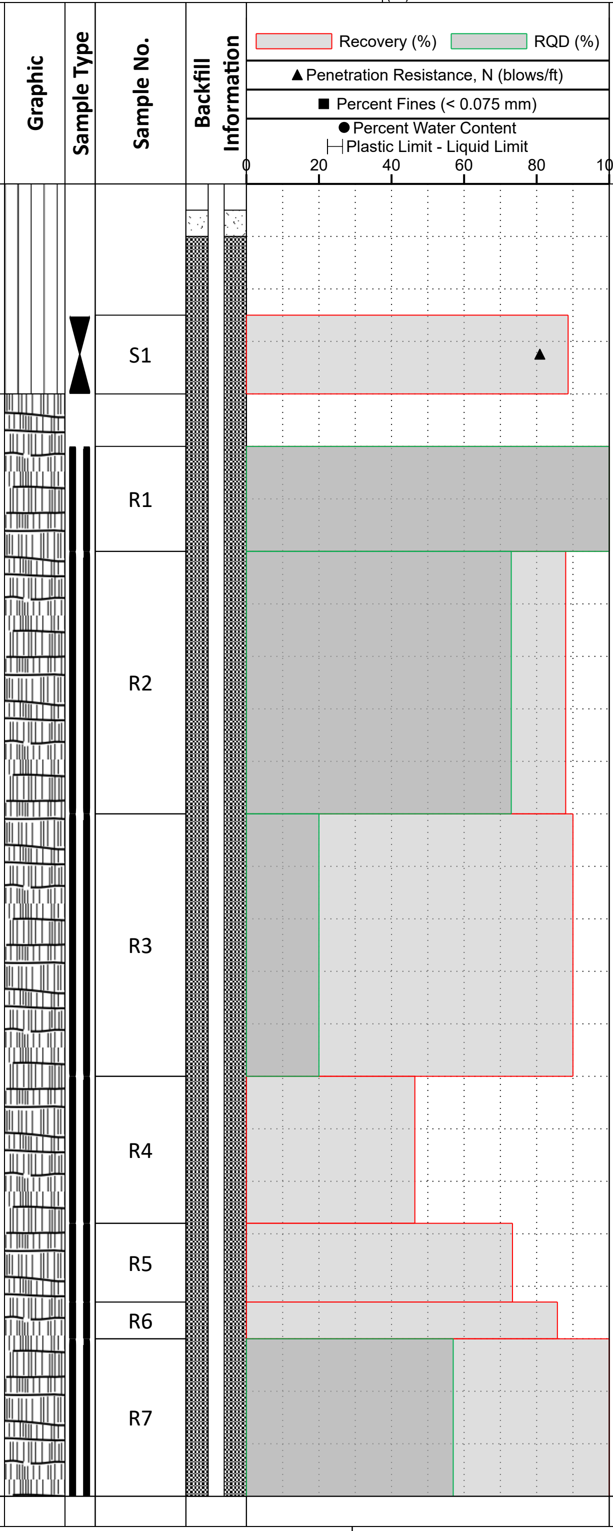
FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-01

Date(s) Drilled	**Dec 03 2018 - Dec 05 2018**	Client	**CDM Smith**	Logged By	**J Fissel**
Checked By	**K Elliott**				
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**45.0 ft.**
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**236.0 ft.**
Location	**Sherwood, OR**	Coordinates	**45.365410 -122.806370**	Hammer Efficiency (%)	

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Sample No.	N Value	Remarks and Tests
	▼	0–4	Very dense, moist, red-brown and gray, Poorly-Graded GRAVEL with cobbles, boulders, clay, and sand (GP-GM); angular gravel-, cobble-, and boulder-size rock fragments, fine-to-coarse sand, low plasticity fines. [Residual Soil]	S1	81	At 1.0 ft bgs, constant drill chattering (rock drill with tricone bit 4 to 5 ft)
231		4–25	BASALT, very strong (R5), fresh, moderately fractured, gray; fine-grained matrix with widely scattered plagioclase phenocrysts up to about 0.2 in. [Columbia River Basalt] *From 5 to 23 ft bgs, joints are ~25°, undulating, smooth; ~50°, planar, smooth; and, ~75°, undulating, rough, iron-oxide stained; typical apertures 0.04-0.08 in.*	R1 R2		PLT est 25,691 psi At 7.0 ft bgs, drill rods obstructed during removal. PLT est 28,285 psi UCS 27,467 psi PLT est 24,651 psi PLT est 32,459 psi
226		10	*Below 9.7 ft bgs, becomes slightly weathered, highly to intensely fractured.*			
		12	*Below 12 ft bgs, core loss generally distributed over runs.*	R3		
221		15	*From 17.6 to 21.3 ft bgs, becomes weak to moderately strong, moderately weathered.*	R4		At 16.5 ft bgs, hole caving, flow contact.
216		20	*From 21.3 to 30 ft bgs, becomes very strong (R5), fresh, highly fractured; fine-grained matrix with scattered plagioclase phenocrysts up to 0.5 in.* *From 23 to 30 ft bgs, joints are ~15° and 30°, undulating, rough, with minor iron-oxide stain; and, near vertical, undulating, rough and iron-oxide stained.* *From 23.3-23.8 ft bgs, zone of light brown and dark brown banded texture.*	R5 R6 R7		



McMILLEN JACOBS ASSOCIATES

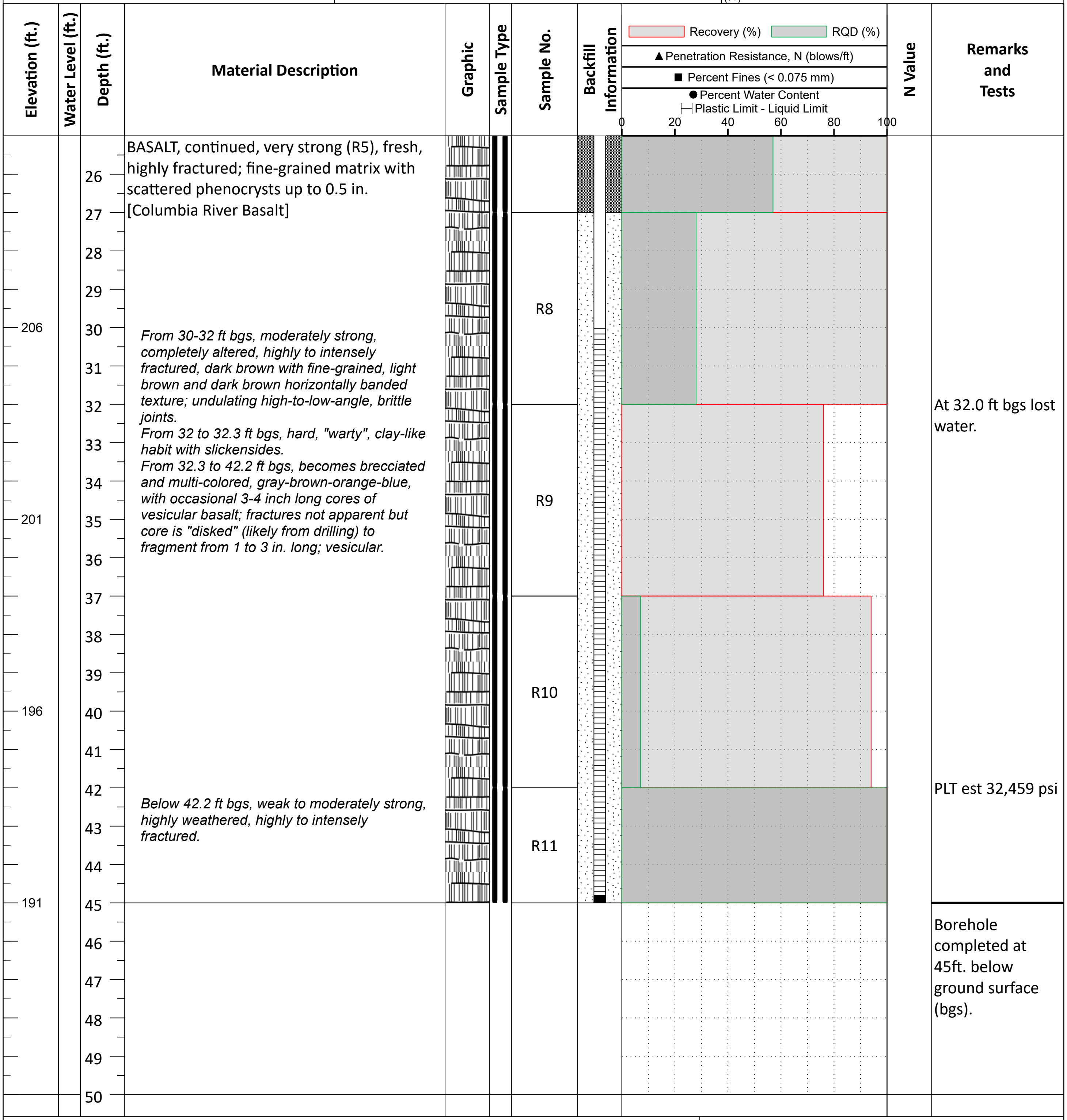
Boring WTP_1.0-B-01

FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-01

Date(s) Drilled	**Dec 03 2018 - Dec 05 2018**	Client	**CDM Smith**	Logged By	**J Fissel**
Checked By	**K Elliott**				
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**45.0 ft.**
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**236.0 ft.**
Location	**Sherwood, OR**	Coordinates	**45.365410 -122.806370**	Hammer Efficiency (%)	



BASALT, continued, very strong (R5), fresh, highly fractured; fine-grained matrix with scattered phenocrysts up to 0.5 in. [Columbia River Basalt]

From 30-32 ft bgs, moderately strong, completely altered, highly to intensely fractured, dark brown with fine-grained, light brown and dark brown horizontally banded texture; undulating high-to-low-angle, brittle joints.
From 32 to 32.3 ft bgs, hard, "warty", clay-like habit with slickensides.
From 32.3 to 42.2 ft bgs, becomes brecciated and multi-colored, gray-brown-orange-blue, with occasional 3-4 inch long cores of vesicular basalt; fractures not apparent but core is "disked" (likely from drilling) to fragment from 1 to 3 in. long; vesicular.

Below 42.2 ft bgs, weak to moderately strong, highly weathered, highly to intensely fractured.

At 32.0 ft bgs lost water.

PLT est 32,459 psi

Borehole completed at 45ft. below ground surface (bgs).

McMILLEN JACOBS ASSOCIATES

Boring WTP_1.0-B-01

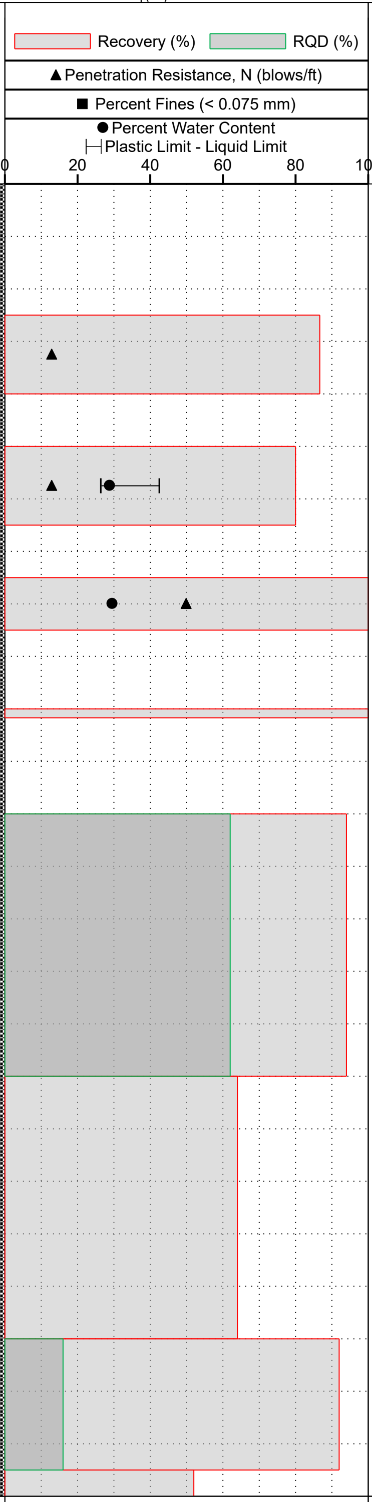
FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-02

Date(s) Drilled	**Dec 11 2018**	Client	**CDM Smith**	Logged By	**K Elliott, A Havekost**	Checked By	**K Elliott**
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**40.0 ft.**		
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**238.0 ft.**		
Location	**Sherwood, OR**	Coordinates	**45.365850 -122.807330**	Hammer Efficiency (%)			

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Sample No.	N Value	Remarks and Tests
		0	Stiff, moist, slightly orange-brown mottled SILT (ML); low-plasticity. [Missoula Flood Deposits]			
		3		S1	13	(3-6-7)
		4	*At 4 ft bgs, grades to yellow to slightly orange-brown with trace roots.*			
233		5				
		6		S2	13	(6-7-6)
		6.5	Stiff to hard, moist, orange-brown, mottled SILT (ML) [Residual Soil]			
		8	*At 8.0 ft bgs, trace fine, gray, weathered Gravel.*	S3	50	At 8.0 ft bgs rods rattle. (6-50/6")
228		10	S-4 is very dense, gray, ROCK CHIPS; with trace orange-brown mottled Silt, trace fine-to-medium sand.	S4	100	(50/2")
		12	BASALT; very strong (R5), fresh to slightly weathered, moderately to highly fractured, medium gray; fine-grained ground mass with diktytaxitic texture and rare plagioclase phenocrysts up to 0.2 in. [Columbia River Basalt] *From 12 to 24 ft bgs, intact joints are ~40° and ~60°, undulating, rough; apertures ≤0.04 in.; iron stains coat surfaces and penetrate up to 0.4 in in some joints.*	R1		At 12.0 ft bgs, switch to rock coring. PLT est 30,889 psi UCS 16,746 psi PLT est 22,741 psi PLT est 29,340 psi
223		15				
		17	*Below 17 ft bgs, strong, fresh to highly weathered, highly to intensely fractured.*	R2		
		19	*Below 19 ft bgs, vesicular with palagonite coatings.*			
218		20				
		22	*From 24.5 to 36 ft bgs, becomes moderately strong to strong, moderately weathered, intensely fractured, slightly vesicular; joints ~40° and ~70°, planar to undulating, rough, iron-oxide stain penetrating up to 0.08 in.*	R3		
		25				



Boring WTP_1.0-B-02

Sheet 1 of 2

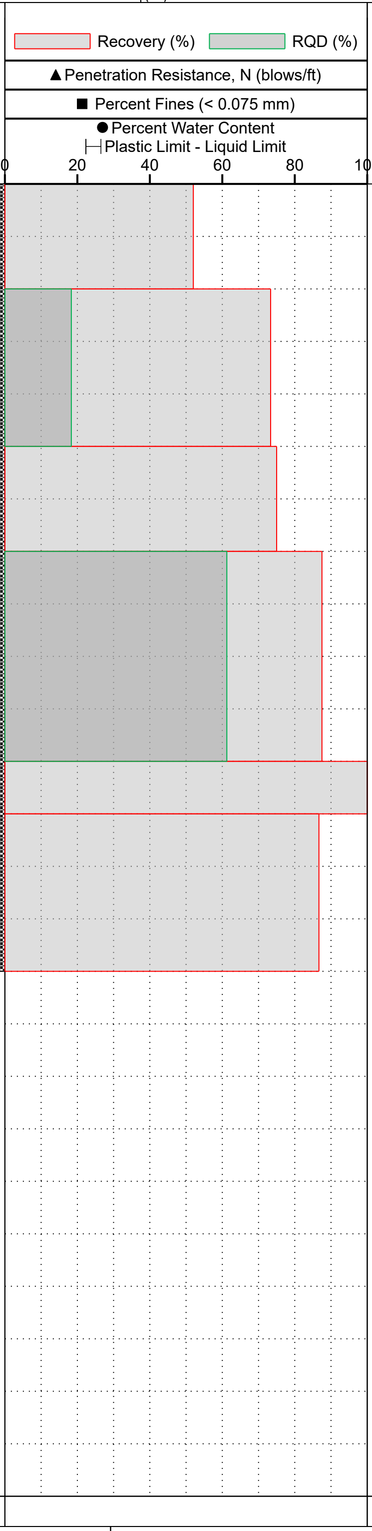
FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-02

Date(s) Drilled	**Dec 11 2018**	Client	**CDM Smith**	Logged By	**K Elliott, A Havekost**
Checked By	**K Elliott**				
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**40.0 ft.**
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**238.0 ft.**
Location	**Sherwood, OR**	Coordinates	**45.365850 -122.807330**	Hammer Efficiency (%)	

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Sample No.	Remarks and Tests
		25–27	BASALT, continued, medium strong to strong, moderately weathered, intensely fractured, slightly vesicular.	R4	PLT est 9,309 psi
		27–30		R5	
208		30–32		R6	
203		32–36	*Below 32.0 ft bgs, plagioclase phenocrysts and glomerocrysts up to 0.3 inches become more abundant.*	R7	
		36–37	BASALT breccia, weak, moderately weathered, highly to intensely fractured, dark grey, fragmented texture with healed fragments, trace vesicles.	R8	
198		37–40	*At 37.0 ft bgs, 4 inch clay layer.*	R9	
193		40–50			Borehole completed at 40ft. below ground surface (bgs).



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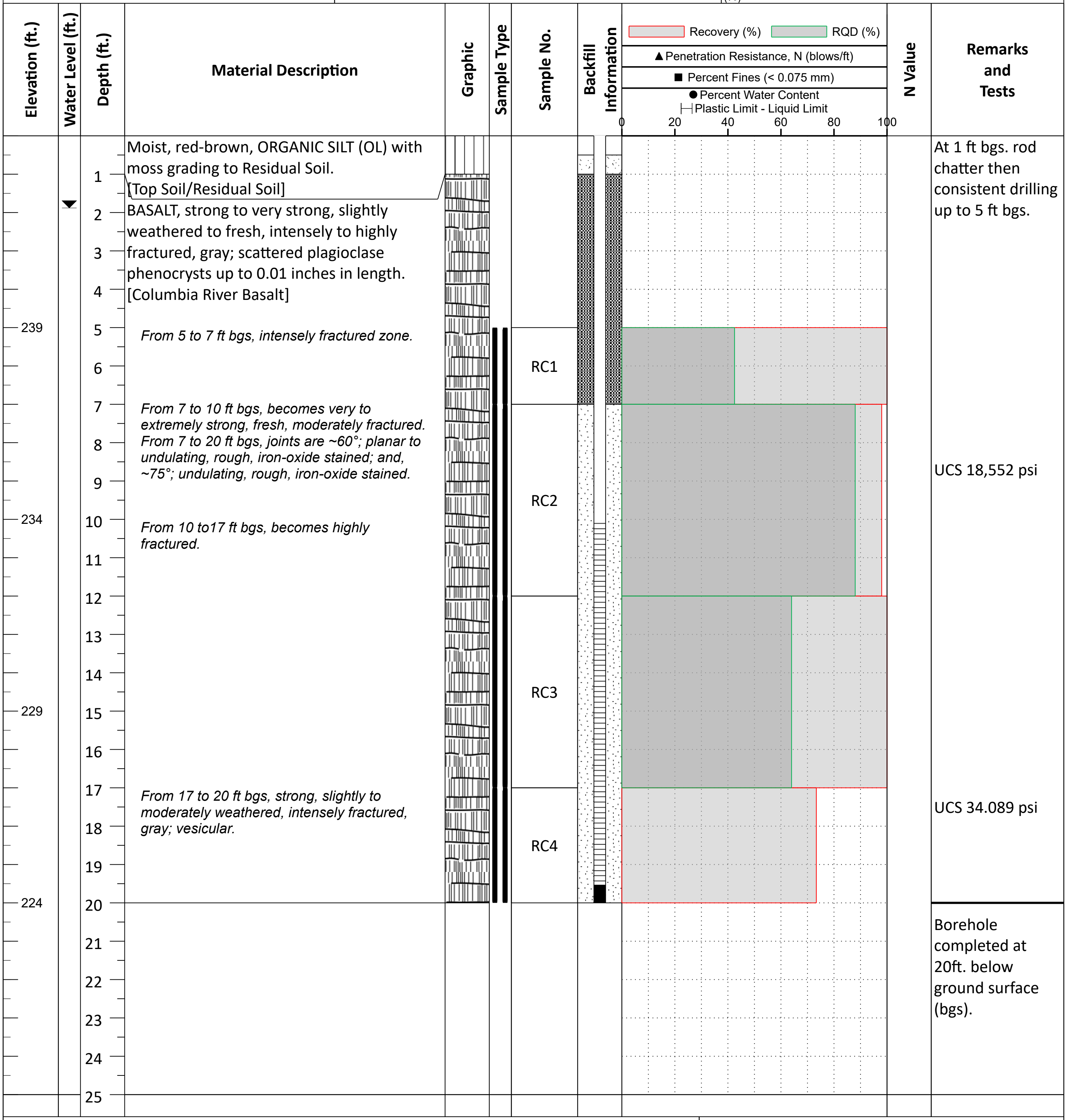
FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-03

Date(s) Drilled	**Feb 07 2019**	Client	**CDM Smith**	Logged By	**A Havekost**
Checked By	**K Elliott**				
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**20.0 ft.**
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type		Ground Surface Elevation/Datum	**244.0 ft.**
Location	**Sherwood, OR**	Coordinates	**45.364850 -122.806852**	Hammer Efficiency (%)	



Moist, red-brown, ORGANIC SILT (OL) with moss grading to Residual Soil. [Top Soil/Residual Soil]

BASALT, strong to very strong, slightly weathered to fresh, intensely to highly fractured, gray; scattered plagioclase phenocrysts up to 0.01 inches in length. [Columbia River Basalt]

From 5 to 7 ft bgs, intensely fractured zone.

From 7 to 10 ft bgs, becomes very to extremely strong, fresh, moderately fractured. From 7 to 20 ft bgs, joints are ~60°; planar to undulating, rough, iron-oxide stained; and, ~75°; undulating, rough, iron-oxide stained.

From 10 to17 ft bgs, becomes highly fractured.

From 17 to 20 ft bgs, strong, slightly to moderately weathered, intensely fractured, gray; vesicular.

Remarks and Tests:

At 1 ft bgs. rod chatter then consistent drilling up to 5 ft bgs.

UCS 18,552 psi

UCS 34.089 psi

Borehole completed at 20ft. below ground surface (bgs).

McMILLEN JACOBS ASSOCIATES

Boring WTP_1.0-B-03

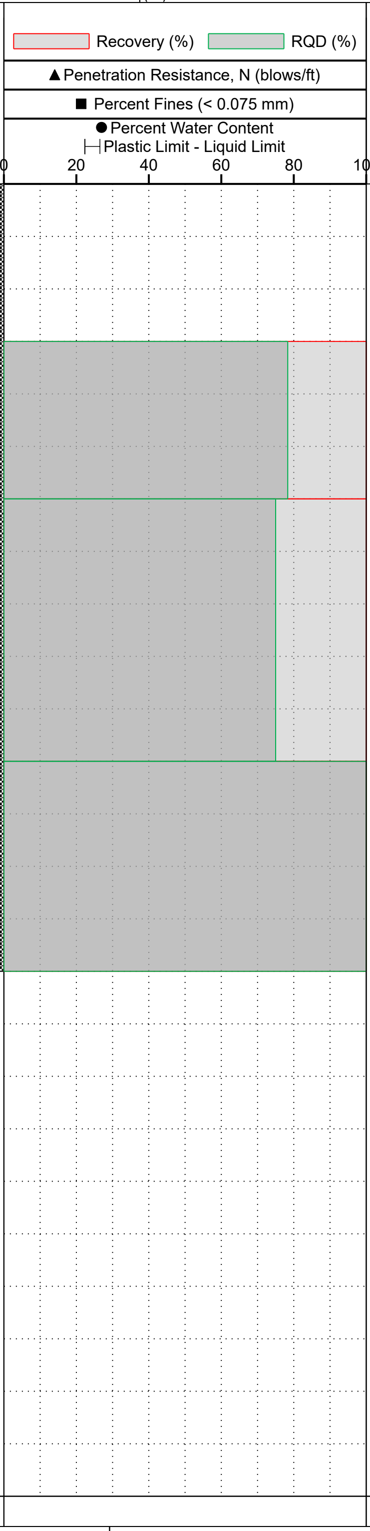
FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-04

Date(s) Drilled	**Feb 05 2019**	Client	**CDM Smith**	Logged By	**A Havekost**	Checked By	**K Elliott**
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**15.0 ft.**		
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type		Ground Surface Elevation/Datum	**269.0 ft.**		
Location	**Sherwood, OR**	Coordinates	**45.363887 -122.808435**	Hammer Efficiency (%)			

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Graphic	Sample Type	Sample No.	Backfill Information	N Value	Remarks and Tests
		0–3	Very wet, brown-black ORGANIC SILT (OL), with roots; grades to Residual Soil. [Top Soil/Residual Soil]						Red-brown drill water and roots returned.
264 (5 ft) 259 (10 ft)		3–15	BASALT, very strong, fresh to slightly weathered, moderately fractured, gray, vesicular; iron-stained joints, scattered plagioclase phenocrysts up to 0.8 in. long. [Columbia River Basalt] *From 3 to 10 ft bgs, highly to intensely fractured; joints range from 35° to 45°, undulating and rough; and, ~75°, heavily iron-oxide stained.* *After 10 ft bgs, joints range from 20° to 30°; planar, undulating, smooth, iron-oxide stained.* *At 10.5 ft bgs, becomes fresh, moderately fractured, gray; diktytaxitic.*			RC1 (3–6) RC2 (6–11) RC3 (11–15)			Rod chatter, gray-brown water and basalt chips. From 10.5-11 ft bgs, dropped core was recovered in Run 3. From 13.5-15 ft, core was stuck in casing, driller hammered and broke core.
254 (15 ft) 249 (20 ft)		15–25							Borehole completed at 15ft. below ground surface (bgs).



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Boring WTP_1.0-B-04

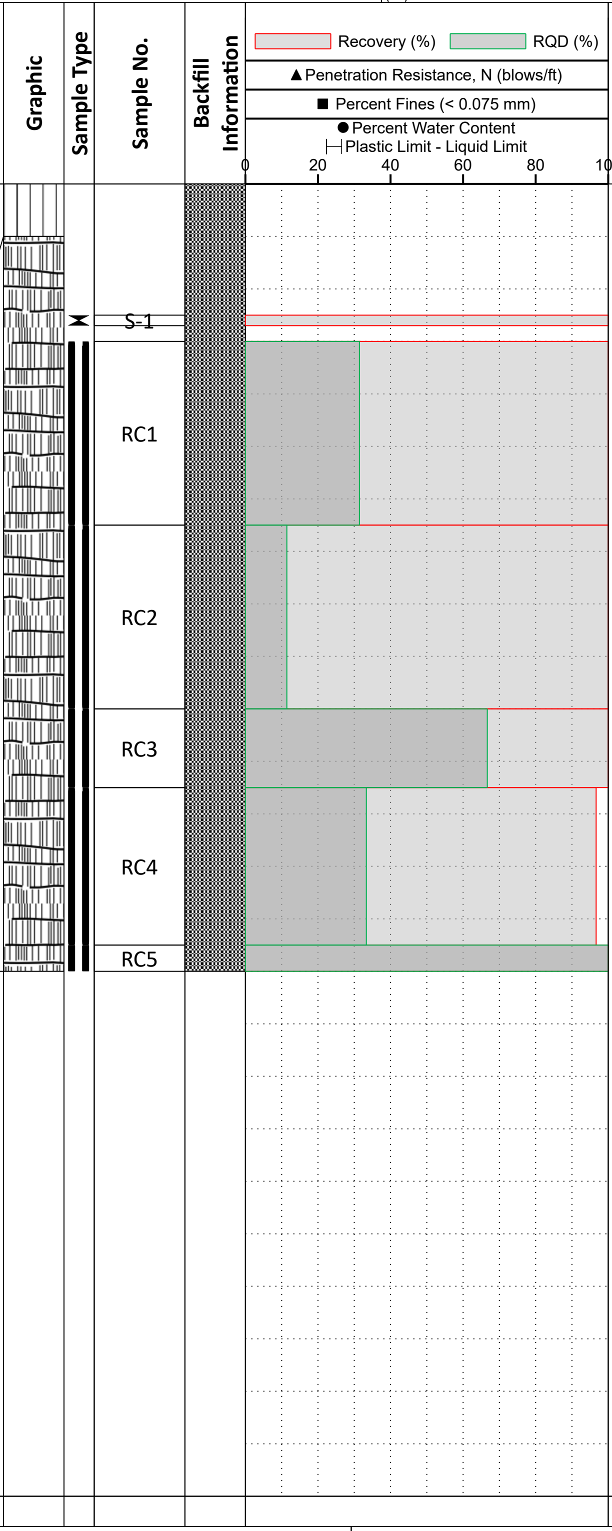
FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-05

Date(s) Drilled	**Feb 06 2019 - Feb 07 2019**	Client	**CDM Smith**	Logged By	**A Havekost**	Checked By	**K Elliott**
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**15.0 ft.**		
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type		Ground Surface Elevation/Datum	**256.0 ft.**		
Location	**Sherwood, OR**	Coordinates	**45.364730 -122.807530**	Hammer Efficiency (%)			

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Sample No.	N Value	Remarks and Tests
		0–1	Moist, brown-black ORGANIC SILT (OL) with roots; grades to red Residual Soil. [Top Soil/Residual Soil]			
		1–15	At 1.0 ft bgs, basalt boulder or block. BASALT, strong to very strong, slightly to moderately weathered, highly to intensely fractured, gray; vesicular; joints are 30° and 60°. [Columbia River Basalt]			At 1 ft bgs, rod chatter, rebound, and kick. Driller hammered 5-inch casing down to 2.5 ft bgs.
		2.5		S-1	100	At 2.5 ft bgs, SPT attempted: 50 blows for 2 inches. At 3 ft bgs, driller switched to rock coring.
251		5		RC1		
		6	*At 6.1 ft bgs, grades to non-vesicular.*			
		7	*From 6.9 to 8.0 ft bgs, intensely fractured zone.*	RC2		
		8	*From 8.3 to 9.5 ft bgs, intensely fractured zone.*			
		9	*At 9.0 ft bgs, becomes very strong, slightly weathered, highly to intensely fractured.*			
246		10	*Below 10 ft bgs, joints range from about 30° to 50°; apertures ≤0.04 in., undulating, iron-oxide stained.*	RC3		At ~9.8 ft bgs, driller loses up to 250 gallons of water; blocked off.
		11.5	*From 11.5 to 11.6 ft bgs, intensely fractured zone.*	RC4		
		13.4	*From 13.4 to 14.4 ft bgs, intensely fractured. zone.*			
241		15		RC5		Borehole completed at 15ft. below ground surface (bgs).
236		20				
		25				



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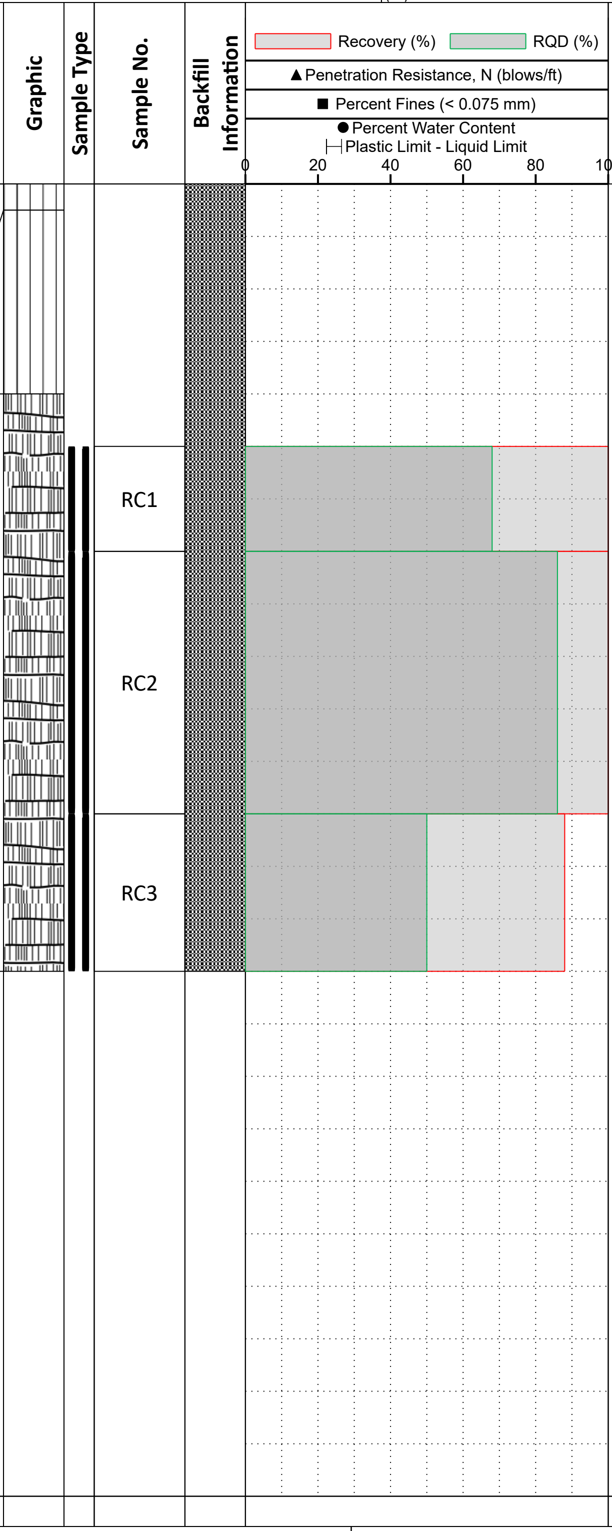
Boring WTP_1.0-B-05

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-06

Date(s) Drilled	**Feb 04 2019**	Client	**CDM Smith**
Logged By	**A Havekost**	Checked By	**K Elliott**
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**
Total Depth of Borehole	**15.0 ft.**	Hole Diameter	**5.00 in.**
Hammer Weight/Drop (lb/in.)/Type		Ground Surface Elevation/Datum	**262.0 ft.**
Location	**Sherwood, OR**	Coordinates	**45.364470 -122.809670**
Hammer Efficiency (%)			

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Remarks and Tests
		0–1	Wet, orange-brown ORGANIC SILT (OL). [Top Soil/Residual Soil]	
		1–4	Silty gravel- and cobble-sized rock fragments grading to highly weathered BASALT. [Residual Soil/Columbia River Basalt]	At 2.5 ft, rod chatter, gravel chips returned. At 4 ft bgs, drilling smooth, rock, changed bit.
257		4–15	BASALT, strong to very strong, slightly weathered to fresh, highly fractured, gray; vesicular, plagioclase phyric; joints range from 20° to 60°; planar, rough, thinly coated with black secondary minerals: iron-oxide stained locally. [Columbia River Basalt] *From 6 to 12 ft bgs, becomes highly to moderately fractured.* *Below 9 ft bgs, becomes nonvesicular.* *Below13 ft bgs, becomes intensely fractured; iron-stained vertical and subvertical joints.*	UCS 23,467 psi
252		10		
247		15		Borehole completed at 15ft. below ground surface (bgs).
242		20		
		25		



N Value

McMILLEN JACOBS ASSOCIATES

Boring WTP_1.0-B-06

Sheet 1 of 1

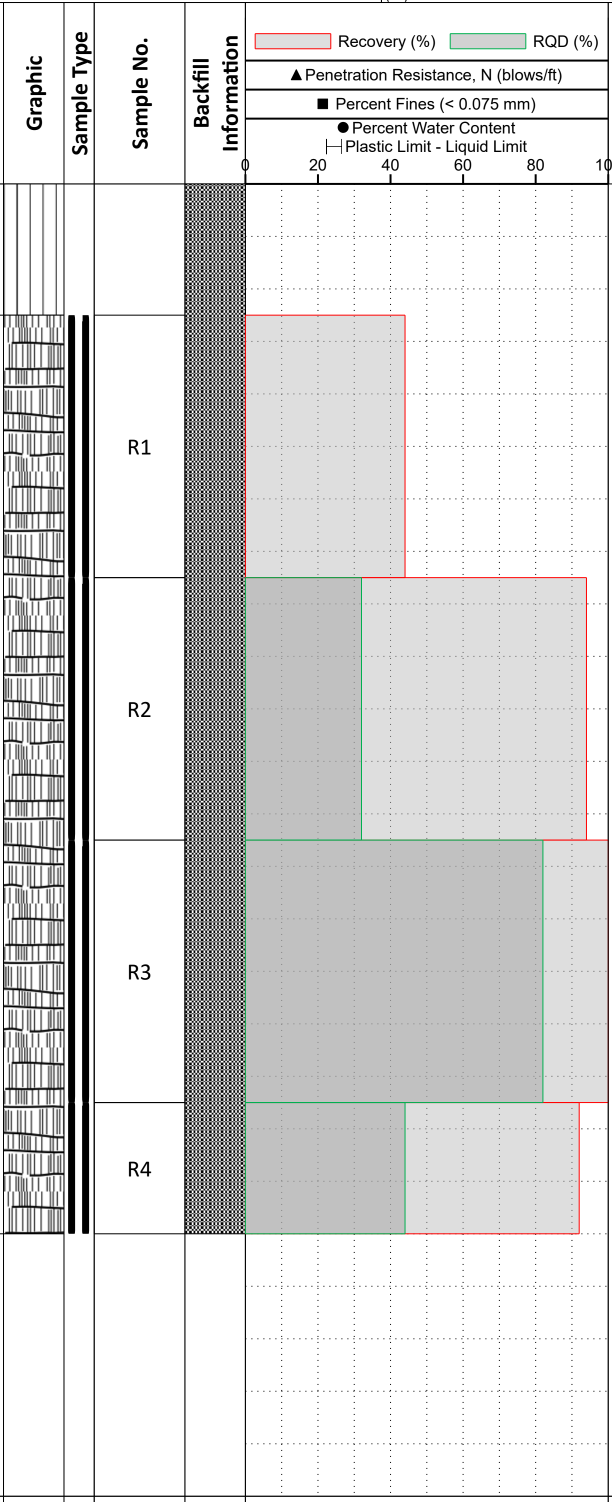
FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-07

Date(s) Drilled	**Dec 10 2018**	Client	**CDM Smith**	Logged By	**K Elliott, A Havekost**
Checked By	**K Elliott**				
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**20.0 ft.**
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type		Ground Surface Elevation/Datum	**253.0 ft.**
Location	**Sherwood, OR**	Coordinates	**45.365120 -122.809510**	Hammer Efficiency (%)	

Elevation (ft.)	Depth (ft.)	Material Description	Sample No.	Remarks and Tests
	0	Moist, dark brown, SILT (ML); organic rich silty soil with scattered angular rock fragments. [Top Soil/Residual Soil]		
	2.5	BASALT, very weak, highly weathered, intensely fractured, with moderately strong, moderately weathered, vesicular core stones with iron-stained joint surfaces. [Columbia River Basalt] *From 2.5 to 12 ft bgs, joint sets at ~ 30° and 60°, planar to undulating, rough, iron-oxide stained.*	R1	At 2.5 ft bgs, switch to rock coring.
248	5			
	7.5	*From 7.5 to 12.5 ft bgs, becomes very strong (R5), slightly to moderately weathered, highly to intensely fractured, gray; fine-grained aphanitic texture.*	R2	At 8.0 ft bgs, drill water returns gray. UCS 18,990 psi
243	10			
	12	*From 12 to 20 ft bgs, joints sets at ~ 40° and 70°, undulating, rough, iron-oxide stained; high angle joints heavily stained.*		
	12.5	*From 12.5 to 20 ft bgs becomes fresh to slightly weathered, highly fractured.*	R3	PLT est 22,409 psi PLT est 27,771 psi UCS 19,466 psi PLT est 22,822 psi
238	15			
	17.5		R4	
233	20			Borehole completed at 20ft. below ground surface (bgs).



N Value

McMILLEN JACOBS ASSOCIATES

Boring WTP_1.0-B-07

Sheet 1 of 1

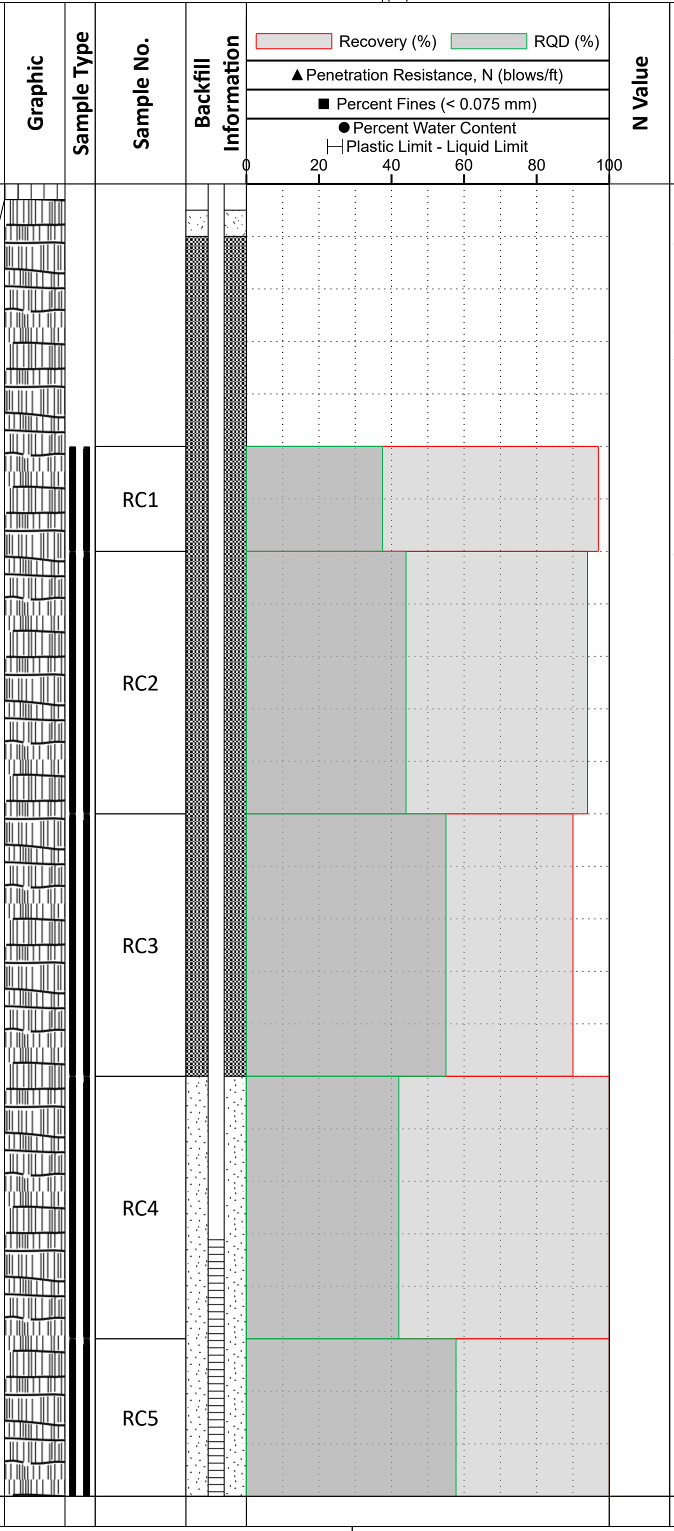
FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-08

Date(s) Drilled	**Feb 05 2019**	Client	**CDM Smith**	Logged By	**A Havekost**	Checked By	**K Elliott**
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**30.0 ft.**		
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type		Ground Surface Elevation/Datum	**276.0 ft.**		
Location	**Sherwood, OR**	Coordinates	**45.364235 -122.809271**	Hammer Efficiency (%)			

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Sample No.	N Value	Remarks and Tests
			Very moist, black ORGANIC SILT (OL). [Top Soil/Residual Soil]			
		1	BASALT, strong to very strong, fresh to moderately weathered, highly to intensely fractured, gray; vesicular with vesicles up to 1 inch elongated horizontally; plagioclase phyric. [Columbia River Basalt]			At ~ 1 ft bgs, drilling water and chips returned is gray; rod chatter. At ~ 2 ft bgs, smooth drilling.
271		5	*From 5 to 14 ft bgs, joints range from about 30° to 40°; undulating, rough.*			
		6		RC1		Losing water, 2-250 gallon tanks.
		9	*From 8.5 to 9 ft bgs, vertical joint, iron-oxide stained.*	RC2		
266		10				
		12	*At 12.2 ft bgs, grades to less vesicular*			
	▼	14	*Below 14 ft bgs, joints range from 40° to 75°; apertures <0.04 to 0.08 in.; undulating, very rough, iron-oxide stained.*	RC3		UCS 11,928 psi
261		15				
		19		RC4		
256		20				
		23	*Below 22.5 ft bgs, very strong, fresh to slightly weathered, highly fractured.*	RC5		
		25				



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Boring WTP_1.0-B-08

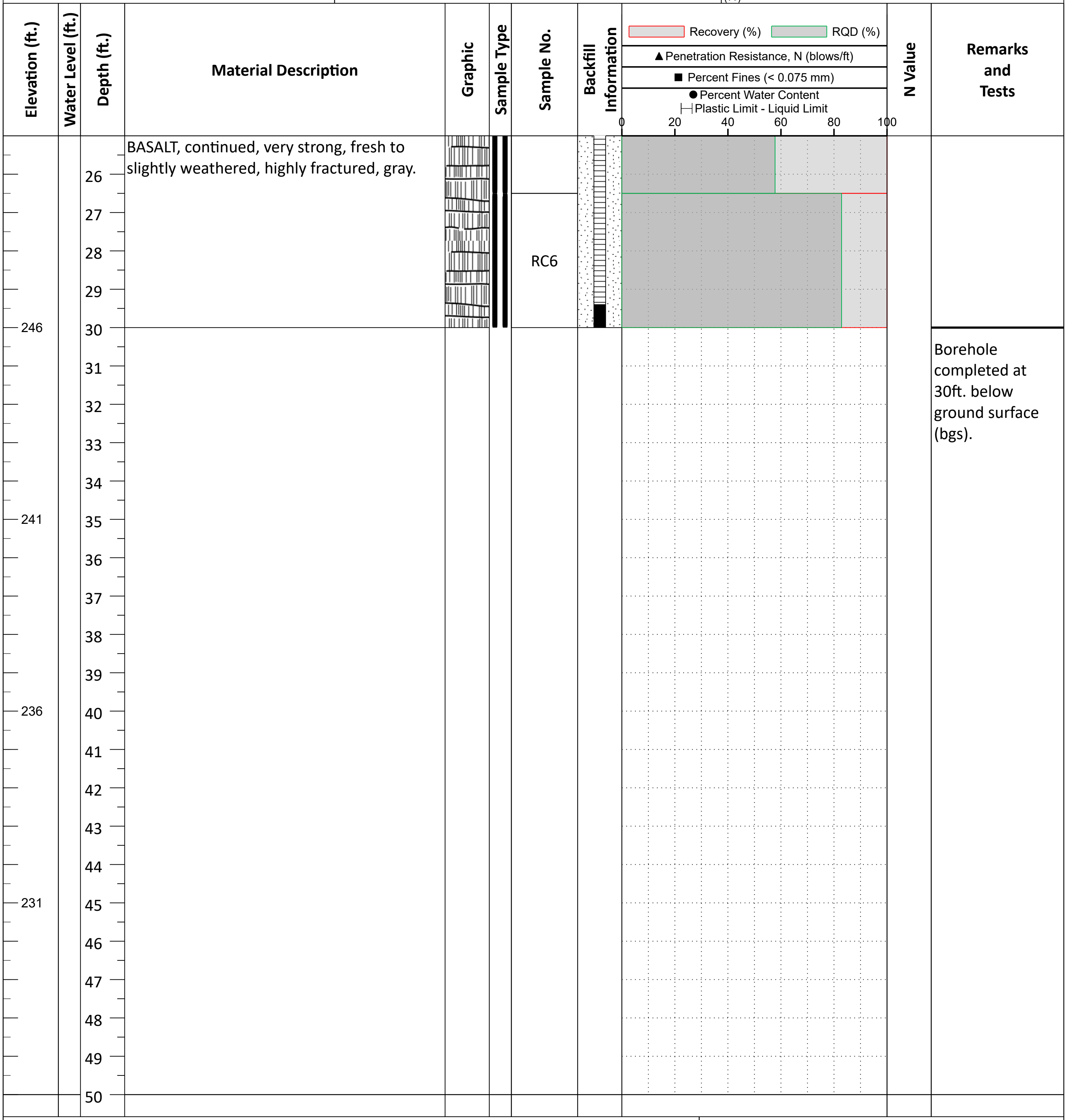
Sheet 1 of 2

FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-08

Date(s) Drilled	**Feb 05 2019**	Client	**CDM Smith**	Logged By	**A Havekost**	Checked By	**K Elliott**
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**30.0 ft.**		
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type		Ground Surface Elevation/Datum	**276.0 ft.**		
Location	**Sherwood, OR**	Coordinates	**45.364235 -122.809271**	Hammer Efficiency (%)			



BASALT, continued, very strong, fresh to slightly weathered, highly fractured, gray.

Borehole completed at 30ft. below ground surface (bgs).

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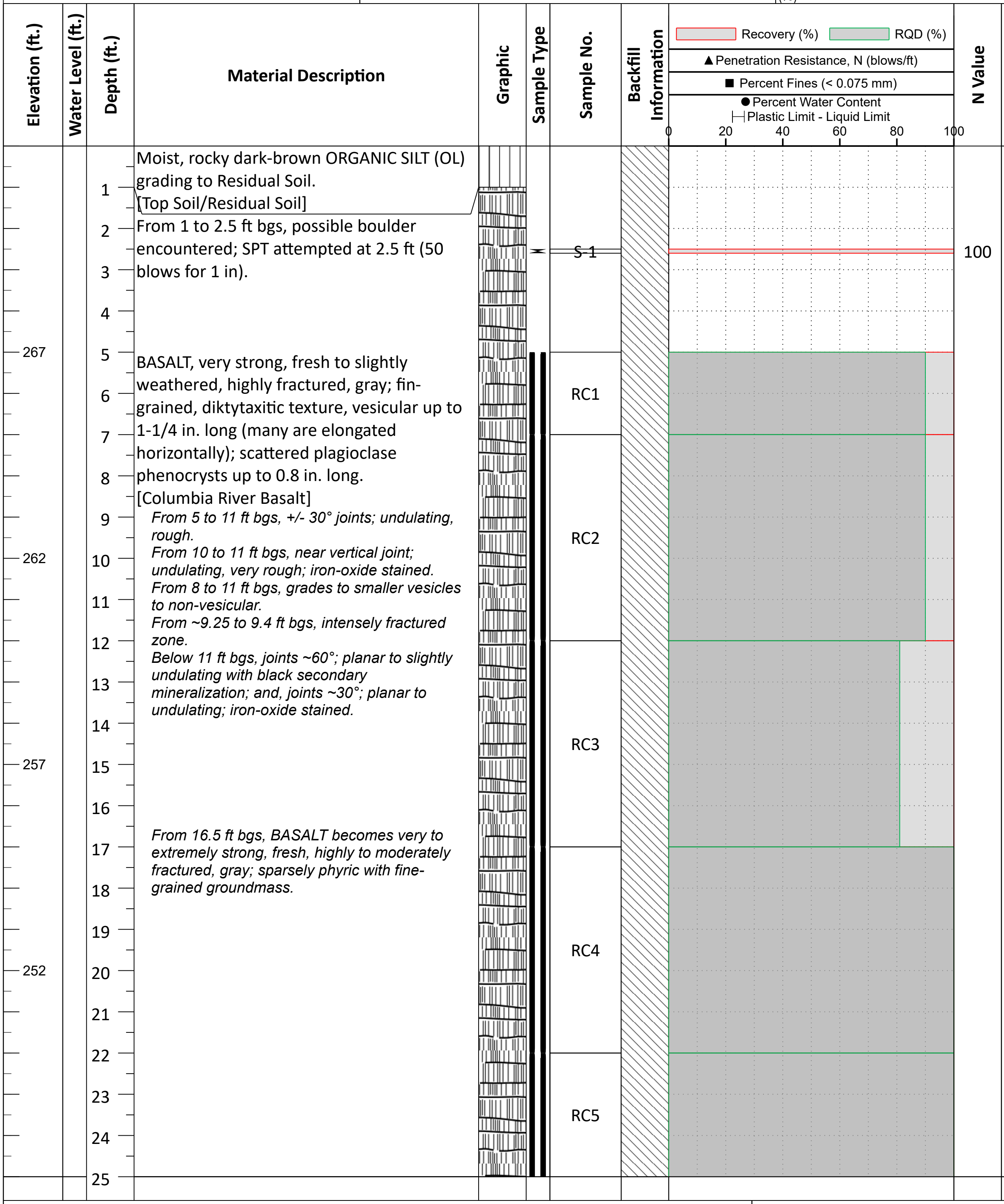
Boring WTP_1.0-B-08

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-09

Date(s) Drilled **Feb 06 2019**
Client **CDM Smith**
Logged By **A Havekost**
Checked By **K Elliott**
Drilling Method/Rig Type **Mud Rotary and HQ Wireline/CME 850 Track Mounted**
Drilling Contractor **Western States Soil Conservation, Inc.**
Total Depth of Borehole **31.5 ft.**
Hole Diameter **5.00 in.**
Hammer Weight/Drop (lb/in.)/Type **140 lb / 30 in / Automatic**
Ground Surface Elevation/Datum **272.0 ft.**
Location **Sherwood, OR**
Coordinates **45.364348 -122.808708**
Hammer Efficiency (%)

Elevation (ft.) | **Water Level (ft.)** | **Depth (ft.)** | **Material Description** | **Graphic** | **Sample Type** | **Sample No.** | **Backfill Information** | **N Value** | **Remarks and Tests**



Moist, rocky dark-brown ORGANIC SILT (OL) grading to Residual Soil.
[Top Soil/Residual Soil]

From 1 to 2.5 ft bgs, possible boulder encountered; SPT attempted at 2.5 ft (50 blows for 1 in).

BASALT, very strong, fresh to slightly weathered, highly fractured, gray; fin-grained, diktytaxitic texture, vesicular up to 1-1/4 in. long (many are elongated horizontally); scattered plagioclase phenocrysts up to 0.8 in. long.
[Columbia River Basalt]

From 5 to 11 ft bgs, +/- 30° joints; undulating, rough.

From 10 to 11 ft bgs, near vertical joint; undulating, very rough; iron-oxide stained.

From 8 to 11 ft bgs, grades to smaller vesicles to non-vesicular.

From ~9.25 to 9.4 ft bgs, intensely fractured zone.

Below 11 ft bgs, joints ~60°; planar to slightly undulating with black secondary mineralization; and, joints ~30°; planar to undulating; iron-oxide stained.

From 16.5 ft bgs, BASALT becomes very to extremely strong, fresh, highly to moderately fractured, gray; sparsely phyric with fine-grained groundmass.

Remarks and Tests

At 1 ft bgs, rod chatter; drill water return is brown.
At 2.5 ft bgs, lost mud; plugged hole with bentonite chips, drilled through, circulation restored.
At 5 ft bgs, drill water returned is gray-brown.
At 7 ft bgs, drill water returned is gray.

McMILLEN JACOBS ASSOCIATES

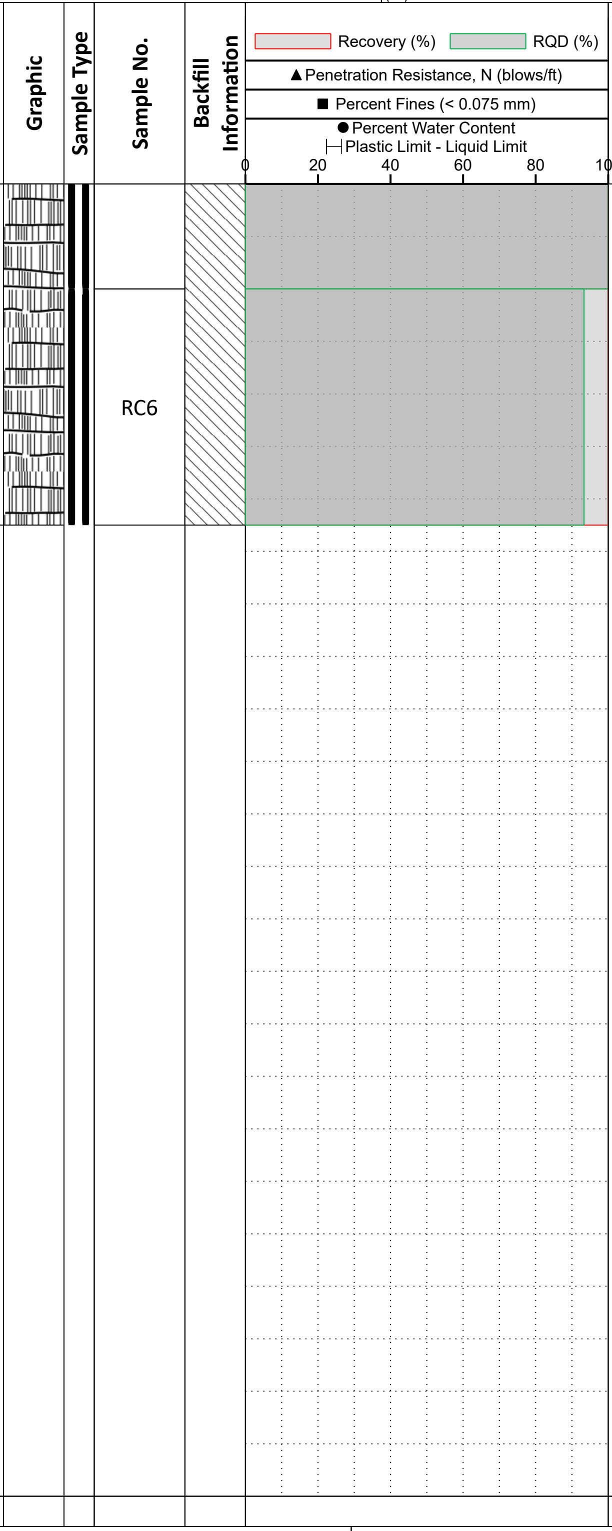
Boring WTP_1.0-B-09

FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-09

Date(s) Drilled	**Feb 06 2019**	Client	**CDM Smith**	Logged By	**A Havekost**	Checked By	**K Elliott**
Drilling Method/ Rig Type	**Mud Rotary and HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**31.5 ft.**		
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**272.0 ft.**		
Location	**Sherwood, OR**	Coordinates	**45.364348 -122.808708**	Hammer Efficiency (%)			



Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	N Value	Remarks and Tests
		26	BASALT, continued, very to extremely strong, fresh, highly to moderately fractured, gray; sparsely phyric with fine-grained groundmass. [Columbia River Basalt]		UCS 18,249 psi
		27			UCS 27,829 psi
		28			
		29			
242		30			UCS 22,316 psi
		31			
		32			Borehole completed at 31.5ft. below ground surface (bgs).
		33			
		34			
237		35			
		36			
		37			
		38			
		39			
232		40			
		41			
		42			
		43			
		44			
227		45			
		46			
		47			
		48			
		49			
		50			

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Boring WTP_1.0-B-09

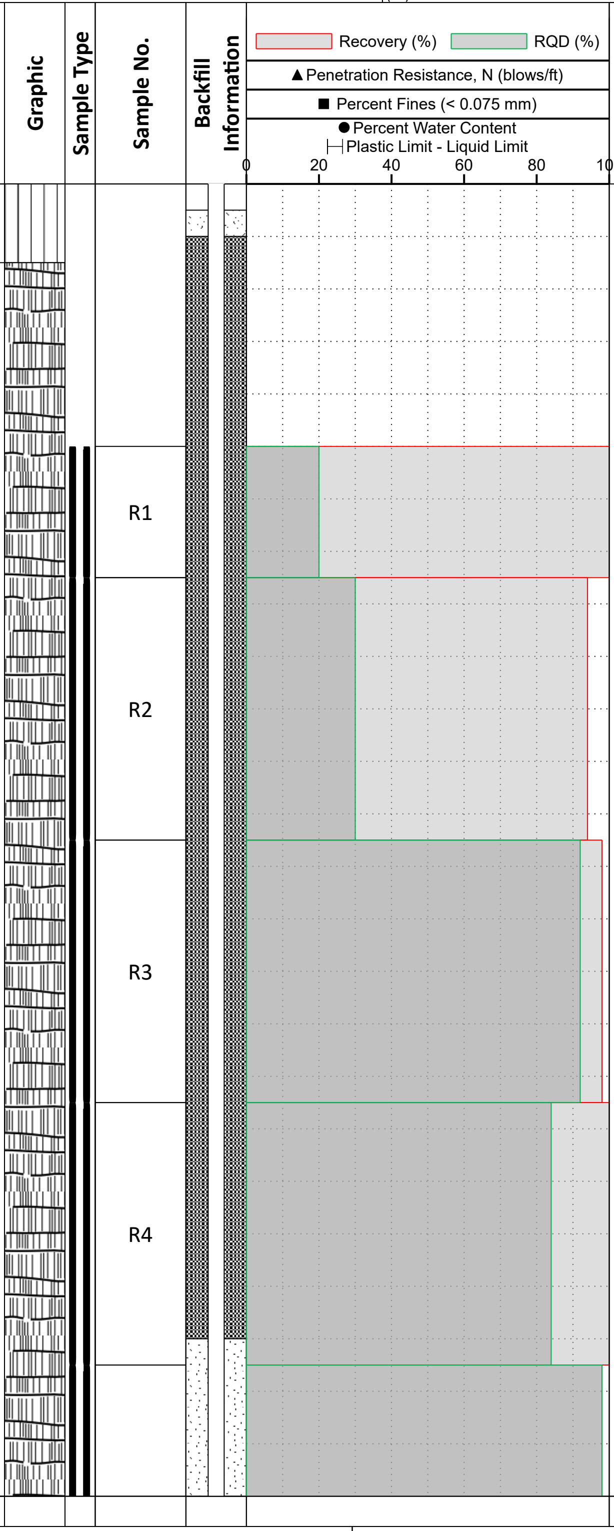
FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-10

Date(s) Drilled	**Dec 07 2018**	Client	**CDM Smith**	Logged By	**F Sariosseri, J Fissel**	Checked By	**K Elliott**
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**40.0 ft.**		
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type		Ground Surface Elevation/Datum	**265.0 ft.**		
Location	**Sherwood, OR**	Coordinates	**45.364530 -122.808590**	Hammer Efficiency (%)			

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Sample No.	Remarks and Tests
		0–1.5	Moist, brown SILT (ML); some cobbles. [Missoula Flood Deposits]		
		1.5	BASALT		
260		5	From 5 to 12.5 ft bgs, very strong, slightly weathered, highly fractured, gray; sparsely vesicular, vesicular, plagioclase phyric with glomerocrysts up to 0.5 in.; joints range from 30 ° to 50°, undulating, rough, iron-oxide stained, apertures 0.04 to 0.08 in. [Columbia River Basalt]	R1	
		7.5		R2	
255		11	*At 11 ft bgs, vertical joint, undulating, rough, iron-oxide stained, aperture ≤0.04 in.* *From 11.0-37.5 ft bgs, vesicles absent.*		
		12.5	*Below 12.5 ft, becomes moderately to highly fractured; phenocrysts rare below 13 ft.* *Below 13 ft bgs, 65° joints set about every 3 ft is planar to undulating, iron-oxide stained; another set at ±25°, undulating, rough, and iron-oxide stained.*	R3	
250		15–17			PLT est 27,004 psi PLT est 24,247 psi UCS 31,235 psi
245		17.5		R4	
		22.5–25			PLT est 22,968 psi



N Value

Boring WTP_1.0-B-10

Sheet 1 of 2

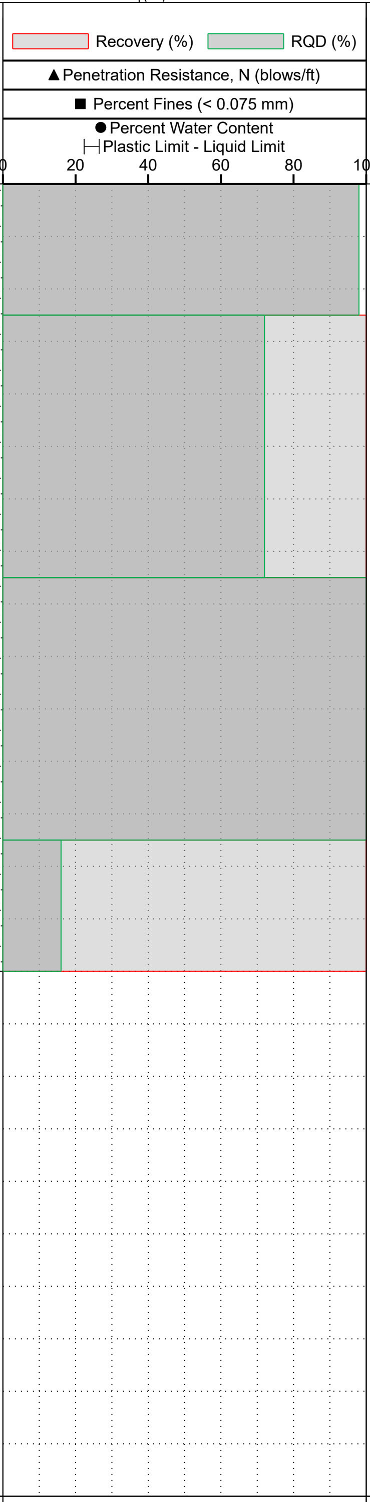
Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-10

Date(s) Drilled	**Dec 07 2018**	Client	**CDM Smith**	Logged By	**F Sariosseri, J Fissel**	Checked By	**K Elliott**
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**40.0 ft.**		
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type		Ground Surface Elevation/Datum	**265.0 ft.**		
Location	**Sherwood, OR**	Coordinates	**45.364530 -122.808590**	Hammer Efficiency (%)			

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Sample No.	Remarks and Tests
		26	BASALT, continued, very strong, fresh to slightly weathered, highly to moderately fractured, gray.	R5	PLT est 25,925 psi UCS 17,446 psi
		27			
		28			
		29			
235		30		R6	
		31			
		32			
		33			
	▼	34			
230	▼	35		R7	
		36			
		37			
		38	*At 37.5 ft, grades to highly fractured; vesicular.*		
		39		R8	
225		40			Borehole completed at 40ft. below ground surface (bgs).
		41			
		42			
		43			
		44			
220		45			
		46			
		47			
		48			
		49			
		50			

Graphic | Sample Type | Backfill Information | N Value

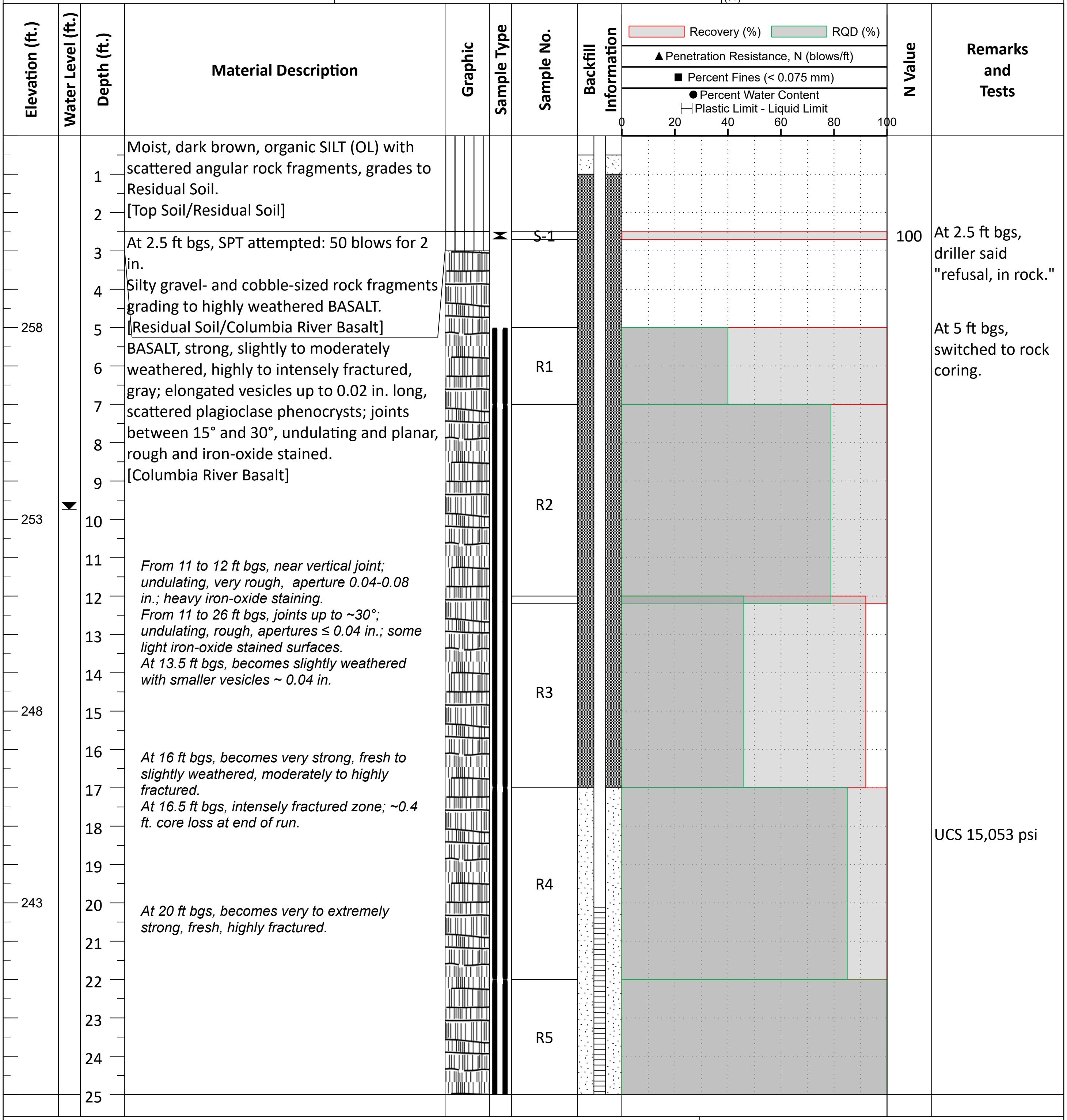


FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-11

Date(s) Drilled	**Feb 08 2019**	Client	**CDM Smith**	Logged By	**A Havekost**	Checked By	**K Elliott**
Drilling Method/ Rig Type	**Mud Rotary and HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**35.0 ft.**		
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**263.0 ft.**		
Location	**Sherwood, OR**	Coordinates	**45.365089 -122.808956**	Hammer Efficiency (%)			



Material Description

Moist, dark brown, organic SILT (OL) with scattered angular rock fragments, grades to Residual Soil.
[Top Soil/Residual Soil]

At 2.5 ft bgs, SPT attempted: 50 blows for 2 in.
Silty gravel- and cobble-sized rock fragments grading to highly weathered BASALT.
[Residual Soil/Columbia River Basalt]

BASALT, strong, slightly to moderately weathered, highly to intensely fractured, gray; elongated vesicles up to 0.02 in. long, scattered plagioclase phenocrysts; joints between 15° and 30°, undulating and planar, rough and iron-oxide stained.
[Columbia River Basalt]

From 11 to 12 ft bgs, near vertical joint; undulating, very rough, aperture 0.04-0.08 in.; heavy iron-oxide staining.
From 11 to 26 ft bgs, joints up to ~30°; undulating, rough, apertures ≤ 0.04 in.; some light iron-oxide stained surfaces.
At 13.5 ft bgs, becomes slightly weathered with smaller vesicles ~ 0.04 in.

At 16 ft bgs, becomes very strong, fresh to slightly weathered, moderately to highly fractured.
At 16.5 ft bgs, intensely fractured zone; ~0.4 ft. core loss at end of run.

At 20 ft bgs, becomes very to extremely strong, fresh, highly fractured.

Sample No.	N Value	Remarks and Tests
S-1	100	At 2.5 ft bgs, driller said "refusal, in rock."
R1		At 5 ft bgs, switched to rock coring.
R2		
R3		
R4		UCS 15,053 psi
R5		

McMILLEN JACOBS ASSOCIATES

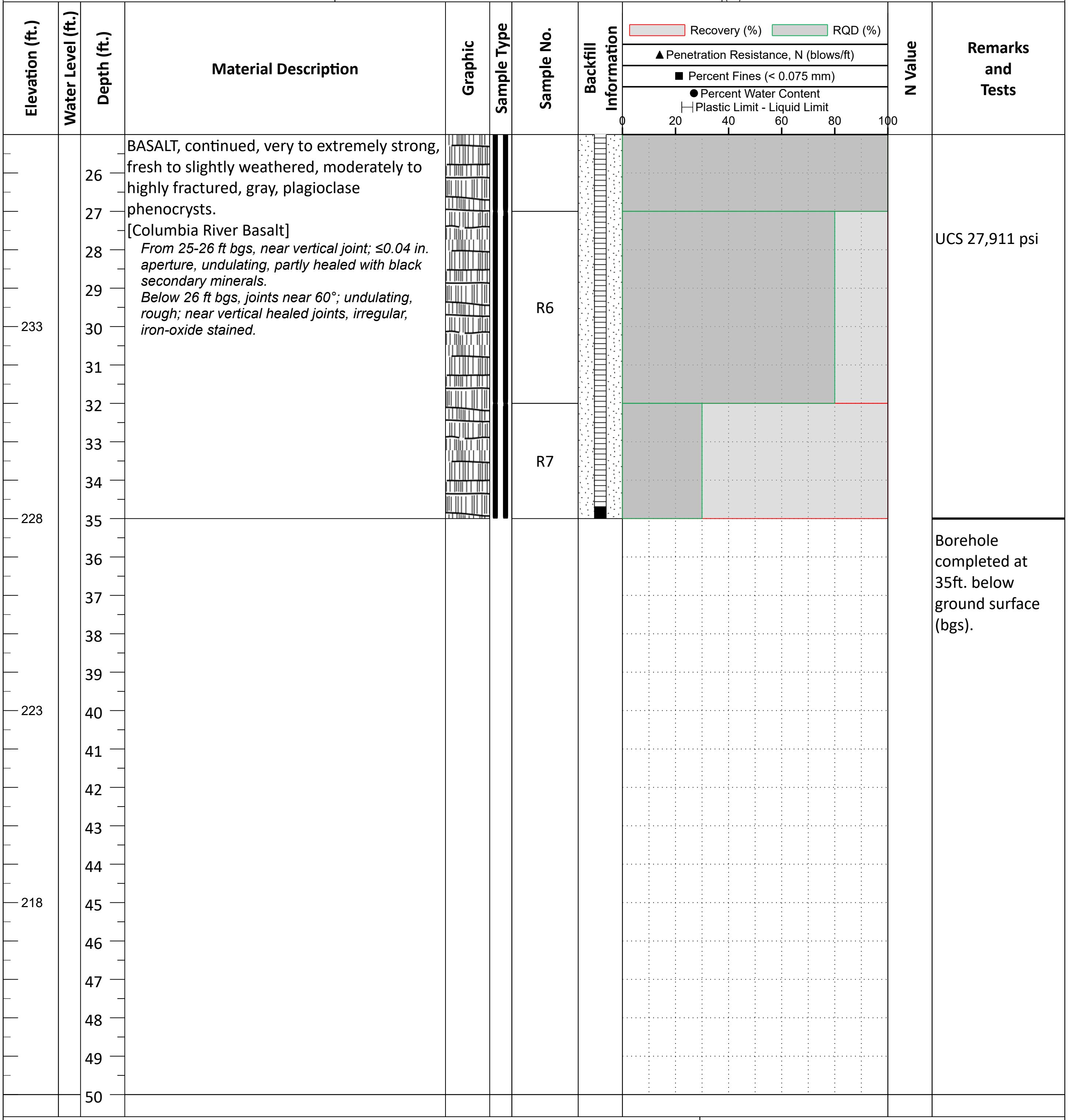
Boring WTP_1.0-B-11

FOR LAND USE PERMITTING (EXHIBIT B)

Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-11

Date(s) Drilled	**Feb 08 2019**	Client	**CDM Smith**	Logged By	**A Havekost**	Checked By	**K Elliott**
Drilling Method/ Rig Type	**Mud Rotary and HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**35.0 ft.**		
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**263.0 ft.**		
Location	**Sherwood, OR**	Coordinates	**45.365089 -122.808956**	Hammer Efficiency (%)			



BASALT, continued, very to extremely strong, fresh to slightly weathered, moderately to highly fractured, gray, plagioclase phenocrysts.
[Columbia River Basalt]

From 25-26 ft bgs, near vertical joint; ≤0.04 in. aperture, undulating, partly healed with black secondary minerals.
Below 26 ft bgs, joints near 60°; undulating, rough; near vertical healed joints, irregular, iron-oxide stained.

UCS 27,911 psi

Borehole completed at 35ft. below ground surface (bgs).

McMILLEN JACOBS ASSOCIATES

Boring WTP_1.0-B-11

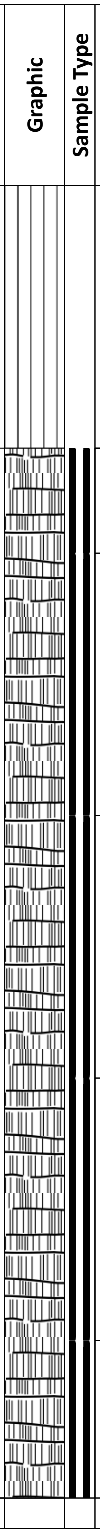
FOR LAND USE PERMITTING (EXHIBIT B)

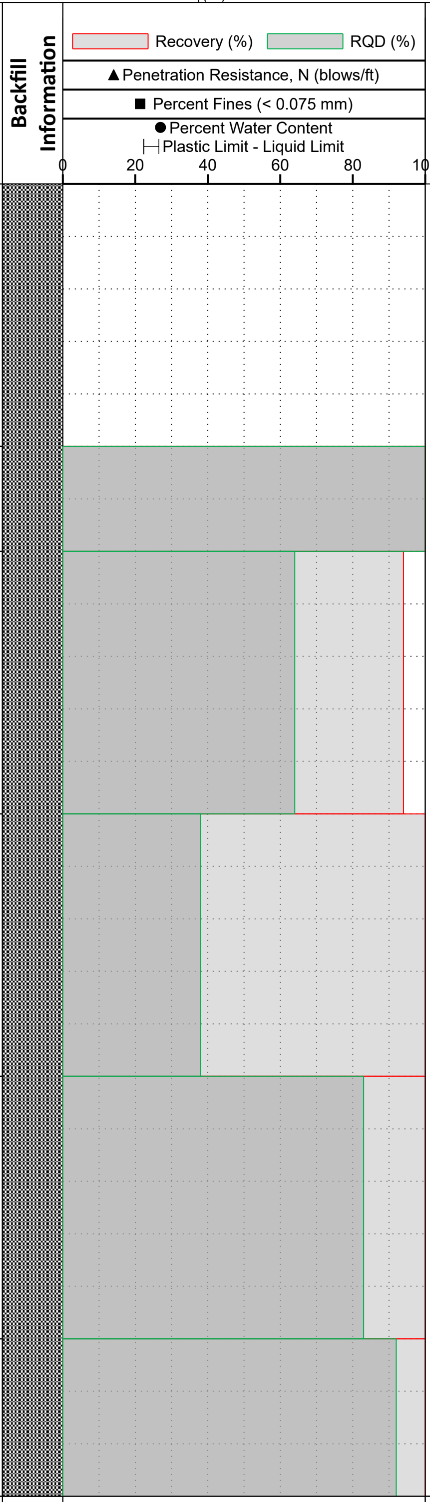
Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-12

Date(s) Drilled	**Dec 05 2018 - Dec 06 2018**	Client	**CDM Smith**	Logged By	**J Fissel**
Checked By	**K Elliott**				
Drilling Method/ Rig Type	**HQ Wireline/CME 850 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**30.0 ft.**
Hole Diameter	**5.00 in.**	Hammer Weight/Drop (lb/in.)/Type		Ground Surface Elevation/Datum	**257.0 ft.**
Location	**Sherwood, OR**	Coordinates	**45.365380 -122.807860**	Hammer Efficiency (%)	

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Sample Type	Sample No.	N Value	Remarks and Tests
		0	Moist, brown SILT (ML); with subangular cobbles up to 10 inches. [Top Soil/Residual Soil]				Drilled using 3-4 tanks (250 gallons/ea.). Started with 4 1/2" casing, advanced to 5 ft bgs.
		1					
		2					
		3					
		4					
252		5	BASALT, very strong (R5), slightly weathered, highly to moderately fractured, gray; joint sets at 15°, 30°, and 40°, planar, rough, iron-oxide stained, aperture ≤0.04 in.; another joint set vertical, undulating, rough, and iron stained, aperture 0.08-0.12 in., minimal penetration of iron-oxide stain into the rock. [Columbia River Basalt]		S1	100	At 5.0 ft bgs. no SPT recovery, switch to coring.
		6			R1		
		7					
		8					UCS 21,764 psi PLT est 22,237 psi
		9					PLT est 30,339 psi
247		10			R2		
		11					
		12					
		13					PLT est 16,586 psi
		14			R3		
242		15					
		16					
		17					
		18					
		19			R4		
237		20					
		21					UCS 19,622 psi
		22					
		23			R5		
		24					
		25					



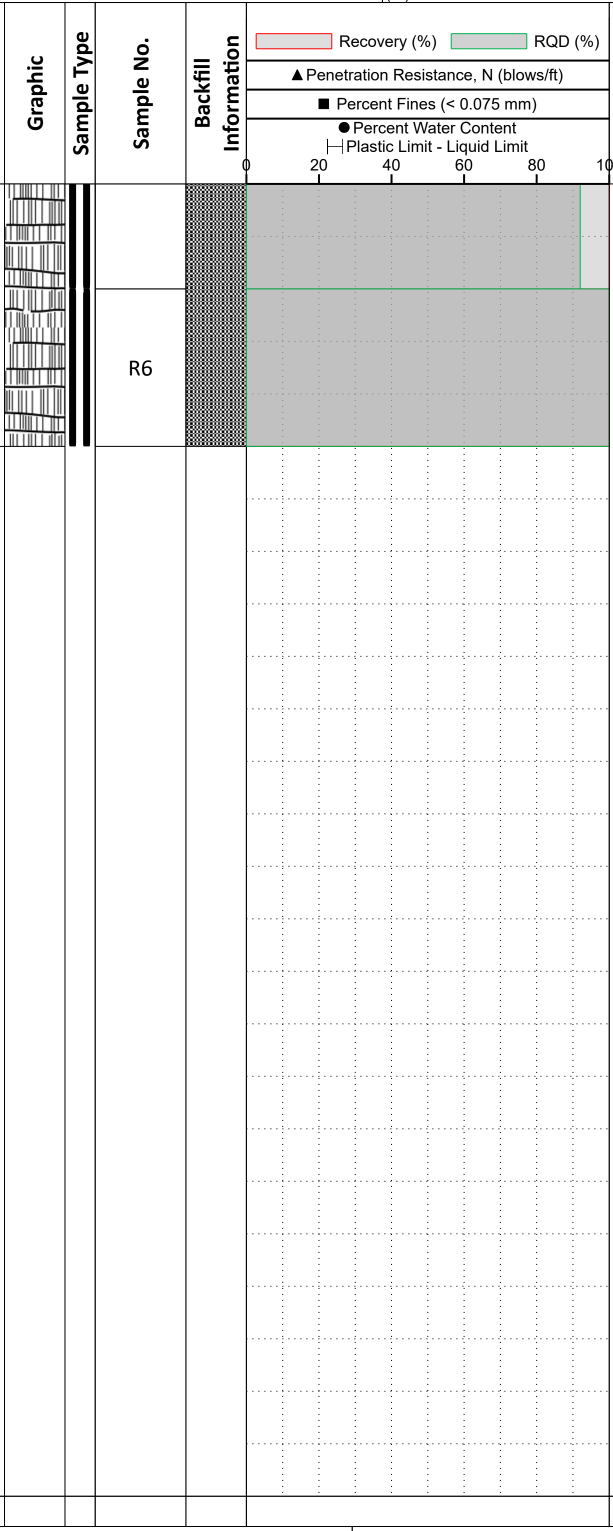


Project: WWSP_WTP_1.0
Project Location: Sherwood, OR
Project Number: 5887.0

Log of Boring WTP_1.0-B-12

Date(s) Drilled **Dec 05 2018 - Dec 06 2018**
Client **CDM Smith**
Logged By **J Fissel**
Checked By **K Elliott**
Drilling Method/ Rig Type **HQ Wireline/CME 850 Track Mounted**
Drilling Contractor **Western States Soil Conservation, Inc.**
Total Depth of Borehole **30.0 ft.**
Hole Diameter **5.00 in.**
Hammer Weight/Drop (lb/in.)/Type
Ground Surface Elevation/Datum **257.0 ft.**
Location **Sherwood, OR**
Coordinates **45.365380 -122.807860**
Hammer Efficiency (%)

Elevation (ft.)	Water Level (ft.)	Depth (ft.)	Material Description	Sample No.	Remarks and Tests
227		26–30	BASALT, very strong (R5), slightly weathered, highly to moderately fractured, gray; joint sets at 15°, 30°, and 40°, planar, rough, iron-oxide stained, aperture ≤0.04 in.; another joint set vertical, undulating, rough, and iron stained, aperture 0.08-0.12 in., minimal penetration of iron-oxide stain into the rock. [Columbia River Basalt]	R6	PLT est 25,824 psi
222, 217, 212		30–50			Borehole completed at 30ft. below ground surface (bgs).



N Value

Probe Hole Exploration Points - Summary Field Logs

Date: All probe holes were drilled in one shift on December 7, 2018.
Contractor: McCallum Rock Drilling and Blasting
Equipment: Furukawa HCR 900 ES (air-track drill)

Logged by: K. Elliott
Weather: 450 - 520 F; clear, becoming overcast

Probe Hole P-1

Depth		Observations		Interpretation
From	**To**	**Driller Log**	**MJA Log**	
0	1	Dirt	Brown Soil	Top Soil
1	2			
2	3		Gray-brown	Residual Soil
3	4			
4	5			
5	6			
6	7	Brown-gray mix	Gray-brown	Moderately weathered rock
7	8			
8	9			
9	10			
10	11			
11	12	Medium gray		
12	13	Brown-gray		
13	14			
14	15			
15	16			
16	17			
17	18			
18	19	Dirt		
19	20			
20	21		Dark brown	Moderate to highly weathered rock
21	22			
22	23			
23	24			
24	25			
25	26	Brown-gray		
26	27			
27	28			
28	29		Dark gray	Chilled rock; basal contact?
29	30	Medium gray		
30	31			
31	32		Gray-brown	Moderately weathered rock
32	33			
33	34			
34	35			

Probe Hole P-2

Depth		Observations		Interpretation
From	**To**	**Driller Log**	**MJA Log**	
0	1	Dirt	Brown	Top Soil grading to Residual Soil
1	2			
2	3			
3	4	Brown-gray mix		
4	5			
5	6		Light gray	Moderately weathered rock
6	7			
7	8			
8	9			
9	10			
10	11			
11	12			
12	13			
13	14			
14	15			
15	16			
16	17			
17	18			
18	19		Red brown	Weathered
19	20		Light gray	Moderately weathered rock
20	21			
21	22			
22	23	Medium gray		
23	24			
24	25			
25	26			
26	27			
27	28			
28	29			
29	30			
30	31			
31	32			
32	33			
33	34	Soft Black		
34	35		Sl. Red-brown	Flow contact?

Probe Hole Exploration Points - Summary Field Logs

Probe Hole P-3

Depth		Observations		Interpretation
From	To	Driller Log	MJA Log	
0	1	Dirt	Gray rock; firm but not hard	Top Soil
1	2			
2	3	Medium gray		Moderately weathered rock
3	4			
4	5			
5	6			
6	7			
7	8			
8	9			
9	10			
10	11			
11	12		Dark gray rock	Slightly weathered rock
12	13			
13	14			
14	15			
15	16			
16	17			
17	18			
18	19		Light gray rock	Slightly to moderately weathered rock
19	20			
20	21			
21	22			
22	23			
23	24			
24	25			
25	26			
26	27			
27	28			
28	29			
29	30			
30	31			
31	32			
32	33			
33	34			
34	35			

Probe Hole P-4

Depth		Observations		Interpretation
From	To	Driller Log	MJA Log	
0	1	Dirt	Brown to yellow-brown soil with gray, soft broken rock	Possible flood silt grading to residual soil
1	2			
2	3			
3	4			
4	5			
5	6	Brown		Residual soil grading to weathered to relatively fresh rock Relatively fresh rock
6	7			
7	8			
8	9			
9	10		Gray rock; Turning light gray at 21 ft Turning gray-brown at 28 ft	
10	11			
11	12			
12	13			
13	14	Medium gray		
14	15			
15	16			
16	17			
17	18			
18	19			
19	20			
20	21			
21	22			
22	23			
23	24			
24	25			
25	26			
26	27			
27	28			
28	29			Relatively fresh rock with iron-stained joints
29	30			
30	31			
31	32			
32	33			
33	34			
34	35			

Probe Hole Exploration Points - Summary Field Logs

Probe Hole P-5

Depth		Observations		Interpretation
From	To	Driller Log	MJA Log	
0	1	Dirt	Moss w/ trace Top Soil at surface; light gray, hard	Top Soil grading to Residual Soil; numerous boulders on surface
1	2			
2	3			
3	4			
4	5			
5	6			
6	7	Medium gray		Slightly weathered rock
7	8			
8	9			
9	10			
10	11			
11	12			
12	13			
13	14			
14	15			
15	16			
16	17			
17	18			
18	19			
19	20			
20	21			
21	22			
22	23			
23	24		Brown	Weathered
24	25		Medium gray	Slightly weathered rock
25	26			
26	27			
27	28			
28	29			
29	30			
30	31		Dark gray	Possible chilled zone; basal contact?
31	32			
32	33			
33	34		Darker gray	
34	35	Black-gray		

Probe Hole P-6

Depth		Observations		Interpretation
From	To	Driller Log	MJA Log	
0	1	Dirt	Brown soil	Top soil grading to residual soil
1	2			
2	3			
3	4			
4	5			
5	6			
6	7	Medium gray	Light gray fine to chunky rock	slightly to moderately weathered, highly to intensely fractured rock
7	8			
8	9			
9	10			
10	11			
11	12			
12	13			
13	14			
14	15			
15	16			
16	17			
17	18			
18	19			
19	20			
20	21			
21	22			
22	23			
23	24			
24	25			
25	26			
26	27			
27	28			
28	29			
29	30			
30	31			
31	32			
32	33			
33	34			
34	35			

Probe Hole Exploration Points - Summary Field Logs

Probe Hole P-7

Depth		Observations		Interpretation
From	To	Driller Log	MJA Log	
0	1	Dirt	Yellow-brown Soil	Top Soil
1	2			
2	3			Fine-grained Missoula Flood Deposits
3	4			
4	5			
5	6			
6	7			
7	8			
8	9		Light Brown Soil	Residual Soil
9	10			
10	11			
11	12	Brown-gray mix		
12	13			
13	14			
14	15			
15	16			
16	17		Boulders or blocky-jointed rock?	Moderately weathered and highly fractured rock
17	18			
18	19			
19	20			
20	21			
21	22			
22	23			
23	24		Dark brown	Highly weathered; possible interflow zone
24	25			
25	26		Orange-brown	
26	27			
27	28			
28	29			
29	30			
30	31		Boulders or blocky-jointed rock	Moderately weathered; blocky-jointed rock
31	32			
32	33			
33	34	Gray		
34	35			

Probe Hole P-8

Depth		Observations		Interpretation
From	To	Driller Log	MJA Log	
0	1	Dirt	Brown soil, moist	Top Soil grading to Missoula Flood Deposits grading to Residual Soil
1	2			
2	3			
3	4			
4	5		Gray rock	
5	6			
6	7			
7	8			
8	9			
9	10	Brown-gray mix		Moderately weathered rock
10	11			
11	12			
12	13	Gray-brown		
13	14			
14	15			
15	16	Dirt		Highly weathered zone; possible flow contact
16	17			
17	18			
18	19			
19	20	Brown-gray		
20	21	Dirt	Dark red-brown	
21	22			
22	23		Gray rock	
23	24			
24	25			
25	26	Brown; bit plugged w/ soil at 32 ft.		Highly weathered and intensely fractured rock
26	27			
27	28			
28	29			
29	30			
30	31		Dark brown	
31	32			

Probe Hole Exploration Points - Summary Field Logs

Probe Hole P-9

Depth		Observations		Interpretation
From	To	Driller Log	MJA Log	
0	1	Dirt	Slightly red-brown soil	Top Soil grading to Residual Soil
1	2			
2	3			
3	4			
4	5	Medium gray		
5	6			
6	7		Brown	Residual Soil
7	8		Gray rock	Slightly to moderately weathered rock
8	9			
9	10			
10	11			
11	12			
12	13			
13	14			
14	15	Brown-gray		
15	16			
16	17		Brown soil with brown rock; Bit plugged w/brown silt at 30 ft.	Moderate to highly weathering
17	18			
18	19			
19	20			
20	21			
21	22			
22	23			
23	24			
24	25			
25	26			
26	27			
27	28			
28	29			
29	30	Soft, gray		
30	31		Dark gray, fine rock cuttings	Chilled zone?
31	32			
32	33			
33	34			
34	35			

Probe Hole P-10

Depth		Observations		Interpretation
From	To	Driller Log	MJA Log	
0	1	Dirt	Red-brown Soil	Top Soil grading to Residual Soil
1	2			
2	3			
3	4			
4	5		Gray	Residual Soil
5	6			
6	7	Medium gray	Light gray rock	Slightly weathered rock
7	8			
8	9			
9	10			
10	11			
11	12		Brown rock	Increased weathering
12	13			
13	14			
14	15			
15	16			
16	17			
17	18			
18	19		Light gray rock chips	Moderately weathered rock
19	20			
20	21			
21	22			
22	23			
23	24		Slightly red-brown	Highly weathered
24	25			
25	26		Dark gray rock chips with soil	Highly weathered rock; chilled zone?
26	27			
27	28	Soft gray mix		
28	29			
29	30			
30	31			
31	32			
32	33			
33	34			
34	35			

Probe Hole Exploration Points - Summary Field Logs

Probe Hole P-11

Depth		Observations		Interpretation
From	**To**	**Driller Log**	**MJA Log**	
0	1	Dirt	Light gray rock chips; no soil cover Bit plugged at 23 ft., but drilled consistent and uniformly to 35 ft.	Slightly to moderately weathered rock with thin highly weathered zones
1	2			
2	3	Medium gray		
3	4			
4	5			
5	6	Brown		
6	7	Medium gray		
7	8			
8	9			
9	10			
10	11			
11	12			
12	13			
13	14			
14	15			
15	16			
16	17			
17	18			
18	19			
19	20			
20	21			
21	22			
22	23			
23	24			
24	25			
25	26			
26	27			
27	28			
28	29			
29	30			
30	31			
31	32			
32	33			
33	34	Soft, black		Possible flow contact
34	35			

Probe Hole P-12

Depth		Observations		Interpretation
From	**To**	**Driller Log**	**MJA Log**	
0	1	Dirt	Brown soil w/ broken gray rock	Residual Soil
1	2			
2	3			
3	4	Medium gray	Gray rock	Moderately weathered rock
4	5			
5	6			
6	7		Brown rock	Increased weathering
7	8			
8	9			
9	10			
10	11			
11	12			
12	13			
13	14		Orange-brown	Significant iron-stained jointing
14	15			
15	16			
16	17			
17	18			
18	19		Dark gray	Moderately weathered rock
19	20	Brown		
20	21	Medium gray		
21	22			
22	23			
23	24			
24	25			
25	26			
26	27			
27	28			
28	29			
29	30	Soft, black		Chilled basal contact?
30	31			
31	32			
32	33			
33	34			
34	35			

Probe Hole Exploration Points - Summary Field Logs

Probe Hole P-13

Depth		Observations		Interpretation
From	To	Driller Log	MJA Log	
0	1	Dirt	Brown soil & broken rock	Residual Soil
1	2			
2	3		Light gray rock Bit plugged at 21 ft.	Slightly to moderately weathered rock
3	4			
4	5			
5	6	Brown		
6	7			
7	8			
8	9			
9	10			
10	11			
11	12			
12	13			
13	14			
14	15			
15	16			
16	17			
17	18			
18	19			
19	20			
20	21			Moderately to highly weathered rock
21	22			
22	23			
23	24		Dark brown soil; soft	
24	25			
25	26			
26	27		Light brown, soft	
27	28			
28	29			
29	30			
30	31			
31	32			
32	33			
33	34	Medium gray	Gray, soft	
34	35			

Probe Hole P-14

Depth		Observations		Interpretation
From	To	Driller Log	MJA Log	
0	1	Dirt	Brown soil, broken rock	Top Soil grading to Residual Soil
1	2			
2	3			
3	4			
4	5	Brown		
5	6			
6	7		Light gray rock; hard	Moderately weathered rock
7	8			
8	9			
9	10			
10	11			
11	12			
12	13			
13	14			
14	15			
15	16			
16	17	Medium gray		Slightly weathered Rock
17	18			
18	19			
19	20			
20	21			
21	22			
22	23			
23	24	Brown-gray mix	Brown, soft	Moderate to highly weathered rock
24	25			
25	26			
26	27			
27	28			
28	29			
29	30			
30	31		Orange-brown	Increased jointing with iron stains
31	32			
32	33			
33	34			
34	35			

FOR LAND USE PERMITTING (EXHIBIT B)

Probe Hole Exploration Points - Summary Field Logs

Probe Hole P-15

Depth		Observations		Interpretation
From	To	Driller Log	MJA Log	
0	1	Dirt	Soil; orange-brown grading to brown	Residual Soil w/ broken rock
1	2			
2	3			
3	4			
4	5	Medium gray		
5	6			
6	7		Light gray rock	
7	8			Slightly weathered rock
8	9			
9	10			
10	11			
11	12			
12	13			
13	14			
14	15			
15	16			
16	17			
17	18			
18	19		Brown, soft	Moderate to highly weathered rock
19	20			
20	21			
21	22			
22	23			
23	24			
24	25			
25	26			
26	27			
27	28			
28	29			
29	30			
30	31			
31	32			
32	33			
33	34	Brown		
34	35			

FOR LAND USE PERMITTING (EXHIBIT B)

Log of Test Pit WTP_1.0-TP-01

<table>
<tr><th></th><th>Test Pit Depth: 4.5 feet
Completed: 12/26/2018</th><th>Equipment: Hitachi 210LC
Contractor: Richter Logging Co.
Logged by: K. Elliott</th></tr>
<tr><th>Depth (feet, bgs)</th><th colspan="2">Material Description</th></tr>
<tr><td>0.0 to 1.0</td><td colspan="2">Very soft, moist, dark brown ORGANIC SILT (OL); numerous fine roots, trace angular fine to coarse gravel-size rock fragments, low plasticity. (Top Soil)</td></tr>
<tr><td>1.0 to 2.0</td><td colspan="2">Soft, moist, slightly orange-brown SILTY GRAVEL with Cobbles (GM); angular coarse gravel- to cobble-size fragments of highly to completely weathered basalt with low plasticity fines. (Residual Soil)</td></tr>
<tr><td>2.0 to 4.5</td><td colspan="2">BASALT; moderately strong, highly weathered, moderately to highly fractured, joint apertures are moderately wide to wide and filled with orange-brown fines, iron oxide stains on the joint surfaces penetrate throughout fragments smaller than boulders; boulder-sizes up to about 3 feet (maximum dimension) of apparent strong and relatively unweathered rock were pulled from the excavation.
Practical refusal of the equipment was reached at a depth of 4.5 feet when the excavator was no longer able to pull the bottom of the test pit.
(Columbia River Basalt)</td></tr>
</table>

Log of Test Pit WTP_1.0-TP-02

<table>
<tr><th></th><th>Test Pit Depth: 4.0 feet
Completed: 12/27/2018</th><th>Contractor: Richter Logging Co.
Equipment: Hitachi 210LC
Logged by: K. Elliott</th></tr>
<tr><th>Depth (feet, bgs)</th><th colspan="2">Material Description</th></tr>
<tr><td>0.0 to 0.5</td><td colspan="2">Very soft, moist, dark brown ORGANIC SILT (OL); numerous fine roots, trace subangular to subrounded fine to coarse gravel-size rock fragments, low plasticity fines. (Top Soil)</td></tr>
<tr><td>0.5 to 3.0</td><td colspan="2">Soft, moist, slightly orange- to red-brown SILTY GRAVEL with Cobbles (GM); subangular coarse gravel- to cobble-size fragments of highly to completely weathered basalt in a low plasticity silt matrix. (Residual Soil)</td></tr>
<tr><td>3.0 to 4.0</td><td colspan="2"><i>Residual Soil, continued</i>. Sub angular to subrounded cobble- and boulder-sizes readily roll out of a soft soil matrix and into the excavation below 3.0 feet; side walls unstable.
Ground water seeped into the excavation at 4.0 feet below ground surface and obscured the bottom of the pit; bottom felt hard; intact rock appears to be just below the ground water seep.
Practical refusal of the equipment was reached at a depth of 4.0 feet when the excavator was no longer able to pull the bottom of the test pit.</td></tr>
</table>

Log of Test Pit WTP_1.0-TP-03

<table>
<tr><th></th><th>Test Pit Depth: 4.0 feet
Completed: 12/27/2018</th><th>Equipment: Hitachi 210LC
Contractor: Richter Logging Co.
Logged by: K. Elliott</th></tr>
<tr><th>Depth (feet, bgs)</th><th colspan="2">Material Description</th></tr>
<tr><td>0.0 to 1.0</td><td colspan="2">Very soft, moist, dark brown ORGANIC SILT (OL); low plasticity, numerous fine roots, estimate 60% coarse angular gravel- to cobble-size basalt rock fragments. (Possible Roadbed Fill)</td></tr>
<tr><td>1.0 to 2.0</td><td colspan="2">Soft, moist, slightly orange-brown SILTY GRAVEL with Cobbles (GM); angular coarse gravel- to cobble-size fragments of highly to completely weathered basalt in a low plasticity silt matrix. (Residual Soil)</td></tr>
<tr><td>2.0 to 4.0</td><td colspan="2">BASALT; moderately strong, highly weathered, moderately to highly fractured, joint apertures are moderately wide to wide and filled with orange-brown fines, iron oxide stains on the joint surfaces penetrate throughout fragments smaller than boulders, boulder-sizes up to at least 3 feet of apparent strong and relatively fresh rock were pulled from the excavation.
Between 2.0 and 2.5 feet an intact portion of rock was observed in the west side wall; interlocking, sharp, angular, fracture-bound, cobble-sized clasts were observed and photographed; fracture apertures were filled with low plasticity fines.
Practical refusal of the equipment was reached at a depth of 4.0 feet when the excavator was no longer able to pull the bottom of the test pit. (Columbia River Basalt)</td></tr>
</table>

Log of Test Pit WTP_1.0-TP-04

<table>
<tr><th></th><th>Test Pit Depth: 4.0 feet
Completed: 12/27/2018</th><th>Contractor: Richter Logging Co.
Equipment: Hitachi 210LC
Logged by: K. Elliott</th></tr>
<tr><th>Depth (feet, bgs)</th><th colspan="2">Material Description</th></tr>
<tr><td>0.0 to 0.8</td><td colspan="2">Very soft, moist, dark brown ORGANIC SILT (OL); numerous fine roots, trace angular fine to coarse gravel-size rock fragments, low plasticity. (Top Soil)</td></tr>
<tr><td>0.8 to 4.0</td><td colspan="2">Soft, slightly yellow-brown, moist, GRAVELLY SILT with Cobbles and Boulders (ML); subangular to subrounded coarse gravel- to boulder-sizes scattered in low plasticity fines.
Groundwater began to seep into the excavation at 3.5 feet bgs; quickly filled the test pit to a level approximately 3.0 feet bgs; sidewalls unstable, cannot support the larger clasts; sidewalls slough; bottom of the test pit obscured by water, but a hard rock surface is present at 4.0 feet bgs.</td></tr>
</table>

Log of Test Pit WTP_1.0-TP-05

	Test Pit Depth: 3.0 feet **Completed: 12/27/2018**	**Equipment: Hitachi 210LC** **Contractor: Richter Logging Co.** **Logged by: K. Elliott**
Depth (feet, bgs)	**Material Description**	
0.0 to 0.8	Very soft, moist, dark brown ORGANIC SILT (OL); low plasticity, numerous fine roots, trace scattered angular gravel- to cobble-size basalt rock fragments. (Top Soil)	
0.8 to 3.0	BASALT; moderately strong, moderately to highly weathered, highly to intensely fractured; joints are moderately wide to wide, filled with low plasticity fines, iron oxide stains on the joint surfaces penetrate throughout fragments smaller than boulders. The rock excavates to Poorly Graded Gravel with Cobbles, Boulders, and Silt (GP-GM). Largest boulder excavated: 4.0 ft X 3.5 ft X 2.0 ft. A hard-continuous rock surface was encountered at 3.0 feet bgs.	

Log of Test Pit WTP_1.0-TP-06

	Test Pit Depth: 0.5 feet **Completed: 12/27/2018**	**Equipment: Hitachi 210LC** **Contractor: Richter Logging Co.** **Logged by: K. Elliott**
Depth (feet, bgs)	**Material Description**	
0.0 to 0.2	Very soft, moist, dark brown ORGANIC SILT (OL); low plasticity; the surface organic layer is about 2 to 3-inches thick and consists mostly of moss with a thin layer of organic soil lying directly on basalt rock. (Top Soil)	
0.2 to 0.5	BASALT; strong, slightly to moderately weathered, vesicular; iron oxide stained at ground surface; jointing not apparent on the surface. The immediate area surrounding this test pit is a surface outcropping of hard rock; bare rock is exposed in places and in others lies beneath a thin organic layer. The excavator was able to penetrate the rock only a few inches; no open jointing was apparent to get the teeth into. (Columbia River Basalt)	

FOR LAND USE PERMITTING (EXHIBIT B)

Appendix B

Geophysical Exploration

FOR LAND USE PERMITTING (EXHIBIT B)

SIEMENS & ASSOCIATES

McMillen Jacobs Associates
1500 SW First Avenue, Suite 750
Portland, Oregon 97201
Attention: Farid Sariosseiri, PhD, PE

March 11, 2019
Siemens Project No. 191011

Project: WWSP – WTP_1.0
Tualatin, Oregon

Subject: Results of Geophysical Reconnaissance

Hello Farid,

This letter presents the results of the geophysical reconnaissance and briefly describes the methods used. The services were provided in general accordance with the agreement prepared by McMillen Jacobs Associates (MJA) and Schedule of Charges dated February 13, 2019. The field work was conducted on February 15, 2019, with guidance in the field provided by Mr. Kim Elliot of MJA. The weather was overcast with occasional light rain.

Project Understanding

Siemens & Associates (SA) understand that MJA is providing geotechnical services to assist with an assessment of the ground conditions for a proposed new water treatment plan. Only a few project details have been provided although SA understands that site development will include mass excavation to depths in excess of 30 feet. As a result, seismic velocity of the subsurface is of interest as one of the predictors regarding excavation characteristics.

Purpose and Methods

Objectives include geophysical characterization of the geotechnical conditions describing the character of the overburden soils and underlying rock in terms of P-wave velocity. SA also recorded and processed shear wave data using the refraction microtremor (ReMi) method. The ReMi results describe important subsurface characteristics and add significant value and basis for developing supportive conclusions.

The seismic methods were used along three lines, the locations of which were chosen to exploit existing areas that were reasonably cleared of brush which includes poison oak. The locations of the lines may not be ideal although for safety reasons, these were the routes available for exploration. The approximate location of the lines are depicted on Figure 100 (Site Plan: Geophysical Exploration) and the geophysical methods used are briefly described as follows:

- P-wave Seismic Refraction (SR): An active seismic method utilizing geophone receivers set along a straight line gathering data from signals induced by a small explosive charge (8 gauge, 400 grain black powder shell detonated using a Betsy Seisgun). Data were processed using forward modeling software developed by Geogiga known as DW Tomo 8.3; a plausible model of the subsurface was developed for each line. SR provides a 2D profile illustrating P-wave velocity with depth. Lower P-wave velocity is related to unconsolidated materials while heavily consolidated materials and rock are illustrated by higher P-wave velocity. P-wave variation within higher velocity layers illustrates the heterogeneity of the rock mass related to fracturing, jointing, and possibly weathering and decomposition.

How it works: When the explosive charge detonates, the receivers are triggered, and the wavelet energy is recorded. The P-wave is the fastest of the various seismic waves that are generated and therefore, only the time of the first arrival wave from the receiver is considered. These "first arrivals" are picked for each record. As the energy travels through the ground, the waves are refracted and the arrival time, combined with distance from the source, is related to both the velocity and distance to the layers promoting refraction. This distance is not necessarily vertical depth; rather the nearest refractor and the image can be skewed when oriented along a dipping refractor. The seismic refraction method takes advantage of a common occurrence: seismic velocities increase as a function of depth. If this assumption is false, critical refraction does not occur as expected and the velocity and depth calculations are inaccurate; hence, the value of conducting more than one geophysical method for effective site exploration.

Data were recorded using a 24 channel DAQ 4 seismograph manufactured by Seismic Source in Ponca City, Oklahoma, USA, connected to an IBM Lenovo laptop computer.

For this project, data were collected using 24 receivers at one time. Shot spacing was set at 20 feet. Shots were induced at intervals equal to twice the receiver spacing and recorded by

4.5 Hz. geophone receivers. Shot locations extended throughout and beyond each end of the receiver arrays.

- Seismic Refraction Microtremor (ReMi): A passive, surface wave seismic method describing the "average" shear-wave velocity depth profile in the vicinity of the geophone spread. Shear-wave velocity can be directly related to the strength of geologic materials and is commonly correlated with other methods to provide confirmation of the interpretation. ReMi data were recorded using the same seismic arrays used for the SR data gather. Only data acquisition parameters were changed. ReMi is an averaging method and for this project, all 24 receivers were used for the analysis such that geologic variations in the vicinity of the receiver array are averaged to a single 1D profile. ReMi can be processed in 2D but this is beyond the scope of this exploration although the data are preserved and 2D processing could be done at a later time.

  ReMi data were processed using Geogiga Microtremor 8.3 software produced by Geogiga Technology Corp., Calgary, Alberta, Canada. Dr. Satish Pullammanappallil, Ph.D., operating from Reno, Nevada, took the lead in processing 1D S-wave profiles for each line. The results are presented as an overlay on each of the SR tomograms and as individual figures with supporting illustrations. The 1D, S-wave profiles extend to 150 feet (BGS) and provide a means of calculating the seismic site classification as defined by ASCE 7, based on the average S-wave velocity through the upper 100 feet of a site (Vs_{100}). These values are also presented on the SR tomograms and vary from 2973 to 4222 f/s with an average of 3632 f/s. This sets the site within the boundaries defining a Seismic Site Class designation "B." The Vs_{100} can be re-calculated in consideration of excavation where foundations will be situated at elevations below ground surface. Excel spreadsheets have been provided to MJA for this purpose.

Conclusions

The P-wave refraction results are presented as Figures SR-1 through SR-3. These tomograms (a Greek word for slice or cut) extend to greater depths than the geotechnical borings and clearly illustrate the character of the geotechnical conditions in terms of P-wave velocity. When in reasonable proximity to a line, the geotechnical borings are plotted on the tomograms.

P-wave and S-wave velocities are high with only a thin veneer of soil indicated along the lines. Through the shallow subsurface (upper 50 feet or so), the tomograms suggest more lateral variation in rock quality than might be inferred from the borings. SA estimates that this feature is likely the result of variation in the fracture and joint pattern of the rock mass.

Rippability

P-wave velocity is often used to predict rock excavation difficulty and this is one of the objectives of the geophysical effort. It is prudent to consider other properties including the frequency of planes of weakness (fractures, joints, faults, laminations, etc.), uniaxial strength, degree of weathering, abrasiveness, and more. Excavator tooth penetration is often the key to ripping success, regardless of seismic P-wave velocity. Experience excavating similar rock from nearby projects is an excellent method if proper correlation to this site can be established. In other words, predicting rippability based on only one parameter, the wave speed, must be enhanced by considering other available information. Physical characteristics that are favorable to ripping include:

- Frequent planes of weakness such as fractures, faults, and laminations
- Weathered rocks
- Rocks with high moisture
- Highly stratified rocks
- Brittle rocks
- Rocks with low shear strength
- Rocks with low seismic velocity (both P-wave and S-wave)

Conditions that make ripping difficult include:

- Massive rocks
- Rocks with no planes of weakness
- Crystalline rocks
- Non-brittle, energy absorbing rock fabric
- Rocks with high shear strength
- Rocks with high seismic velocity (both P-wave and S-wave, especially when the ratio of V_s/V_P approaches 1

Caterpillar industry charts provide a summary of rippability based on field trials with a primary correlation to rippability related to seismic P-wave velocity. Along with factors previously discussed, it is important to recognize that the Caterpillar research was completed mostly in the mid-western United States and may not be applicable to the conditions encountered near Tualatin, Oregon. A review of "Handbook of Ripping" published by Caterpillar (Twelfth Edition) provides many charts and performance estimates that are helpful and the document is submitted with this report. Note that ripping is often considered more of an art than science and SA encourages the consideration of other known geologic features to base a conclusion regarding rippability. This said, SA is reasonably confident that rock offering P-wave velocity on the order of 4000 f/s and lower is quite likely to excavate without need for drilling and blasting. Production is likely to vary similar to the variation illustrated by lateral velocity variation in the tomograms. Although with less certainty, velocities ranging from 4000 to around 6500 f/s are considered to represent the “marginal” range and light blasting may be an effective way to enhance production. P-wave velocity 6500 f/s and higher are likely to be representative of strong rock and excavation characteristics will improve by applying drilling and blasting methods. The conditions interpreted along the tomograms suggest only a thin zone (typically less than 10 feet) offers P-wave suggesting opportunity for ripping. Most of the rock velocities observed in this report suggest drilling and blasting will be necessary for effective, efficient removal of rock for any significant depth of excavation.

It is important to point out that the results are reported in 2D while the data were generated from and influenced by a 3D environment. When subsurface conditions are changing rapidly along the survey route, this effect can skew the 2D model. The methods tend to average subsurface conditions and rapid or minor changes may not be interpreted. For these reasons and others as previously discussed, the results may or may not consistently compare with 1D data such as a geotechnical boring.

Survey Line Locations, Data Acquisition, and Gear

The location of each geophysical line is illustrated on Figure 100. The end points of each geophysical survey were marked with a wooden lath to define the location in the field. Ground elevation along the array was surveyed by SA using a theodolite and grade rod with elevations referenced to temporary benchmarks set by the project surveyor. Elevations of these reference points were provided to SA by MJA. Vertical resolution is estimated to be within a few tenths of a foot. Horizontal positions as shown on Figure 100 were plotted on the Google Earth base map from onsite data recorded using a hand-held GPS (Garmin 755t) and are estimated to be within about 10 feet of actual.

Seismic data were collected using a 24 channel DAQLink 4 digital seismograph manufactured by Seismic Source, Ponca City, Oklahoma. Seismic receivers were 4.5 Hz. geophones manufactured by the GeoSpace, Houston, Texas.

FOR LAND USE PERMITTING (EXHIBIT B)

Confidence

The seismic data were gathered at a relatively "quiet" site and this led to accurate interpretation of first arrival waveforms for P-wave analysis. The long records for the ReMi data gather collected adequate variations in frequency content from ambient signals induced by area road traffic, distant construction, and plate and hammer near each array. The results are judged to be robust and of high quality.

Limitations

This report has been prepared for the exclusive use of MJA for specific application to the project known as WWSP WTP_1.0 in Tualatin, Oregon. SA has endeavored to complete the services in accordance with generally accepted geophysical practice consistent with similar work done near Tualatin, Oregon, by geophysical practitioners at this time. No other warranty, expressed or implied, is made.

The information presented is based on data obtained from the field explorations described in this report. The explorations indicate geophysical conditions only at specific locations and times, and only to the depths penetrated. They do not necessarily reflect variations that may exist between exploration locations. The subsurface at other locations may differ from conditions interpreted at these explored locations. Also, the passage of time may result in a change in conditions. If any changes in the nature, design, or location of the project are implemented, the information contained in this report should not be considered valid unless the changes are reviewed by SA to address the implications and benefit of enhancing the work as necessary. SA is not responsible for any claims, damages, or liability associated with outside interpretation of these results, or for the reuse of the information presented in this report for other projects.

SA has included "Important Information About this Geotechnical Report" prepared by GBA (Geoprofessional Business Association, of which SA is a member) which also applies to geophysical services, to assist you and others in the use and limitations of this report.

SA appreciates the opportunity to conduct this exploration and present the results and conclusions. We trust that the services are in line your expectation. Please contact us with questions.

Prepared by,
Siemens & Associates

16,377
JAN. 19, 1993
ANDREW SIEMENS

J. Andrew Siemens, P.E., G.E.
Principal

Addressee: 1 electronic (pdf)
Enclosures: Figure 100, SR-1 through SR -3, RM1 through RM-3
GBA document

FOR LAND USE PERMITTING (EXHIBIT B)

Important Information about This

Geotechnical-Engineering Report

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

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

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

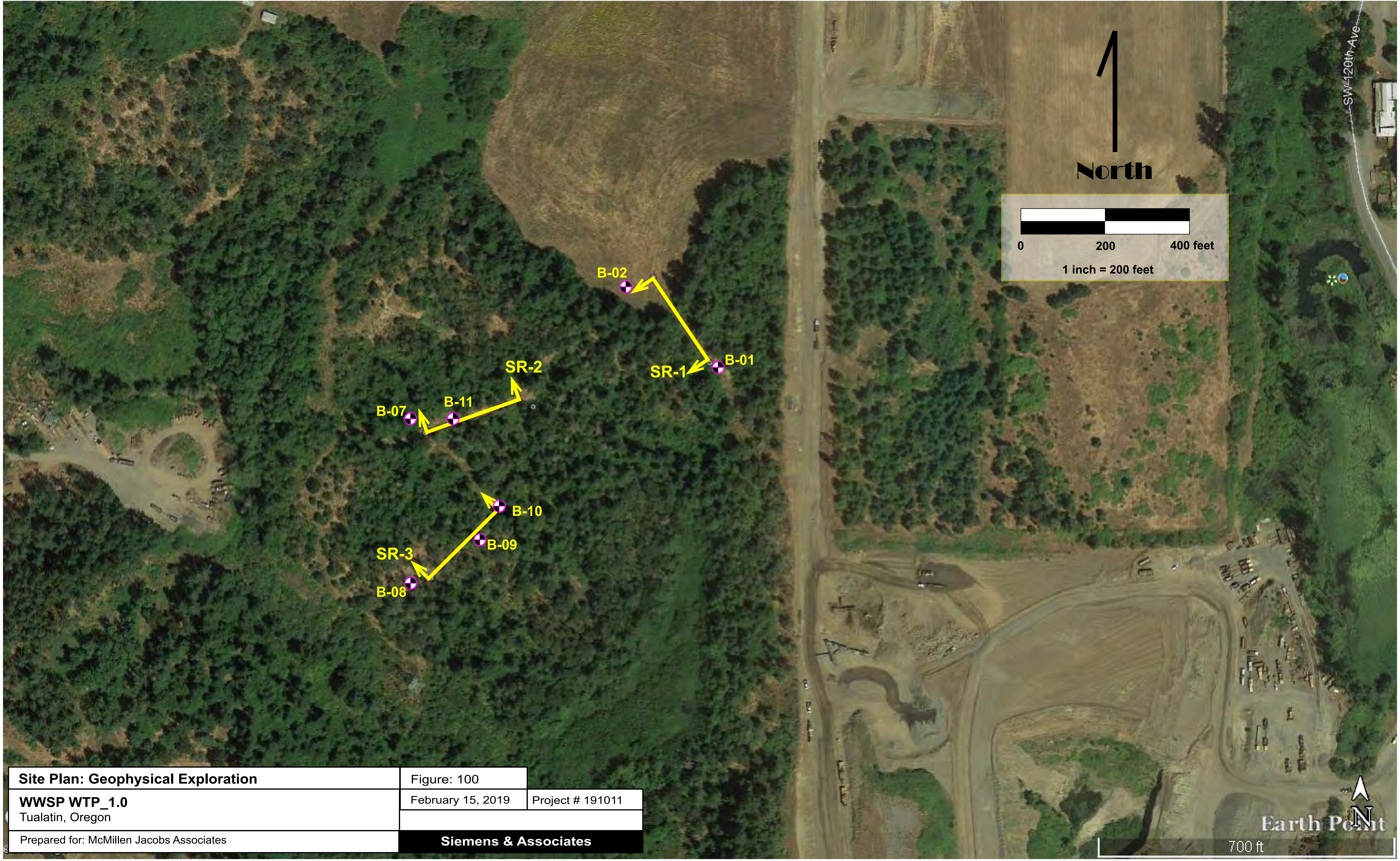
Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

GBC GEOTECHNICAL BUSINESS COUNCIL
of the Geoprofessional Business Association

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org

FOR LAND USE PERMITTING (EXHIBIT B)



Site Plan: Geophysical Exploration

Figure: 100

WWSP WTP_1.0
Tualatin, Oregon

February 15, 2019

Project # 191011

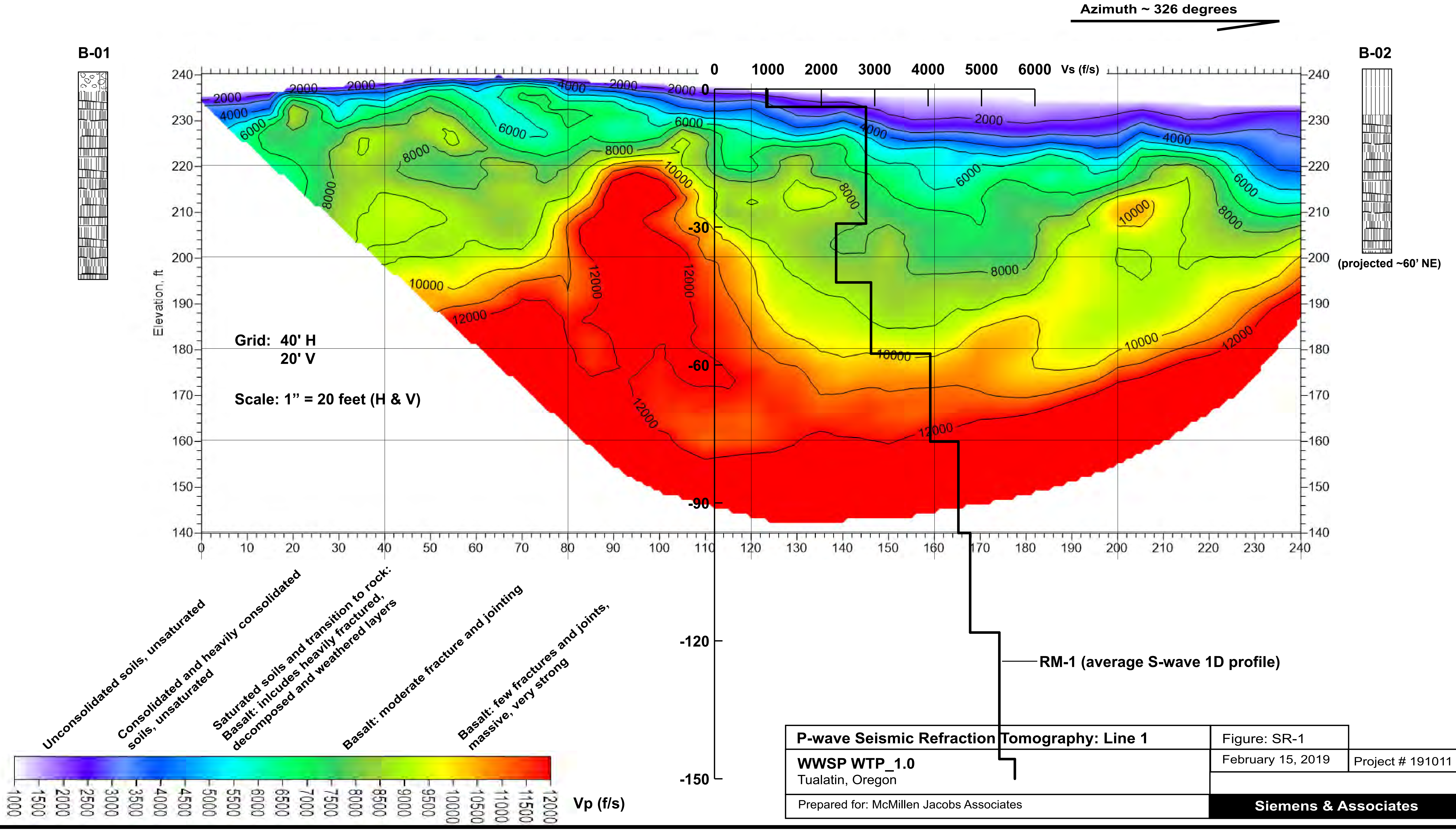
Prepared for: McMillen Jacobs Associates

Siemens & Associates

FOR LAND USE PERMITTING (EXHIBIT B)

P-wave Seismic Refraction Tomography (SR): Line 1

(24, 4.5 Hz. receivers on 10 foot spacing, 13 shots, 20 foot spacing)



P-wave Seismic Refraction Tomography: Line 1

Figure: SR-1

WWSP WTP_1.0
Tualatin, Oregon

February 15, 2019

Project # 191011

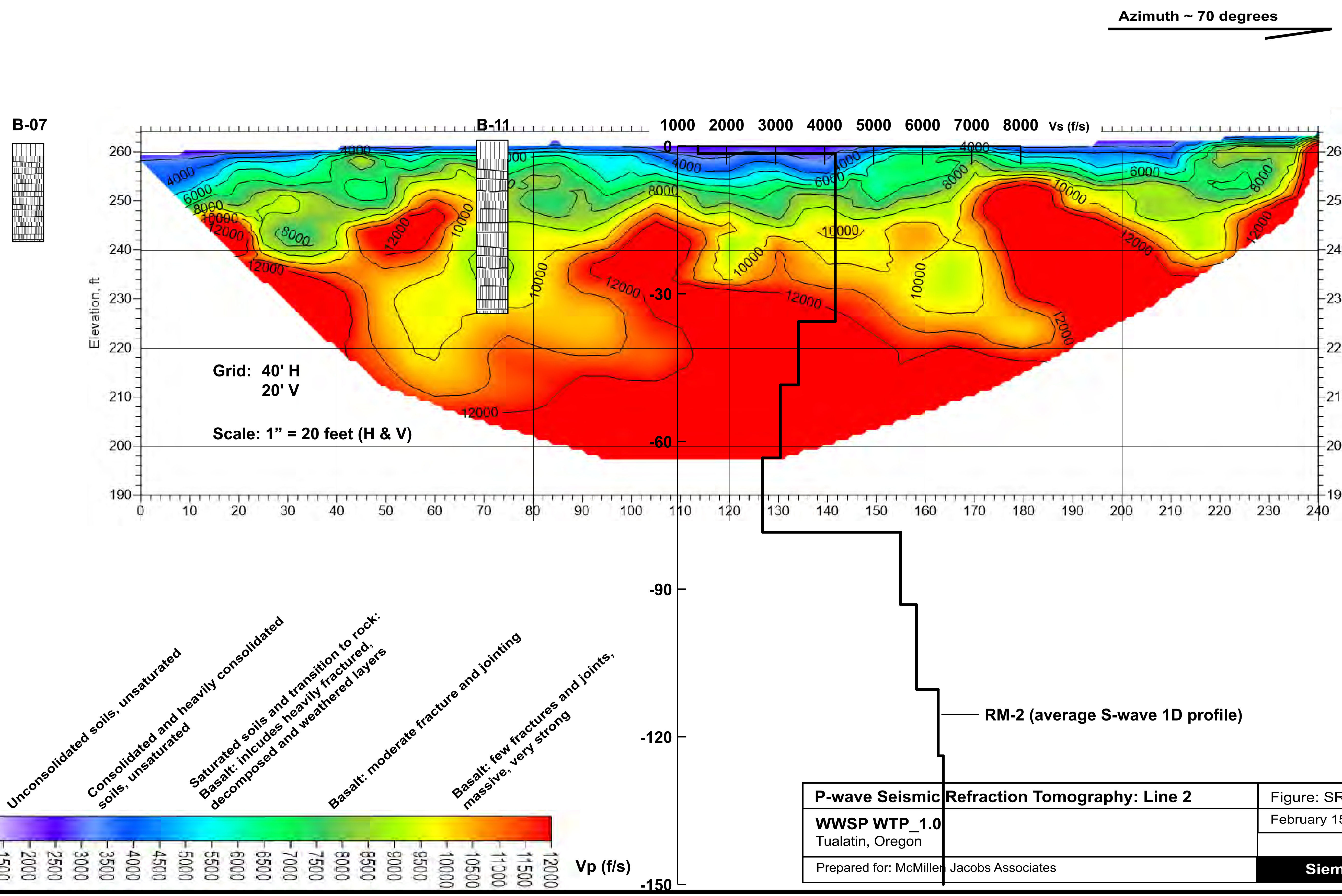
Prepared for: McMillen Jacobs Associates

Siemens & Associates

FOR LAND USE PERMITTING (EXHIBIT B)

P-wave Seismic Refraction Tomography (SR): Line 2

(24, 4.5 Hz. receivers on 10 foot spacing, 13 shots, 20 foot spacing)

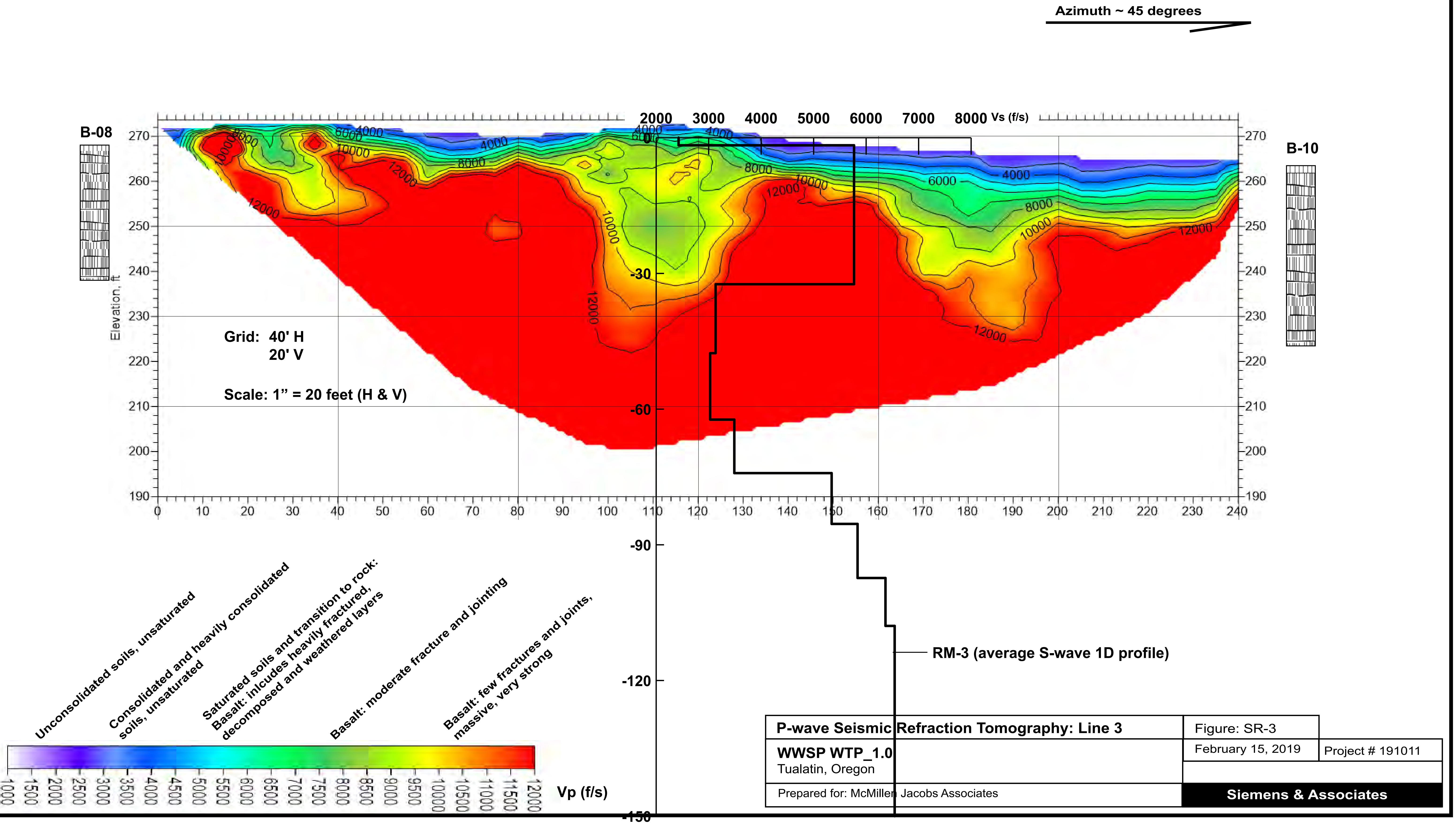


P-wave Seismic Refraction Tomography: Line 2	Figure: SR-2	
WWSP WTP_1.0 Tualatin, Oregon	February 15, 2019	Project # 191011
Prepared for: McMillen Jacobs Associates	**Siemens & Associates**	

FOR LAND USE PERMITTING (EXHIBIT B)

P-wave Seismic Refraction Tomography (SR): Line 3

(24, 4.5 Hz. receivers on 10 foot spacing, 13 shots, 20 foot spacing)



P-wave Seismic Refraction Tomography: Line 3	Figure: SR-3	
WWSP WTP_1.0 Tualatin, Oregon	February 15, 2019	Project # 191011
Prepared for: McMillen Jacobs Associates	**Siemens & Associates**	

FOR LAND USE PERMITTING (EXHIBIT B)

WWSP WTP_1.0
Tualatin, Oregon

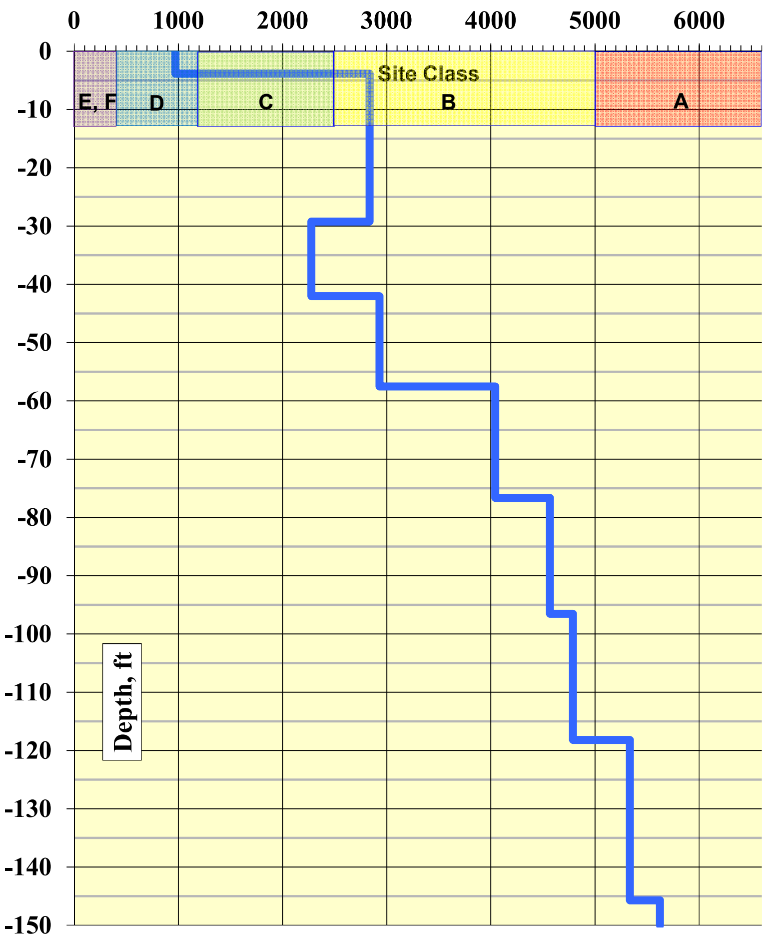
Project Number: 191011

Refraction Microtremor: RM #1

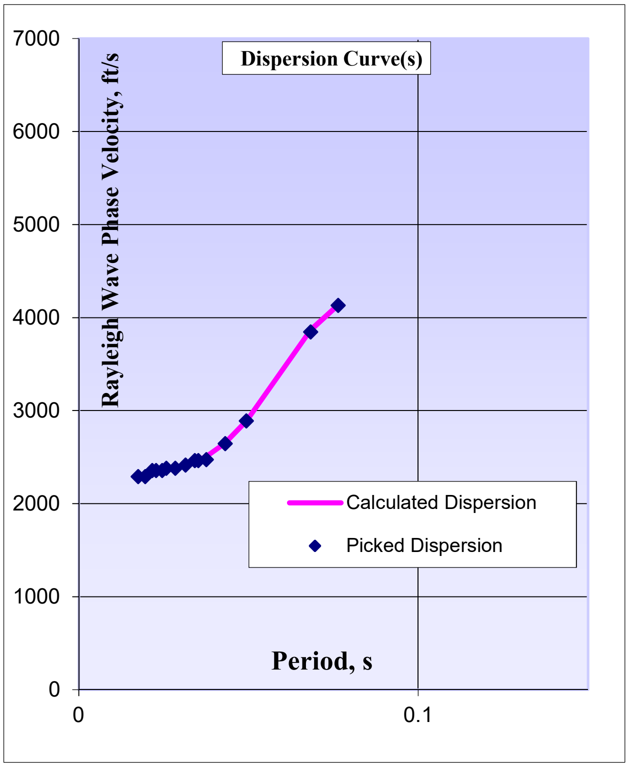
24, 4.5 Hz. Receivers on 10 foot spacing

(NE area of site: Azimuth ~ 326^0)

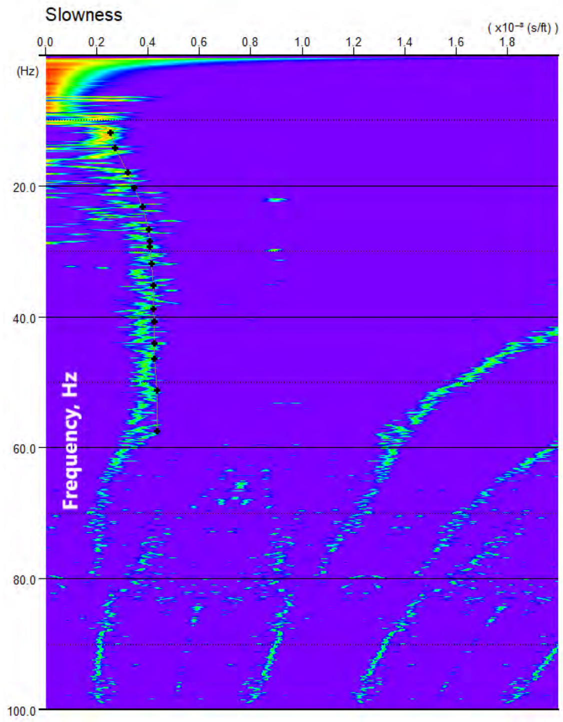
Shear Wave Velocity Profile: V100 = 2973 f/s,
ASCE 7: Site Class B (from BGS)



Supportive Illustrations:



ReMi Spectral Ratio (p-f image)
w/ Modeling Picks



Prepared By: Siemens & Associates
Geogiga Surface Plus 8.3 Analysis by SubterraSeis, LLC
February 15, 2019

FOR LAND USE PERMITTING (EXHIBIT B)

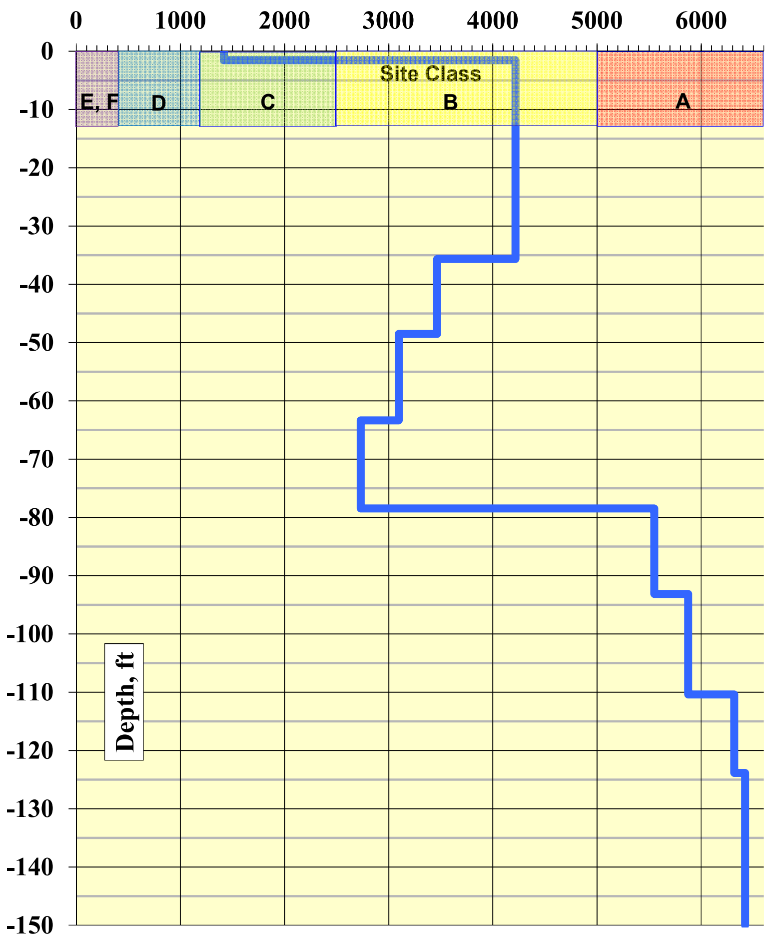
WWSP WTP_1.0
Tualatin, Oregon

Project Number: 191011

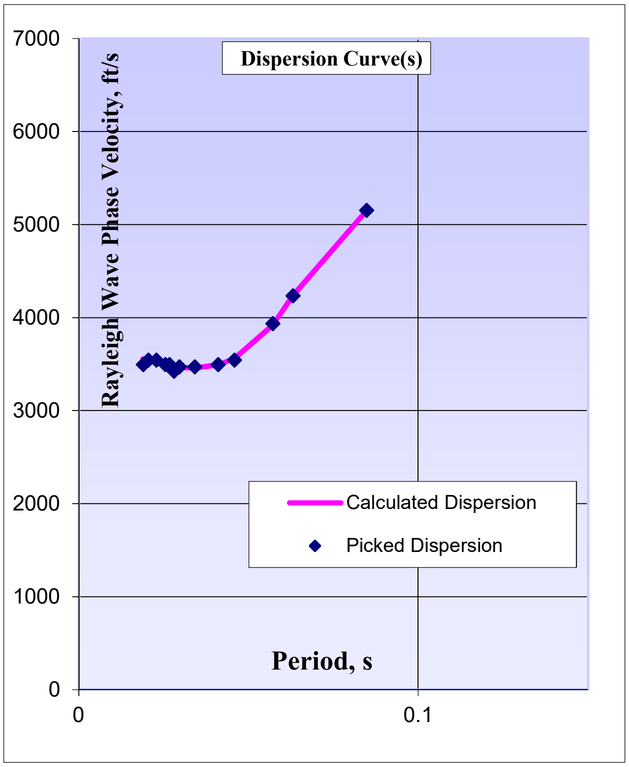
Refraction Microtremor: RM #2

24, 4.5 Hz. Receivers on 10 foot spacing

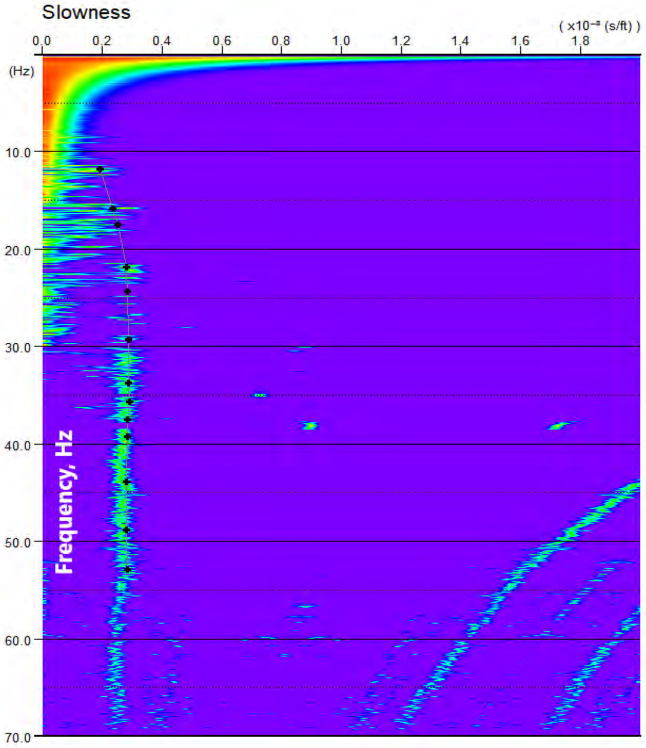
(N center area of site: Azimuth ~ 70^0)



Supportive Illustrations:



ReMi Spectral Ratio (p-f image)
w/ Modeling Picks



Prepared By: Siemens & Associates
Geogiga Surface Plus 8.3 Analysis by SubterraSeis, LLC
February 15, 2019

FOR LAND USE PERMITTING (EXHIBIT B)

WWSP WTP_1.0
Tualatin, Oregon

Project Number: 191011

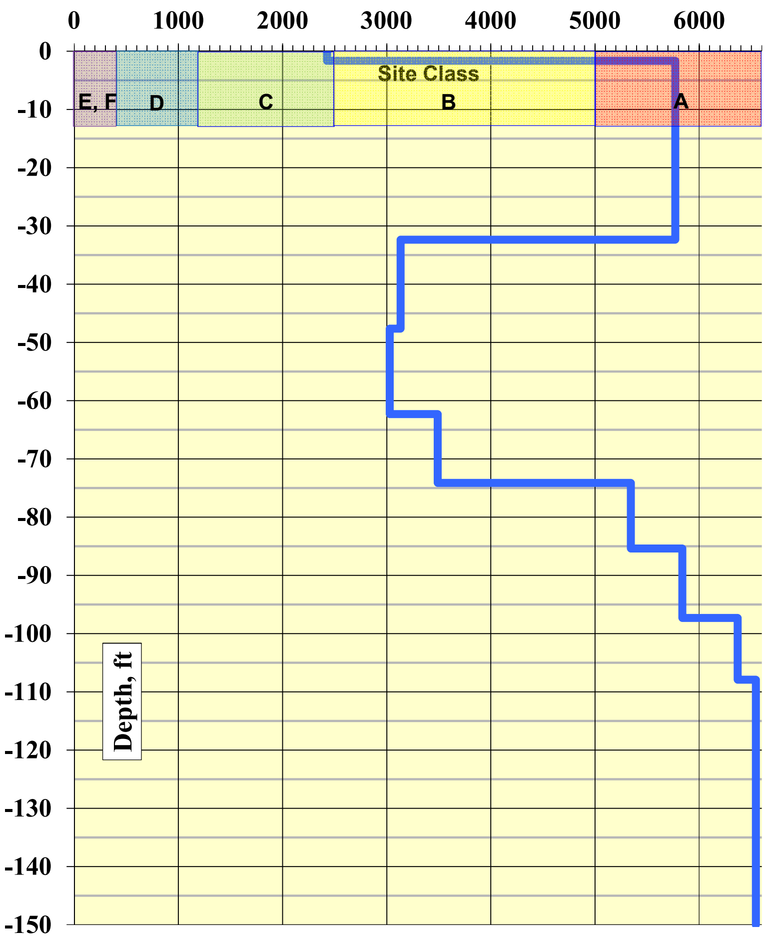
Refraction Microtremor: RM #3

24, 4.5 Hz. Receivers on 10 foot spacing

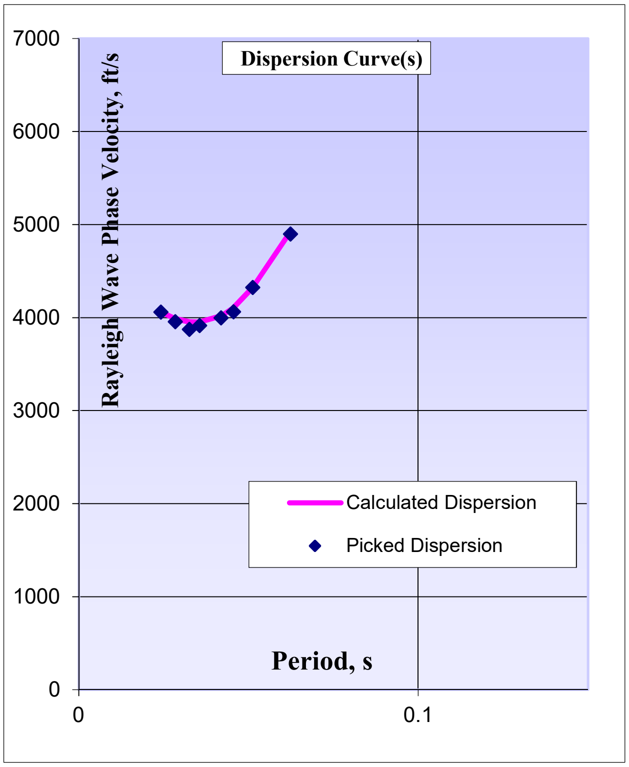
(Approximate center area of site: Azimuth ~ 45^0)

Shear Wave Velocity Profile: V100 = 4222 f/s,
ASCE 7: Site Class B (from BGS)

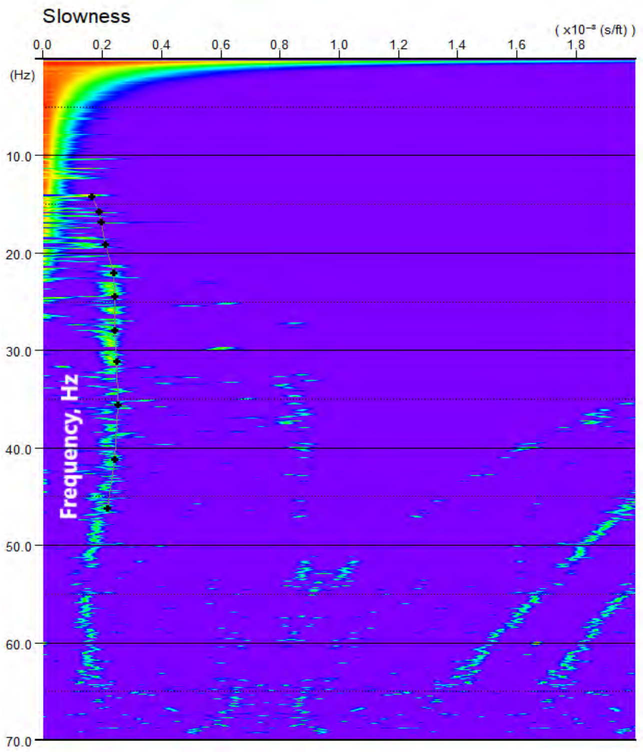
Vs (f/s)



Supportive Illustrations:



ReMi Spectral Ratio (p-f image)
w/ Modeling Picks



Prepared By: Siemens & Associates
Geogiga Surface Plus 8.3 Analysis by SubterraSeis, LLC
February 15, 2019

Appendix C

Laboratory Test Data

Table C-1. Summary of Unconfined Compressive Strength Testing

Sample Location or ID				Strength		
Boring ID	**Sample No.**	**Top Depth (feet)**	**Bottom Depth (feet)**	**Uniaxial Compressive (psi)**	**Grade[1]**	**Term[1]**
B-01	C-01	7.1	7.8	27,467	R5	Very Strong
B-01	C-02	22.5	23.4	23,727	R5	Very Strong
B-02	C-03	13.4	14.3	16,746	R5	Very Strong
B-03	C-10	8.65	9.0	18,552	R5	Very Strong
B-03	C-11	17.3	18.0	34,089	R5	Very Strong
B-06	C-14	9.1	9.7	23,467	R5	Very Strong
B-07	C-04	8.3	9.3	18,990	R5	Very Strong
B-07	C-05	15.8	16.7	19,466	R5	Very Strong
B-08	C-17	14.6	15.1	11,928	R4	Strong
B-09	C-13	22.9	23.6	18,249	R5	Very Strong
B-09	C-15	27.0	27.5	27,829	R5	Very Strong
B-09	C-16	29.7	30.2	22,316	R5	Very Strong
B-10	C-06	16.5	17.5	31,235	R5	Very Strong
B-10	C-07	26.3	27.4	17,446	R5	Very Strong
B-11	C-18	17.95	18.55	15,053	R4	Strong
B-11	C-19	27.65	28.15	27,911	R5	Very Strong
B-12	C-08	7.8	8.6	21,764	R5	Very Strong
B-12	C-09	20.7	27.7	19,622	R5	Very Strong

Note:

1. Rock Strength grades and terms from ODOT (1987)

Table C-2. Summary of Point Load Strength Testing

Boring ID	Depth to Top (feet)	Sample ID	Failure Load, P (kN)	Corrected Point Load Strength Index, Is(50) (PSI)	Estimated Uniaxial Compressive Strength, UCS (PSI)	Strength Grade[1]	Strength Term[1]
B-01	5.0	S-1	33.57	1,468	25,691	R5	Very Strong
B-01	6.6	S-2	36.96	1,616	28,285	R5	Very Strong
B-01	7.8	S-3	32.21	1,409	24,651	R5	Very Strong
B-01	8.2	S-4	42.41	1,855	32,459	R5	Very Strong
B-02	12.0	S-5	40.36	1,765	30,889	R5	Very Strong
B-02	14.0	S-21	37.01	1,618	28,322	R5	Very Strong
B-02	14.3	S-6	29.71	1,299	22,741	R5	Very Strong
B-02	15.8	S-7	38.34	1,677	29,340	R5	Very Strong
B-02	26.3	S-8	12.16	532	9,309	R4	Strong
B-04	3.3	S-22	10.35	453	7,924	R3	Medium Strong
B-04	4.0	S-23	28.35	1328	23,240	R5	Very Strong
B-07	13.4	S-10	29.28	1,281	22,409	R5	Very strong
B-07	13.9	S-11	36.29	1,587	27,771	R5	Very strong
B-07	16.7	S-12	29.82	1,304	22,822	R5	Very strong
B-10	15.7	S-13	35.28	1,543	27,004	R5	Very strong
B-10	16.1	S-14	31.68	1,386	24,247	R5	Very strong
B-10	23.9	S-15	30.01	1,312	22,968	R5	Very strong
B-10	26.0	S-16	33.87	1,481	25,925	R5	Very strong
B-12	8.6	S-17	29.06	1,271	22,237	R5	Very strong
B-12	9.7	S-18	39.64	1,734	30,339	R5	Very strong
B-12	13.3	S-19	21.67	948	16,586	R5	Very strong
B-12	26.1	S-20	33.74	1,476	25,824	R5	Very strong

Note:

1. Rock strength grades from ODOT (1987).

Table C-3. Summary of Moisture Content and Atterberg Limits Testing

Sample Location or ID			Soil Description			Moisture %	Atterberg Limits		
Boring	Sample, No.	Depth Interval (feet)	Geologic Unit	USCS	Soil Description		Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
B-02	S-2	5-6.5	Missoula Flood Deposits	ML	Brown silt with sand	28.9	42.7	26.6	16.1
B-02	S-3	7.5-8	Missoula Flood Deposits	ML	Brown silt	29.6	-	-	-

Table C-4. Summary of Corrosivity Testing

Sample Location or ID			Corrosivity Testing								
			Resistivity @ 15.5 °C	Chloride	Sulfate		pH	ORP		Sulfide	Moisture
			(Ohm-cm)	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test
Boring	Sample, No.	Depth Interval (feet)	Sat.	Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test Temp °C	by Lead Acetate Paper	%
B-01	S-1	2.5-4.0	2,354	7	36	0.0036	6.3	502	15	Negative	17.9
B-02	S-1	2.5-4.0	7,884	7	3	0.0003	5.9	513	15	Negative	24.1
TP-04	S-1	2.0-3.0	79,679	14	2	0.0002	6.3	520	15	Negative	35.9

nt Load Strength Index Test Results ASTM D-5731

McMILLEN JACOBS ASSOCIATES

PROJECT:	**WWSP WTP_1.0**	**LAB SAMPLE NO.:**	B-01, -02, -04, -07, -10, -12
PROJECT NO.:	**5887.0**	**SAMPLE NO.:**	S1-S20
PROJECT LOCATION:	**Sherwood, Oregon**	**SAMPLE DESCRIP:**	BASALT
SAMPLED BY:	**K. Elliott, J. Fissel**	**DATE REPORTED:**	3/11/2019
DATE SAMPLED:	**December 5-12, 2018**	**REPORTED BY:**	A. Havekost

Site-specific Correlation Factor, C= 17.5

Sample No., Boring	Test Number**	Rock Type	Depth or Diameter, D (mm)	D' (mm)	ΔD/D, penetration ratio (%)	Width, W (mm)	Area, A (mm^2)	Failure Load, P (kN)	Failure Load, P (N)	De^2 (mm^2)	Equivalent Diameter, De (mm)	Uncorrected Point Load Strength Index, I_s (Mpa)	Uncorrected Point Load Strength Index, I_s (PSI)	Size Correction Factor, F	Corrected Point Load Strength Index, $I_{s(50)}$ (Mpa)	Corrected Point Load Strength Index, $I_{s(50)}$ (PSI)	Estimated Uniaxial Compressive Strength, UCS (Mpa)	Estimated Uniaxial Compressive Strength, UCS (PSI)
B-01-5	1	basalt	60.00	55.00	8	125	6875	33.57	33568	3600	60.0	9	1352	1.09	10.1	1468	177	25691
B-01-6.55*	2	basalt	60.00	54.00	10	130	7020	36.96	36958	3600	60.0	10	1489	1.09	11.1	1616	195	28285
B-01-7.76*	3	basalt	60.00	55.00	8	140	7700	32.21	32210	3600	60.0	9	1298	1.09	9.7	1409	170	24651
B-01-8.22	4	basalt	60.00	55.00	8	65	3575	42.41	42412	3600	60.0	12	1709	1.09	12.8	1855	224	32459
B-02-12	5	basalt	60.00	55.00	8	115	6325	40.36	40360	3600	60.0	11	1626	1.09	12.2	1765	213	30889
B-02-14	21	basalt	60.00	55.00	8	91	5005	37.01	37006	3600	60.0	10	1491	1.09	11.2	1618	195	28322
B-02-14.3*	6	basalt	60.00	55.00	8	140	7700	29.71	29714	3600	60.0	8	1197	1.09	9.0	1299	157	22741
B-02-15.8	7	basalt	60.00	55.00	8	110	6050	38.34	38336	3600	60.0	11	1544	1.09	11.6	1677	202	29340
B-02-26.25	8	basalt	60.00	57.00	5	180	10260	12.16	12164	3600	60.0	3	490	1.09	3.7	532	64	9309
B-04-3.3	22	basalt	60.00	56.00	7	120	6720	10.35	10354	3600	60.0	3	417	1.09	3.1	453	55	7924
B-04-4	23	basalt	61.00	54.00	11	122	6588	28.35	28346	3294	57.4	9	1248	1.06	9.2	1328	160	23240
B-07-13.4	10	basalt	60.00	55.00	8	130	7150	29.28	29280	3600	60.0	8	1180	1.09	8.8	1281	155	22409
B-07-13.9	11	basalt	60.00	56.00	7	125	7000	36.29	36286	3600	60.0	10	1462	1.09	10.9	1587	191	27771
B-07-16.7*	12	basalt	60.00	56.00	7	120	6720	29.82	29820	3600	60.0	8	1201	1.09	9.0	1304	157	22822
B-10-15.7	13	basalt	60.00	56.00	7	90	5040	35.28	35284	3600	60.0	10	1422	1.09	10.6	1543	186	27004
B-10-16.05*	14	basalt	60.00	56.00	7	115	6440	31.68	31682	3600	60.0	9	1276	1.09	9.6	1386	167	24247
B-10-23.85	15	basalt	60.00	56.00	7	100	5600	30.01	30010	3600	60.0	8	1209	1.09	9.0	1312	158	22968
B-10-25.96*	16	basalt	60.00	55.00	8	73	3988	33.87	33874	3600	60.0	9	1365	1.09	10.2	1481	179	25925
B-12-8.6	17	basalt	60.00	56.00	7	155	8680	29.06	29056	3600	60.0	8	1171	1.09	8.8	1271	153	22237
B-12-9.65	18	basalt	60.00	56.00	7	75	4200	39.64	39642	3600	60.0	11	1597	1.09	12.0	1734	209	30339
B-12-13.25	19	basalt	60.00	57.00	5	145	8265	21.67	21672	3600	60.0	6	873	1.09	6.5	948	114	16586
B-12-26.1	20	basalt	60.00	56.00	7	165	9240	33.74	33742	3600	60.0	9	1359	1.09	10.2	1476	178	25824

Minimum 1:	453		7924
Minimum 2:	532		9309
Maximum 1:	1855		32459
Maximum 2:	1765		30889
Average excluding Min 1, Min 2, Max 1, Max 2:	1439		25187

Notes:

*Samples used for site-specific UCS correlation factor calculation

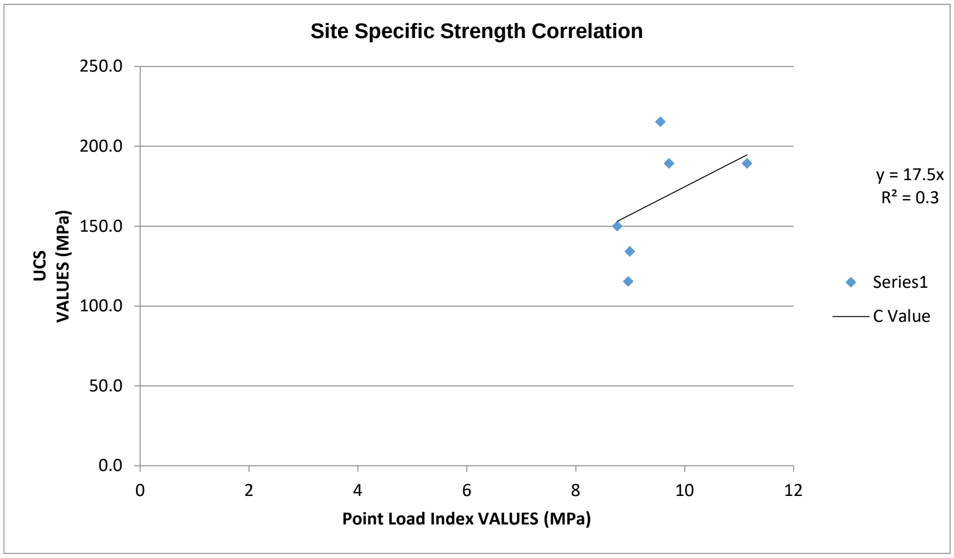
**Sample 9 invalid and not included in this report; sample broke on pre-existing plane of weakness.

Point Load Strength Index Calculation Explanation ASTM D-5731

PROJECT: WWSP WTP_1.0
PROJECT NO.: 5887
PROJECT LOCATION: Sherwood, Oregon
SAMPLED BY: K. Elliot, J. Fissel
DATE SAMPLED: December 5-12, 2018

LAB SAMPLE NO.: B-01, -02, -07, -10, -12
SAMPLE NO.: S1-S20
SAMPLE DESCRIP: BASALT
DATE REPORTED: 1/9/2019
REPORTED BY: A. Havekost

UCS Sample ID, Depth	PLT Sample ID, Depth	Uniaxial Compressive Strength (psi)	UCS (Mpa)	PLS(Mpa)
B-01 C-01 @ 7.12 – 7.76 ft.	B-01 S2 @ 6.55 ft	27467	189.43	11.14
B-01 C-01 @ 7.12 – 7.76 ft.	B-01 S3 @ 7.76 ft.	27467	189.43	9.71
B-02 C-03 @ 13.35 – 14.3 ft.	B-02 S6 @ 14.3 ft	16746	115.49	8.96
B-07 C-05 @ 15.8 – 16.7 ft.	B-07 S12 @ 16.7 ft.	19466	134.25	8.99
B-10 C-06 @ 16.5 – 17.5 ft.	B-10 S14 @ 16.05 ft.	31235	215.41	9.55
B-12 C-08 @ 7.8 – 8.6 ft.	B-12 S17 @ 8.6 ft.	21764	150.10	8.76



FOR LAND USE PERMITTING (EXHIBIT B)

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

TECHNICAL REPORT

Report To: Ms. Annie Havekost
McMillen Jacobs Associates
1500 SW First Avenue, Suite 750
Portland, Oregon 97201

Date: 1/10/19

Lab No.: 19-004

Project: Laboratory Testing – WWSP-WTP-1.0

Project No.: 2286.1.1

Report of: Compressive strength of rock

Sample Identification

NTI completed compressive strength of rock testing on samples delivered to our laboratory on January 7, 2018. Testing was performed in accordance with the standards indicated. Our laboratory test results are summarized on the following table.

Laboratory Testing

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

Sample ID	Diameter (inches)	Height (inches)	Rate of Loading (lbs/s)	Uniaxial Compressive Strength (psi)
B-01 C-01 @ 7.12 – 7.76 ft.	2.40	4.80	100	27,467
B-01 C-02 @ 22.5 – 23.35 ft.	2.40	4.81	100	23,727
B-02 C-03 @ 13.35 – 14.3 ft.	2.40	4.83	100	16,746
B-07 C-04 @ 8.3 – 9.3 ft.	2.40	4.83	100	18,990
B-07 C-05 15.8 – 16.7 ft.	2.40	4.80	100	19,466
B-10 C-06 @ 16.5 – 17.5 ft.	2.40	4.80	100	31,235
B-10 C-07 @ 26.26 – 27.4 ft.	2.40	4.83	100	17,446
B-12 C-08 @ 7.8 – 8.6 ft.	2.40	4.81	100	21,764
B-12 C-09 @ 20.7 – 21.7 ft.	2.40	4.80	100	19,622

Attachments: Laboratory Test Results

Copies: Addressee

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SHEET 1 of 1 REVIEWED BY: Bridgett Adame

TECHNICAL REPORT

Report To: Ms. Annie Havekost McMillen Jacobs Associates
1500 SW First Avenue, Suite 750
Portland, Oregon 97201

Date: 3/6/19
Lab No.: 19-047

Project: Laboratory Testing – WWSP-WTP-1.0 **Project No.:** 2286.1.1

Report of: Compressive strength of rock

Sample Identification

NTI completed compressive strength of rock testing on samples delivered to our laboratory on March 4, 2019. Testing was performed in accordance with the standards indicated. Our laboratory test results are summarized on the following table.

Invalid

Laboratory Testing

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

Sample ID	Diameter (inches)	Height (inches)	Rate of Loading (lbs/s)	Uniaxial Compressive Strength (psi)
B-03 C-10 @ 8.65 – 9.1 ft.	2.39	4.86	100	18,552
B-03 C-11 @ 17.3 – 18 ft.	2.38	4.80	100	34,089
~~B-04 C-12 @ 6.3 – 7 ft.~~	~~2.34~~	~~4.72~~	~~100~~	~~60,353~~
B-06 C-14 @ 9.1 – 9.7 ft.	2.33	4.70	100	23,467
B-08 C-17 14.6 – 15.1 ft.	2.33	4.68	100	11,928
B-09 C-13 @ 22.9 – 23.6 ft.	2.38	4.82	100	18,249
B-09 C-15 @ 27 – 27.5 ft.	2.38	4.80	100	27,829
B-09 C-16 @ 29.7 – 30.2 ft.	2.38	4.76	100	22,316
B-11 C-18 @ 17.95 – 18.55 ft.	2.39	4.81	100	15,053
B-11 C-19 @ 27.65 – 34.65 ft.	2.38	4.83	100	27,911

Copies: Addressee

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REVIEWED BY: Bridgett Adame

TECHNICAL REPORT

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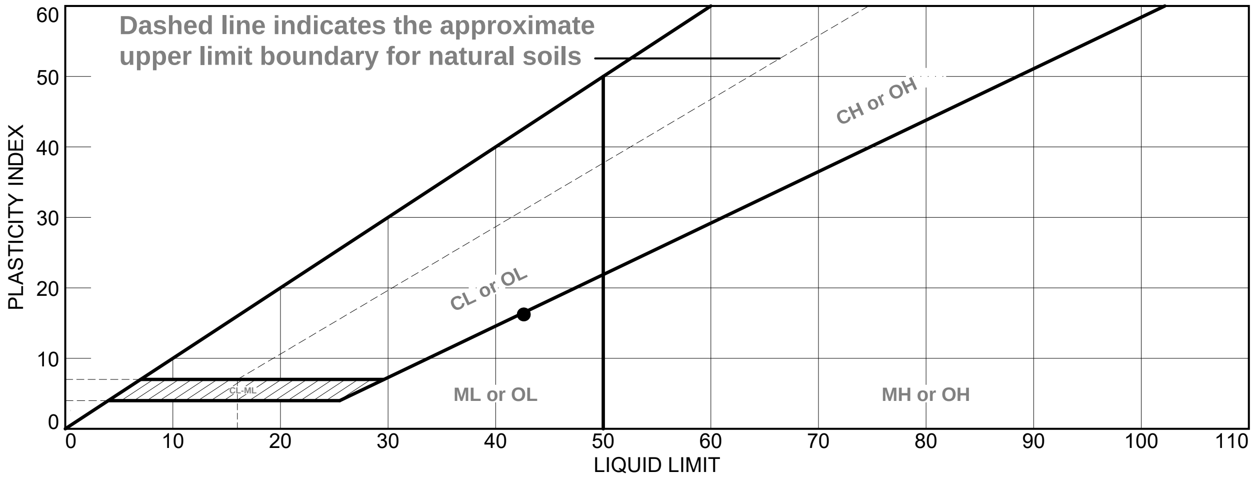
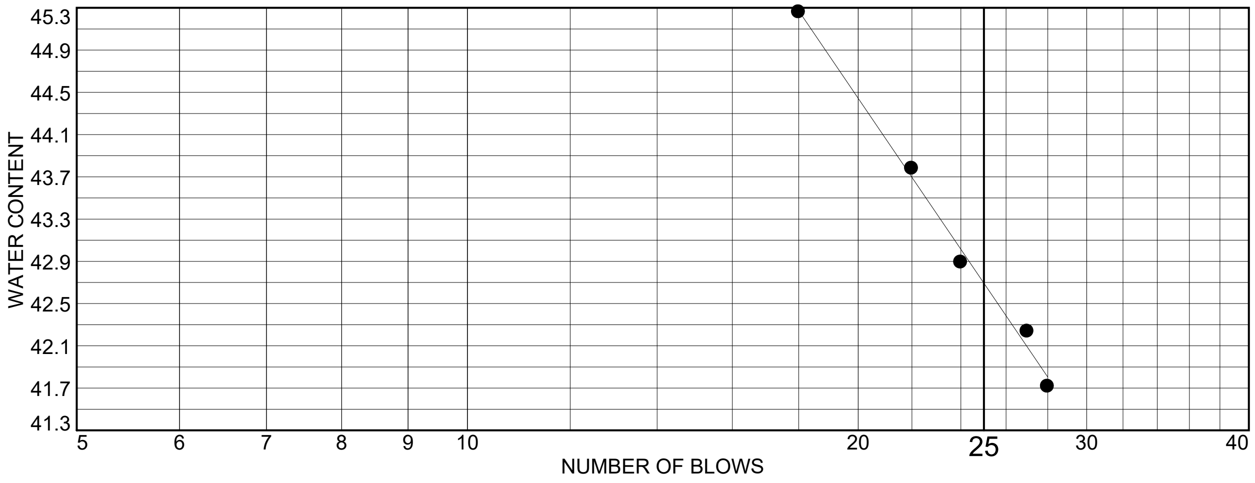
BKA

FOR LAND USE PERMITTING (EXHIBIT B)

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown SILT w/ Sand	42.7	26.6	16.1			

Project No. 024-011 **Client:** McMillen Jacobs Associates

Project: WWSP WTP_1.0 - 5887.0

● **Source of Sample:** B-2 **Depth:** 5-6.5 **Sample Number:** S-2

Remarks:

Benchmark Geolabs, LLC

Figure

BENCHMARK GEOLABS

Corrosivity Tests Summary

BGL # 024-011 **Date:** 1/16/2019 **Tested By:** PJ **Checked:** PJ

Client: McMillen Jacobs Associates **Project:** WWSP WTP_1.0 **Proj. No:** 5887.0

Remarks: The unusually high resistivity value for TP-4, S-1 was confirmed.

Sample Location or ID			**Resistivity @ 15.5 °C (Ohm-cm)**			**Chloride**	**Sulfate**		**pH**	**ORP**		**Sulfide**	**Moisture**	**Soil Visual Description**
			As Rec.	**Min**	**Sat.**	**mg/kg**	**mg/kg**	**%**		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	**%**	
Boring	**Sample, No.**	**Depth, ft.**	ASTM G57	Cal 643	ASTM G57	EPA 300.0	EPA 300.0	EPA 300.0	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
B-1	S-1	2.5-4	-	-	2,354	7	36	0.0036	6.3	502	15	Negative	17.9	Brown Silty SAND
B-2	S-1	2.5-4	-	-	7,884	7	3	0.0003	5.9	513	15	Negative	24.1	Dark Reddish Brown SILT
TP-4	S-1	2-3	-	-	79,679	14	2	0.0002	6.3	520	15	Negative	35.9	Dark Reddish Brown SILT

FOR LAND USE PERMITTING (EXHIBIT B)

BENCHMARK GEOLABS

Moisture-Density-Porosity Report
(ASTM D7263b)

BGL Job No: 024-011 **Project No.** 5887.0 **By:** PJ

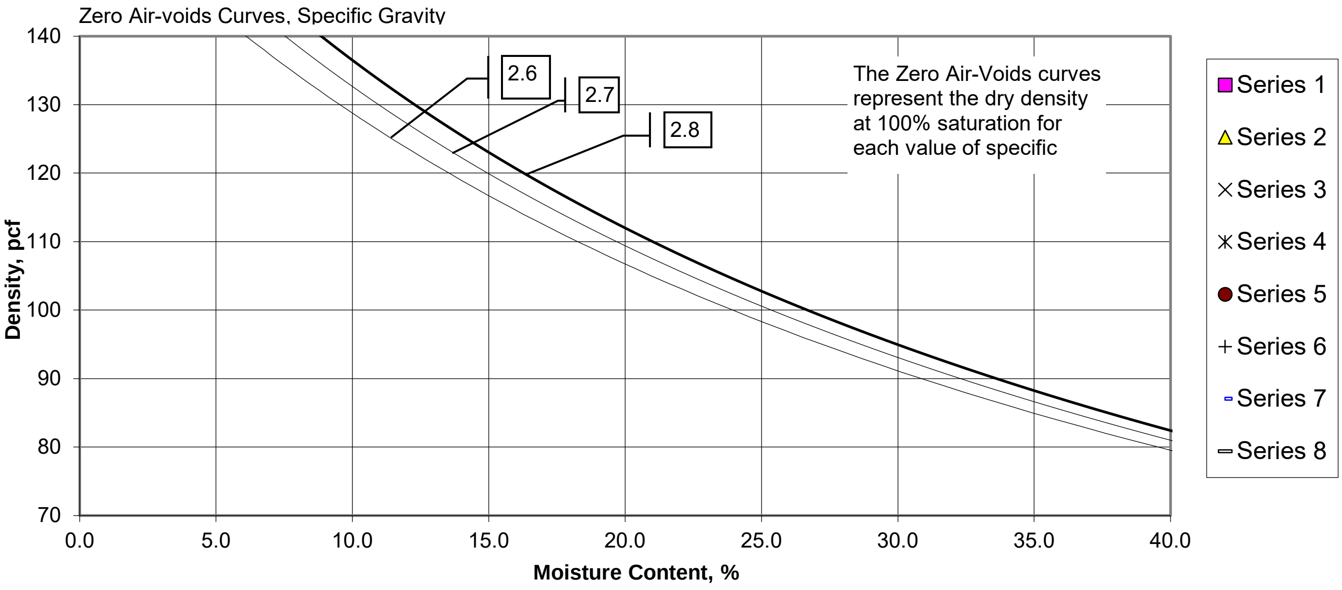
Client: McMillen Jacobs Associates **Date:** 01/06/19

Project Name: WWSP WTP-1.0 **Remarks:**

Boring: **Sample:** **Depth, ft:**	B-2 S-2 5-6.5	B-2 S-3 7.5-8						
Visual Description:	Brown SILT w/ Sand	Brown SILT						
Actual G_s								
Assumed G_s								
Moisture, %	28.9	29.6						
Wet Unit wt, pcf								
Dry Unit wt, pcf								
Dry Bulk Dens.pb, (g/cc)								
Saturation, %								
Total Porosity, %								
Volumetric Water Cont,Θw,%								
Volumetric Air Cont., Θa,%								
Void Ratio								
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered to be approximate.

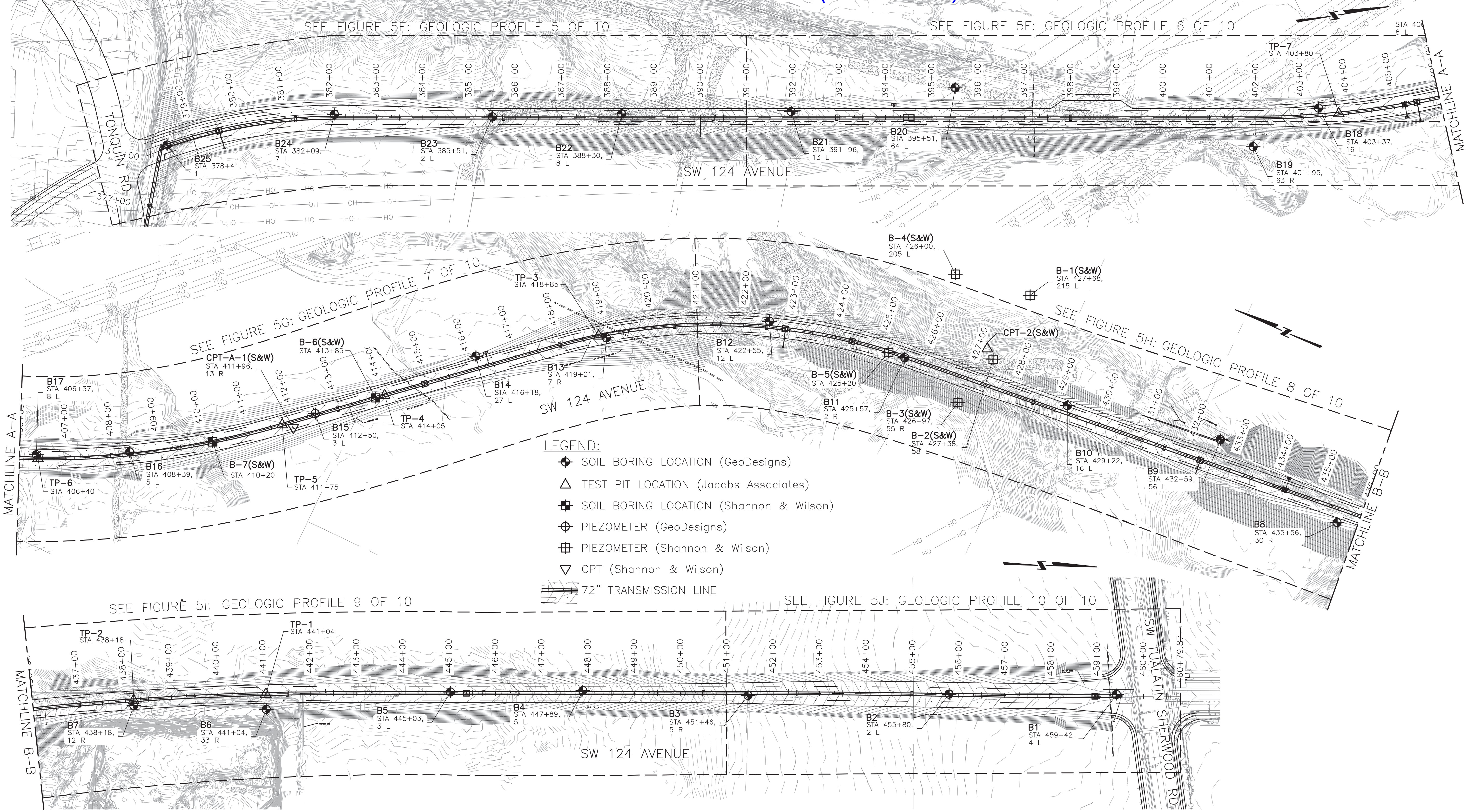
Moisture-Density



Appendix D

Previous Explorations

FOR LAND USE PERMITTING (EXHIBIT B)



GENERAL LAYOUT PLAN

SCALE: 1" = 200'–0"

NOTE: LOCATION OF SHANNON & WILSON BORINGS IS APPROXIMATE

PROJECT # 5106.0

JACOBS ASSOCIATES
Engineers/Consultants

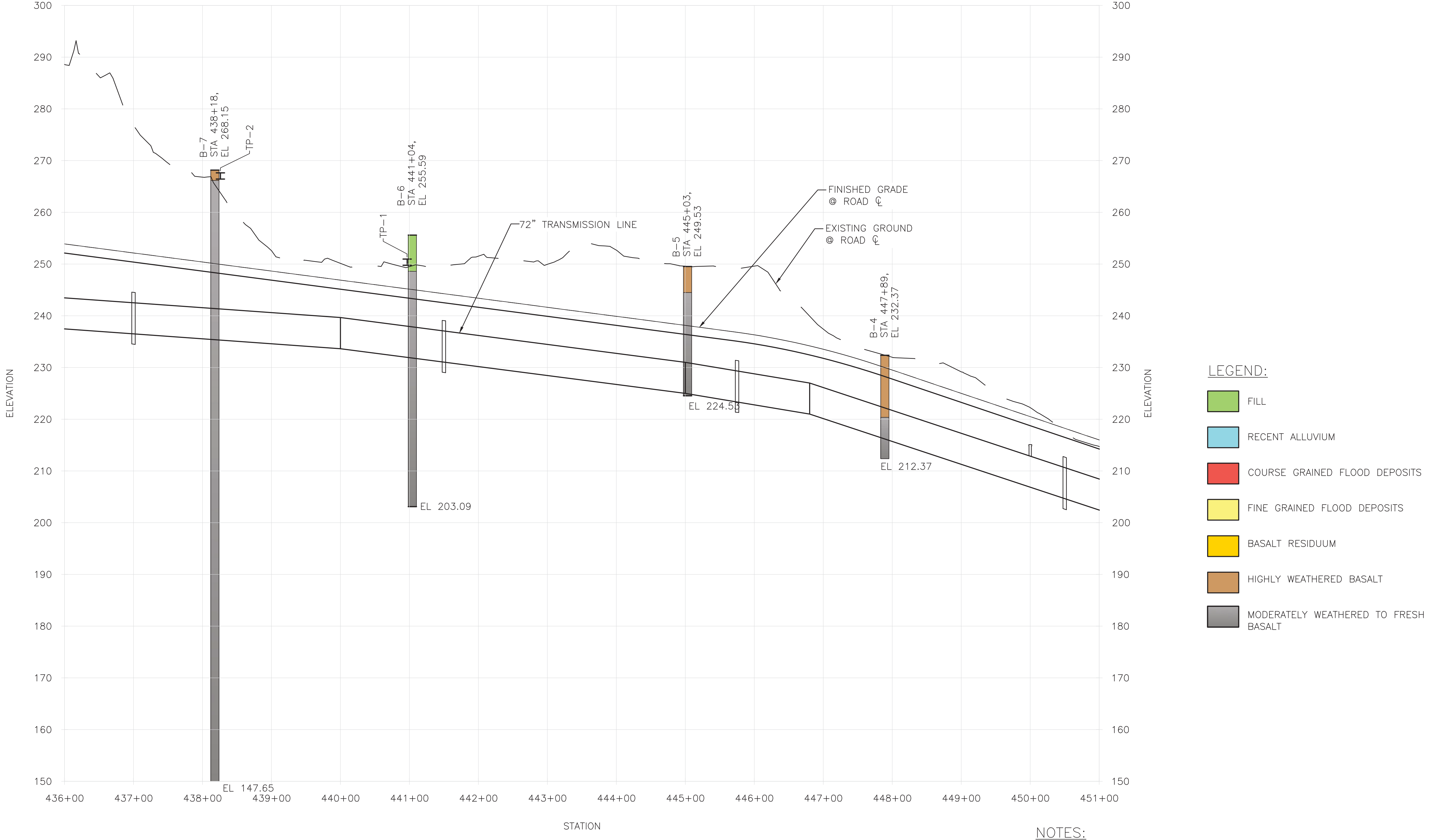
SW 124TH AVENUE TRANSMISSION LINE DESIGN
GEOLOGIC PROFILE

SITE EXPLORATION PLAN

DATE
DECEMBER
2014

FIGURE
4B

FOR LAND USE PERMITTING (EXHIBIT B)



GEOLOGY PARTIAL PROFILE

HORIZ. SCALE: 1" = 150'–0" VERT.SCALE: 1" = 20'–0"

NOTES:

1. BOREHOLE COLLAR ELEVATIONS ARE BASED ON ACTUAL GROUND SURVEY.
2. FOR GROUNDWATER ELEVATIONS AT THE EXPLORATION LOCATIONS, SEE TABLE 2 OF GDR AND INDIVIDUAL BORING LOGS IN APPENDIX A.

PROJECT # 5106.0

JACOBS ASSOCIATES
Engineers/Consultants

SW 124TH AVENUE TRANSMISSION LINE DESIGN
GEOLOGIC PROFILE

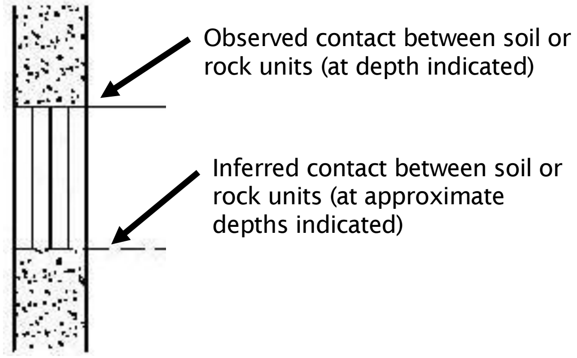
GEOLOGY PROFILE 9 OF 10
STA 436+00 TO 451+00

DATE
DECEMBER
2014

FIGURE
5I

FOR LAND USE PERMITTING (EXHIBIT B)

SYMBOL	SAMPLING DESCRIPTION
◧	Location of sample obtained in general accordance with ASTM D 1586 Standard Penetration Test with recovery
◧	Location of sample obtained using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D 1587 with recovery
▤	Location of sample obtained using Dames & Moore sampler and 300-pound hammer or pushed with recovery
▤	Location of sample obtained using Dames & Moore and 140-pound hammer or pushed with recovery
⧖	Location of sample obtained using 3-inch-O.D. California split-spoon sampler and 140-pound hammer
⊠	Location of grab sample
▮	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown



Graphic Log of Soil and Rock Types

Observed contact between soil or rock units (at depth indicated)

Inferred contact between soil or rock units (at approximate depths indicated)

GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	PP	Pocket Penetrometer
CBR	California Bearing Ratio	P200	Percent Passing U.S. Standard No. 200 Sieve
CON	Consolidation		
DD	Dry Density	RES	Resilient Modulus
DS	Direct Shear	SIEV	Sieve Gradation
HYD	Hydrometer Gradation	TOR	Torvane
MC	Moisture Content	UC	Unconfined Compressive Strength
MD	Moisture-Density Relationship	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal
P	Pushed Sample		

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

FOR LAND USE PERMITTING (EXHIBIT B)

RELATIVE DENSITY - COARSE-GRAINED SOILS

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 - 11	0 - 4
Loose	4 – 10	11 - 26	4 - 10
Medium Dense	10 – 30	26 - 74	10 - 30
Dense	30 – 50	74 - 120	30 - 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOILS

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 - 4	3 – 6	2 - 5	0.25 - 0.50
Medium Stiff	4 - 8	6 – 12	5 - 9	0.50 - 1.0
Stiff	8 - 15	12 – 25	9 - 19	1.0 - 2.0
Very Stiff	15 - 30	25 – 65	19 – 31	2.0 - 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVELS (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVELS WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SANDS (<5% fines)	SW or SP	SAND
		SANDS WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SANDS WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOILS			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soils	Coarse-Grained Soils		Fine-Grained Soils	Coarse-Grained Soils
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

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SOIL CLASSIFICATION SYSTEM

TABLE A-2

FOR LAND USE PERMITTING (EXHIBIT B)

HARDNESS	DESCRIPTION
Extremely Soft (R0)	Indented by thumbnail
Very Soft (R1)	Can be peeled by pocket knife or scratched with finger nail
Soft (R2)	Can be peeled by a pocket knife with difficulty
Medium Hard (R3)	Can be scratched by knife or pick
Hard (R4)	Can be scratched with knife or pick only with difficulty
Very Hard (R5)	Cannot be scratched with knife or sharp pick
WEATHERING	**DESCRIPTION**
Decomposed	Rock mass is completely decomposed
Predominantly Decomposed	Rock mass is more than 50% decomposed
Moderately Weathered	Rock mass is decomposed locally
Slightly Weathered	Rock mass is generally fresh
Fresh	No discoloration in rock fabric
JOINT SPACING	**DESCRIPTION**
Very Close	Less than 2 inches
Close	2 inches to 1 foot
Moderate Close	1 foot to 3 feet
Wide	3 feet to 10 feet
Very Wide	Greater than 10 feet
FRACTURING	**FRACTURE SPACING**
Very Intensely Fractured	Chips and fragments with a few scattered short core lengths
Intensely Fractured	0.1 foot to 0.3 foot with scattered fragments intervals
Moderately Fractured	0.3 foot to 1 foot with most lengths 0.6 foot
Slightly Fractured	1 foot to 3 feet
Very Slightly Fractured	Greater than 3 feet
Unfractured	No fractures
HEALING	**DESCRIPTION**
Not Healed	Discontinuity surface, fractured zone, sheared material or filling not re-cemented
Partly Healed	Less than 50% of fractured or sheared material
Moderately Healed	Greater than 50% of fractured or sheared material
Totally Healed	All fragments bonded

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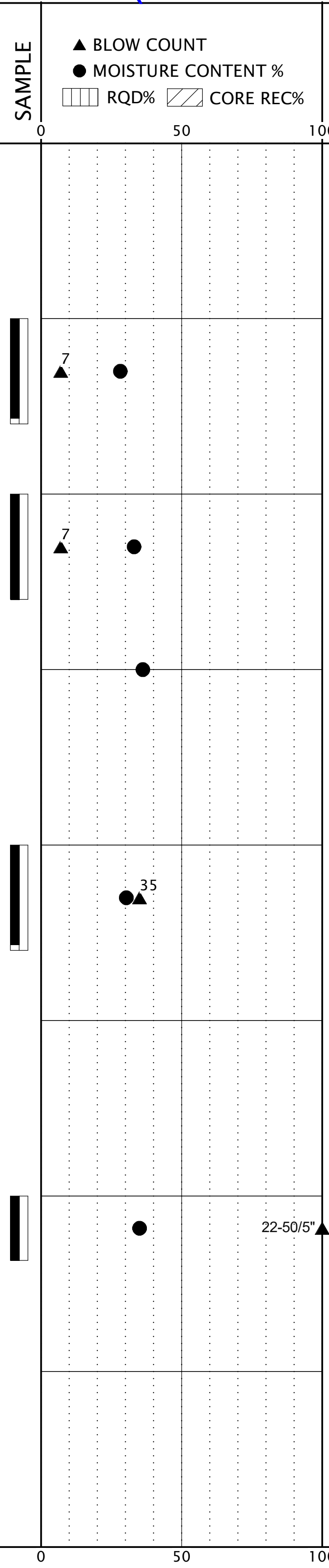
ROCK CLASSIFICATION SYSTEM

TABLE A-3

FOR LAND USE PERMITTING (EXHIBIT B)

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	INSTALLATION AND COMMENTS
0.0		Medium stiff, brown CLAY (CL), trace sand; moist, sand is fine (topsoil to 6 inches, 2-inch-thick root zone; alluvium).	212.2			
2.5						
5.0						LL = 40% PL = 25%
		minor sand at 5.5 feet		ATT		
7.5						
10.0		Dense, gray with orange mottled, clayey SAND with gravel (SC); moist, fine to coarse, relict rock structure (decomposed basalt).	202.2 10.0	SIEV		Driller comment: Harder at 10.0 feet.
12.5						
		Very dense, gray with orange mottled, silty GRAVEL with sand (GM); moist (weathered basalt).	198.2 14.0			
15.0						
		Very dense, dark gray GRAVEL (GP), trace sand; moist.	195.7 16.5			Increase in drill resistance at 16.5 feet.
17.5						
20.0						



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: CR

COMPLETED: 06/19/13

BORING METHOD: mud rotary (see report text)

BORING BIT DIAMETER: 4 7/8-inch

BORING LOG DEA-118-02-5.12-B1_32_36-38_56-TWWD1_13-INF1_2.GPJ GEODESIGN.GDT PRINT DATE: 11/3/14:KT

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BORING B-3

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-3

FOR LAND USE PERMITTING (EXHIBIT B)

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% 0 50 100	INSTALLATION AND COMMENTS
20.0		(continued from previous page) Exploration completed at a depth of 20.2 feet.	192.0 20.2			50/2" ▲	
22.5							
25.0							
27.5							
30.0							
32.5							
35.0							
37.5							
40.0						0 50 100	

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: CR COMPLETED: 06/19/13

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

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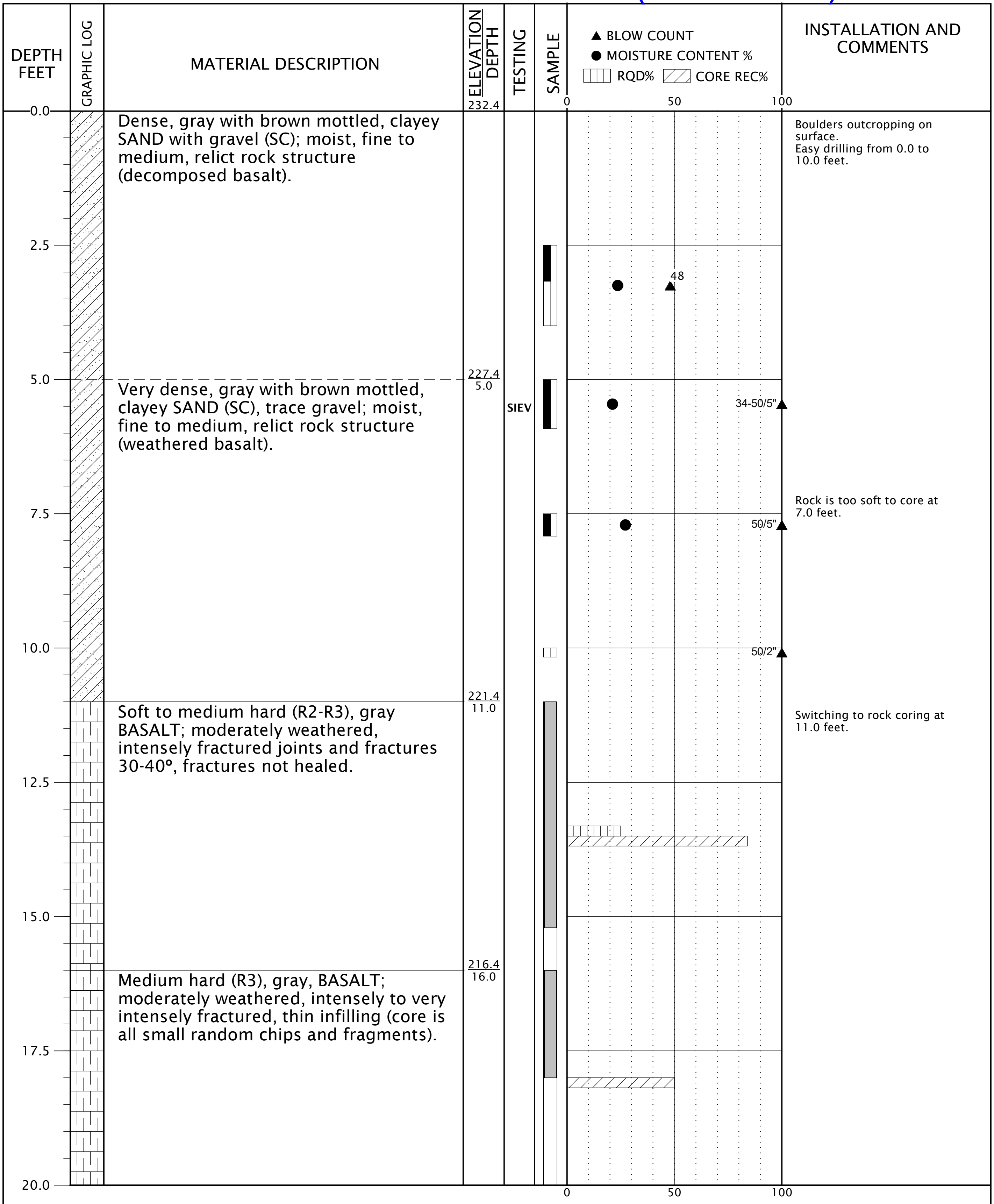
BORING B-3
(continued)

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-3

BORING LOG DEA-118-02-5.12-B1_32_36-38_56-TVWD1_13-INF1_2.GPJ GEODESIGN.GDT PRINT DATE: 11/3/14:KT

FOR LAND USE PERMITTING (EXHIBIT B)



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Dense, gray with brown mottled, clayey SAND with gravel (SC); moist, fine to medium, relict rock structure (decomposed basalt).	232.4		Boulders outcropping on surface. Easy drilling from 0.0 to 10.0 feet.
5.0	Very dense, gray with brown mottled, clayey SAND (SC), trace gravel; moist, fine to medium, relict rock structure (weathered basalt).	227.4 5.0	SIEV	Rock is too soft to core at 7.0 feet.
11.0	Soft to medium hard (R2-R3), gray BASALT; moderately weathered, intensely fractured joints and fractures 30-40°, fractures not healed.	221.4 11.0		Switching to rock coring at 11.0 feet.
16.0	Medium hard (R3), gray, BASALT; moderately weathered, intensely to very intensely fractured, thin infilling (core is all small random chips and fragments).	216.4 16.0		

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: CR

COMPLETED: 06/19/13

BORING METHOD: mud rotary (see report text)

BORING BIT DIAMETER: 4 7/8-inch

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BORING B-4

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-4

FOR LAND USE PERMITTING (EXHIBIT B)

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% 0 50 100	INSTALLATION AND COMMENTS
20.0		Exploration completed at depth of 20.0 feet.	212.4 20.0				
22.5							
25.0							
27.5							
30.0							
32.5							
35.0							
37.5							
40.0						0 50 100	

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: CR

COMPLETED: 06/19/13

BORING METHOD: mud rotary (see report text)

BORING BIT DIAMETER: 4 7/8-inch

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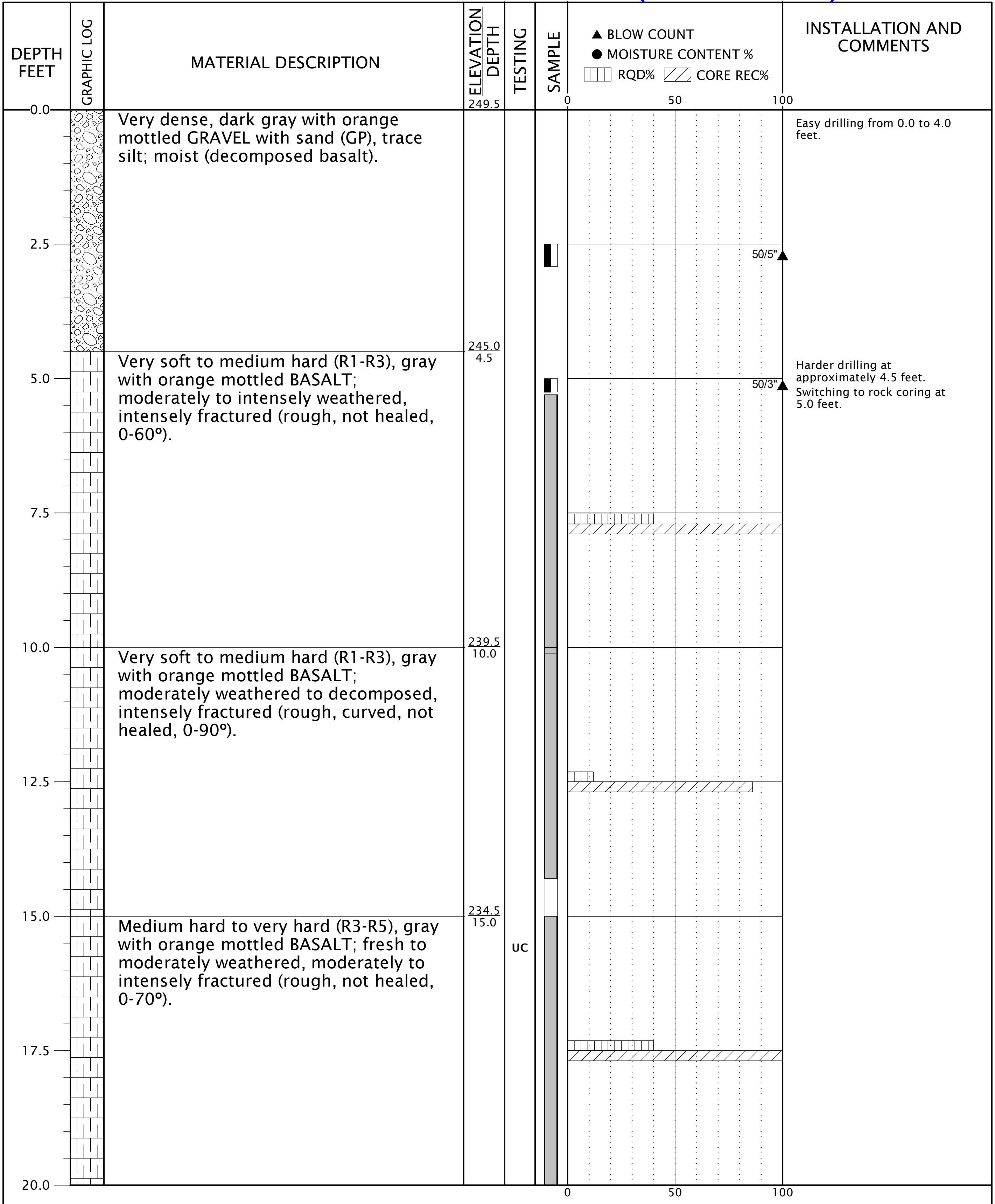
BORING B-4
(continued)

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-4

BORING LOG DEA-118-02-5.12-B1_32_36-38_56-TVWD1_13-INF1_2.GPJ GEODESIGN.GDT PRINT DATE: 11/3/14:KT

FOR LAND USE PERMITTING (EXHIBIT B)



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Very dense, dark gray with orange mottled GRAVEL with sand (GP), trace silt; moist (decomposed basalt).	249.5		Easy drilling from 0.0 to 4.0 feet.
4.5	Very soft to medium hard (R1-R3), gray with orange mottled BASALT; moderately to intensely weathered, intensely fractured (rough, not healed, 0-60º).	245.0 4.5		Harder drilling at approximately 4.5 feet. Switching to rock coring at 5.0 feet.
10.0	Very soft to medium hard (R1-R3), gray with orange mottled BASALT; moderately weathered to decomposed, intensely fractured (rough, curved, not healed, 0-90º).	239.5 10.0		
15.0	Medium hard to very hard (R3-R5), gray with orange mottled BASALT; fresh to moderately weathered, moderately to intensely fractured (rough, not healed, 0-70º).	234.5 15.0	UC	

DRILLED BY: Western States Soil Conservation, Inc.

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COMPLETED: 06/20/13

BORING METHOD: mud rotary (see report text)

BORING BIT DIAMETER: 4 7/8-inch

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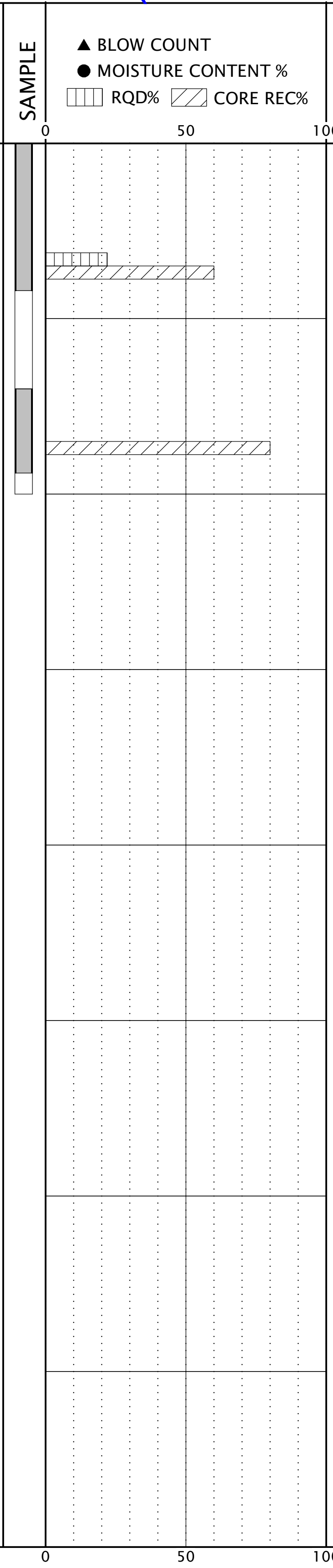
BORING B-5

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-5

FOR LAND USE PERMITTING (EXHIBIT B)

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
20.0	Medium hard to hard (R3-R), gray BASALT; slightly weathered, intensely fractured (joints not healed, 0-45°, light orange infilling).	229.5 20.0	
22.5	becomes vesicular at 23.0 feet		
	Medium hard to hard (R3-R4), gray BASALT; slightly weathered to fresh, very intensely fractured (joints rough, not healed, 0-80°), vesicular.	226.0 23.5	Possible flow contact at 24.7 feet with brown clay with some sand.
25.0	Exploration completed at a depth of 25.0 feet.	224.5 25.0	
27.5			
30.0			
32.5			
35.0			
37.5			
40.0			



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: CR
COMPLETED: 06/20/13

BORING METHOD: mud rotary (see report text)
BORING BIT DIAMETER: 4 7/8-inch

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BORING B-5
(continued)

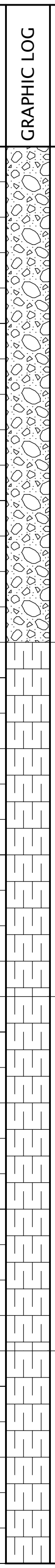
SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

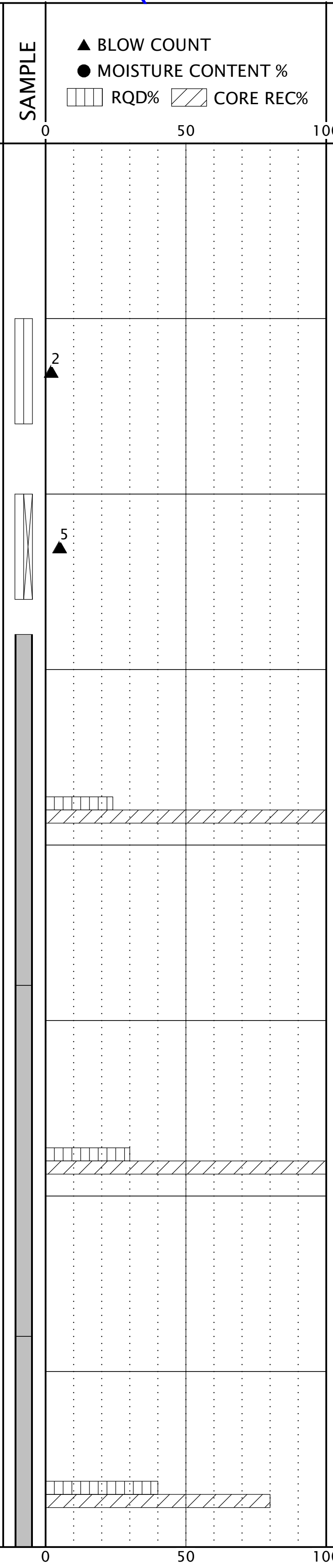
FIGURE A-5

BORING LOG DEA-118-02-5.12-B1_32_36-38_56-TVWD1_13-INF1_2.GPJ GEODESIGN.GDT PRINT DATE: 11/3/14:KT

FOR LAND USE PERMITTING (EXHIBIT B)

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Loose, gray GRAVEL with boulders and cobbles (GP); moist - **FILL**.	255.6		
2.5				Driller comment: softer at 2.0 feet.
				Loss of circulation at 3.0 feet. Possible void space within boulder fill.
5.0				Bent 3-inch SPT during sample at 5.0 feet.
				Driller comment: hard rock at 6.5 feet.
	Medium hard to hard (R3-R4), gray with orange mottled BASALT; fresh to moderately weathered, intensely fractured (very thin to moderate fracture filling, tight to open rough, not healed, 0-55°).	248.6 7.0		Switch to rock coring at 7.0 feet.
7.5			UC	▽ 8.9 feet, during drilling
10.0				
	Medium hard to hard (R3-R4), gray with orange mottled BASALT; fresh to moderately weathered, intensely fractured (clean to moderate infilling, tight to open, planar to curved, rough, 0-85°).	243.6 12.0		
12.5				
15.0				
	Medium hard to hard (R3-R4), gray with orange mottled BASALT; slightly weathered to fresh, moderately fractured (clean to very thin filling, tight to open, not healed, 0-45°). becomes vesicular at 18.0 feet	238.6 17.0		
17.5				
20.0				





DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: CR

COMPLETED: 06/21/13

BORING METHOD: mud rotary (see report text)

BORING BIT DIAMETER: 4 7/8-inch

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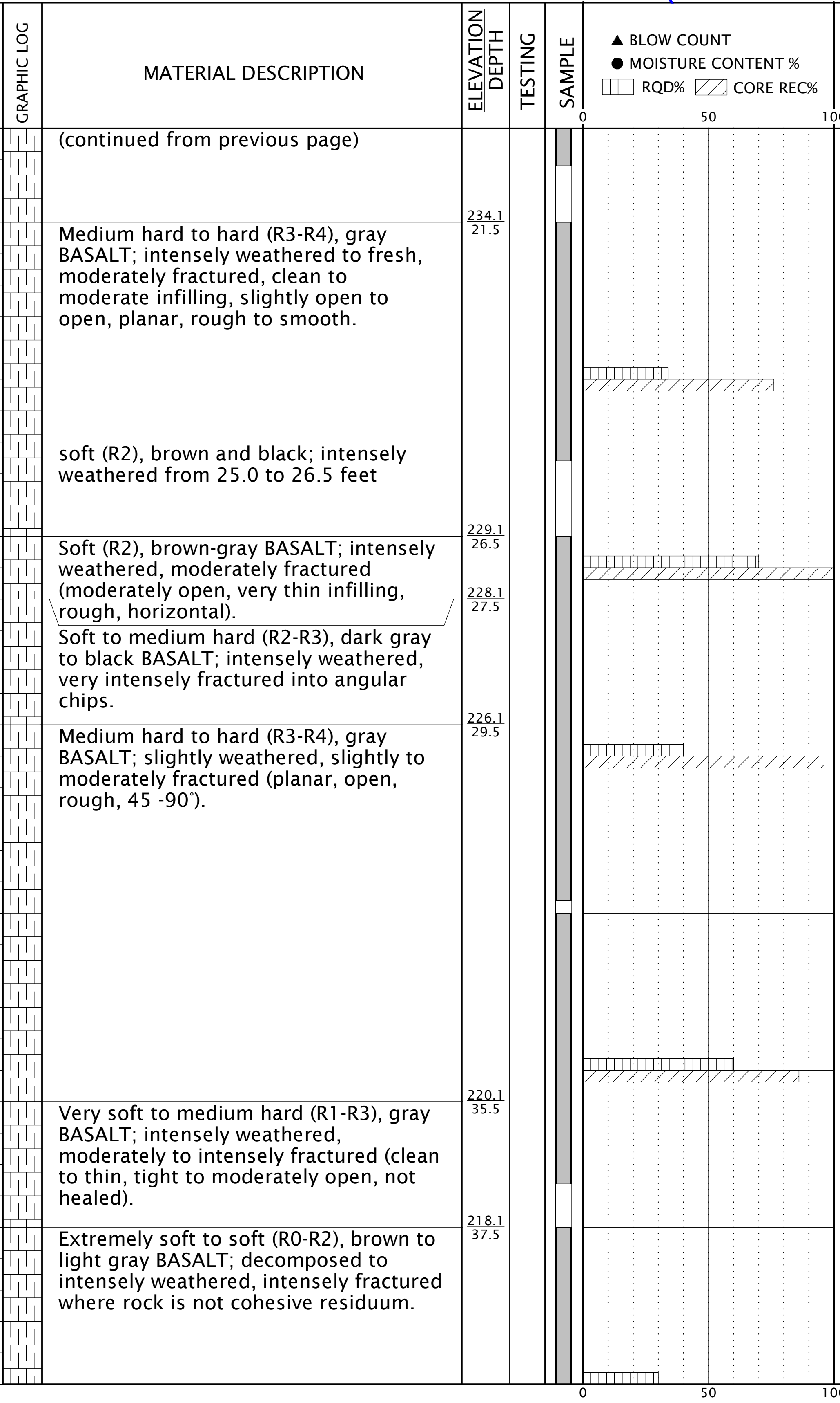
BORING B-6

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-6

FOR LAND USE PERMITTING (EXHIBIT B)

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH
20.0	(continued from previous page)	
	Medium hard to hard (R3-R4), gray BASALT; intensely weathered to fresh, moderately fractured, clean to moderate infilling, slightly open to open, planar, rough to smooth.	234.1 21.5
22.5		
25.0	soft (R2), brown and black; intensely weathered from 25.0 to 26.5 feet	
	Soft (R2), brown-gray BASALT; intensely weathered, moderately fractured (moderately open, very thin infilling, rough, horizontal).	229.1 26.5
27.5	Soft to medium hard (R2-R3), dark gray to black BASALT; intensely weathered, very intensely fractured into angular chips.	228.1 27.5
	Medium hard to hard (R3-R4), gray BASALT; slightly weathered, slightly to moderately fractured (planar, open, rough, 45 -90°).	226.1 29.5
30.0		
32.5		
35.0		
	Very soft to medium hard (R1-R3), gray BASALT; intensely weathered, moderately to intensely fractured (clean to thin, tight to moderately open, not healed).	220.1 35.5
37.5	Extremely soft to soft (R0-R2), brown to light gray BASALT; decomposed to intensely weathered, intensely fractured where rock is not cohesive residuum.	218.1 37.5
40.0		



INSTALLATION AND COMMENTS

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: CR
COMPLETED: 06/21/13

BORING METHOD: mud rotary (see report text)
BORING BIT DIAMETER: 4 7/8-inch

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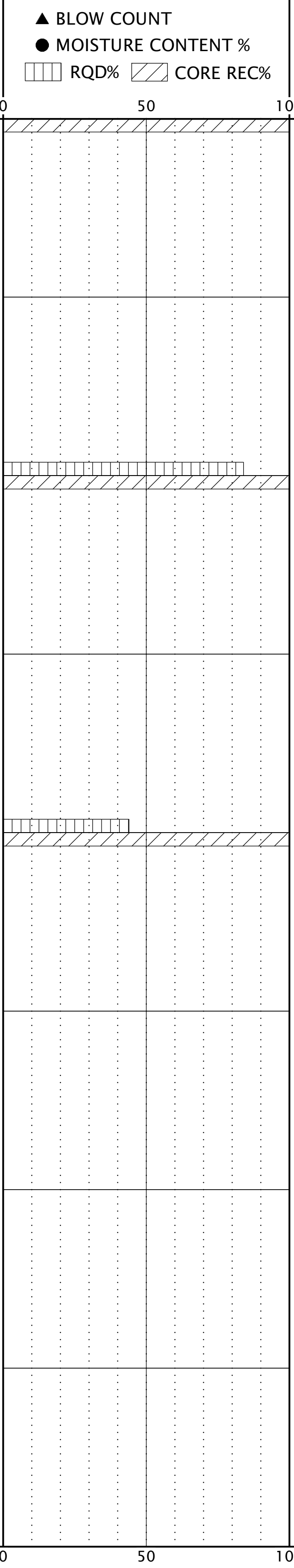
BORING B-6
(continued)

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-6

FOR LAND USE PERMITTING (EXHIBIT B)

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	INSTALLATION AND COMMENTS
40.0	(continued from previous page)				
42.5	Very soft to medium hard (R1-R3), gray with orange mottled BASALT; intensely to moderately weathered, moderately fractured (rough, tight to moderately open, clean to very thin infilling, not healed, 0-45°). vesicular at 43.0 feet	213.1 42.5			
45.0					
47.5	Medium hard (R3), gray with orange mottled BASALT; slightly weathered, moderately fractured (slightly open to open, clear to very thin infilling, rough, not healed), vesicular.	208.1 47.5			
50.0					
52.5	Exploration completed at a depth of 52.5 feet.	203.1 52.5			
55.0					
57.5					
60.0					



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: CR
COMPLETED: 06/21/13

BORING METHOD: mud rotary (see report text)
BORING BIT DIAMETER: 4 7/8-inch

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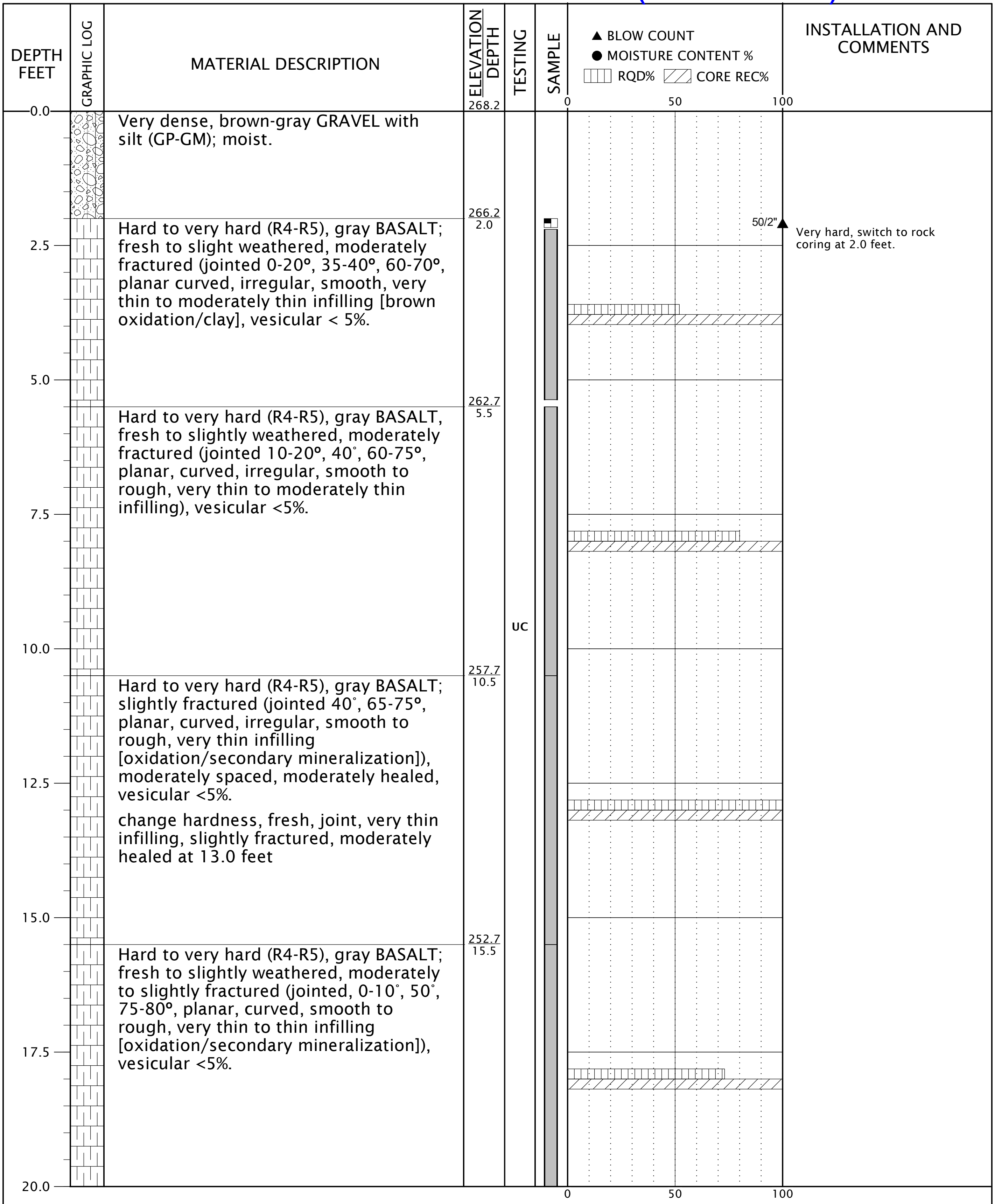
BORING B-6
(continued)

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-6

FOR LAND USE PERMITTING (EXHIBIT B)



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Very dense, brown-gray GRAVEL with silt (GP-GM); moist.	268.2		
	Hard to very hard (R4-R5), gray BASALT; fresh to slight weathered, moderately fractured (jointed 0-20°, 35-40°, 60-70°, planar curved, irregular, smooth, very thin to moderately thin infilling [brown oxidation/clay], vesicular < 5%.	266.2 2.0		Very hard, switch to rock coring at 2.0 feet.
	Hard to very hard (R4-R5), gray BASALT, fresh to slightly weathered, moderately fractured (jointed 10-20°, 40˚, 60-75°, planar, curved, irregular, smooth to rough, very thin to moderately thin infilling), vesicular <5%.	262.7 5.5	UC	
	Hard to very hard (R4-R5), gray BASALT; slightly fractured (jointed 40˚, 65-75°, planar, curved, irregular, smooth to rough, very thin infilling [oxidation/secondary mineralization]), moderately spaced, moderately healed, vesicular <5%. change hardness, fresh, joint, very thin infilling, slightly fractured, moderately healed at 13.0 feet	257.7 10.5		
	Hard to very hard (R4-R5), gray BASALT; fresh to slightly weathered, moderately to slightly fractured (jointed, 0-10˚, 50˚, 75-80°, planar, curved, smooth to rough, very thin to thin infilling [oxidation/secondary mineralization]), vesicular <5%.	252.7 15.5		
20.0				

DRILLED BY: Western States Soil Conservation, Inc.   LOGGED BY: JGH   COMPLETED: 06/26/13

BORING METHOD: mud rotary and HQ rock coring (see report text)   BORING BIT DIAMETER: 4 7/8-inch

GEODESIGN INC

15575 SW Sequoia Parkway - Suite 100
Portland OR 97224
Off 503.968.8787 Fax 503.968.3068

DEA-118-02-5.12

NOVEMBER 2014

BORING B-7

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

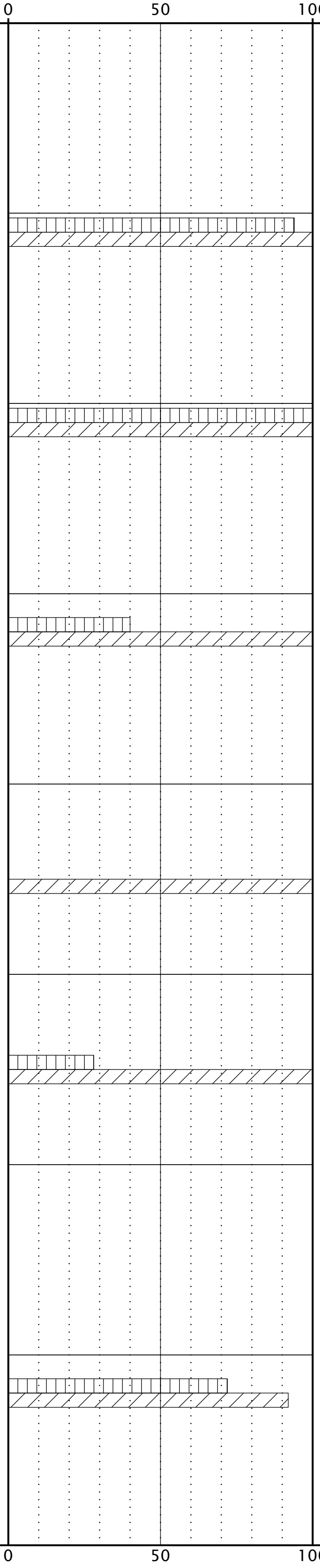
FIGURE A-7

FOR LAND USE PERMITTING (EXHIBIT B)

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING
20.0	(continued from previous page)		
	Hard to very hard (R4-R5), gray BASALT; fresh, slightly fractured (jointed, 15˚ and 40º [top and bottom], thin infilling [iron oxide/secondary white mineralization] curved, smooth to rough), vesicular <5%.	247.7 20.5	
22.5			UC
25.0	Hard to very hard (R4-R5), gray BASALT; fresh, moderately fractured (jointed 40˚ and 60º).	243.2 25.0 242.7 25.5	
27.5	Hard to very hard (R4-R5), gray BASALT; fresh to slightly weathered, slightly to moderately fractured (jointed 25˚, 40˚, 65˚ to vertical, curved, undulating, planar, smooth to rough, very thin infilling), vesicular <5%. becomes medium hard (R3), gray-brown; moderately to intensely weathered, intensely fractured, vesicular 15%, hydrothermal alteration/brecciated at 29.0 feet		
30.0	Soft to medium hard (R2-R3), gray-brown BASALT; moderately weathered, moderately to intensely fractured (jointed 30º, 50º, 80º, planar, curved, undulating, smooth to rough with very thin to thin infilling [<1 mm to 3/8-inch]), vesicular 10%.	237.7 30.5	
32.5	Very soft to medium hard (R1-R3), brown-gray BASALT; moderately to intensely weathered, moderately to intensely fractured, brecciated vesicles <5%, flow bottom. light brown interflow, soil horizon from 34.0 to 35.5	236.2 32.0	
35.0	Very soft to soft (R1-R2), dark gray to brown-light gray BASALT; intensely weathered to decomposed, intensely fractured (jointed 10º, 30º, curved, planar, smooth, thin, brecciated with clay), vesicular with clay 5-10% vesicles, hydrothermal alteration.	232.7 35.5	
37.5			
40.0			

SAMPLE

▲ BLOW COUNT
● MOISTURE CONTENT %
▥ RQD%
▨ CORE REC%



INSTALLATION AND COMMENTS

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: JGH
COMPLETED: 06/26/13

BORING METHOD: mud rotary and HQ rock coring (see report text)
BORING BIT DIAMETER: 4 7/8-inch

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DEA-118-02-5.12

NOVEMBER 2014

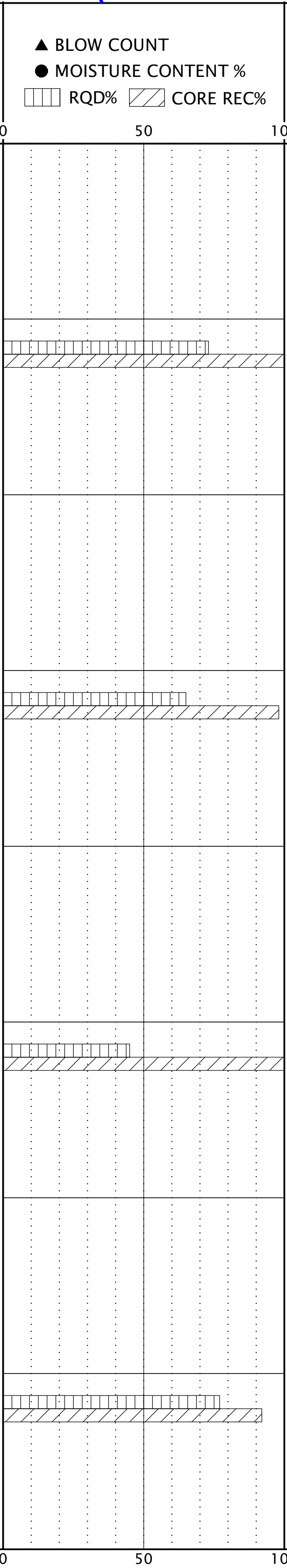
BORING B-7
(continued)

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-7

FOR LAND USE PERMITTING (EXHIBIT B)

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE
40.0	(continued from previous page)			
	Alternating zones of soft to hard (R2-R4), brown-gray BASALT; decomposed to fresh, slightly to moderate fracture (jointed 20-30º, 50-60º, planar curved, rough, very thin to thin infilling [iron oxide/clay]), vesicular 5-20%, partially infilled with clay, brecciated zones.	227.7 40.5		
42.5				
	Soft (R2), brown-gray BASALT; moderately to intensely weathered, intensely fracture (jointed 10˚, 30˚, 50º, planar, curved, undulating, rough, clean to moderate infilling [iron oxide/dark gray secondary mineralization]), vesicular 20-25%.	223.7 44.5		
45.0				
	Hard to very hard (R4-R5), gray BASALT; fresh to slightly weathered, slightly to moderately fractured, vesicular 5%.	221.7 46.5		
47.5				
50.0				
52.5				
	Soft to medium hard (R2-R3), gray to red-brown BASALT; slightly to intensely weathered, intensely to moderately fracture (jointed 0-10˚, 50º, 60º, planar, curved, undulating, smooth to rough, very thin infilling [iron oxide/clay]), vesicular 15-20% mm scale.	215.2 53.0		
	Very soft to soft (R1-R2), gray-brown BASALT; intensely weathered, intensely to very intensely fracture (jointed 0-20˚, 40˚, 60˚ to vertical with intersecting joints, moderate to thin infilling [non oxide and clay]), vesicular 20% to 1/4-diameter. becomes red-brown at 55.0 feet	214.0 54.2		
55.0				
	Soft to medium hard (R2-R3), orange-brown to gray BASALT; moderately to intensely weathered, moderately to slightly fractured (0-10˚, 20-30˚, planar, irregular, rough), vesicular 25-35%,	212.7 55.5		
57.5				
60.0				



INSTALLATION AND COMMENTS

BORING LOG DEA-118-02-5.12-B1_32_36-38_56-TWWD1_13-INF1_2.GPJ GEODESIGN.GDT PRINT DATE: 11/3/14:KT

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/26/13

BORING METHOD: mud rotary and HQ rock coring (see report text) BORING BIT DIAMETER: 4 7/8-inch

GEODESIGN INC

15575 SW Sequoia Parkway - Suite 100
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Off 503.968.8787 Fax 503.968.3068

DEA-118-02-5.12

NOVEMBER 2014

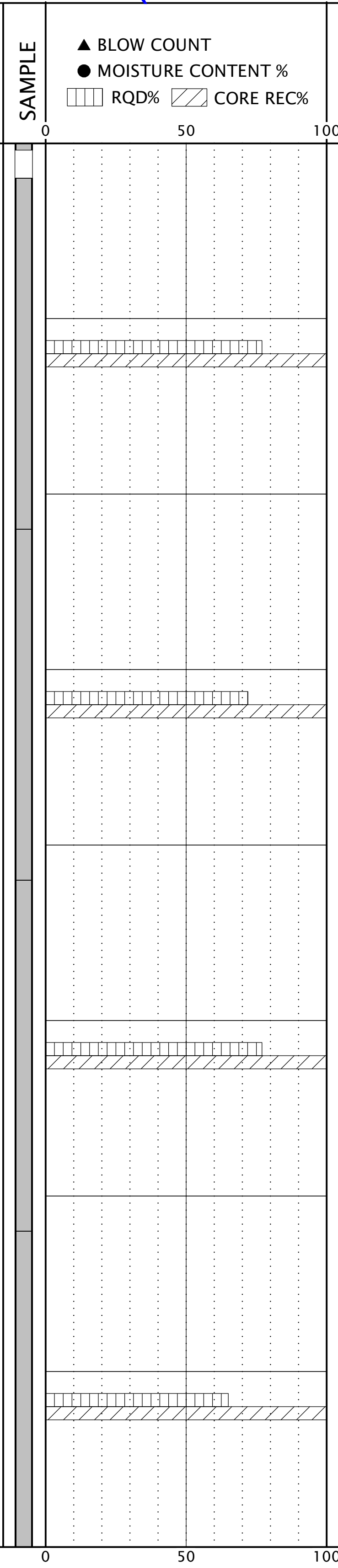
BORING B-7
(continued)

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-7

FOR LAND USE PERMITTING (EXHIBIT B)

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING
60.0	brecciated, hydrothermally altered, partially infilled with clay to 3/4-inch diameter. (continued from previous page)	207.7 60.5	
62.5	Hard to very hard (R4-R5), gray-brown BASALT; fresh to slightly weathered, slightly to moderately fractured (jointed 20˚, 35˚, 60º, planar, curved, undulating, very thin to moderate infilling {thickness <1 mm to <0.1 foot with iron oxide clay/breccia], smooth to rough with with slickensides. vesicular 5-10% at 62.0 feet vesicular <5% at 64.5 feet		
65.0	Soft to medium hard (R2-R3), brown-red to gray BASALT; moderately to intensely weathered, moderately fractured (jointed 30˚, 50˚, 60-70˚, planar, curved, undulating, thin to moderate infilling [iron oxide and clay breccia to 0.1 foot]), vesicular 20-30%, 1/2-inch diameter (60% infilled with clay), brecciated, hydrothermal alteration.	202.7 65.5	
67.5			
70.0	Medium hard to hard (R3-R4), gray-brown BASALT; slightly to moderately weathered, moderately fractured (jointed 20˚, 40º, 60º, and 80º to vertical, planar, curved, and undulating, very thin to thin infilling [iron oxide and clay to 3/8-inch]), vesicular 20-25% to 3/4-inch diameter with approximately 50% filled with clay, brecciated in zones with possible faint slickensides on wider joint.	197.7 70.5	
72.5			
75.0	Soft to medium hard (R2-R3), brown-orange to gray BASALT; moderately to intensely weathered, moderately fracture (jointed 10˚, 20˚, 50˚, 65-70º, planar, curved undulating, smooth to rough with moderately thin to thin infilling [clay iron oxide]), vesicular <10% in zones, brecciated in zones. very soft to soft (R1-R2) altered zone from 78.8 to 79.4 feet (interflow?)	192.7 75.5	
77.5			
80.0			



INSTALLATION AND COMMENTS

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/26/13

BORING METHOD: mud rotary and HQ rock coring (see report text) BORING BIT DIAMETER: 4 7/8-inch

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DEA-118-02-5.12

NOVEMBER 2014

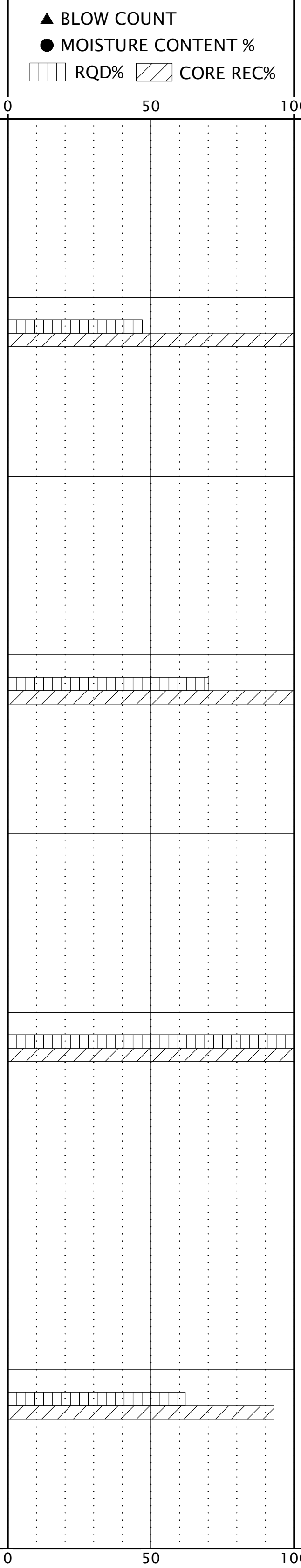
BORING B-7
(continued)

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-7

FOR LAND USE PERMITTING (EXHIBIT B)

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE
80.0		(continued from previous page)			
		Medium hard to hard (R3-R4), gray BASALT; slightly to moderately weathered, moderately fractured (jointed 20˚, 35-40˚, 60˚, 75° planar, curved, undulating, stepped, rough, moderately thin infilling [iron oxide and clay]), vesicular20% to 1/4-inch diameter.	187.7 80.5		
82.5					
		gray-red interflow, clay from 84.0 to 84.5 feet			
85.0		soft (R2); intensely fractured at 85.0 feet			
		Medium hard to hard (R3-R4), gray-brown BASALT; fresh to slightly weathered, slightly to moderately fractured (jointed 20˚, 40˚, 60-70°, very thin to thin infilling [iron oxide/clay] smooth to rough, with intersecting joints), vesicular 30% to 1/2-inch diameter. gray at 87.0 feet	182.7 85.5		
87.5					
90.0					
		Hard to very hard (R4-R5), gray BASALT; fresh, moderately to slightly fractured (jointed 10˚, 20˚, 55°, planar undulating/curved, rough, very thin iron oxide staining), vesicular 25-30% to 3/4-inch diameter with rare zeolites.	177.7 90.5		
92.5					
		vesicular <10% at 94.0 feet			
95.0					
		Hard to very hard (R4-R5), gray BASALT; fresh to slightly weathered, moderately fractured (jointed 10-20˚, 30°, 65-80°, curved, planar, undulating, smooth to rough with slickensides on 70° joints, very thin infilling [iron oxide] with planar joint at 97.0 feet [approximately 3 inches of clay], with intersecting joints), vesicular <5%. zone of yellow alteration (hydrothermal?) from 96.5 to 97.9 feet	172.7 95.5		
97.5					
100.0					



INSTALLATION AND COMMENTS

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: JGH

COMPLETED: 06/26/13

BORING METHOD: mud rotary and HQ rock coring (see report text)

BORING BIT DIAMETER: 4 7/8-inch

BORING LOG DEA-118-02-5.12-B1_32_36-38_56-TWWD1_13-INF1_2.GPJ GEODESIGN.GDT PRINT DATE: 11/3/14:KT

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DEA-118-02-5.12

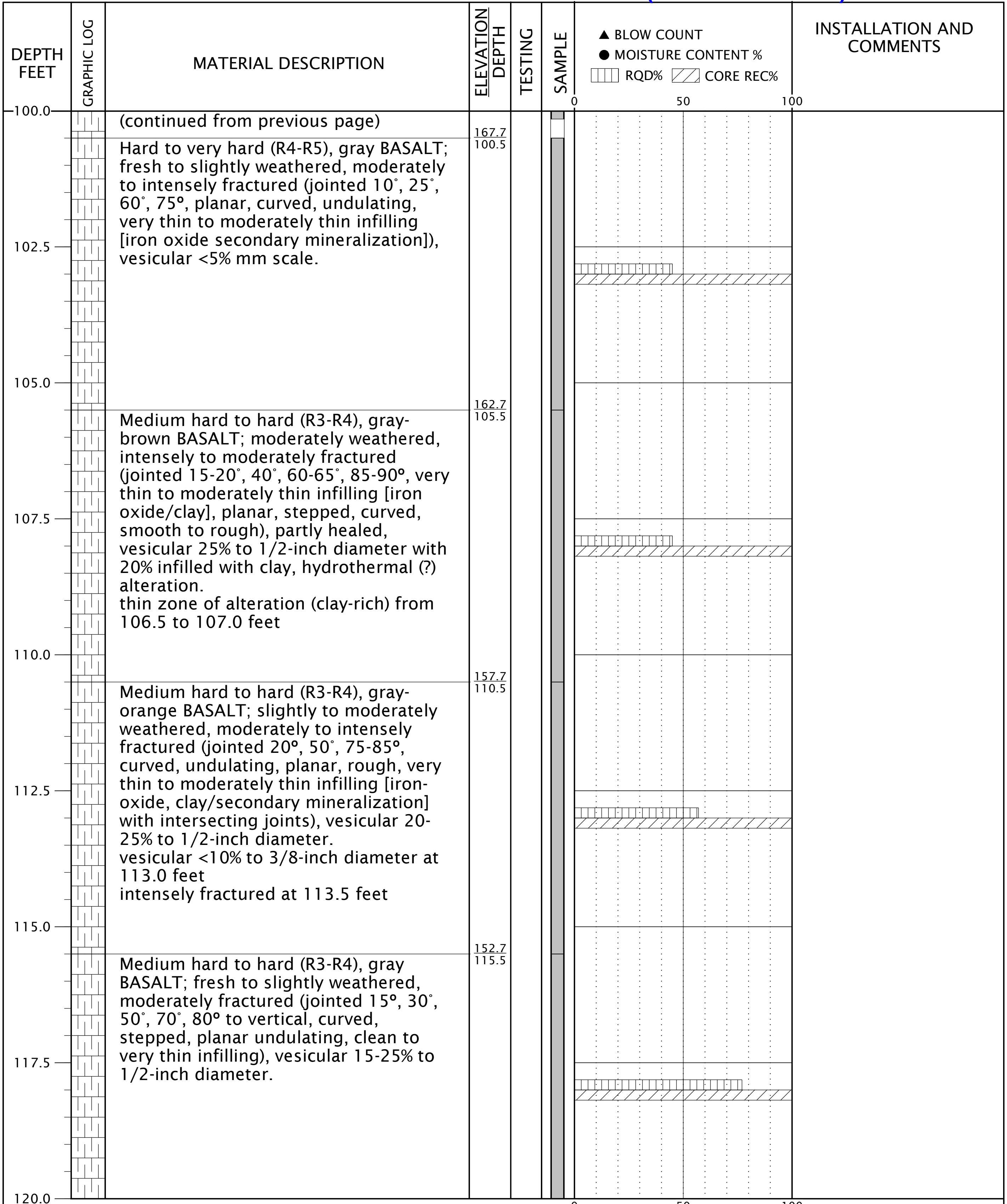
NOVEMBER 2014

BORING B-7
(continued)

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-7

FOR LAND USE PERMITTING (EXHIBIT B)



ELEVATION DEPTH	MATERIAL DESCRIPTION
	(continued from previous page)
167.7 100.5	Hard to very hard (R4-R5), gray BASALT; fresh to slightly weathered, moderately to intensely fractured (jointed 10˚, 25˚, 60˚, 75°, planar, curved, undulating, very thin to moderately thin infilling [iron oxide secondary mineralization]), vesicular <5% mm scale.
162.7 105.5	Medium hard to hard (R3-R4), gray-brown BASALT; moderately weathered, intensely to moderately fractured (jointed 15-20˚, 40˚, 60-65˚, 85-90°, very thin to moderately thin infilling [iron oxide/clay], planar, stepped, curved, smooth to rough), partly healed, vesicular 25% to 1/2-inch diameter with 20% infilled with clay, hydrothermal (?) alteration. thin zone of alteration (clay-rich) from 106.5 to 107.0 feet
157.7 110.5	Medium hard to hard (R3-R4), gray-orange BASALT; slightly to moderately weathered, moderately to intensely fractured (jointed 20°, 50˚, 75-85°, curved, undulating, planar, rough, very thin to moderately thin infilling [iron-oxide, clay/secondary mineralization] with intersecting joints), vesicular 20-25% to 1/2-inch diameter. vesicular <10% to 3/8-inch diameter at 113.0 feet intensely fractured at 113.5 feet
152.7 115.5	Medium hard to hard (R3-R4), gray BASALT; fresh to slightly weathered, moderately fractured (jointed 15°, 30˚, 50˚, 70˚, 80° to vertical, curved, stepped, planar undulating, clean to very thin infilling), vesicular 15-25% to 1/2-inch diameter.

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/26/13

BORING METHOD: mud rotary and HQ rock coring (see report text) BORING BIT DIAMETER: 4 7/8-inch

GEODESIGN INC

15575 SW Sequoia Parkway - Suite 100
Portland OR 97224
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DEA-118-02-5.12

NOVEMBER 2014

BORING B-7
(continued)

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-7

BORING LOG DEA-118-02-5.12-B1_32_36-38_56-TWWD1_13-INF1_2.GPJ GEODESIGN.GDT PRINT DATE: 11/3/14:KT

FOR LAND USE PERMITTING (EXHIBIT B)

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% 0 50 100	INSTALLATION AND COMMENTS
120.0		(continued from previous page)					
		Exploration completed at a depth of 120.5 feet.	147.7 120.5				
122.5							
125.0							
127.5							
130.0							
132.5							
135.0							
137.5							
140.0						0 50 100	

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: JGH
COMPLETED: 06/26/13

BORING METHOD: mud rotary and HQ rock coring (see report text)
BORING BIT DIAMETER: 4 7/8-inch

GEODESIGN INC
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Portland OR 97224
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DEA-118-02-5.12

NOVEMBER 2014

BORING B-7
(continued)

SW 124TH AVENUE EXTENSION PROJECT
WASHINGTON COUNTY, OR

FIGURE A-7

BORING LOG DEA-118-02-5.12-B1_32_36-38_56-TWVD1_13-INF1_2.GPJ GEODESIGN.GDT PRINT DATE: 11/3/14:KT

FOR LAND USE PERMITTING (EXHIBIT B)

Log of Test Pit TP-1

SHEET 1 of 1

PROJECT: 124th Avenue Transmission Line
PROJECT NUMBER: 5106.0
LOCATION:

DATE OF EXCAVATION: 8/25/2014
TEST PIT DEPTH: 2 FT

ELEVATION: FT
DATUM:
COORDINATES: N
E

DEPTH (FT)	WATER LEVEL	ELEV. (FT)	DESCRIPTION	USCS	GRAPHIC LOG	WELL	SAMPLES NUMBER	SAMPLES TYPE/ REC%	SAMPLES BLOWS	SAMPLES N	PENETRATION RESISTANCE BLOWS/FT ■ 20 40 60 80 WATER CONTENT - ATTERBERG PL MC LL 20 40 60 80	NOTES
			Silty GRAVEL (GM), brown, dry, loose, angular clasts less than 12-inch in diameter. [Fill]	GM								
			Basalt, brown-grey, slightly weathered, strong, closely spaced joints.									

Excavator refusal at 2.0 feet.
BOTTOM OF TEST PIT AT 2 FT

DRILL METHOD: Test Pits
DRILLING CONTRACTOR: Tigard Sand and Gravel
DRILLER:

EQUIPMENT: Kobelco Mark III SK300 LC
LOGGED BY: JDS
CHECKED BY:

JACOBS ASSOCIATES
Engineers/Consultants

FIGURE A1

FOR LAND USE PERMITTING (EXHIBIT B)

Log of Test Pit TP-2

SHEET 1 of 1

PROJECT: 124th Avenue Transmission Line
PROJECT NUMBER: 5106.0
LOCATION:

DATE OF EXCAVATION: 8/25/2014
TEST PIT DEPTH: 2 FT

ELEVATION: FT
DATUM:
COORDINATES: N
E

DEPTH (FT) WATER LEVEL ELEV. (FT)	DESCRIPTION	USCS	GRAPHIC LOG	WELL	SAMPLES				PENETRATION RESISTANCE BLOWS/FT ■ 20 40 60 80 WATER CONTENT - ATTERBERG PL MC LL 20 40 60 80	NOTES
					NUMBER	TYPE/ REC%	BLOWS	N		
	Silty GRAVEL (GM), brown, dry, loose, angular clasts less than 12-in in diameter. [Fill]	GM	▨							
	Basalt, brown-grey, Slightly weathered, strong, closely spaced joints.		▨							

Excavator refusal at 2.0 feet.
BOTTOM OF TEST PIT AT 2 FT

DRILL METHOD: Test Pits
DRILLING CONTRACTOR: Tigard Sand and Gravel
DRILLER:

EQUIPMENT: Kobelco Mark III SK300 LC
LOGGED BY: JDS
CHECKED BY:

JACOBS ASSOCIATES
Engineers/Consultants

BNSF BRIDGE TEST PIT LBNE TEST PIT LOGS.GPJ UNIVERSITY LINK PROGRESS.GPJ 9/3/14 REV-

FIGURE A1

FOR LAND USE PERMITTING (EXHIBIT B)

GRI
9750 SW Nimbus Avenue
Beaverton, OR 97008-7172
p | 503-641-3478 f | 503-644-8034

MEMORANDUM

To: Ken Leahy / Ken Leahy Construction, Inc.

Date: April 13, 2016

GRI Project No.: 5838

From: Michael Reed, PE, GE; Brian Bayne, PE; and Seth Reddy, PhD, EIT

Re: Preliminary Subsurface Investigation for Pre-purchase Due Diligence
90-Acre Site
12900 SW Tualatin-Sherwood Road
Sherwood, Oregon

DRAFT

At your request, GRI has conducted a preliminary subsurface investigation as part of a pre-purchase due diligence evaluation for a 90-acre site at 12900 SW Tualatin-Sherwood Road in Sherwood, Oregon. Our services included a review of existing geotechnical data for the area and limited subsurface explorations. This memorandum describes the work accomplished and provides a site plan with table showing approximate depths to basalt and groundwater encountered in the borings.

PROJECT DESCRIPTION

Ken Leahy Construction, Inc. (Leahy) is considering acquiring the 90-acre site for development into multiple buildable lots for commercial development. Preliminary site grades for the lots are unavailable.

SITE DESCRIPTION

Topography and Surface Conditions

The existing ground surface elevation varies significantly across the site from about elevation 192 ft (North American Vertical Datum of 1988 [NAVD88]) on the northern edge of the site to about elevation 280 ft near the southeast corner of the site. An existing farmhouse and several out-buildings are located near the north edge of the property. The northeast portion of the site is covered with grass in an area that was previously used for agricultural purposes. The remainder of the site is heavily wooded with mature trees and shrubs. Basalt outcroppings were observed most predominately near the northwest quarter and middle of site, but are present throughout the heavily wooded areas. Cobbles and boulders are present at the ground surface in the wooded areas.

Geology

This site is at the northern edge of an area known as the Tonquin Scablands, where Pleistocene-age catastrophic floods from the Columbia River scoured away the soil, leaving rock exposed at the ground surface or covered by only a thin layer of soil. Portions of the area may be mantled with a thin layer of Pleistocene-age lacustrine (floodplain) deposits of the Columbia River, consisting of interlayered sand, silt, and gravel. Below the thin zone of surficial soil, the site is underlain by Columbia River Basalt, a thick sequence of dark gray, basalt lava flows of mid-Miocene age. Based on our experience with other nearby projects and our observations while onsite, we anticipate basalt is present at relatively shallow depths.

SUBSURFACE CONDITIONS

General

Subsurface materials and conditions were investigated on a preliminary basis on March 28 and 29, 2016, with 48 borings, designated B-1 through B-48. The borings were advanced to depths of 15 to 30 ft at the approximate locations shown on the Site Plan, Figure 1. The borings were completed by McCallum Rock Drilling, Inc. of Albany, Oregon using a track-mounted FRD Furukawa HCR 900-ES II drilling rig. The rock drilling rig used open-hole air-rotary impact drilling methods typically used for production blasting in aggregate quarries and large rock cuts. The driller was contracted directly by Leahy, and the exploration locations and depths of the borings were selected by a representative of Leahy. The drill cuttings were diverted to a cyclone to allow collection of disturbed samples of soil and rock. GRI was on site on a full-time basis during drilling and recorded the GPS coordinates and depth to basalt at each boring location. Disturbed soil and rock cuttings were collected as bulk samples removed by hand from the cyclone on an intermittent basis, saved in airtight jars and bags, and returned to our lab for further examination. The depth to rock and estimates of rock weathering and hardness were approximated based on rate of advancement of the drill, color of the drill cuttings, and evaluation of the cuttings samples collected. Following drilling, each hole was left open to allow measurements of static groundwater.

Subsurface Conditions

Based on the disturbed soil cuttings collected during drilling and our observations while onsite, the site is typically mantled with silt or sand soils with varying percentages of clay. We anticipate fill soils may be encountered locally. Basalt was encountered at the ground surface in borings B-28, B-29, B-31, B-33, B-35, and B-39 and beneath the silt and sand soils at depths ranging from 0.5 to 15 ft in borings B-5 through B-8, B-12 through B-19, B-22, B-23, B-30, B-32, B-34, B-36 through B-38, and B-40 through B-48. Basalt was not encountered in borings B-1 through B-4, B-9 through B-11, B-20, B-21, and B-24 through B-27. The approximate depths and relatively hardness of the basalt is presented in a table on Figure 1. Terms used to describe the soil and rock are defined on Tables 1 and 2. For the purpose of discussion, the basalt has been grouped into two categories: very soft (R1) to medium hard (R3), moderately weathered to predominantly decomposed basalt, and soft (R2) to hard (R4), slightly weathered to fresh basalt.

Based on the rate of advancement of the drill rig, color of the drill cuttings, and subsequent evaluation of the cutting samples collected during drilling, the surface of the basalt is typically very soft (R1) to soft (R2), moderately weathered to predominately decomposed, and likely contains some medium hard (R3) zones. Drill cuttings in the moderately weathered to predominately decomposed basalt are typically red-brown to brown and contain few angular pieces of basalt. Borings B-5, B-6, B-18, B-19, B-22, and B-23 were terminated in the moderately weathered to predominantly decomposed basalt at depths ranging from 15 to 20 ft. Zones of moderately weathered to predominantly decomposed basalt were encountered below fresh to slightly weathered basalt at depths of 6 to 21 ft in borings B-14, B-28, B-46, and B-47.

Fresh to slightly weathered, soft (R2) to medium hard (R3) basalt was generally encountered beneath the more weathered basalt at depths ranging from 9 to 18 ft and likely contains zones of hard (R4) basalt. Drill cuttings in the basalt were typically light gray silt- and sand-sized pieces with frequent small fine gravel-sized rock fragments.

Following completion of the drilling, the holes were left open to allow measurements of depth to groundwater. Groundwater depths are provided on Figure 1 and vary considerably across the site. All groundwater measurements were taken in the afternoon of March 29 and indicate perched groundwater conditions.

EXCAVATION METHODS

Final site grading and depth of utilities for the proposed development are currently unknown. We anticipate conventional excavation equipment can be used to excavate the silty and sandy soils overlying the basalt. We anticipate some of the near-surface very soft to soft (R1 to R2) basalt can be excavated with a sufficiently large track-mounted excavator equipped with a rock excavation bucket and rock teeth, or by ripping with a CAT D8 bulldozer, or equivalent, equipped with a single-shank ripper. It should be noted that although the slightly weathered to predominately decomposed basalt is typically very soft (R1) to soft (R2), zones of medium hard (R3) basalt are likely present within this unit. Rock excavation methods, such as hydraulic splitters and chippers or pneumatic hammers, may be needed to excavate the rock in these areas of medium hard (R3) rock. We anticipate the fresh to slightly weathered, soft (R2) to medium hard (R3) basalt with zones of hard (R4) basalt will likely require blasting or other rock excavation methods to excavate.

Rock hardness designations provided in this memorandum are based on visual observation of drilling spoils and the rate of drilling. If significant excavation into the basalt is planned, coring of the basalt should be performed to obtain samples for completion of compressive strength testing and to evaluate fracture spacing.

OTHER CONSIDERATIONS

Properties to the south of this site have previously been quarried for aggregate production. We anticipate that some of the rock removed during site grading could be crushed for aggregate. In general, the quality of aggregate decreases as weathering of the source rock increases. The proportion of clay, silt, and sand produced during crushing for aggregate will typically increase as the weathering in the source rock increases. Reduced material strength and chemical changes in the rock mineralogy can result in decreased durability of aggregates produced from weathered rock. In general, a rock mass that is classified as moderately weathered using the relative rock weathering scale on Table 2 can be considered marginal to poor for aggregate production. Rock weathered to the range of predominantly decomposed or decomposed is unsuitable for aggregate production.

LIMITATIONS

This preliminary memorandum has been prepared to aid in the pre-purchase evaluation of the subject property described herein. The findings, conclusions, and recommendations presented in this memorandum are based on our interpretation of the information obtained through the assessment procedures described in this memorandum, based on 48 widely spaced borings advanced at the locations shown on Figure 1. It should be noted that there are significant limitations associated with using air-rotary percussion methods to characterize subsurface conditions. While slower than the air rotary drill, conventional geotechnical drilling methods, especially rock coring, would more accurately characterize rock hardness, fracture spacing, and rock weathering. Due to the method of drilling used for this preliminary evaluation, the estimated thickness, degree of weathering, and hardness of the rock at each exploration should be considered approximate.

In the performance of subsurface investigations, specific information is obtained at specific times, and variations in subsurface conditions may exist across the site. This preliminary report does not reflect any variations that may occur between exploration locations. The nature and extent of variation may not become evident until site development is underway. Consequently, any material volume estimates developed using the information provided in this memorandum should be considered approximations intended for planning purposes only.

The information presented herein is preliminary and provides our general conclusions regarding the depth to rock and excavation methods with respect to the observed site conditions. This information is intended for preliminary planning purposes. Additional geotechnical investigation should be completed as specific projects are developed for specific locations on the property.

Please contact the undersigned if you have any questions.

Submitted for GRI,

Michael W. Reed, PE, GE
Principal

Brian J. Bayne, PE
Senior Engineer

Seth C. Reddy, PhD, EIT
Staff Engineer

5838-PRELIM EVAL MEMO

FOR LAND USE PERMITTING (EXHIBIT B)

Table 1: GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows per foot	Torvane or Undrained Shear Strength, tsf
Very Soft	0 - 2	less than 0.125
Soft	2 - 4	0.125 - 0.25
Medium Stiff	4 - 8	0.25 - 0.50
Stiff	8 - 15	0.50 - 1.0
Very Stiff	15 - 30	1.0 - 2.0
Hard	over 30	over 2.0

Grain-Size Classification

Boulders:
- $>$12 in.

Cobbles:
- 3 - 12 in.

Gravel:
- $^{1}/_{4}$ - $^{3}/_{4}$ in. (fine)
- $^{3}/_{4}$ - 3 in. (coarse)

Sand:
- No. 200 - No. 40 sieve (fine)
- No. 40 - No. 10 sieve (medium)
- No. 10 - No. 4 sieve (coarse)

Silt/Clay:
- pass No. 200 sieve

Modifier for Subclassification

	Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
Adjective	Percentage of Other Material (by weight)	
trace:	5 - 15 (sand, gravel)	5 - 15 (sand, gravel)
some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
trace:	$<$5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
some:	5 - 12 (silt, clay)	
silty, clayey:	12 - 50 (silt, clay)	

Table 2: GUIDELINES FOR CLASSIFICATION OF ROCK

RELATIVE ROCK WEATHERING SCALE

Term	Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 in. into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominantly Decomposed	Rock mass is more than 50% decomposed. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to soil with hand pressure.

RELATIVE ROCK HARDNESS SCALE

Term	Hardness Designation	Field Identification	Approximate Unconfined Compressive Strength
Extremely Soft	R0	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	< 100 psi
Very Soft	R1	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocket knife and scratched with fingernail.	100 - 1,000 psi
Soft	R2	Can be peeled by a pocket knife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.	1,000 - 4,000 psi
Medium Hard	R3	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.	4,000 - 8,000 psi
Hard	R4	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.	8,000 - 16,000 psi
Very Hard	R5	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	> 16,000 psi

RQD AND ROCK QUALITY

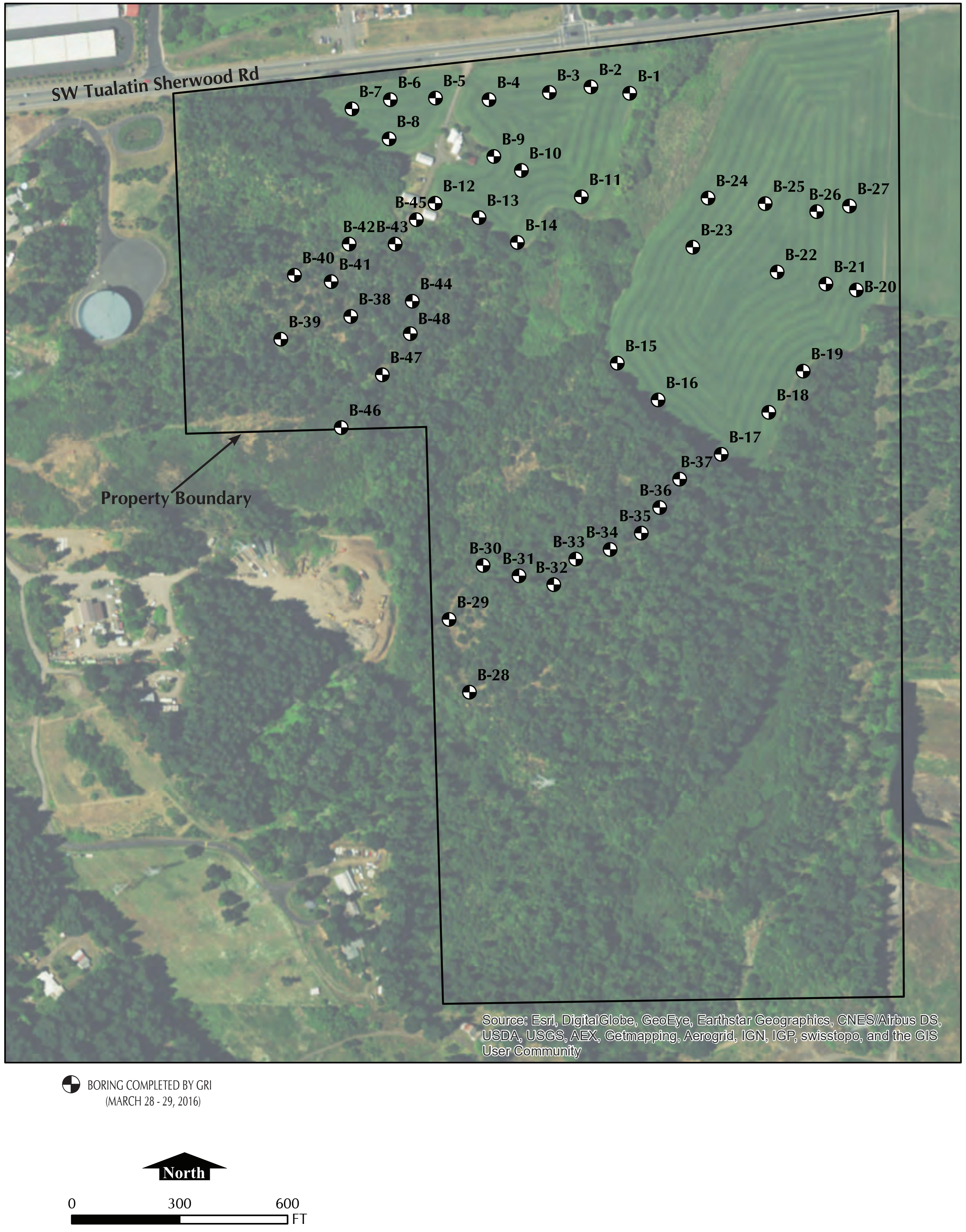
Relation of RQD and Rock Quality

RQD (Rock Quality Designation), %	Description of Rock Quality
0 - 25	Very Poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Terminology for Planar Surface

Bedding	Joints and Fractures	Spacing
Laminated	Very Close	< 2 in.
Thin	Close	2 in. – 12 in.
Medium	Moderately Close	12 in. – 36 in.
Thick	Wide	36 in. – 10 ft
Massive	Very Wide	> 10 ft

FOR LAND USE PERMITTING (EXHIBIT B)



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

Boring	Latitude$^{(5)}$	Longitude$^{(5)}$	Ground Surface Elev. (ft)$^{(6)}$	Total Depth (ft)	Depth to Very Soft (R1) to Medium Hard (R3), Moderately Weathered to Predominantly Decomposed Basalt (ft)	Depth to Soft (R2) to hard (R4), Slightly Weathered to Fresh Basalt (ft)	Depth to Groundwater (ft)
B-1	45.36863	-122.80848	195.1	15	>15	>15	3
B-2	45.36867	-122.80890	194.3	15	>15	>15	N/A
B-3	45.36862	-122.80935	195.3	15	>15	>15	N/A
B-4	45.36855	-122.81000	197.1	15	>15	>15	2.5
B-5	45.36855	-122.81058	200.1	15	9	>15	7
B-6	45.36853	-122.81107	203.4	15	8	>15	>15
B-7	45.36845	-122.81148	205.7	15	5	12	>15
B-8	45.36823	-122.81107	209.3	15	N/A	13	9.7
B-9	45.36812	-122.80993	202.6	15	>15	>15	N/A
B-10	45.36802	-122.80963	201.0	15	>15	>15	N/A
B-11	45.36783	-122.80897	199.6	15	>15	>15	N/A
B-12	45.36775	-122.81055	215.6	20	7	12	10.8
B-13	45.36765	-122.81007	214.2	20	6	11	18.2
B-14	45.36747	-122.80965	215.6	20	18	5$^{(2)}$	14.8
B-15	45.36657	-122.80853	227.0	20	5	9	19.1
B-16	45.36630	-122.80808	232.8	20	5$^{(4)}$	9	10.9
B-17	45.36590	-122.80738	241.1	20	N/A	6	15.9
B-18	45.36623	-122.80688	234.1	20	4	>20	16
B-19	45.36655	-122.80652	228.5	20	7	>20	8.3
B-20	45.36718	-122.80597	213.8	20	>20$^{(3)}$	>20	N/A
B-21	45.36722	-122.80630	215.1	20	>20	>20	6.8
B-22	45.36730	-122.80683	214.6	20	12	>20	7.7
B-23	45.36747	-122.80775	204.4	20	12	>20	N/A
B-24	45.36785	-122.80760	200.1	15	>15	>15	2.2
B-25	45.36782	-122.80698	203.5	15	>15	>15	N/A
B-26	45.36777	-122.80642	205.1	15	>15	>15	0.4
B-27	45.36782	-122.80607	204.3	15	>15	>15	1.1
B-28	45.36403	-122.81003	256.8	23	21	0$^{(2)}$	18.6
B-29	45.36458	-122.81027	249.2	23	N/A	0	>23
B-30	45.36500	-122.80992	246.6	23	2	7	7.9
B-31	45.36493	-122.80953	257.7	30	N/A	0	6.2
B-32	45.36487	-122.80915	264.7	30	2	18	9.5
B-33	45.36507	-122.80892	266.1	30	0	10	15.5
B-34	45.36515	-122.80855	267.1	30	N/A	1.5	26.3
B-35	45.36528	-122.80822	265.6	30	N/A	0	11
B-36	45.36548	-122.80803	260.1	23	N/A	3	4.9
B-37	45.36570	-122.80782	254.3	23	N/A	0.5	10.2
B-38	45.36687	-122.81143	244.5	30	N/A	2	1.9
B-39	45.36668	-122.81218	243.5	30	N/A	0	15.9
B-40	45.36717	-122.81205	248.1	30	N/A	3	1.9
B-41	45.36713	-122.81165	244.7	30	N/A	2	27.7
B-42	45.36742	-122.81147	235.6	30	N/A	1	14.7
B-43	45.36743	-122.81097	231.8	30	4	10	28.6
B-44	45.36700	-122.81077	243.5	30	N/A	3	22.4
B-45	45.36762	-122.81075	222.8	30	6	11	29.5
B-46	45.36602	-122.81150	228.7	30	6$^{(1)}$	2$^{(2)}$	18
B-47	45.36643	-122.81107	233.3	30	20$^{(1)}$	2$^{(2)}$	22
B-48	45.36675	-122.81078	242.6	30	N/A	1	>30

Notes:

1. Clay seams were observed in B-46 and B-47 at a depth of about 20 ft.
2. Zones of moderately weathered to predominantly decomposed basalt beneath fresh to slightly weathered basalt in B-14, B-28, B-46, and B-47.
3. Boulder encountered between 8 and 14 ft in boring B-20.
4. Very soft (R1) to Medium hard (R3) basalt encountered below 18 ft in boring B-16.
5. Geographic Coordinate System: North American Datum of 1983 (NAD 83). Accuracy within 15 ft horizontal for hand held unit.
6. Elevation Datum: North American Vertical Datum of 1988 (NAD 88). Accuracy within 1 ft vertical for GIS lidar.

GRI

KEN LEAHY CONSTRUCTION
90-ACRE SITE

SITE MAP

APR. 2016 JOB NO. 5838 FIG. 1

Appendix E

Unconfined Compressive Strength and RQD Data

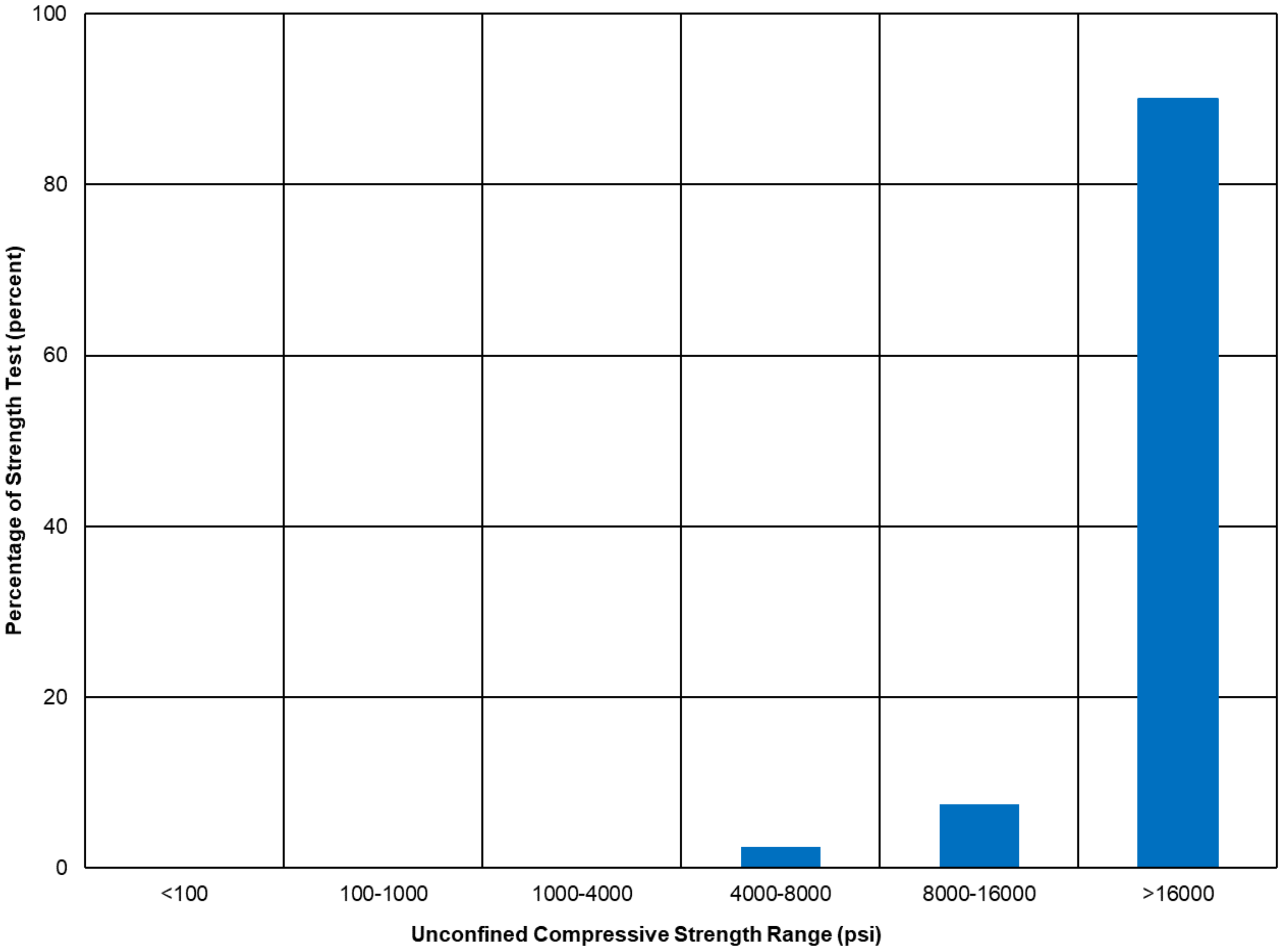


Figure includes UCS test results and estimated UCS from point load tests.

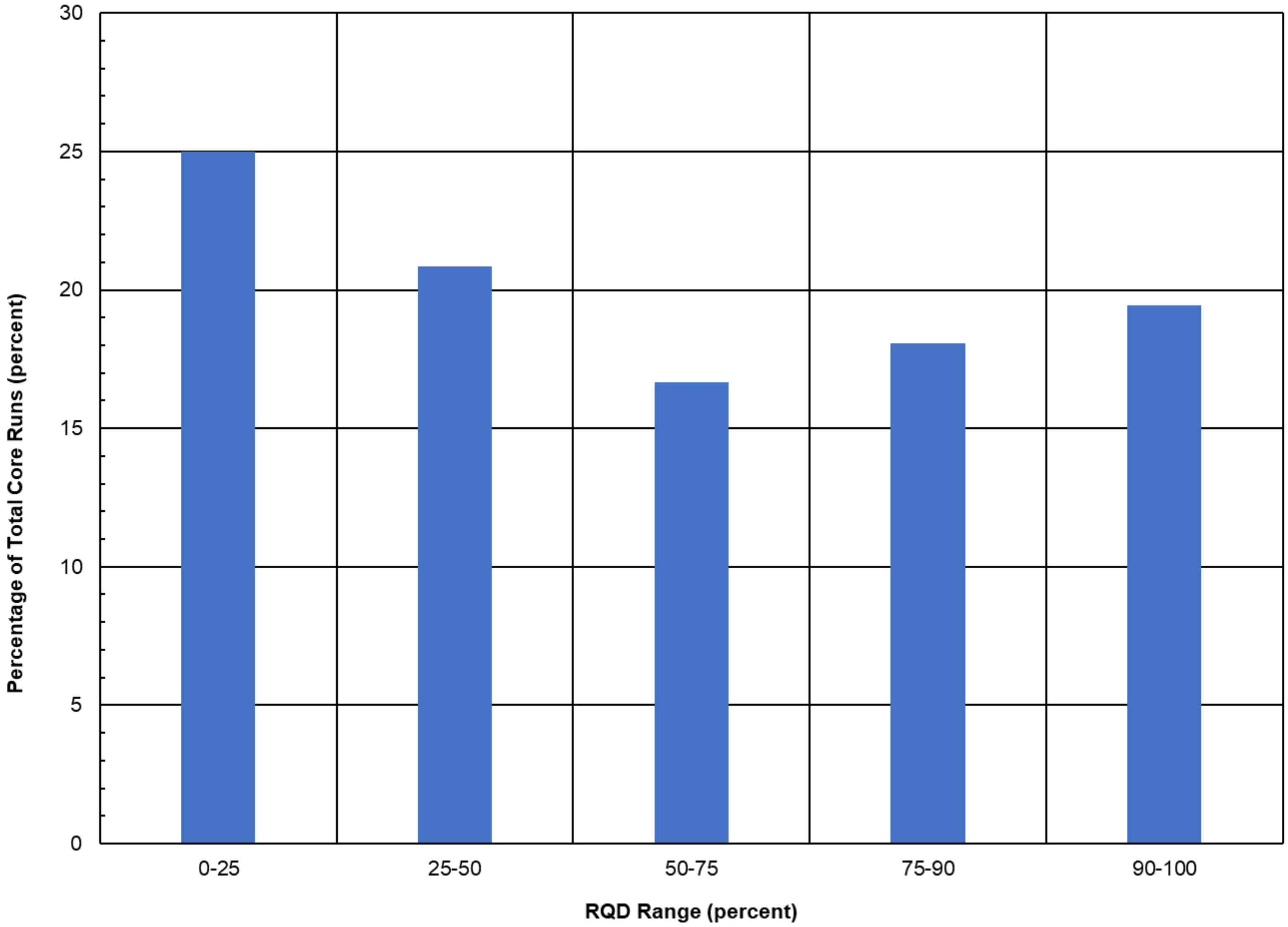
McMILLEN JACOBS ASSOCIATES

WILLAMETTE WATER SUPPLY PROGRAM
WTP_1.0

UNCONFINED COMPRESSIVE STRENGTH FREQUENCY

FIGURE
1E

DECEMBER
2019



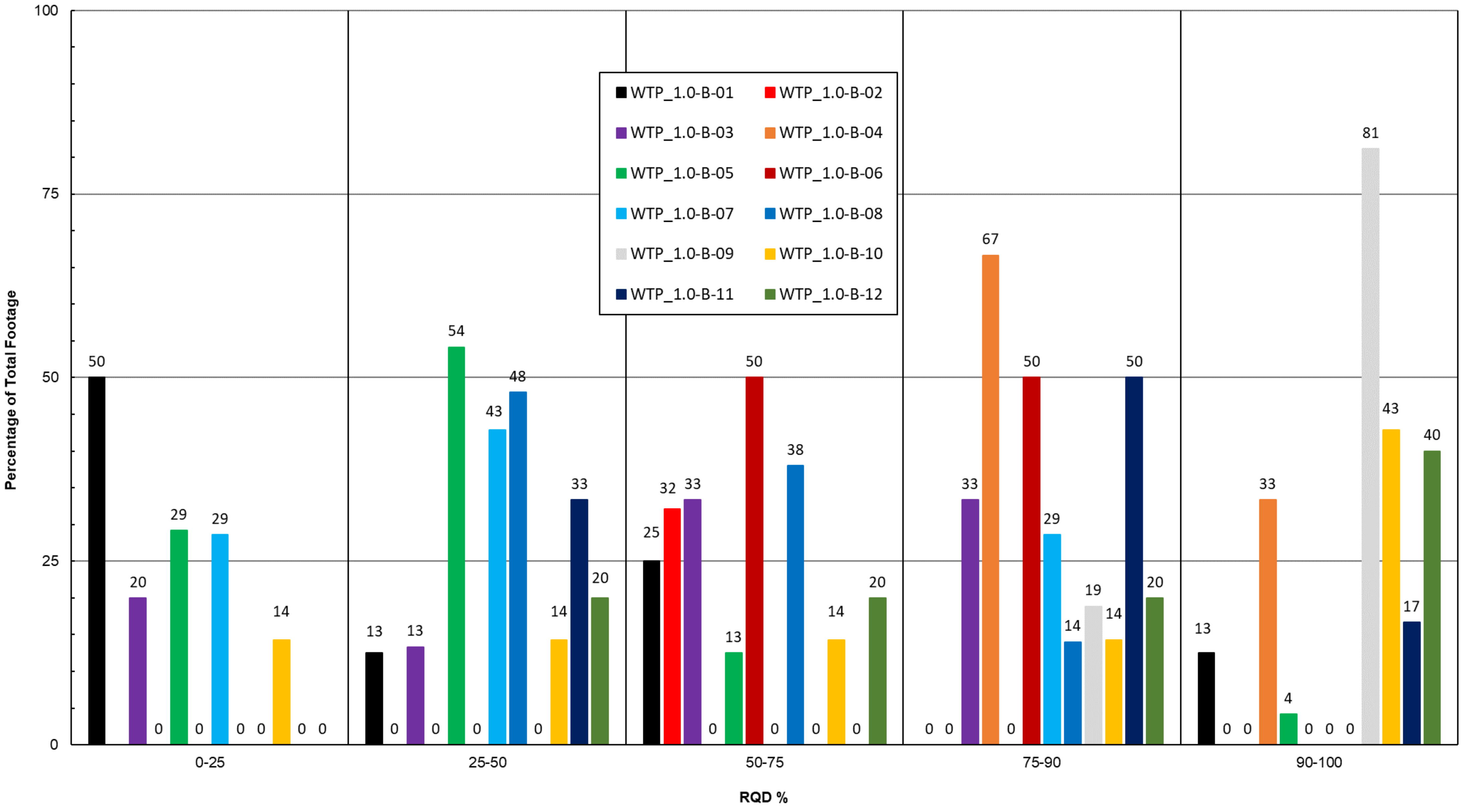
McMILLEN JACOBS ASSOCIATES

WILLAMETTE WATER SUPPLY PROGRAM
WTP_1.0

RQD FREQUENCY

FIGURE 2E

DECEMBER 2019



McMILLEN JACOBS ASSOCIATES

WILLAMETTE WATER SUPPLY PROGRAM
WTP_1.0

FIGURE
3E

SUMMARY OF RQD BY BOREHOLE

DECEMBER
2019

Appendix F

Fault Location Study Technical Memorandum

Technical Memorandum

To:	Greg Lindstadt, PE Michael Hyland, PE	Project:	WWSP WTP_1.0
From:	Kim Elliott, CEG Wolfe Lang, PE, GE	cc:	
Date:	December 20, 2019	Job No.:	5887.0
Subject:	Willamette Water Supply Program - WTP_1.0 Fault Location Study		

Revision Log

Revision No.	Date	Revision Description
0	February 15, 2019	Draft submitted for CDM Smith review
1	March 11, 2019	Incorporated CDM Smith's comments
	December 20, 2019	Final submittal

1.0 Introduction

1.1 General

McMillen Jacobs Associates (MJA) has prepared this technical review and characterization of faults and seismicity within an approximately 100-kilometer (km) (60-mile [mi]) radius of the Willamette Water Supply Program (WWSP) Water Treatment Plant WTP_1.0. In addition to the Cascadia Subduction Zone (CSZ), a total of 28 faults have been catalogued. The location of WTP_1.0 project and its relation to the identified faults are shown in Figure 1. The faults and their characteristics are presented in Table 1.

1.2 Information Sources

This study and evaluation is based on a review of available geological maps and research literature, and Light Detection and Ranging (LiDAR or "lidar") imagery. The maps and literature were published by Oregon Department of Geology and Mineral Industries (DOGAMI), U.S. Geological Survey (USGS), and a variety of scientific publications.

Lidar data, covering the USGS 7.5-minute Sherwood Quadrangle available from the Oregon Lidar Consortium was processed with ArcMap Geographic Information System (GIS) software to create hillshade images for visual scanning. The hillshade tool uses elevation data to create an image that looks more like real world topography (without the vegetation cover), making it easier to pick out important geomorphic features.

We also had brief consultations with Dr. Ian P. Madin, Chief Scientist with the Oregon Department of Geology and Mineral Industries (DOGAMI) and Dr. Ray Wells, Project Chief Emeritus with the U.S. Geological Survey (USGS). Dr. Madin is the author and co-author of numerous geologic map studies and technical publications on faulting and earthquake hazards in Oregon. Dr. Wells is an expert on Pacific Northwest geology and tectonics, geology of the Cascadia forearc, and crustal deformation and neotectonics of convergent margins. One of Dr. Wells' current projects is geologic mapping of Urban Corridor fault zones in the Pacific Northwest, which includes the Portland Hills and Gales Creek Fault zones in northwest Oregon.

2.0 Geologic Setting

2.1 General

The Willamette Valley of Oregon is part of the northwest regional Puget-Willamette Lowland that lies between the Cascade Range on the east and the Coast Range on the west. The Puget-Willamette Lowland and the Coast Range together constitute the Cascadia forearc, the area that lies between the Cascadia Subduction Zone (CSZ) at the Pacific (Juan de Fuca)-North America plate boundary and the Cascade Range volcanic archipelago ("arc").

The compressional forces that exist between the colliding Pacific and North America plates cause the denser oceanic plate to descend, or subduct beneath the less dense continental plate. The compression also causes folds and faults to form in the sediment and rock that make up the forearc. Earthquakes are generated where slippage occurs at a fault.

The Juan de Fuca plate is being thrust northeastward and oblique to North America. The oblique subduction has created a complex, seismically active convergent margin and volcanic arc in the Pacific Northwest. Such oblique convergence commonly produces arc-parallel migration of the forearc and often creates an additional seismic hazard from the relative motions of forearc blocks (Wells and others, 1998). Great subduction earthquakes have occurred along the Cascadia margin (Atwater and Hemphill-Haley, 1997), but the potential for damaging upper-plate (crustal) earthquakes is poorly known because of the short record of historical seismicity, sparse data on regional deformation rates, and poor exposure of active structures.

2.2 Local Geology

The structure of northwestern Oregon is dominated by a broad, north plunging anticlinorium[1] centered over the Coast Range (Yeats, et al, 1991) that deforms Eocene through early Miocene volcanic and marine sedimentary rocks. About 16-14.5 Ma (mega annum or million years) basalt flows of the Columbia River Basalt Group (CRBG) flowed westward through a structural lowland in the Cascade Range between the Columbia River and the Clackamas River into the northern Willamette Valley (Beeson, et al., 1989). At this time, the Willamette Valley must have been a broad plain with little topographic relief because the basalts now underlie the entire northern Willamette valley from north of

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[1] An anticlinorium is a large anticline with superimposed smaller folds.

the Columbia River to south of Salem and from the Cascade Range to the Coast Range. The CRBG also crossed the Coast Range, probably as intracanyon flows (Beeson, et al., 1989).

Continued uplift of the Coast and Cascade Ranges and smaller scale fault and open fold structures superimposed on the anticlinorium have further defined the Willamette Valley and attracted a continuing stream of sediment, eroded from the flanking mountain ranges, that has filled the lowest areas of the valley with fluvial and lacustrine deposits (Yeats, et al., 1996). Structural deformation, including sub-basin subsidence and faulting, has created uplands of CRBG rock across this synclinorium between Albany and Salem that separate the northern Willamette Valley from the southern Willamette Valley (Crenna, et al., 1994). A similar CRBG uplift, the Chehalem Mountains and Parrett Mountain, which rise north of Wilsonville, separates the Tualatin Basin from the northern Willamette Valley while upland-forming faults and Pliocene-Pleistocene basalt flows near Oregon City separate the northern Willamette Valley from the Portland Basin (Beeson, et al., 1989; Madin, 1990, 1994). The Portland and Tualatin basins are separated from each other by the Tualatin Mountains (“Portland Hills”), another faulted and folded CRBG uplift.

Alluvial deposits that post-date the CRBG are the oldest strata to be confined principally to the present lowland basins. In the Tualatin basin, these deposits include the Hillsboro Formation (Wilson, 1997, 1998); in the Portland basin, the Sandy River Mudstone (Trimble, 1963) and Troutdale Formation (Lowery and Baldwin, 1952); and in the northern Willamette basin, the Troutdale Formation of Hampton (1972). From Pliocene to Pleistocene time, basaltic lava, breccia and volcanic ash of the Boring Lavas were erupted from numerous vents in the Portland, Tualatin and northern Willamette basins (Treasher, 1942).

During the Quaternary Period, windblown silt, named the Portland Hills Silt (Lowry and Baldwin, 1952; Baldwin, 1964) because of its prominence on Portland’s West Hills, mantled most of the uplands around the Portland and Tualatin basins. The loess is probably not a major component of local sediments, but it has probably made its way into the northern Willamette basin through erosion by streams draining the Chehalem Mountains and Parrett Mountain uplifts. Near the end of the Pleistocene epoch, catastrophic glacial-outburst floods from the upper Columbia River basin repeatedly back-flooded up the Willamette River valley as far south as Eugene and deposited the fine-grained sand and silt of the Willamette Formation (Balster and Parsons, 1969; Allison, 1978).

Most of the fold and fault structures identified in this study are thought to have either appeared or reactivated from late Miocene to Pliocene time coincident with post-CRBG basin forming. All the fault structures listed in Table 1 are either known or suspected of offsetting the CRBG; some are suspected of offsetting the Hillsboro Formation and the Missoula Flood deposits, but no offsets have been confirmed.

The present north-south compressional stress regime may have resulted in significant shortening of the forearc in northwestern Oregon. Geologic field mapping and analysis of offset magnetic anomalies have suggested that about 10 km (6 mi) of north-south shortening has been accommodated by movements on the Gales Creek-Mt. Angel Fault zone alone and additional shortening is likely to have been taken up on the Portland Hills, Sylvan-Oatfield, East Bank, Molalla-Canby, and Frontal fault zones (Beeson et al., 1985; Blakely et al., 2000; Wells et al., 1995, 2009; Wong et al., 2000). Motions on these faults were also likely to have had a significant influence on the formation of the present Tualatin and Portland basins in the middle Miocene (McPhee et al., 2014).

2.3 Evidence for Quaternary Faulting

Although numerous faults have been recognized and mapped within the study area (see Figure 1), documented deformations in geologic units younger than the CRBG are scarce. The mapped faults listed in Table 1 are considered to have been “active” during the Quaternary Period (the past 1.6 Ma) (USGS, 2006, 2014), but evidence of post--CRBG movements have been identified on only the Portland Hills Fault and the Gales Creek Faults. However, the evidence of such movement is inconclusive.

Geophysical evidence of deformed Pliocene to Pleistocene (5 Ma to 10 ka [kilo annum or thousand years]) sediments were recognized on the Portland Hills Fault (Wong et al., 2001; Liberty et al., 2003), but there was no direct evidence of deformation of Holocene (<10 ka) deposits.

Using Lidar imagery, a USGS research team identified an apparent fault scarp along a strand of the Gales Creek Fault. They then investigated the site by excavating a trench across the feature and found Yamhill Formation sandstone (middle to late Eocene, between 48 and 34 Ma) thrust over <250 ka loess deposits (Wells, pers. commun., 2017). This finding indicated a minimum age of the most recent movement to be about 250 ka. They found no evidence, however, of more recent activity, such as offset of younger geologic deposits. Dr. Wells anticipates that findings from this investigation will be published soon.

Evidence of Holocene faulting in the Pacific Northwest is difficult to find for the following reasons:

- Ground surface erosion and blanket deposition of fine sediments by the Missoula floods may have hidden surface ruptures;
- Potentially fault-deformed geomorphic features (e.g., topographic scarps) cannot be conclusively dated;
- Strike-slip fault movements often do not have significant vertical movements; and
- Most undisturbed ground surfaces in the Pacific Northwest are naturally covered by thick vegetation which can obscure ground surface features.

3.0 Regional Seismicity

3.1 General

Earthquakes in the Pacific Northwest occur in response to active convergence of the Juan de Fuca oceanic plate and the North American continental plate. Stresses build with friction between the plates as the Juan de Fuca plate is subducted beneath the over-riding continental plate in the CSZ. Both plates break periodically along fault lines as a result of the stress. Faulting occurs both between the plates (interplate) and within the plates (intraplate). In northwest Oregon, earthquakes can be generated from three primary sources:

- Megathrust interplate earthquake events are generated along the boundary (the CSZ) between the subducting Juan de Fuca plate and the overriding North American plate,
- Deep intraplate earthquake events are generated within the subducted portion of the Juan de Fuca plate, and
- Shallow intraplate crustal earthquake events occur along faults that form in the continental crust and accretionary wedge of sediments that accumulate along continental shelf and slope.

The largest (megathrust) earthquakes occur on the interface between the two plates within the CSZ and could generate magnitudes of 8 to more than 9. These earthquakes could cause shaking that lasts for several minutes and could generate tsunamis. Moderately large intraplate earthquakes occur deep within the subducting Juan de Fuca plate. These "intraplate" earthquakes could range from magnitude 6.5 to 8.0. Crustal earthquakes are smaller in magnitude (usually less than 7.0) and because they are shallower than the others and occur in the continental crust east of the CSZ, they are likely to be closer to urban areas. Shaking associated with crustal earthquakes usually lasts less than one minute. Although crustal earthquakes are smaller in magnitude and the period of shaking is much shorter than the interplate and intraplate earthquakes, because they would likely occur closer to urban areas, they are still very significant in terms of the potential hazards they pose to populated areas of the Pacific Northwest.

3.2 Earthquake history

The Portland area has exhibited a low to moderate level of historical seismicity compared to other areas of the Pacific Northwest, but it might be the most active area in Oregon for events of moment magnitude (M_W) ≥ 3.0 (Wong et al., 2001). Based on the historical earthquake record, seven felt earthquakes greater than Richter (local) magnitude (M_L) 3.5 have occurred near Portland since 1850 (Bott and Wong, 1993). At least 20 earthquakes less than M_L=3.0 have been recorded in the greater Portland metropolitan area since 1980, many of which were too small to be felt. None of the earthquakes recorded and located in the Portland area can be located with certainty on any known fault. However, because numerous small earthquakes have occurred in this vicinity, the Portland Hills Fault is a likely source for future large earthquakes (Wong et al., 2001).

The most recent CSZ megathrust event occurred in AD 1700 and is believed to have ruptured the entire length of the subduction zone. This event has been estimated to be M_W=9 based on Japanese tsunami records (Satake et al., 1996; 2003).

The CSZ Intraslab region has been the source of numerous historical earthquakes in the region including the M_L =7.1 Olympia earthquake in 1949 and the M_L =6.8 Nisqually earthquake in 2001.

3.3 Evaluating Crustal Faults for Holocene Deformation

The USGS Quaternary Fault and Fold Database of the United States (USGS, 2006[2]) contains information on faults and associated folds in the United States that are believed to be sources of M>6 earthquakes

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[2] Although the website requests the use of a citation with a publication date of 2006, individual faults are updated as new data becomes available. For example, the Lacamas Creek fault was last updated in 2002, the Mt. Angel fault was last updated in 2011, and the Gales Creek fault was last updated in 2017.

during the Quaternary Period (the past 1.6 Ma). Maps of these geologic structures are linked to a database containing detailed descriptions of the faults and reference sources. The database is intended to be the USGS's archive for historic and ancient earthquake sources and is used in current and future probabilistic seismic-hazard analyses.

The USGS has also developed the National Seismic Hazard Map (NSHM) program, which uses a subset of fault data from the USGS Quaternary Fault database and recorded earthquake data to model seismic hazards. The NSHM is updated periodically; the current version was published in 2014 while a draft version was released in 2018.

Using the Quaternary Fault and Fold Database and the 2014 NSHM data set as guides, we have evaluated 29 faults that are or might have been active during the Quaternary Period. We have included all the northwestern Oregon faults included in the 2014 NSHM and all Quaternary faults included in the USGS Quaternary Fault database within a radius of about 100 km (60 mi) of WTP_1.0. Table 1 summarizes the evaluated faults, data sources, and interpretation of their significant characteristics. Figure 1 shows the locations of the faults with respect to WTP_1.0

4.0 Fault Descriptions

4.1 Crustal Faults

In order of increasing distance from WTP_1.0, the following sections summarize the significant facts and characteristics of the 29 faults evaluated in this study and upon which we are basing our conclusions. These faults are also presented in Table 1.

4.1.1 Sherwood-Lake Oswego Fault

The Sherwood-Lake Oswego Fault is a Class C fault (fault classes are defined in Table 1 Notes). It is not included in the 2014 NSHM. The USGS Fault database lists the fault but concludes that it has not been active in the Quaternary Period; no information for this fault, other than its probable location, is available. McPhee et. al. (2014) have interpreted a broad, northeast-trending gravity high along the north side of Parrett Mountain as suggestive of a concealed fault-bounded ridge coincident with the Parrett Mountain uplift. The Sherwood Fault is approximately on strike with the Lake Oswego Fault and together, these faults tend to form the southeastern boundary of the Tualatin basin. A mapped fault is defined as “Class C” when geologic evidence is insufficient to demonstrate either that a tectonic fault exists, or that Quaternary slip or deformation is associated with the feature (USGS, 2006). The Sherwood-Lake Oswego Fault is included here only because it is the nearest mapped fault to the WTP_1.0.

4.1.2 Canby-Molalla Fault

The Canby-Molalla Fault is a Class A (fault classes are defined in Table 1 Notes) fault about 50 km (31 mi) in length; it strikes northwest and it is located about 8 km (5 mi) east of WTP_1.0. Aeromagnetic data suggests that the fault offsets the CRBG and possibly Missoula Flood deposits. This fault was not included in the 2014 NSHM. The USGS fault database gives the estimated age of last activity on this fault as less than 15 ka (Personius, 2002a).

4.1.3 Bolton Fault

The Bolton Fault is a Class B (fault classes are defined in Table 1 Notes) fault located about 11 km (7 mi.) east of WTP_1.0 along the east front of the West Hills south of Lake Oswego. It is a northwest trending fault of about 9 km (5.6 mi) in length. The Bolton Fault is included in the 2014 NSHM and modeled with a slip rate of <0.2 millimeters per year (mm/yr) and a maximum (Richter or local) magnitude M_L =6.19. There is a high fault escarpment in CRBG, but no evidence of Quaternary offset (Personius, 2002b).

4.1.4 Beaverton Fault

The Beaverton Fault is a Class A fault about 15 km (9 mi) in length; it strikes east-west and it is about 12 km (7 mi) north of WTP_1.0. This fault apparently offsets CRBG and late Pleistocene sediments of the Hillsboro Formation as interpreted from geophysical surveys and water well logs. The Beaverton Fault was not included in the 2014 NHSM. The USGS fault database gives the age of last activity on this fault as less than 750 ka (Personius, 2002c).

4.1.5 Newberg Fault

The Newberg Fault is a Class A fault located near the town of Newberg about 15 km (9 miles) southwest of WTP_1.0. This fault trends northwest and is about 7 km (4 mi) in length. It is part of the Gales Creek-Mt. Angel structural zone. The Newberg Fault was included in the 2014 NSHM and modeled with a slip rate of <0.2 mm/yr and a maximum M_L =6.85. There is no surface expression of this fault, but an offset in the CRBG surface has been confirmed by comparison of the logs of water wells that penetrate the basalt and in aeromagnetic and gravity data. The Newberg Fault is considered active although there is no evidence of activity post-15 ka (Personius, 2002d).

4.1.6 Oatfield Fault

The Oatfield Fault is a Class A fault of about 29 km (18 mi) in length located on the western flank of the West Hills; it is about 15 km (9 mi) from WTP_1.0. The strike of the Oatfield Fault is parallel to the Portland Hills Fault and the trend of the West Hills. The Oatfield Fault might be structurally connected to the Portland Hills Fault (Wong, et al., 2001). The Oatfield Fault is not included in the 2014 NSHM. The Oatfield Fault was observed offsetting Boring Lava in Portland's light rail tunnel, but no offset of Quaternary units was observed (Walsh et al., 2011). The USGS Fault database lists the age of the Oatfield Fault at <1.6 Ma (Personius, 2002e).

4.1.7 Portland Hills Fault

The Portland Hills Fault is a Class A fault located about 16 km (10 mi) north of WTP_1.0 in the Portland basin. The Portland Hills Fault is about 49 km (30 mi) in length and marks the western boundary of the Portland basin. There are surface features on the east face of the West Hills that suggest the presence of this fault, and a trench excavation has exposed disturbed Missoula Flood sediments, but no offset. The disturbed sediments might suggest liquefaction during a prehistoric earthquake. However, the limited historical earthquake records do not place any known earthquake on the Portland Hills Fault. Many small magnitude historic earthquakes have been recorded and located near the Portland Hills Fault suggesting that there are active structures nearby; "the presence of small earthquakes, more often than not, delineates areas where larger earthquakes are likely to occur" (Wong et al., 2001). This fault was included in the 2014 NSHM and modeled with a slip rate of <0.2 mm/yr and a maximum M_L =7.05. The USGS Fault database lists the age of last activity on this fault as <15 ka (Personius and Haller, 2017a).

4.1.8 Damascus-Tickle Creek Faults

The Damascus-Tickle Creek Faults consist of numerous short strands within the Boring Hills about 21 km (13 mi) east of WTP_1.0 in the Portland basin. These faults are Class A faults, but they were not included in the 2014 NSHM Project. Some fault strands offset Boring Lava and the Troutdale Formation. The USGS Fault database gives the age of last activity on this fault as <750 ka (Personius, 2002f).

4.1.9 East Bank Fault

The East Bank Fault is a Class A fault located about 22 km (7 mi) northeast of WTP_1.0 in the Portland basin; it is about 29 km (18 mi) in length. There is no surface expression of this fault because it is buried by Missoula Flood deposits, but deep-water wells drilled on opposite sides of the fault and seismic geophysical studies have identified offsets in the CRBG and Troutdale Formation. This fault was not included in the 2014 NSHM. The USGS Fault database lists the age of the most recent activity on this fault as <15 ka, although no surface offsets in the Missoula Flood deposits that overlie and conceal this fault have been recognized (Personius, 2002g).

4.1.10 Helvetia Fault

The Helvetia Fault is a Class A fault located about 22 km (14 miles) northwest of WTP_1.0. This fault is a northwest trending fault of about 7 km (4 mi) in length and forms part of the northeast boundary of the Tualatin basin. The Helvetia Fault was included in the 2014 NSHM and modeled with a slip rate of <0.2 mm/yr and a maximum M_L =6.4. There is no surface expression of this fault; it has been mapped primarily by comparison of water wells that penetrate and offset the CRBG. The Helvetia Fault is considered active; its age of last activity is given as <1.6 Ma in the USGS Fault database (Personius, 2002h).

4.1.11 Grant Butte Fault

The Grant Butte Fault is a Class A fault located about 23 km (14 mi) east of WTP_1.0 in the Portland basin. The Grant Butte Fault was included in the 2014 NSHM and modeled with a slip rate of <0.2 mm/yr and a maximum M_L =6.21. The Pleistocene Boring Lava and Springwater Formation are apparently offset by this fault, but the offset does not cut the overlying late Pleistocene Missoula Flood deposits. The age of last activity on this fault is therefore given as <750 ka (Personius, 2002i).

4.1.12 Gales Creek Fault Zone

The Gales Creek Fault Zone is a Class A fault located about 27 km (17 mi) northwest of WTP_1.0 and it is about 73 km (45 mi) in length (Personius and Haller, 2017b). This fault is included in the 2014 NSHM and modeled with a slip rate of <0.2 mm/yr and a maximum M_L =6.75. This fault is a northwest-trending, right-lateral, strike-slip fault that marks the boundary between the Tualatin basin and the Coast Range uplift. The Gales Creek Fault is a very old fault and appears to have been active for about 60 Ma. A recent trench excavation across a strand of this fault found evidence of Tertiary marine sandstone being thrust over <250 ka loess deposits (R.E. Wells, pers. commun., 2017). Although this observation is strongly suggestive of Quaternary activity, it is inconclusive for activity during the Holocene epoch. The USGS Fault database gives an age of <1.6 Ma for the Gales Creek Fault.

4.1.13 Mt. Angel Fault

The Mt. Angel Fault is a Class A fault located about 29 km (18 mi) south of WTP_1.0. It is approximately 30 km (19 mi) in length, with an age of last activity that is estimated at less than 15 ka. This fault lies on, and might be connected to, the Gales Creek Fault Zone, also a Class A fault. The 1993 M_L =5.6 Scotts Mill earthquake was located near the Mt. Angel Fault, and though the earthquake cannot confidently be located on the fault, there is a strong suggestion that the fault is currently active (Thomas et. al., 1996). This fault was included in the 2014 NSHM and modeled with a slip rate of <0.2 mm/yr and a maximum M_L =6.5 earthquake (Personius and Lidke, 2011).

4.1.14 Sandy River and Lacamas Lake Faults

The Sandy River and Lacamas Lake faults are sometimes referred to as the Frontal Fault. The Sandy River strand is a Class C fault, while the Lacamas Lake strand, with an apparent age of less than 750 ka, is listed as a Class A fault due to the presence of shear zones and an apparent offset in Boring Lava. These two faults were included as the Frontal Fault in the 2014 NSHM and modeled as a right-lateral strike slip fault with a slip rate of <0.2 mm/yr and a maximum M_L =6.5. These faults are about 40 km (25 mi) northeast of WTP_1.0. The Lacamas Lake strand is about 24 km (15 mi) in length and the Sandy River strand is about 17 km (11 mi) in length. The age of the Sandy River strand is estimated at <1.6 Ma (Personius, 2002j; Peterson et al., 2014).

4.1.15 Salem-Eola Hills Homocline

The northwest-striking Salem-Eola Hills homocline is a Class A structure located approximately 49 km (30 mi) south of WTP_1.0, it's about 32 km (20 mi) in length, and its age is estimated at <1.6 Ma. This structure is not included in the 2014 NSHMP. The homocline structure deforms Miocene rocks of the Columbia River Basalt Group along the Salem Hills and Eola Hills in the central Willamette Valley. It is located at the southwestern margin of deposition of the CRBG in this part of Oregon. In the late Miocene, the fold acted as a tectonic dam, causing the obstruction of the ancestral Willamette River and deposition of a thick sequence of basin-fill sediment in the southern Willamette Valley. The fold is not shown on most geologic maps of the region, but it appears to be significantly offset across the Mill Creek fault (Personius, 2002k).

4.1.16 Waldo Hills Fault

The northeast-striking, southeast-dipping Waldo Hills reverse fault is a Class A fault that offsets Miocene rocks of the CRBG along the northwestern margin of the Waldo Hills in the central Willamette Valley. The fault is about 50 km (31 mi) south of WTP_1.0, about 12 km (7.5 mi) in length, and its age is estimated at <1.6 Ma. It is not included in the 2014 NSHM. The Waldo Hills Fault is coincident with a steep, linear range front that marks the northwestern margin of the Waldo Hills and the eastern margin of the central Willamette Valley. No fault scarps on surficial Quaternary deposits have been described along its trace (Personius, 2002l).

4.1.17 Turner / Mill Creek Faults

Originally mapped as two separate faults, the Mill Creek Fault is believed by Yeats et al., (1996) to be an extension of the Turner Fault. These are Class A faults located about 51 km (32 mi) south of WTP_1.0. The faults are about 18 km (11 mi) in length and strike northeast. These faults are modeled in the NSHM as having reverse slip at a rate <0.2 mm/yr and a maximum M_L= 6.59. The faults offset Miocene rocks of the CRBG in the Salem and Waldo Hills in the central Willamette Valley. The Turner and Mill Creek faults are coincident with a CRBG range front along the southern margin of the Waldo Hills, and may deform middle Pleistocene(?) deposits near the Mill Creek water gap (Personius, 2002m).

4.1.18 Tillamook Bay Fault Zone

The Tillamook Bay Fault Zone is a Class A fault and a major northwest-striking fault that offsets the Eocene Tillamook Volcanics on the west flank of the Coast Range. The fault is located approximately 65 km (40 mi) west of WTP_1.0 and it is about 32 km (20 mi) in length. This fault is not included in the NSHM. The fault zone has about 4 km (2.5 mi) of down-southwest vertical separation and about 20 km (12 mi) of left-lateral strike-slip displacement in Eocene Tillamook Volcanics. No displacements in Quaternary deposits have been documented, but the fault zone parallels the mountain front that controls

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the northeastern margin of Tillamook Bay, and thus has geomorphic expression consistent with Quaternary displacement. As with other folds and faults located in the Cascadia forearc, it is unknown if coseismic displacements on this fault are always related to great megathrust earthquakes on the subduction zone, or whether some displacements are related to smaller earthquakes in the North American Plate (Personius, 2002n).

4.1.19 Mount Hood Fault

The Mount Hood Fault is a Class C fault that is located about 75 km (47 mi) east of WTP_1.0 and is modeled in the NSHM as being a normal fault, 11 km (7 mi) in length, with a slip rate of <0.3 mm/yr and capable of generating a maximum magnitude of M_L =6.29 (Peterson, et al., 2014). The Mount Hood fault is shown on numerous Quaternary fault compilations based on a 1982 field mapping project that shows the fault as offsetting Pleistocene to Holocene lava flows from Mount Hood. However, the mapped fault is restricted to Miocene bedrock or is not shown at all on recently published geologic maps. Recent investigations in the area found no evidence of tectonic faulting along the trace originally mapped, but rather found evidence of landslide slip features, which may have been related to deglaciation (Sherrod and Scott, 1995; Scott et al., 1997). Given that recent field investigations have restricted fault movement to only Miocene rocks, the Mount Hood Fault is classified as a Class C structure in the current USGS Fault database. Nevertheless, we have included it here for completeness because it is also included in the current NSHM.

4.1.20 Clackamas River Fault Zone

Located about 80 km (50 mi) from WTP_1.0, the Class A Clackamas River Fault Zone is a broad zone, approximately 29 km (18 mi) in length, of mostly northwest-striking normal and right-lateral strike-slip faults that offset early Pleistocene, Pliocene, and Miocene volcanic rocks in the Cascade Range. The age of these faults is given as <1.6 Ma and the slip rate as <0.2 mm/yr; no evidence of fault scarps on Quaternary deposits has been described in published literature. The Clackamas River Fault Zone is not included in the 2014 NSHM. These faults are part of a regional structural zone that controlled the distribution of CRBG lava flows in western Oregon and may form a link between similarly striking Brothers or Sisters fault zones to the southeast and the Portland Hills Fault Zone to the northwest (Personius, 2002o).

4.1.21 Happy Camp Fault

The Happy Camp Fault is located approximately 89 km (55 mi) west of WTP_1.0. The age of this fault is given as <1.6 Ma and the assigned slip rate is <0.2 mm/yr. The Happy Camp Fault is included in the NSHM as a reverse fault, 20 km in length and capable of generating a maximum M_L =6.58. This fault is an east-striking thrust fault that offsets the Miocene sedimentary rocks of the Astoria Formation on the west flank of the Coast Range uplift. The fault might project offshore as the Nehalem Bank Fault. Locally, the fault thrusts Miocene CRBG over poorly dated Quaternary deposits in sea cliffs near Happy Camp, at the north end of Netarts Bay. As with other folds and faults located in the Cascadia forearc, it is unknown if coseismic displacements on this fault are always related to great megathrust earthquakes on the subduction zone or whether some displacements are related to smaller earthquakes in the North American Plate (Personius, 2002p).

4.1.22 Owl Creek Fault

The steeply east-dipping Owl Creek Fault is a Class A reverse fault associated with an anticline in the Eocene Spencer Formation mapped in the subsurface east of Corvallis. This fault is not included in the NSHM. The north trending fault is located about 90 km (56 mi) south of WTP_1.0. Fault length is about 15 km (9 mi); age and slip rate are given as <750 ka and <0.2 mm/yr, respectively (Personius, 2002q). The fault, which has no geomorphic expression, apparently offsets the middle- to late-Pleistocene Rowland Formation, but it does not offset the latest Pleistocene Willamette Formation (Graven, 1990; Yeats et al., 1996). Madin and others (2001) infer late Quaternary offset, and Geomatrix Consultants, Inc. (1995) and Madin and Mabey (1996) infer middle and late Quaternary (<780 ka) displacement.

4.1.23 Hood River Fault Zone

The Hood River Fault zone is approximately 96 km (60 mi) east of WTP_1.0. The Hood River Fault zone is a Class A fault with normal to right-lateral displacement, about 44 km (27 miles) in length which has been assigned a slip rate of <0.2 mm/yr and an age of < 1.6 Ma. The Hood River Fault Zone is not modeled in the NSHM. The zone defines the eastern margin of a half graben that forms the Upper Hood River Valley in the High Cascades of northern Oregon. This structure may be part of an extensive group of graben structures formed in response to subsidence related to extrusion of extensive volcanic rocks in the early Pliocene. The area is underlain by Miocene volcanic rocks of the Columbia Plateau and Pliocene through Quaternary volcanic rocks of the Cascade Range. No fault scarps on Quaternary deposits have been described, but prominent escarpments on Neogene (comprises the Miocene and Pliocene epochs, 23 to 2.6 Ma) volcanic rocks and a minimum offset of 600 m (1,969 ft.) in Pliocene volcanic rocks suggest that some displacement occurred in the Quaternary (Personius, 2002r).

4.1.24 Cascadia Fold and Thrust Belt

The Cascadia Fold and Thrust Belt is a group of north-striking, Class A folds and faults that form a broad fold and thrust belt of deformed sediments on the continental shelf and slope off the Oregon Coast. There is no detailed published information on these structures, but many of the them do offset Pleistocene and Holocene sediments. Their age is given as <15 ka. They are not included in the NSHM. The fold and thrust belt consists of two primary domains differentiated based on fold wavelength: (1) a continental slope domain underlain by a thick sequence of accretionary wedge sediments deformed by closely-spaced thrust faults and short-wavelength folds, and (2) a continental shelf domain underlain by a rigid basement of Siletz River Volcanics deformed by more broadly spaced folds and thrusts. As with other folds and faults located in the Cascadia forearc, it is unknown if coseismic displacements on these structures are always related to great megathrust earthquakes on the subduction zone or whether some independent displacements are related to smaller earthquakes in the overriding North American Plate (Personius, 2002s).

4.1.25 Faults Near the Dalles

Faults near The Dalles in northern Oregon and southern Washington are northwest-striking, right-lateral strike-slip and minor normal faults. The nearest of these 6 fault strands are located about 100 km (60 mi) east of WTP_1.0. These faults have been assigned an age of <1.6 Ma and a slip rate of <0.2 mm/yr. These faults are not modeled in the NSHM. These faults span the Columbia River Gorge and offset Miocene and Pliocene volcanic and sedimentary rocks near the southern margin of the Yakima fold belt. The nearest fault strand is about 9 km (5.6 mi) long. No scarps on Quaternary deposits have been described, but one of these faults may offset Quaternary basalt. These faults form prominent regional lineaments that also suggest they may have undergone Quaternary displacement (Personius and Lidke, 2003).

4.1.26 Nehalem Bank Fault

The Nehalem Bank Fault is approximately 99 km (62 mi) west of WTP_1.0. The north-and northwest-striking, right-lateral reverse Nehalem Bank Fault is mapped as multiple fault strands and anticlinal axes in Miocene through Holocene sediment on the continental shelf. Cumulative length is fault is about 52 km (32 miles). This is a Class A fault, age of most recent deformation is thought to be <15 ka; slip rate is <0.2 mm/yr. This fault is not modeled in the NSHM. The fault may form the boundary between the Eocene Siletz Volcanics to the east and Miocene and younger accretionary wedge sediment to the west. Offsets of 10 to 20 m in Holocene sediment, apparent in side-scan sonar and seismic records, probably post-date the late Pleistocene sea-level low-stand suggesting most recent movement in the latest Quaternary (Goldfinger, 1994). As with other folds and faults located in the Cascadia forearc, it is unknown if coseismic displacements on this fault are always related to great megathrust earthquakes on the subduction zone, or whether some independent displacements are related to smaller earthquakes in the overriding North American Plate (Personius, 2002t).

4.1.27 Unnamed Offshore Faults

The Unnamed Offshore Faults are a group of Class A faults located about 100 km (62 mi) west of WTP_1.0. They total about 287 km (178 miles) in length and are not included in the NSHM. These faults offset accretionary wedge sediments on the continental shelf and slope off-shore near the Tillamook area. Some faults also offset the underlying oceanic basalts of the subducting Juan de Fuca Plate. These faults are mapped as left- and right-lateral strike-slip faults and normal and reverse faults, but most have strikes oblique to the Cascadia deformation front, suggesting a strong lateral component of slip. No detailed information on age of offset deposits is available, but similarities with better-studied offshore faults suggest most recent movement in the latest Quaternary (<15 ka) on most of these structures. As with other folds and faults located in the Cascadia forearc, it is unknown if coseismic displacements on these faults are always related to great megathrust earthquakes on the subduction zone, or whether some independent displacements are related to smaller earthquakes in the overriding North American Plate (Personius, 2002u).

4.1.28 CSZ Megathrust

The CSZ extends from Vancouver Island to Northern California and forms the boundary between the overriding North American plate and the subducting Juan de Fuca Plate. The most recent event, which occurred in AD 1700 (Atwater and Hemphill-Haley, 1997), is believed to have ruptured the entire length of the subduction zone and has been estimated to be M_W =9 based on Japanese tsunami records (Satake, et al., 1996). This fault is considered capable of generating an event of up to M_L =9.3 at 50-75 km (30-50 mi) from the site (Shannon & Wilson, 2017).

4.1.29 CSZ Intraslab

CSZ intraslab earthquakes are generated within the subducting Juan de Fuca Plate as it is stressed and deformed. The CSZ Intraslab region has been the source of numerous historical earthquakes in

Washington State including the M_W =7.1 Olympia earthquake in 1949 and the M_W =6.8 Nisqually earthquake in 2001. This type of fault is considered capable of generating a M_L =8 earthquake 50-75 km from the site (Shannon & Wilson, 2017).

5.0 Conclusion

From the research cited above we find that in the greater Portland Metropolitan area there are no historically recorded earthquakes that can be conclusively located on a known fault, and no conclusive evidence for activity on a known fault in the past 12,000[3] years. In addition, no known active faults cross the WTP_1.0 site.

The nearest known, potentially active faults to the WTP_1.0 site are the Sherwood-Lake Oswego and Canby-Molalla faults, which are located at approximately 2 km (1 mi) north and 8 km (5 mi) east respectively. Therefore, it is our opinion that the risk of fault rupture on the WTP_1.0 site is negligible.

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[3] We have retained the 12,000-year date as the beginning of the time interval of interest because it is consistent with the dates used in other published investigations of local faulting (Wells et al., 1998; Wong et al., 2001; Madin and Hemphill-Haley, 2001; Liberty et al., 2003; McPhee et al., 2014). This date is the approximate end of the period of Missoula flooding. Sediments deposited by the Missoula floods obscure surface evidence that might exist of prior fault rupture.

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ATTACHMENTS

Table 1	Quaternary Faults in Northwest Oregon
Figure 1	Regional Fault Map – 100 km Radius

Table 1. Quaternary Faults in Northwest Oregon

Fault Name	Fault ID[1]	Fault Class[2]	Distance to WTP_1.0 (km)	Evidence for Quaternary Activity	Evidence for <12 ka Activity	Age[3]	Slip Rate (mm/yr)	Sense of Slip	Strike (azimuth degrees)	Dip Direction	Length (km)	NSHMP Model 2014		Data Sources
												Included?	Max. Mag.[4]	
Sherwood-Lake Oswego Fault	-	C	2	Concordant w/ broad gravity high suggestive of a concealed fault-bound basement ridge coincident w/ Parrett Mtn uplift.	No	-	-	-	-	-	-	No	-	McPhee et., al., 2014
Canby-Molalla Fault	716	A	8	Aeromagnetic anomalies suggest offset of CRBG; seismic reflection suggests possible offset of Missoula Flood Deposits.	No	<15 ka	<0.2	Right Lateral; Reverse	326	-	50	No	-	Personius, 2002a
Bolton Fault	874	B	11	150 m high escarpment of CRBG; no evidence of Quaternary offset.	No	<1.6 Ma	<0.2	Reverse, Right Lateral	307	SW	9	Yes	6.19	Madin, 1990; Personius, 2002b
Beaverton Fault	715	A	12	Offset of late Pleistocene Hillsboro Fm. sediments interpreted from geophysical surveys and subsurface explorations.	No	<750 ka	<0.2	Reverse	86	Unk	15	No	-	Madin, 1990; Popowski, 1996; Personius, 2002c; McPhee and others 2014
Newberg Fault	717	A	15	Offset CRBG noted in subsurface exploration data identifies location of fault; no surface expression; undeformed Missoula Flood deposits overlie the fault.	No	<1.6 Ma	<0.2	Right Lateral, Reverse	318	-	5	Yes	6.85	Personius, 2002d
Oatfield Fault	875	A	15	Fault identified by offset CRBG; offset 1 Ma Boring lava exposed in Light Rail Tunnel; no offset Quaternary units noted.	No	<1.6 Ma	<0.2	Reverse, Right Lateral	319	NE	29	No	-	Personius, 2002e; Walsh and others, 2011
Portland Hills Fault	877	A	16	Prominent escarpment along W. Hills; seismic reflection and ground penetrating radar suggest possible offset at depth.	No; trenching data identified disturbed, but not offset late Pleistocene flood sediments.	<1.6 Ma	<0.2	Right Lateral, Reverse	323	SW	49	Yes	7.05	Madin, 1990; Beeson, et. al., 1991; Unruh et. Al., 1994; Pratt et. al., 2001; Liberty et. al., 2003; Personius and Haller, 2017a
Damascus-Tickle Creek Faults	879	A	21	Numerous short strands; some offset Pleistocene Boring Lava; folds and offsets in Troutdale Fm; one strand might have offset Missoula Flood deposits while others are buried by flood deposits.	No; paleoseismic trench was excavated across a fault strand, but results were inconclusive.	<750 ka	<0.2	Right Lateral, Left Lateral, Reverse	0	-	16	No	-	Madin, 1990, 1994; Personius, 2002f
East Bank Fault	876	A	22	CRBG and Troutdale Fm offset at fault; no surface offset in Missoula Flood deposits, but seismic reflection suggests offset at depth.	No	<750 ka	<0.2	Reverse,	314	NE	29	No	-	Beeson, et. al., 1991; Madin, 1990; Personius, 2002g
Helvetia Fault	714	A	22	Identified in subsurface explorations as offsetting CRB and overlying Pleistocene basin-fill deposits.	No	<1.6 Ma	<0.2	Right Lateral, Reverse	334	-	7	Yes	6.4	Personius, 2002h
Grant Butte Fault	878	A	23	Pleistocene Boring lava and Springwater Fm mapped as offset at fault trace; overlying Missoula Flood deposits are not deformed.	No	<750 ka	<0.2	Normal	10	N	10	Yes	6.21	Madin, 1990, Personius, 2002i

Table 1. Quaternary Faults in Northwest Oregon

Fault Name	Fault ID[1]	Fault Class[2]	Distance to WTP_1.0 (km)	Evidence for Quaternary Activity	Evidence for <12 ka Activity	Age[3]	Slip Rate (mm/yr)	Sense of Slip	Strike (azimuth degrees)	Dip Direction	Length (km)	NSHMP Model 2014		Data Sources
												Included?	Max. Mag.[4]	
Gales Creek Fault Zone	718	A	27	Paleoseismic trenches of lidar-identified fault scarps yielded deformation age of <250 ka loess; modern streams appear to be offset across the fault zone.	No; trenching data is suggestive, but not conclusive.	<1.6 Ma	<0.2	Right Lateral, Reverse	319	-	73	Yes	6.75	Bemis and Wells, 2012; McPhee and others, 2014; R.A. Wells pers. common., 2017; Personius and Haller, 2017b
Mt. Angel Fault	873	A	29	Subsurface explorations indicate CRBG offset; parallel to a strong gravity anomaly; late Pleistocene (<125 ka) fluvial deposits possibly deformed across the fault; fault location coincident with 1993 Scotts Mill EQ.	No; the 1993 Scotts Mill EQ was located nearby, but not confidently located on the fault.	<15 ka	<0.2	Right Lateral, Reverse	43	NE	30	Yes	6.8	Unruh et. al., 1994; Personius and Lidke, 2011; McPhee and others, 2014
Sandy River Fault	-	C	38	Fault identified in gravity data; no evidence for offset of Quaternary deposits.	No	-	< 0.02	Strike Slip	-	Vertical	17	Yes	6.5	Peterson et. al., 2014
Lacamas Lake Fault	880	A	42	Shear zones mapped on surface exposures of the fault; subsurface explorations indicate offset of Pleistocene Boring Lava; overlying Missoula Flood deposits undeformed; Columbia River morphology appears to have been influenced by the fault.	No	<750 ka	<0.2	Right Lateral, Normal	317	SW	24	Yes	6.67	Anderson et. al., 2013; Evarts, 2006; Personius, 2002j
Salem-Eola Hills homocline	719	A	49	Older, possibly Quaternary, gravel might be deformed; evidence equivable.	No	<1.6 Ma	<0.2	Homocline	334	NE	32	No		Pezzopane, 1993; Unruh et. al., 1994; Crenna and Yeats, 1994; Yeats, et. al., 1993; Personius, 2002k
Waldo Hills Fault	872	A	50	Offsets CRBG and older, possibly Quaternary, gravel deposits; evidence equivocal.	No	<1.6 Ma	<0.2	Normal	45	NW	12	No		Mabey and Madin, 1996; Yeats et. al., 1991, 1993; 1996; Yeats and Levi, 1994; Crenna and Yeats, 1994; Personius, 2002l
Turner Fault (TF) and Mill Creek Fault (MCF)	871	A	51	Deformation of mid-Pleistocene sediments is inferred. Originally mapped as two faults, but Yeats et. al., (1996) believe the MCF might be an extension of the TF.	No	<1.6 Ma	<0.2	Reverse	66	SE	20	Yes	6.59	Yeats et. al., 1991, 1993, 1996; Yeats and Levi, 1994; Crenna and Yeats, 1994; Personius, 2002m
Tillamook Bay Fault Zone	881	A	65	The fault has a geomorphic expression consistent with Quaternary displacement.	No	<1.6 Ma	<O.2	Reverse, Left Lateral	304	NE	32	No		Pezzopane, 1993; Madin and Mabey, 1996; Personius, 2002n
Mount Hood Fault	-	C	75	None. Recent investigations found evidence of land slip, but faulting only in Miocene-age rocks.	No	-	<0.3	Normal	-	NE	11	Yes	6.29	Sherrod and Scott, 1995; Scott et. al., 1997; Peterson et. al., 2014
Clackamas River Fault Zone	864	A	80	No offsets in Quaternary units are described, but early Pleistocene volcanics might be offset.	No	<1.6 Ma	<0.2	Normal, Right lateral	341	W, E, varies	29	No		Anderson, 1978; Sherrod and Smith, 1989; Sherrod and Scott, 1995; Personius, 2002o

Table 1. Quaternary Faults in Northwest Oregon

Fault Name	Fault ID[1]	Fault Class[2]	Distance to WTP_1.0 (km)	Evidence for Quaternary Activity	Evidence for <12 ka Activity	Age[3]	Slip Rate (mm/yr)	Sense of Slip	Strike (azimuth degrees)	Dip Direction	Length (km)	NSHMP Model 2014		Data Sources
												Included?	Max. Mag.[4]	
Happy Camp Fault	882	A	89	Col. River Basalt Gp thrust over deposits thought to be Quaternary age.	No	<1.6 Ma	<0.2	Reverse	287	N	20	Yes	6.58	Parker, 1990; Wells et. al., 1994; McNeill, et. al., 1998; Personius, 2002p
Owl Creek Fault	870	A	90	Offsets the late Pleistocene Rowland Formation.	No	<750 ka	<0.2	Reverse	5	E?	15	No		Craven, 1990; Madin and Mabey, 1996; Yeats et. al., 1996; Madin, et. al., 2001; Personius, 2002q
Hood River Fault Zone	866	A	96	Offsets Col. River Basalt and Pliocene volcanics; no scarps have been described in Quaternary units.	No	<1.6 Ma	<0.2	Normal, Right lateral	329	W	44	No		Williams et. al., 1982; Sherrod and Pickthorn, 1989; Pezzopane, 1993; Madin and Mabe, 1996; Weldon et. al., 2002; Personius, 2002r
Cascadia Fold and Thrust Belt	784	A	97	No detailed info published on these structures, but many off-shore structures do offset Pleistocene and Holocene sediments.	No	<15 ka	1.0 - 5.0	Thrust	330	W, E	484	No		Goldfinger, et. al, 1992, 1994, 1997; Pezzopane, 1993; Personius, 2002s
Faults near The Dalles	580	A	98	Offsets Col. River Basalt and Pliocene sedimentary units; 0.84 Ma basalt flows might possibly be offset.	No	<1.6 Ma	<0.2	Right lateral, Normal	322	Vertical	69	No		Beaulieu, 1977; Swanson, et. al., 1981; Bela, J.L., 1982; Anderson, J.L., 1987; Korosec, 1987; Walsh, et. al., 1987; Walker and MacLeod, 1991; Weldon, et. al., 2002; Personius and Lidke, 2003
Nehalem Bank Fault	789	A	99	Offsets Miocene through Holocene sediments on continental shelf.	Sea floor offsets of 10-20 m probably post-date the late Pleistocene sea level lowstand.	<15 ka	1.0 - 5.0	Right lateral, Reverse	345	Vertical, NE	101	No		Goldfinger, et. al., 1992a, 1992b; Goldfinger, 1994; Madin and Mabey, 1996; McNeill et. al., 1998; Personius, 2002t
Unnamed Offshore Faults	785	A	109	No detailed info published on this structure, but many off-shore structures do offset Pleistocene and Holocene sediments.	No	<15 ka	1.0 - 5.0	Left lateral, Right lateral, Normal, Reverse	349	-	280	No		Goldfinger, et. al, 1992, 1994, 1997; Personius, 2002u
Cascadia Subduction Zone	781	A	200	In N. America, radiocarbon and tree ring dating limit most recent rupture between Aug 1699 and May 1700; in Japan written history of widespread tsunami of remote origin in Jan 1700.	Yes	<15 ka	>5.0	Thrust	356	9^0-11^0 E	754 km			Atwater et al., 1995; Atwater and Hemphill-Haley, 1997; Satake, et al., 2003; Personius and Nelson, 2006

Notes:

1. Fault ID number in the USGS Quaternary Fault and Fold Database
2. Fault Classes:

Class A= Geologic evidence demonstrates the existence of a Quaternary fault of tectonic origin, whether the fault is exposed for mapping or inferred from liquefaction or other deformational features.

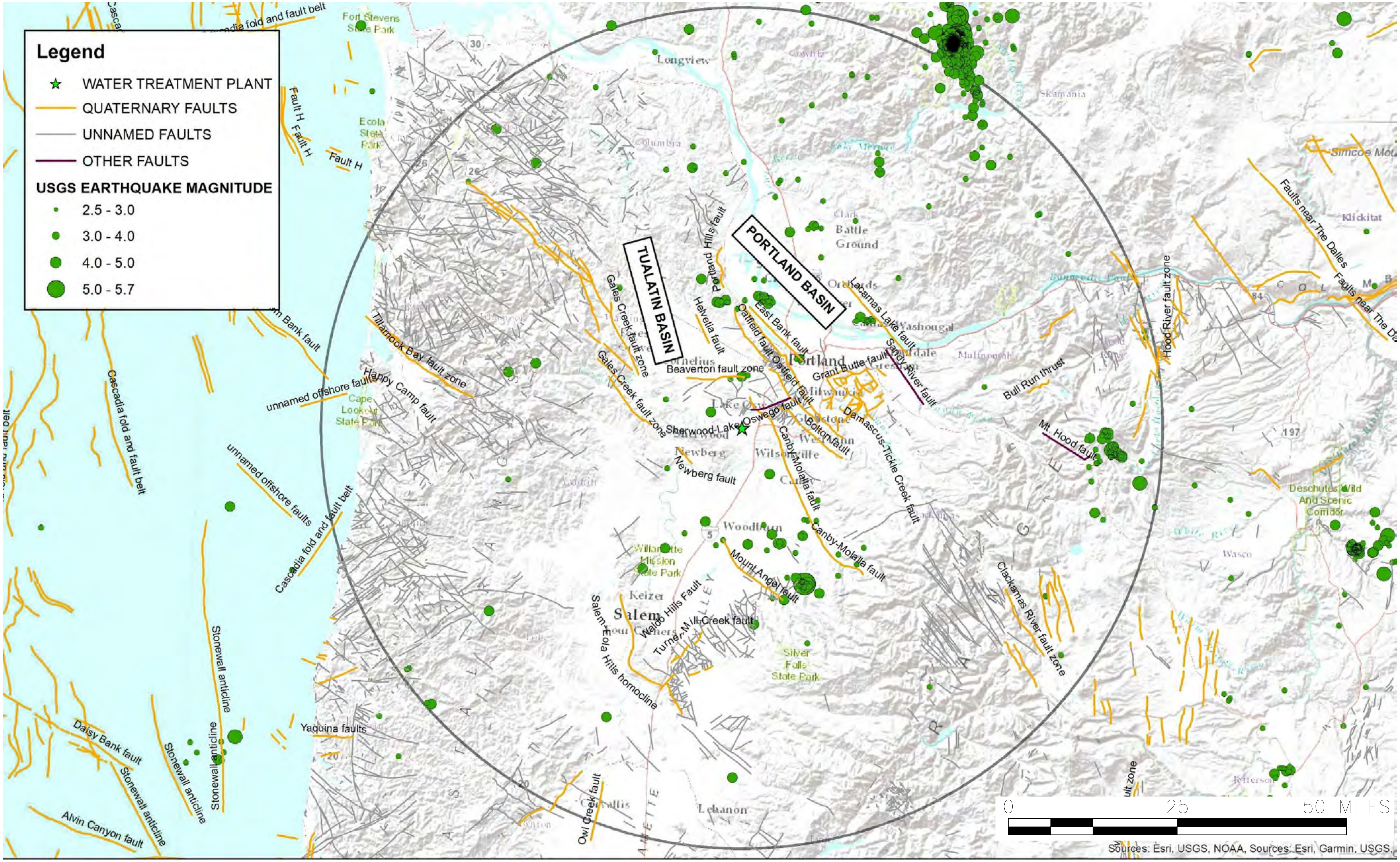
Class B= Geologic evidence demonstrates the existence of a fault or suggests Quaternary deformation, but either (1) the fault might not extend deeply enough to be a potential source of significant earthquakes, or (2) the currently available geologic evidence is too strong to confidently assign the feature to Class C but not strong enough to assign it to Class A.

Class C= Geologic evidence is insufficient to demonstrate (1) the existence of a tectonic fault, or (2) Quaternary slip or deformation associated with the feature.

Class D= Geologic evidence demonstrates that the feature is not a tectonic fault or feature; this category includes features such as demonstrated joints or joint zones, landslides, erosional or fluvial scarps, or landforms resembling fault scarps, but of demonstrable non-tectonic origin.

3. Ma= Mega annum or one million years; ka= kilo annum or one thousand years;
4. Maximum Magnitude used in the 2014 NSHMP model.
5. - = no data; unknown.

FOR LAND USE PERMITTING (EXHIBIT B)



NOTES:

1. QUATERNARY FAULTS ARE FROM U.S. GEOLOGICAL SURVEY (USGS), 2006, QUATERNARY FAULT AND FOLD DATABASE FOR THE UNITED STATES, ACCESSED 01/08/2019 FROM USGS WEBSITE: HTTPS://EARTHQUAKE.USGS.GOV/HAZARDS/QFAULTS/.
2. THE UNNAMED FAULTS ARE FROM THE OREGON DEPARTMENT OF GEOLGOY AND MINERAL INDUSTRIES (DOGAMI), DIGITAL SERIES NO. OGDC-6 (2015, ACCESSED 05/05/2017 FROM HTTPS://WWW.OREGONGEOLOGY.ORG/PUBS/DDS/P-OGDC-6.HTM.
3. OTHER FAULTS REFER TO TEXT AND TABLE 1 FOR REFERENCES.
4. EARTHQUAKES ARE FROM U.S. GEOLOGICAL SURVEY (USGS), 2019, EARTHQUAKE CATALOG, ACCESSED 01/17/2019, HTTPS://EARTHQUAKE.USGS.GOV/EARTHQUAKES/SEARCH.
5. CIRCLE REPRESENTS AN APPROXIMATE 100 KM RADIUS FROM WTP_1.0

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WILLAMETTE WATER SUPPLY PROGRAM

WTP_1.0

FAULT LOCATION STUDY
FAULT MAP

FIG.1

DEC 2019

Path: C:\Users\cburke\Box\Jobs\5887.0 WWSP WTP 1.0\CADD_FaultStudy_Fig.1.dwg Plot date: Dec 10, 2019, CAD User: cburke

Appendix G

Geological Hazards Report (GHR)

FOR LAND USE PERMITTING (EXHIBIT B)

Willamette Water Supply
Our Reliable Water

Willamette Water Supply Program

WTP_1.0

Geological Hazards Report

FINAL

Prepared For:

CDM Smith

May 6, 2020

McMILLEN JACOBS ASSOCIATES

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Table of Contents

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Acronyms and Abbreviations

bgs	below ground surface
bpf	blows per foot
CM/GC	Construction Manager/General Contractor
CRB	Columbia River Basalt
ID	Inside Diameter
GDR	Geotechnical Data Report
GER	Geotechnical Engineering Report
GHR	Geologic Hazards Report
HQ3	A triple-tube wireline rock core sampling system
LF	Linear feet
MA	Mega annum (one million years)
McMillen Jacobs	McMillen Jacobs Associates
MFD	Missoula Flood Deposits
MGD	Million Gallons Per Day
PGE	Portland General Electric
Project	WWSP_WTP_1.0 Project
psi	pounds per square inch
PVC	Polyvinylchloride
RQD	Rock Quality Designation
SPT	Standard Penetration Test
TVWD	Tualatin Valley Water District
WTP_1.0	Water Treatment Plant Project 1.0
WWSP	Willamette Water Supply Program

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Revision Log

Revision No.	Date	Revision Description
Final Submittal	May 6, 2020	Incorporated CDM Smith’s comments.

1.0 Introduction

1.1 General

McMillen Jacobs Associates (McMillen Jacobs) has been retained by CDM Smith to provide geotechnical services for the Willamette Water Supply Program (WWSP) Water Treatment Plant (WTP_1.0) Project. Tualatin Valley Water District (TVWD), the City of Hillsboro, and the City of Beaverton are the project owners. The project is located in Washington County, Oregon, and shown in Figure 1. This Geological Hazards Report (GHR) summarizes the geologic and geotechnical data collected from the subsurface investigations performed for the WTP_1.0 Project.

1.2 Project Description

The Willamette Water Supply Program (WWSP) is a drinking water infrastructure program implemented by the TVWD, the City of Hillsboro, and the City of Beaverton to provide a seismically resilient water supply for their service areas. The WWSP includes more than 30 miles of transmission pipelines, ranging from 36 to 66 inches in diameter, extending from the Willamette River Water in Wilsonville to the TVWD service areas in Washington County, including the cities of Hillsboro and Beaverton. The WWSP also includes two 15-million-gallon water storage tanks, a new water treatment plant, and a raw water pumping station. The new system elements are being designed to meet future demand and to provide redundancy in the event of an emergency.

The WWSP has been divided into multiple design packages and work is proceeding with a phased approach. The WTP_1.0 is a new water treatment plant with an initial treated water design capacity of 60 million gallons per day (mgd) and a future build-out treated water design capacity of 120 mgd, located near the City of Sherwood, Oregon. The project includes construction of several access roads within the treatment plan and construction of a portion of SW Blake Street that connects the SW 124th Avenue to the plant’s western property line. The project involves mitigation and/or protection for the existing natural areas (i.e. wetlands).

WTP_1.0 will need to be connected to the existing infrastructure including the raw water pipeline, treated water pipeline, sewer, and storm drain in SW Tualatin-Sherwood Road and SW 124th Avenue. Connection to the existing infrastructure will require coordination with PLM_4.0, the City of Sherwood, Washington County, and the developer of the parcel north of WTP_1.0 site.

The owners have selected a Construction Manager/General Contractor (CM/GC) project delivery method. The CM/GC contractor will be involved throughout the design from the preliminary design to the final detailed design phase. The construction is anticipated to begin in early 2022.

1.3 Purpose and Scope of Work

The purpose of this report is to evaluate geologic and seismic hazards based on anticipated ground motions consistent with building and project seismic design guidelines. Specifically, the scope of work for this report includes the following:

- Summary of the regional geologic, site surface, and site subsurface conditions;

- Summary of regional faulting and seismic sources that control the seismic hazard;
- Identification of the anticipated ground shaking associated with the design seismic event;
- Summary of seismic hazards including liquefaction, liquefaction settlement, and ground rupture;
- Summary of slope stability across the project area considering static and seismic conditions;
- Evaluation of other seismic hazards such as lateral spreading, loss of bearing capacity, etc.;
- Impact of flooding for 100-year and 500-year events, and seismically induced dam failure; and
- Development of this report summarizing our findings and analyses.

1.4 Project Geotechnical Reports

This Geotechnical Hazard Report has been developed for the use of the design team to summarize geologic and seismic hazards in support of the project design. Several related geotechnical documents have been developed for this project and are referred to in this report. These documents are as follows:

- *Willamette Water Supply Program – WTP_1.0 Fault Location Study Technical Memorandum (McMillen Jacobs, 2019a);*
- *Willamette Water Supply Program – WTP_1.0 Geotechnical Data Report (McMillen Jacobs, 2019b),*
- *Willamette Water Supply Program –WTP_1.0 Geotechnical Engineering Report (McMillen Jacobs, 2020);*
- *Geotechnical Report – Probabilistic Seismic Hazard Analysis, Willamette Water Supply Program, Clackamas and Washington County, Oregon (Shannon & Wilson, Inc., 2017).*

1.5 Authorization

The Tualatin Valley Water District authorized the WTP_1.0 project work on behalf of the Owners, under the terms and conditions of an agreement between the Owners and CDM Smith dated July 24, 2018. McMillen Jacobs has been retained by CDM Smith to provide geotechnical design services for, or in connection with, the Willamette Water Supply Program WTP_1.0 Project (Project) per their Subconsultant Agreement dated August 17, 2018.

2.0 Site Description

2.1 Site Location

The WTP_1.0 is located on an approximately 90-acre site at 12900 SW Tualatin-Sherwood Road in unincorporated Washington County. The project site is located south of SW Tualatin-Sherwood Road and west of SW 124th Avenue on a vacant and largely wooded parcel. The project's north property line is approximately 1,100 feet south of SW Tualatin Sherwood Road; intervening land, currently vacant, was formerly in agricultural use. SW 124th Avenue borders the project's eastern property boundary. The western boundary is largely bordered by vacant woodland and a construction contractor's equipment yard. The southern boundary is a woodland which is crossed diagonally, northwest to southeast, by a Portland General Electric (PGE) powerline easement.

A woodland and former agricultural land lie east of SW 124th Avenue from the northern portion of the project site. An active rock quarry operation is present east of SW 124th Avenue opposite the southeastern portion of the project site. A rock quarry site is also present south beyond the southern property line.

The project location is shown on Figure 1, Vicinity Map.

2.2 Surface Conditions

The existing ground surface elevation varies across the site from elevation El 284 feet and descends in all directions away from the plant to a low of approximately elevation of 192 feet. Most of the WTP_1.0 future-site and immediately adjacent areas are wooded with thick underbrush, including large thick patches of Himalayan blackberry and pervasive poison oak. The over story incudes numerous large Douglas fir, Oregon white oak, and madrone, among others. Several decomposing saw-cut stumps suggest that the site was logged in the past. An existing farmhouse and several out-buildings are located near the northern edge of the property.

Bare rock outcrops were noted at several locations near the WTP_1.0 site and a prominent rock face with approximately a 10-foot step down toward the north was observed north of the WTP footprint. Although difficult to verify through the thick underbrush, this rock face appears to be continuous from the northeast corner of the site westward and around to the southwest corner. Seven wetland areas have been identified by others within the site. Two small wetland areas, one northeast of the WTP footprint and another near the midpoint of the WTP footprint were saturated at the time of our explorations in December 2018 and February 2019.

2.3 Geology

Regionally, the site lies within the Willamette Valley, a structural lowland between uplifted marine rocks of the Coast Range and volcanic rocks of the Cascade Range. The Coast Range, to the west of the lowland, consists of several thousand feet of Tertiary marine sandstone, siltstone, shale, and associated volcanic and intrusive rocks. The Cascade Range, to the east of the lowland, consists of volcanic lava flows, ash-flow tuffs, and pyroclastic and epiclastic debris. Marine and continental strata interfinger beneath and adjacent to the Willamette Lowland.

Four major depositional basins were formed from folding and faulting during and after arrival of the Columbia River Basalt Group. These depositional basins include: the southern Willamette basin, northern Willamette basin, Portland basin, and the Tualatin basin. These basins, separated in most places by folded or faulted uplands of the Columbia River Basalt, have locally accumulated more than 1,600 feet of fluvial sediment derived from adjacent uplifted blocks of Columbia River Basalt, the Cascade and Coast Ranges, and transported into the region by the Columbia River.

Locally, WTP_1.0 lies on the broad summit of the Parrett Mountain uplift which forms the boundary between the Tualatin Valley and the northern Willamette Valley. Parrett Mountain is an uplifted block of Columbia River Basalt. During the period of slow upward movement, exposed surficial basalt gradually developed a deep profile of weathering that graded from decomposed silt and clay soils at the surface through iron-stained and open-jointed weathered rock to relatively fresh dark gray basalt at depth. In the latest Pleistocene, about 15,000 and 12,000 years ago, a series of catastrophic floods reoccurring as outbursts caused by the melting of Glacial Lake Missoula. These flood waters inundated the Columbia River system and back flooded up the Willamette River. The flood waters also surged through the Tualatin Mountains (“Portland Hills”) gap at Lake Oswego, dumping boulders and coarse gravel at the mouth of the gap, which grade westward to sand and then to micaceous, clayey to fine sandy silt across the Tualatin Valley. Many of the floods that entered the Tualatin Valley were sufficiently large enough to overtop the Parrett Mountain ridge crest. The flood waters then cascaded south into the Willamette Valley, scouring and eroding the soil and weathered basalt surface of the ridge crest in the process. This area of flood-scoured over-flow channels is now referred to as the Tonquin Scabland. The proposed WTP_1.0 is located on the Parrett Mountain ridge crest and within the area overtopped and scoured by the catastrophic floods.

2.4 Subsurface Conditions

Subsurface explorations completed in December 2018 and February 2019. The explorations included twelve borings, six test pits, fifteen probe holes, and geophysical explorations at three locations. Locations of the explorations are shown on Figure 2. Details of explorations are provided in the GDR (McMillen Jacobs, 2019b)

We identified several geological units consisting of Topsoil, Missoula Flood Deposits, Residual Soil, and Columbia River Basalt. These units were identified based on their geologic origin, stratigraphic position, engineering properties, and their distribution in the subsurface. Variations in subsurface conditions may exist between the locations of the borings.

Brief descriptions of the identified geologic units are provided below. Detailed descriptions of the units and accompanying laboratory test data are included in the GDR for WTP_1.0 (McMillen Jacobs, 2019b).

- **Topsoil**: Consists predominantly of 3- to 12-inches of very soft, dark brown to black, low plasticity Organic Silt;

- **Missoula Flood Deposits (MFD)**: Consists of two facies; (1) Valley Fill deposits; stiff moist slightly yellow to orange-brown mottled Silt (ML); which occur at lower elevations in the former agricultural field at the northeast corner of the project site, and (2) Channel Fill deposits; soft to stiff slightly yellow-brown Silt with scattered subangular cobbles and boulders which occur in the flood-scoured uplands in and adjacent to the areas now occupied by wetlands;

- **Residual Soil**: Generally, consists of very dense or stiff to hard mixtures of silt with trace sand and scattered to numerous angular, iron-stained gravel- to cobble-sized rock fragments; and
- **Columbia River Basalt (CRB)**: The Columbia River Basalt (CRB) Unit includes basalt that is highly weathered to fresh. The basalt is generally weak to very strong, moderately to slightly weathered, moderately to intensely fractured with iron-stained joint surfaces. The Rock Quality Designation (RQD) of the basalt rock ranged from 0 to 100 percent and averaged about 58 percent. Unconfined compressive strength ranged from approximately 12,000 psi to 34,000 psi, with an average of 22,000 psi. Corrected Point Load Strength Index ($I_{S(50)}$) ranged from approximately 530 psi and 1,860 psi, with an average value of 1,460 psi.

2.5 Groundwater

Groundwater measurements were made between December 2018 and November 2019 at locations where piezometers were installed. Results of the groundwater measurements and details of piezometer construction are provided in Table 3.4. Groundwater measurements will be continued through the upcoming spring, summer, fall, and winter seasons.

Table 2-1. WTP_1.0 Groundwater Measurement Summary

Boring ID WTP_1.0-	Depth to Groundwater(feet)									Groundwater Elevation Range (feet)
	12/27/2018	12/28/2018	01/03/2019	02/07/2019	02/08/2019	02/14/2019	05/6/2019	07/01/2019	11/05/2019	
B-01	-	2.0	1.7	3.0	-	0[1]	4.5	7.1	8.3	227.7 – 236
B-03					6.7	1.8	9.9	12.7	15.2	228.8 – 242.2
B-08						14.4	17.1	See Note 2		<256 – 261.6
B-10	33.0	-	34.0	35.7	-	34.9	35.5	36.1	37.6	227.4 – 232
B-11						9.7	13.9	16.2	25.8	237.2 – 253.3

Notes:
1. The monument was underwater, we assumed groundwater at the ground surface.
2. Groundwater was below piezometer screen interval.

Evaluation of nearby water well logs indicate static groundwater level is approximately 100 feet below the ground surface. In our opinion, the measured groundwater elevations in piezometers represent perched groundwater conditions, and likely coincide with groundwater levels in the wetlands.

Groundwater levels can vary with precipitation, the time of year, and/or other factors. Generally, 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.

3.0 Seismic and Geologic Hazards Evaluation

3.1 General

Seismic and geologic hazard analyses are summarized in this section. Detailed discussion of local faults and seismic sources are presented in the technical memorandum for WTP_1.0 Fault Location Study (McMillen Jacobs, 2019a). The seismic hazards evaluation has been performed in general accordance with the International Building Code 2018 and American Society for Civil Engineers' (ASCE) Minimum Design Loads for Buildings and Other Structures, 2016 Edition (ASCE 7-16). Design ground motions presented herein are based on the system-wide Probabilistic Seismic Hazard Analysis (PSHA) performed for the WWSP (Shannon & Wilson, 2017). Project design criteria (WWSP, 2017) requires that the new pipeline be designed for the 2,475-year period event.

3.2 Regional Seismicity

The Pacific Northwest is a seismically active region that has three principle seismic sources: (1) the Cascadia Subduction Zone (CSZ) megathrust, which represents the interface between the subducting Juan de Fuca plate and the overriding North American plate; (2) deep 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). Faulting and seismicity associated with Cascade volcanoes are also potential sources of seismicity, though they generally do not impact sites in the Willamette Valley. Seismic sources are further described in the Fault Location Study (McMillen Jacobs, 2019a). These sources are all considered in the system-wide PSHA (Shannon & Wilson, 2017).

3.3 Site Classification

The project site was assigned a seismic site class following code-based procedures in ASCE 7-16, Chapter 20. Site class is used to categorize common subsurface conditions into broad classes to which ground motion attenuation and amplification effects are assigned. Site class accounts for the conditions encountered at the upper 100 feet of subsurface profile. Shallow bedrock was encountered during the subsurface investigation and most of the structures are anticipated to be supported on the bedrock. Therefore, a Site Class B is appropriate for the design purposes.

3.4 Design Ground Motions

A system-wide PSHA has been performed for the WWSP (Shannon and Wilson, 2017). The PSHA provides the 2,475-year return period uniform hazard spectral values for site classes BC, C, CD, D, and E assuming 5-percent damping ratio. For seismic hazard analyses of ground deformation and liquefaction potentials, the peak ground acceleration (PGA) is the most critical property of the spectrum. Spectral accelerations for Site Class B were not provided in PSHA, therefore Site Class BC was used for seismic evaluation of the project.

Table 3-1. WTP_1.0 Seismic Design Parameters

Parameter	Value
Soil Profile Class	BC
Peak Bedrock Acceleration (g)	0.463
SA Peak Ground Acceleration (g)	0.463
SA 0.2-sec Period Spectral Acceleration (S_{MS})	1.062
SA 1.0-sec Period Spectral Acceleration (S_{M1})	0.400
SA 2.0-sec Period Spectral Acceleration	0.175
SA Peak Ground Velocity (PGV) (cm/sec)	40

Note: All spectral accelerations are adjusted for site class.

3.5 Seismic Sources and Hazards Deaggregation

The PSHA produces a mean source event that generates the spectral accelerations included in Table 3-1. The mean magnitude ranges from 7.8 to 8.7 and the mean site-to-source distance ranges from 51 to 86 km.

The deaggregation data identify the earthquake sources, magnitudes, and distances that contribute to the ground motion hazard for a particular return interval and spectral period. The deaggregation results indicate that multiple earthquake sources are significant contributors to the ground motion hazards at the project site. The seismic sources and the percentage of relative contribution of the three primary sources are:

- Large megathrust events (between magnitudes 8.6 and 9.4) at distances of 50 to 150 km – 60-65% contribution to hazard;
- Shallow crustal events (up to magnitude 7.5) at distances of less than 25 km – 30-35% contribution to hazard; and
- Deep intraslab CSZ events (up to magnitude 7.5) at distances of about 45 to 100 km - <5% contribution to hazard.

3.6 Liquefaction

Liquefaction is a phenomenon affecting saturated, loose sandy and low-plastic silty soils in which cyclic rapid shearing from an earthquake results in a loss of shear strength and a transformation from a solid mass to a viscous, heavy fluid mass. The results of soil liquefaction potentially include loss of shear strength, loss of soil materials through sand boils, flotation of buried chambers/pipes, and post liquefaction settlement.

The site is underlain by shallow bedrock basalt, which is not susceptible to liquefaction. Therefore, liquefaction is not considered a hazard at the site.

3.7 Slope Stability

Areas surrounding the site are relatively flat to gently sloping. The site is underlain by shallow bedrock and the majority of structures are founded on rock. Slope stability is not considered a hazard.

3.8 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 free face. Liquefaction is not anticipated at the site. Therefore, lateral spreading is not considered a hazard.

3.9 Fault Rupture

There are no known active faults that cross the site. The nearest faults to the site are Sherwood-Lake Oswego fault located approximately 2 kilometers north of the site and Canby-Mollala fault located approximately 8 kilometers east of the site. Therefore, the risk of fault rupture across the WTP_1.0 is negligible.

3.10 Buoyancy and Flotation

When pipes or hollow structures are installed under the groundwater table it is possible that they can float, if the upward buoyant forces on the pipe exceed the downward gravitational forces from the soil cover or weight of the structures. Buoyancy and flotation of hollow deep structure and mitigation options are discussed in the GER (McMillen Jacobs, 2020). For pipes which their cover depth is greater than O.D., factor of safety against flotation is greater than 1.5, meeting the requirements of Seismic Guidelines and Minimum Design Requirements (WWSP, 2018).

3.11 Abrupt Settlement

Abrupt settlement generally occurs due to liquefaction or where structures (i.e. buildings and pipelines) founded on the transition between soil and rock. Liquefaction is not anticipated at the site and most of the structures will be founded on the bedrock. Therefore, abrupt settlement is not considered a hazard.

3.12 Other Hazards

No significant geologic hazards such as landslides, slope instabilities, tsunamis, seiches, debris flows, or collapsible soils were identified within the proposed WTP_1.0 area

4.0 Flooding Hazard

4.1 General

There are two flood hazard sources for the WTP_1.0 site: (1) precipitation events causing flooding along the Willamette River and its tributaries; and (2) a breach of the Scoggins Dam, which is located approximately 20 miles northwest of the project alignment and impounds Henry Hagg Lake. These two sources were considered in the evaluation of the flooding hazard at the WTP_1.0 site.

4.2 Precipitation-Induced Flooding

The Federal Emergency Management Agency (FEMA) has published maps with estimated flood inundation limits in the project area for 100-year and 500-year floods. These flood maps were reviewed to evaluate the precipitation induced flooding hazard along the WTP_1.0 site. Figure 3 shows the published flood inundation limits for the 100-year and 500-year floods. The maps indicate that the flood water surface elevation is between 140 and 150 feet. The maps also indicate the WTP_1.0 site is outside of the range of the 100-year and 500-year floods. Therefore, the risk of flooding resulted from precipitation is negligible and is not considered a hazard.

4.3 Scoggins Dam Breach Inundation

A breach of Scoggins Dam would result in a large outflow of water and flooding along the Tualatin River due to the rapid draining of Henry Hagg Lake. The WTP_1.0 site is topographically isolated from the Tualatin River basin and therefore the risk of flooding due to the Scoggins Dam breach is negligible and is not considered a hazard.

5.0 Conclusions

Based on the results of our geologic hazards assessments and analyses, we make the following conclusions with respect to the WWSP WTP_1.0 project:

- The risk of liquefaction and lateral spreading is negligible.
- The risk of slope instability is negligible.
- No known active fault crosses the WTP_1.0 site.
- Considering WTP_1.0 includes hollow structures below groundwater level, flotation may be an issue. Discussions about flotation and potential mitigation options are provided in the project GER (McMillen Jacobs, 2020).
- The risk of abrupt settlement is negligible.
- No geologic hazards such as landslide, debris flows, tsunami, and seiche were identified.
- The risk of flooding negligible.

6.0 Closure

This Geologic Hazards Report has been prepared for the Willamette Water Supply Program WTP_1.0 Project located in Washington County, 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, as well as 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 advised at once, 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.

MCMILLEN JACOBS ASSOCIATES

REGISTERED PROFESSIONAL ENGINEER
86968
OREGON
Mar. 12, 2013
FARID SARIOSSEIRI
EXPIRES: 6/30/2020

Farid Sariosseiri

Farid Sariosseiri, PE
Associate

Jeremy Fissel, PE
Project Engineer

7.0 References

American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads for Buildings and other Structures. ASCE/SEI Standard 7-16.

McMillen Jacobs Associates, 2019a, Willamette Water Supply Program, WTP_1.0 – Fault Location Study Memorandum, December 2019.

McMillen Jacobs Associates, 2019b, Willamette Water Supply Program, WTP_1.0 – Geotechnical Data Report, December 2019.

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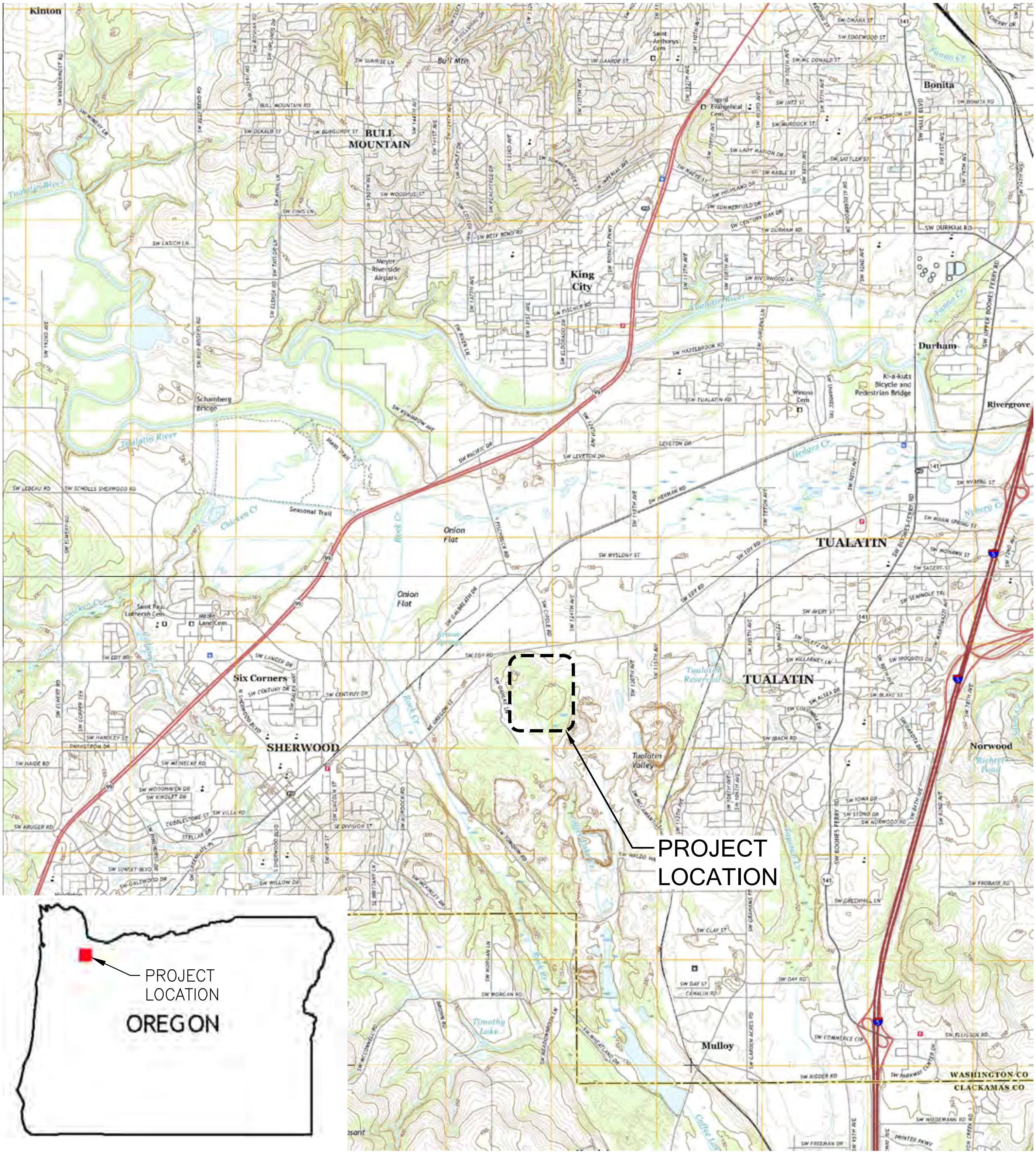
Figures

FOR LAND USE PERMITTING (EXHIBIT B)



PROJECT VICINITY MAP

SCALE: NTS

N

McMILLEN JACOBS ASSOCIATES

WILLAMETTE WATER SUPPLY PROGRAM

WTP_1.0

GEOTECHNICAL HAZARDS REPORT
PROJECT VICINITY MAP

FIG.1

MAY 2020

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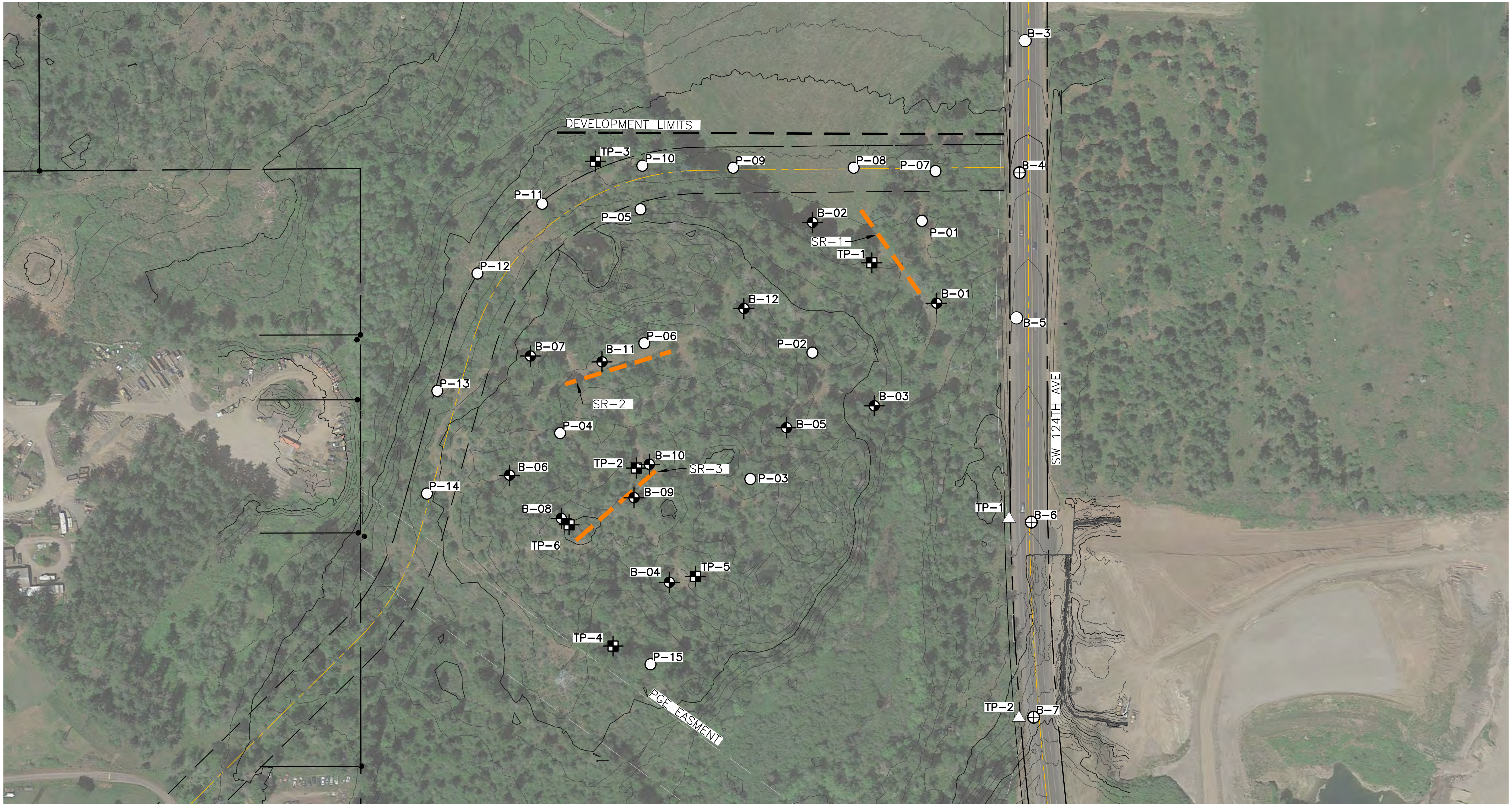

LEGEND:

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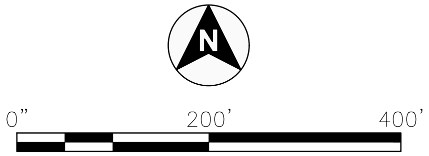
WTP_1.0-B-1	⊕	BOREHOLE LOCATION
WTP_1.0-TP-1	⊞	TEST PIT LOCATION
B-1	⊕	BOREHOLE COMPLETED AS PART OF PLM_3.0
TP-1	△	TEST PIT COMPLETED AS PART OF PLM_3.0
SR-1	▬ ▬ ▬	GEOPHYSICAL EXPLORATIONS

NOTES:

1. BASE MAP PROVIDED BY CDM SMITH IN JAN 2019.
2. EXPLORATION LOCATIONS ARE APPROXIMATE.



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WILLAMETTE WATER SUPPLY PROGRAM

WTP_1.0

GEOTECHNICAL HAZARDS REPORT
EXPLORATION PLAN

FIG.2

MAY 2020

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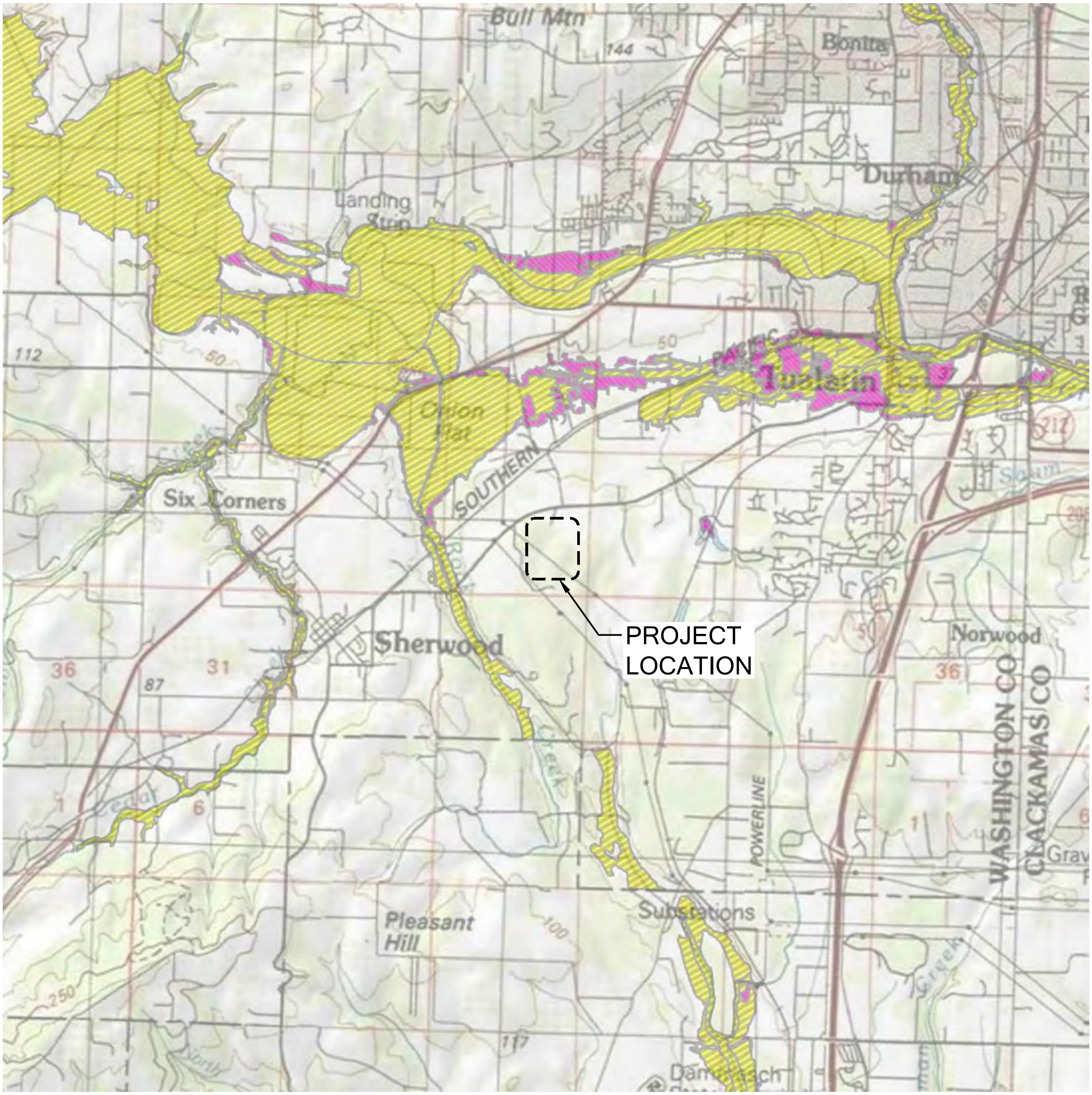
FOR LAND USE PERMITTING (EXHIBIT B)



LEGEND:

🟨 100 YEAR FLOOD

🟪 500 YEAR FLOOD

FLOOD MAP

SCALE: NTS

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WILLAMETTE WATER SUPPLY PROGRAM

WTP_1.0

GEOTECHNICAL HAZARDS REPORT
FLOOD MAP

FIG.3

MAY 2020

Path: C:\Users\cburke\Box\Jobs\5887.0 WWSP WTP 1.0\CADD_GHRY_Fig.3.dwg Plot date: Dec 10, 2019, CAD User: cburke

Appendix H

Site-Specific Seismic Response Spectrum for WTP_1.0

FOR LAND USE PERMITTING (EXHIBIT B)

Technical Memorandum

To:	Greg Lindstadt, PE, CDM Smith Mike Hyland, CDM Smith	Project:	WWSP WTP_1.0
From:	Wolfe Lang, PE, GE	cc:	File
Prepared by:	Farid Sariosseiri, PE Todd LaVielle	Job No.:	5887.0
Date:	May 6, 2020		
Subject:	Site-Specific Seismic Response Spectrum		

Revision Log

Revision No.	Date	Revision Description
Final Submittal	May 6, 2020	Incorporated CDM Smith’s comments.

1.0 Introduction

The Willamette Water Supply Program (WWSP) is a reginal drinking water infrastructure project being implemented by the Tualatin Valley Water District (TVWD) and the Cities of Hillsboro and Beaverton to provide a seismically resilient water supply for their service areas. The WWSP includes more than 30 miles of transmission pipelines, ranging from 24 to 66 inches in diameter, extending from the Willamette River Water Treatment Plant, in Wilsonville, to the TVWD and Hillsboro service areas in Washington County, which include the cities of Hillsboro and Beaverton. The WWSP also includes two 15-million-gallon water storage tanks, a new water treatment plant, and an expansion of the existing Willamette River Water Treatment Plant. The new system elements are being designed to meet future demand and to provide redundancy in the event of an emergency.

The WTP_1.0 is a new water treatment plant with an initial treated water design capacity of 60 million gallons per day (mgd) and a future build-out treated water design capacity of 120 mgd. The project includes construction of several access roads within the treatment plan and construction of a portion of SW Blake Street that connects the SW 124th Avenue to the plant’s western property line. The project involves mitigation and/or protection for the existing natural wetland areas.

The project is located west of SW 124th Avenue, approximately 2,000 feet south of the intersection of SW 124th Avenue and SW Tualatin-Sherwood Road in Sherwood, Oregon. The site is currently undeveloped. The existing ground surface elevation is near 260 feet and descends in all directions away from the site within the proposed development. Most of the WTP_1.0 site and immediately adjacent areas are wooded with thick underbrush, including large thick patches of Himalayan blackberry and pervasive poison oak

This memorandum presents the horizontal and vertical site-specific seismic response spectra for design.

2.0 Geotechnical Data and Seismic Site Classification

Subsurface data from the site investigation of WTP_1.0 project and other projects in the immediate vicinity of the pipeline were reviewed for site characterization. These data are presented in the Geotechnical Data Report for WTP_1.0 (McMillen Jacobs Associates, 2019)

The site classification was developed following the procedures outlined in ASCE 7-16, Section 20 (2016). Site classification is used to categorize common subsurface conditions into broad classes to which ground motion attenuation and amplification effects are assigned. Site classification is based on the weighted average of the shear wave velocity in the upper 100 feet of subsurface profile. Shallow bedrock was encountered during the subsurface investigation. Most of the structures are anticipated to be supported on the bedrock. Therefore, Site Class B is appropriate for the design.

3.0 Spectral Acceleration

To provide an update to the USGS 2014 seismic hazard maps, a program-wide probabilistic seismic hazard analysis (PSHA) report was prepared (Shannon & Wilson, 2017). The study found that the ground motion hazard for the project area was adequately represented by PSHAs completed at two locations: Site A and Site B. The results from Site A should be applied to the portion of the WWSP alignment north of latitude 45.44° N, and the results from Site B should be applied south of latitude 45.44° N. The PSHA report provides a horizontal, 2,475-year return period, mean uniform hazard acceleration spectra for site classes BC, C, CD, D, and E assuming 5 percent damping ratio, and represent the risk-targeted maximum considered earthquake (MCE_R) which is typically used for structure design, and some geotechnical seismic evaluations.

Because the WTP_1.0 site is located south of latitude 45.44° N the Site B spectrum should be used (Shannon & Wilson, 2017). The PSHA did not include a spectrum for Site Class B, therefore an acceleration spectrum for Site Class BC should be used. Table 1 and Figure 1 summarize the acceleration spectrum.

The PSHA was performed for horizontal ground motions only. A vertical response spectrum was developed based on the horizontal PSHA, following the code-based procedures outlined in ASCE 7-16, Section 11.9. The vertical acceleration response spectrum for Site Class B/C are presented in Table 2 and shown on Figure 1. ASCE 7-16 requires spectral acceleration values for periods beyond 2 seconds be developed by performing site-specific procedures. Because it is unlikely that any of the proposed structures have a period greater than 2 seconds, a site-specific vertical, analysis was deemed unnecessary and was not performed. The provided vertical acceleration spectrum represent the risk-targeted maximum considered earthquake (MCE_R).

Table 1. Horizontal, 2,475-Year Return Period, Mean Uniform Hazard Spectra for Site Class B/C (5 percent damping ratio)[1]

Period[2] (seconds)	Spectral Acceleration (g)
0.01	0.463
0.02	0.487
0.03	0.530
0.05	0.620
0.075	0.809
0.10	0.965
0.15	1.088
0.20	1.062
0.30	0.928
0.50	0.703
0.75	0.505
1.0	0.400
1.5	0.257
2.0	0.175
3.0	0.093
5.0	0.0391
7.5	0.0174
10	0.0098

Notes:
1. From Shannon & Wilson PSHA Report (2017)
2. Spectral acceleration at 0.01 second may be used for peak ground acceleration.

Table 2. Vertical, 2,475-Year Return Period, Mean Uniform Hazard Spectra for Site Class B/C (5 percent damping ratio)[1]

Period (seconds)	Spectral Acceleration (g)
0.01	0.29
0.02	0.29
0.02	0.29
0.05	0.76
0.10	0.76
0.15	0.76
0.20	0.62
0.25	0.52
0.50	0.31
0.75	0.23
1.00	0.18
1.25	0.16
1.50	0.14
1.75	0.12
2.00	0.11

Note:
Spectral acceleration at 0.01 second may be used for peak ground acceleration.

4.0 References

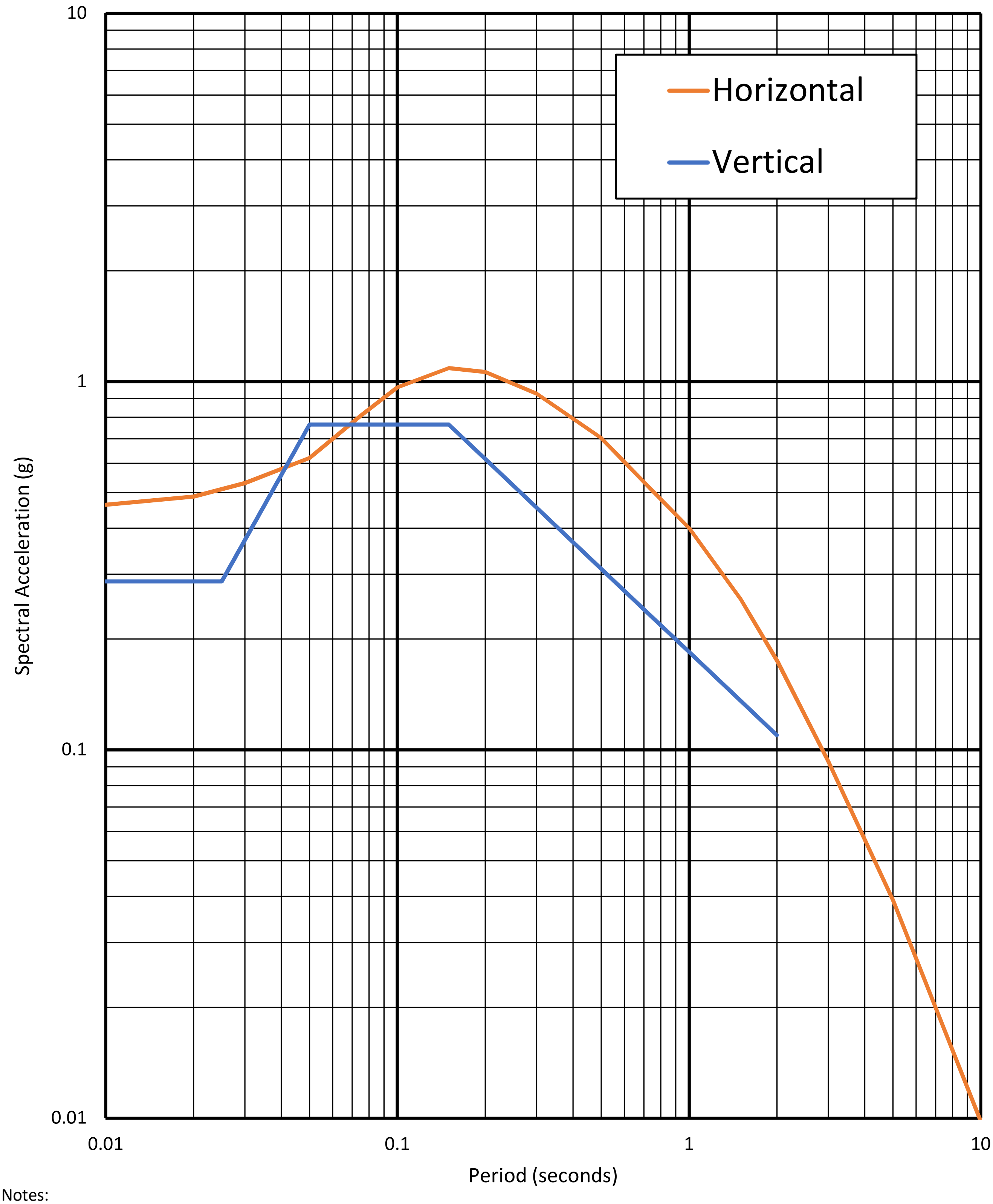
American Society of Civil Engineers (ASCE), 2016. Minimum Design Loads for Buildings and Other Structures. ASCE/SEI Standard 7-16.

American Society of Civil Engineers (ASCE), 2006. Seismic Rehabilitation of Existing Buildings. ASCE/SEI Standard 41-06

McMillen Jacobs Associates, 2019. Willamette Water Supply Program, WTP_1.0 – Geotechnical Data Report, May 2019.

Shannon & Wilson, Inc., 2017. Geotechnical Report – Probabilistic Seismic Hazard Analysis, Willamette Water Supply Program, Clackamas and Washington County, Oregon, August 2017

FOR LAND USE PERMITTING (EXHIBIT B)



Notes:

1. Horizontal acceleration spectrum is from the program wide PSHA (Shannon & Wilson, 2017)
2. The vertical response spectrum was developed using procedures defined in ASCE 7-16 Section 11.9.

McMILLEN JACOBS ASSOCIATES

WILAMETTE WATER SUPPLY PROGRAM

SEGMENT WTP_1.0

SITE-SPECIFIC SEISMIC REPONSE SPECTRA

FIG. 1

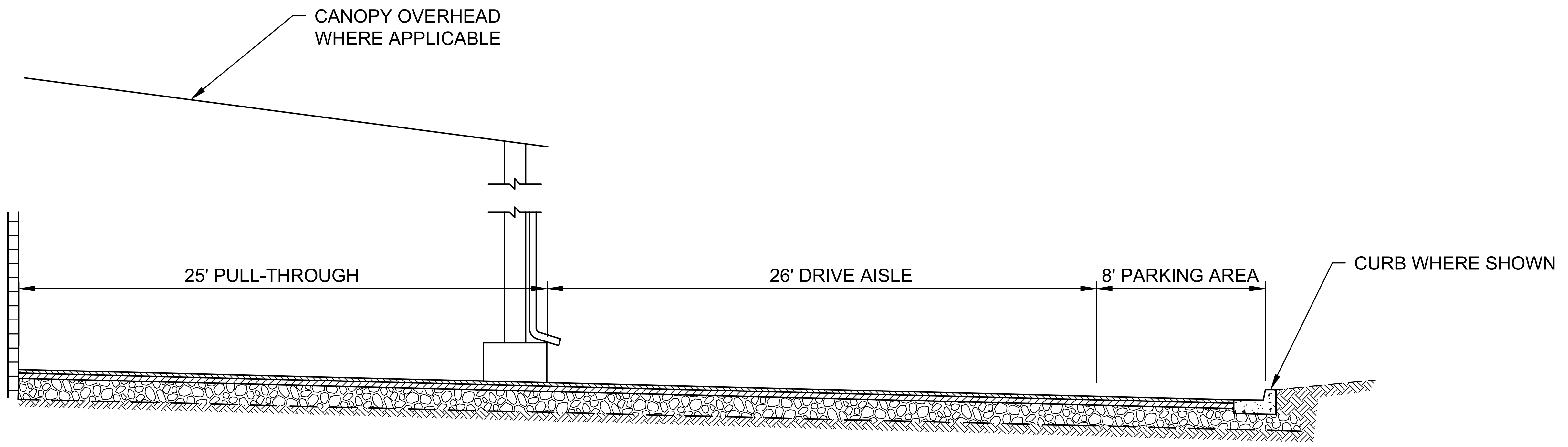
MAY 2020

file:C:\Users\lavielle\Box\Jobs\5887.0 WWSP WTP 1.0\Project Work\07 - Deliverables\Site-Specific Response Spectrum\Final\[Response Spectrum.xlsx]Figure

Appendix I

SW Blake Street 60-Percent Design Drawing

FOR LAND USE PERMITTING (EXHIBIT B)



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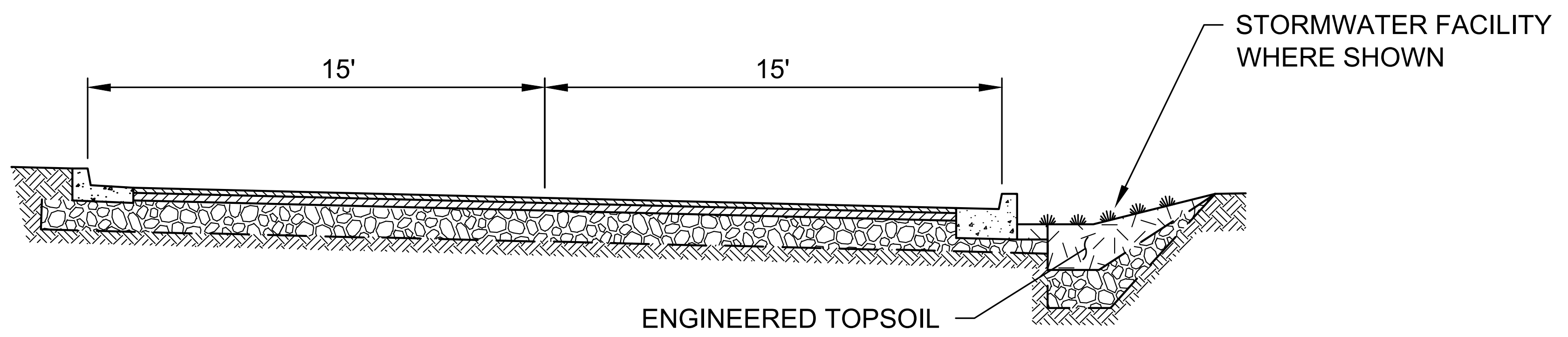

ROAD A, B, C & D

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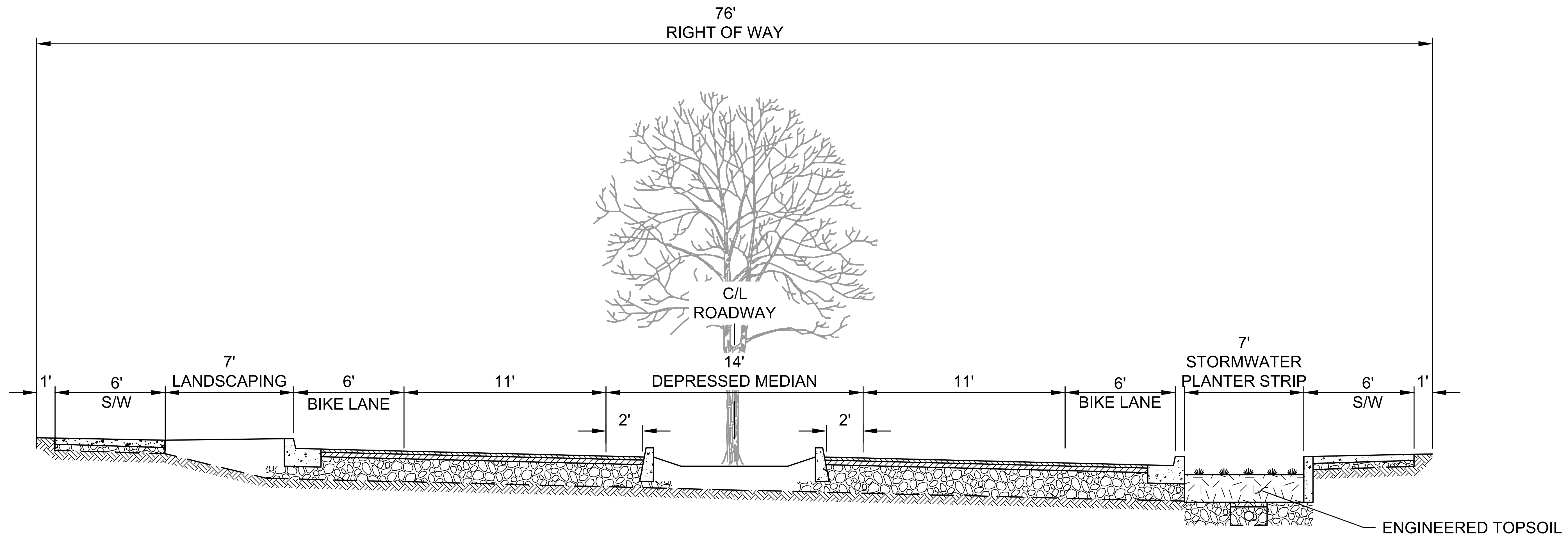

SW BLAKE STREET WITH DEPRESSED MEDIAN

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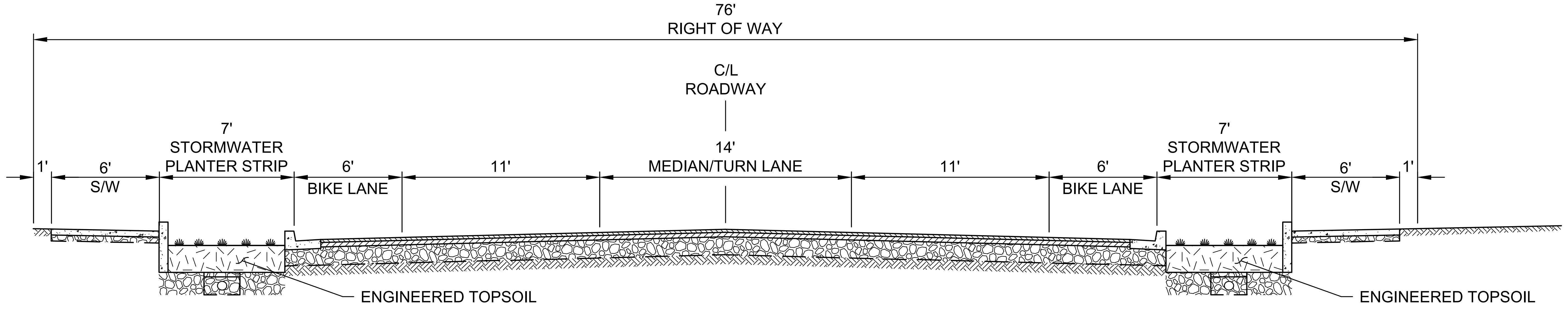

SW BLAKE STREET WITH MEDIAN/LEFT TURN LANE

SCALE: 1"=5'-0"

NOTES:

1. ONSITE STORMWATER FACILITIES, CURB AND GUTTER AND FENCING WHERE SHOWN ON PLANS.
2. CURB CUTS TO BE DETERMINED IN COORDINATION WITH STORMWATER DRAINAGE AND TREATMENT DESIGN.
3. SW BLAKE STREET DEPRESSED MEDIAN TO BE USED AS A STORMWATER PLANTER FROM STA 12+50± TO STA 19+50±.

30% DESIGN - NOT FOR CONSTRUCTION

BY: MATT ESTEP

PLOT DATE: Wednesday, May 25, 2016 1:37:44 PM

DWG FILE: C:\cdmext\matt.estep@murraysmith.us\d0274278\WTP1-03-GR-30000.dwg

DSGN	C JAIN
DR	M ESTEP
CHK	NA
APVD	NA

NO.	DATE	REVISION	BY	APVD

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SITE
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ROADWAY TYPICAL SECTIONS

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DATE	MARCH 2019
PROJ	WTP_1.0

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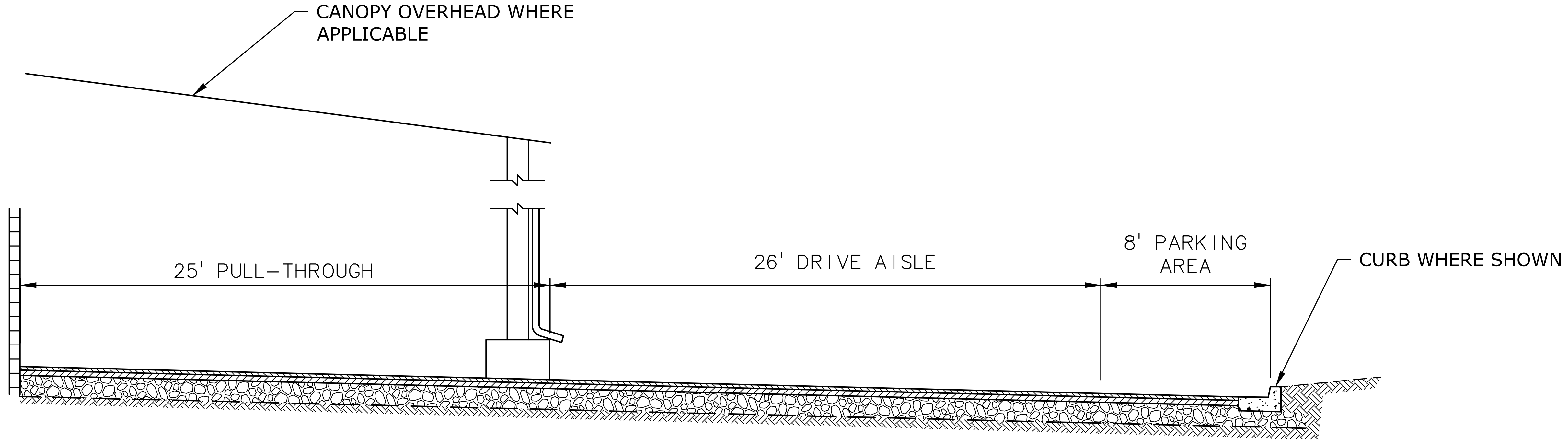
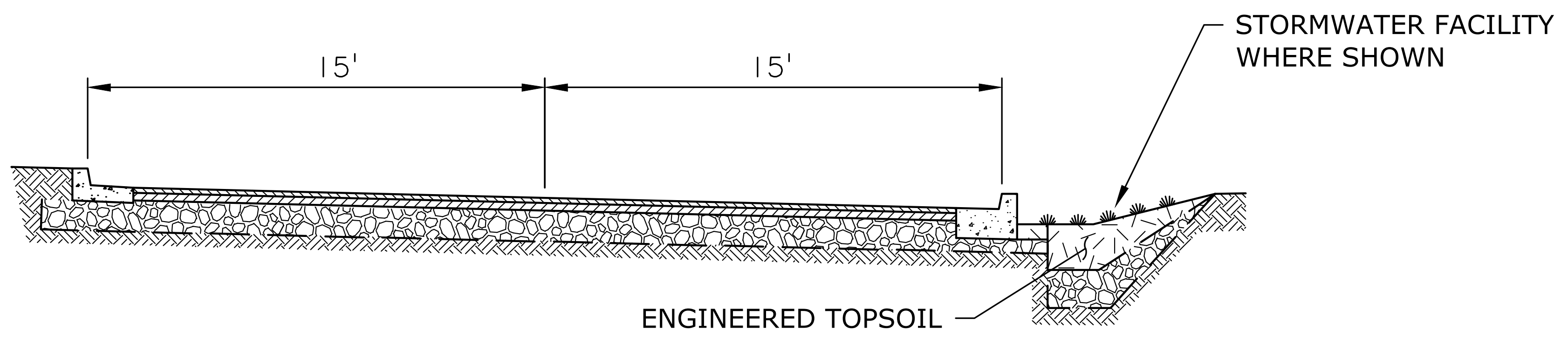


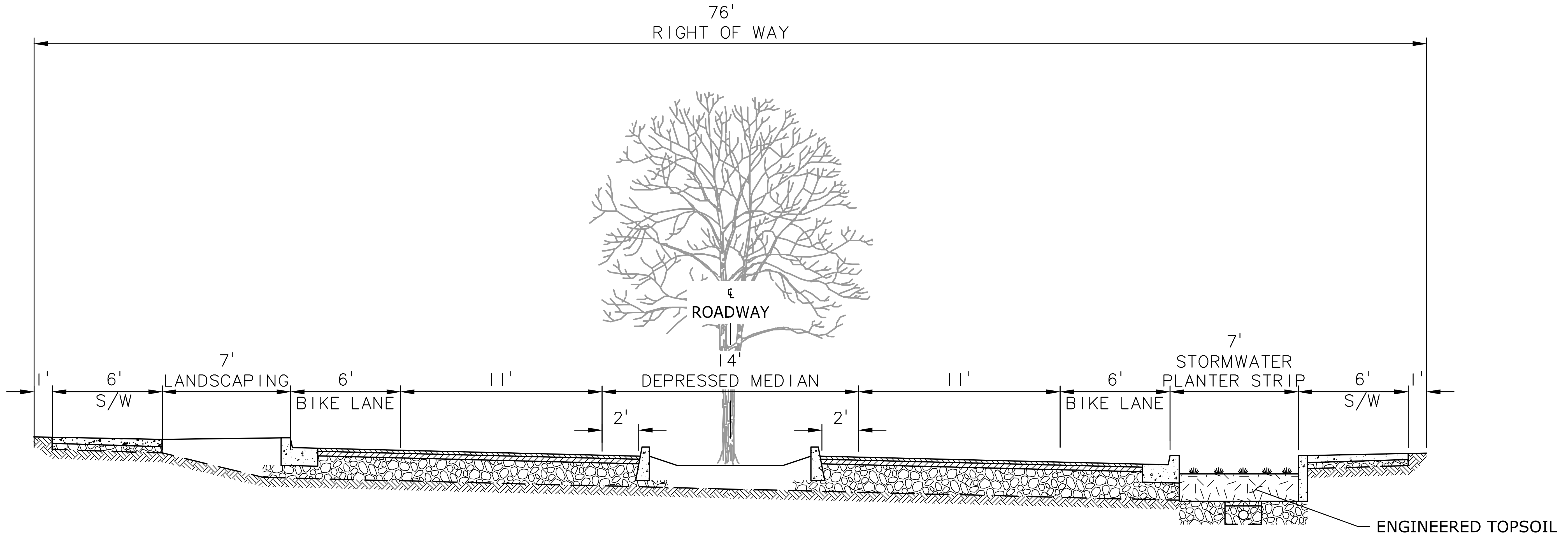
ROAD A AT KOLK POND

SCALE: 1"=5'-0"



ROAD A, B, C & D

SCALE: 1"=5'-0"

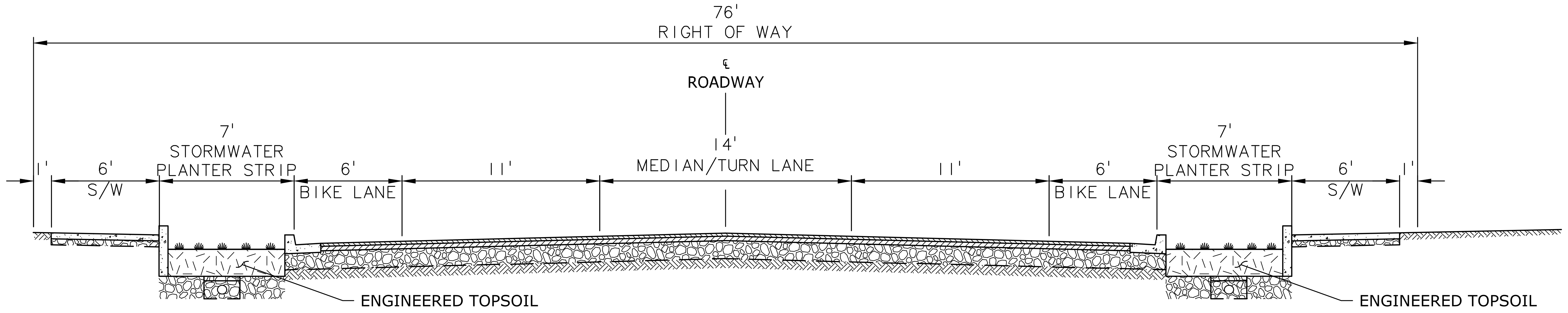


SW BLAKE STREET WITH DEPRESSED MEDIAN

SCALE: 1"=5'-0"

NOTES:

1. ONSITE STORMWATER FACILITIES, CURB AND GUTTER AND FENCING WHERE SHOWN ON PLANS.
2. CURB CUTS TO BE DETERMINED IN COORDINATION WITH STORMWATER DRAINAGE AND TREATMENT DESIGN.
3. SW BLAKE STREET DEPRESSED MEDIAN TO BE USED AS A STORMWATER PLANTER FROM STA 12+50± TO STA 19+50±.



SW BLAKE STREET WITH MEDIAN/LEFT TURN LANE

SCALE: 1"=5'-0"

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APVD NA

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DATE MARCH 2019
PROJ WTP_1.0

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BY: ERICA RODRIGUEZ
PLOT DATE: Wednesday, May 25, 2016 1:37:44 PM
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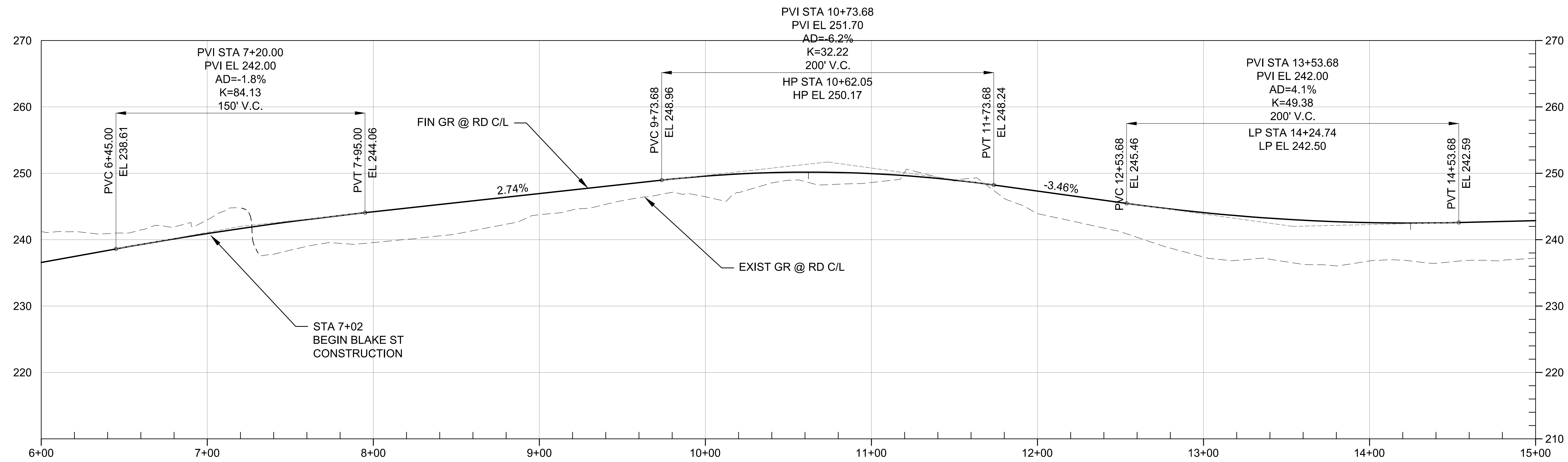

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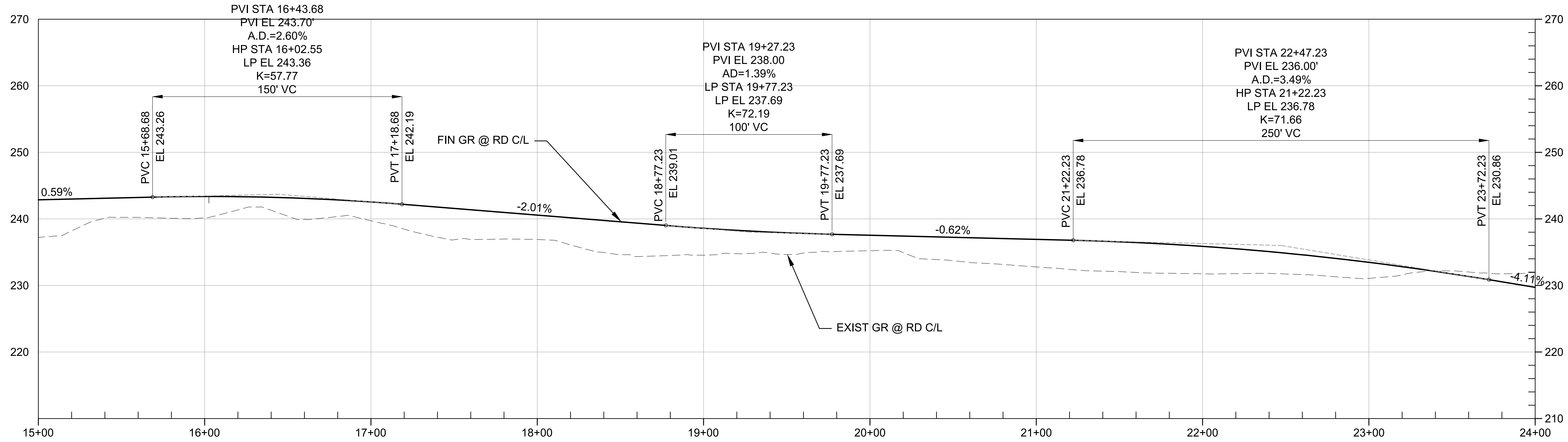

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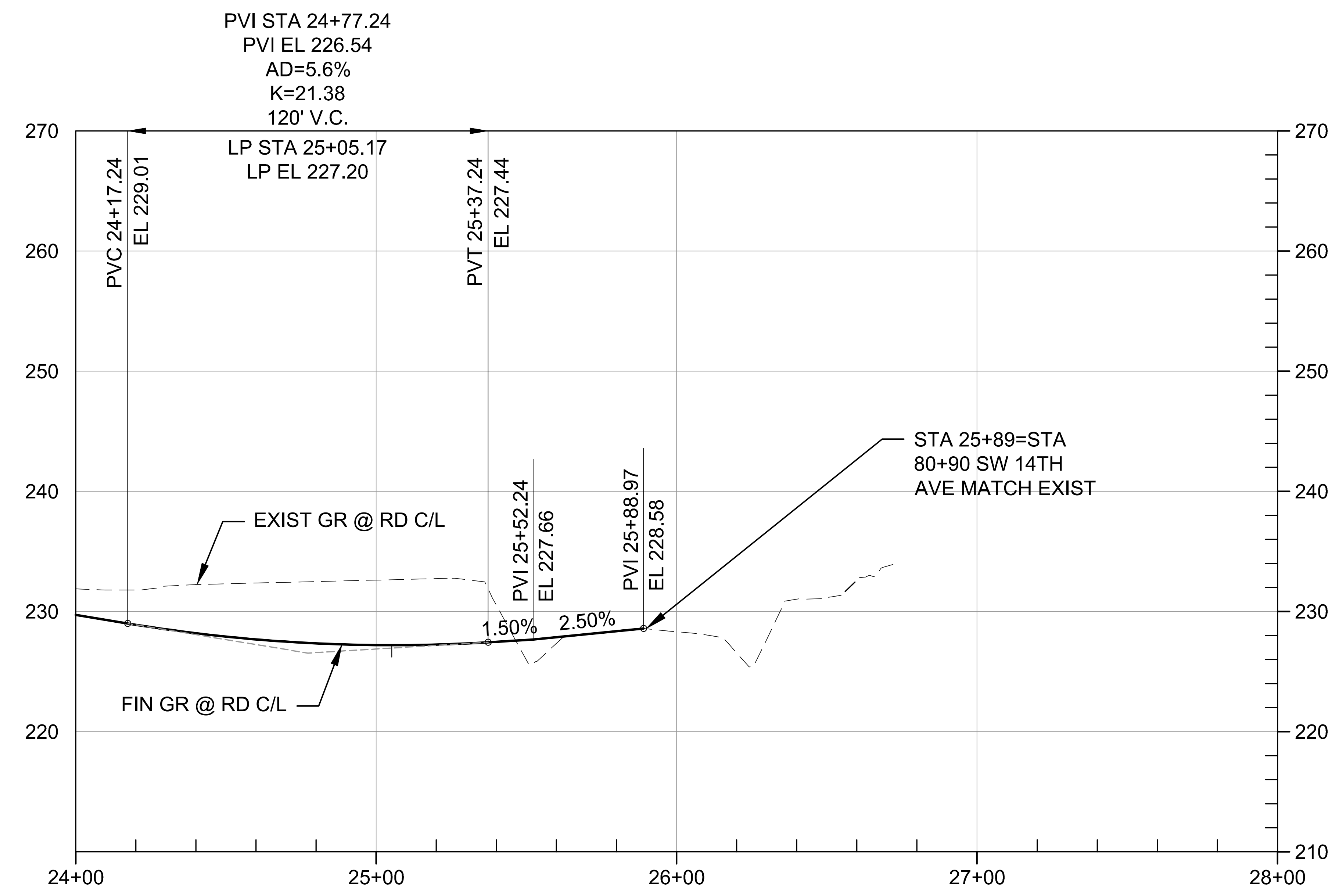

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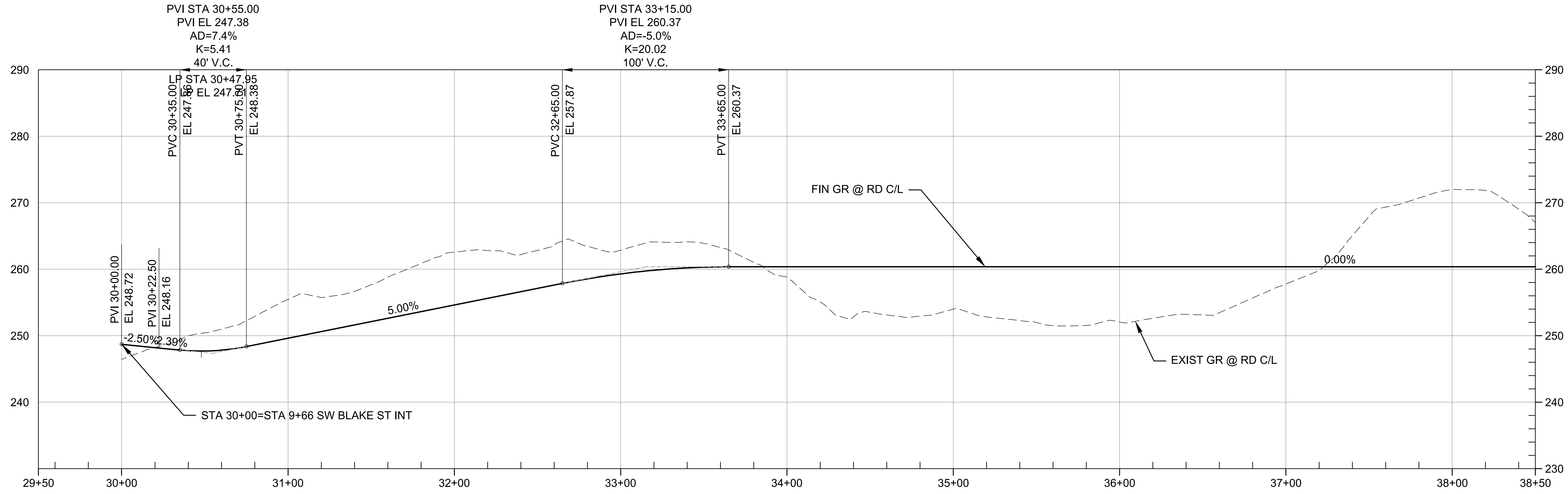

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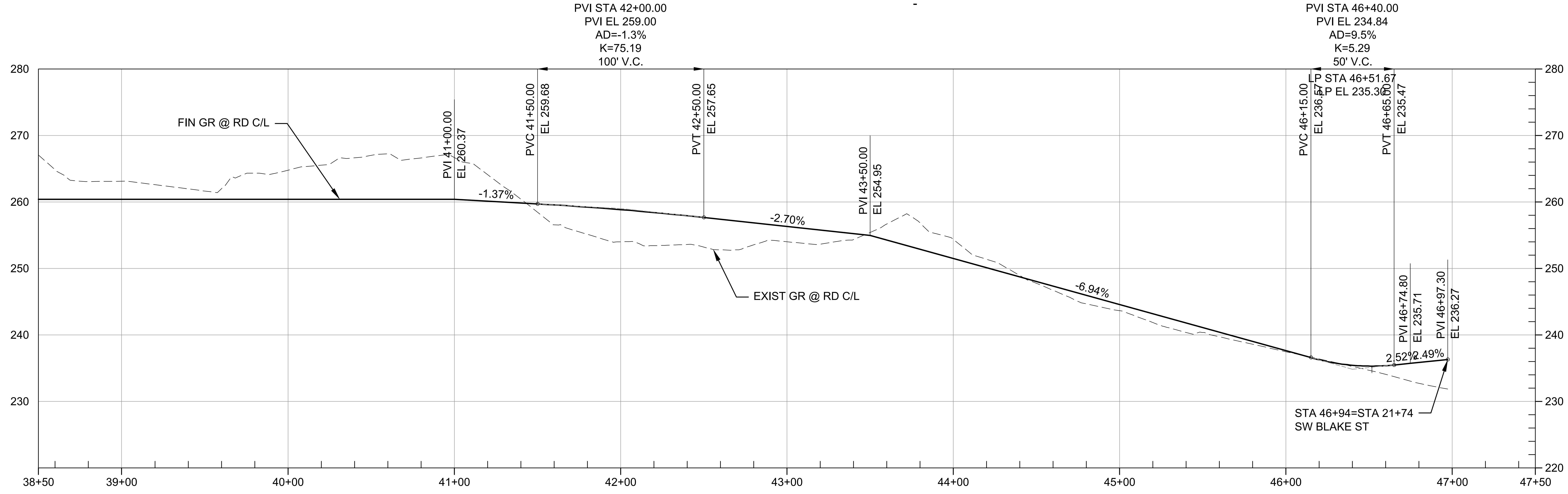

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CHK	NA						
APVD	NA	NO.	DATE	REVISION	BY	APVD	

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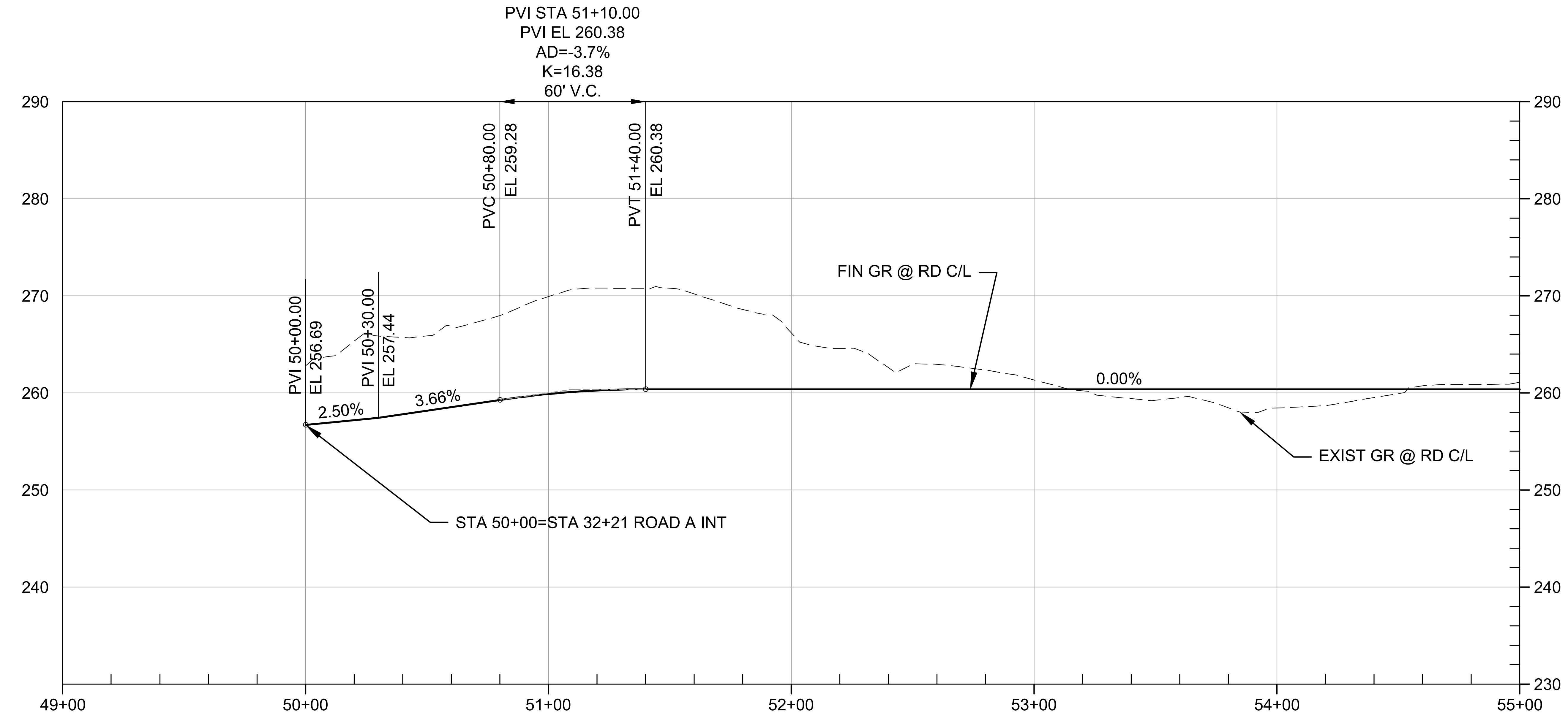

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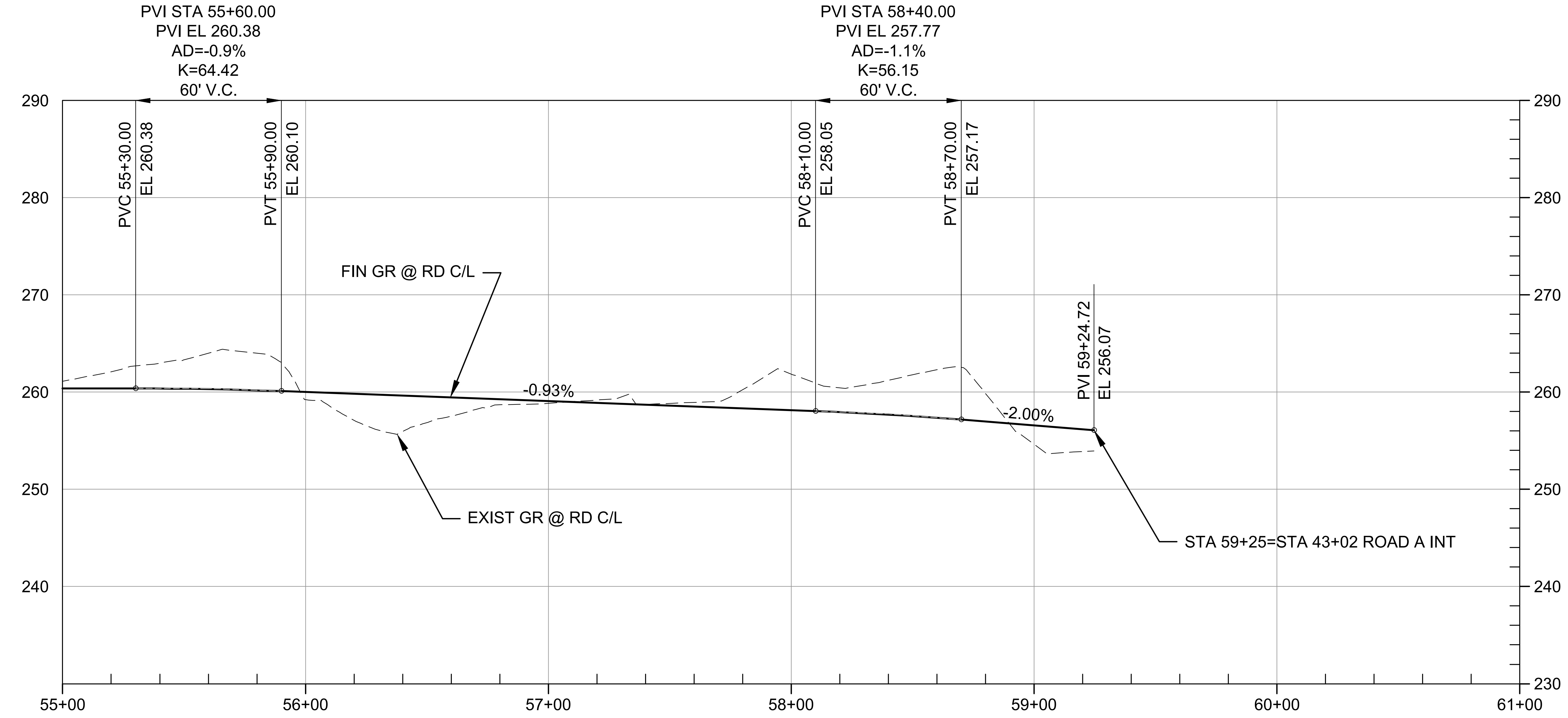

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BY: LANCE GUBERINA

PLOT DATE: Wednesday, May 25, 2016 1:37:44 PM

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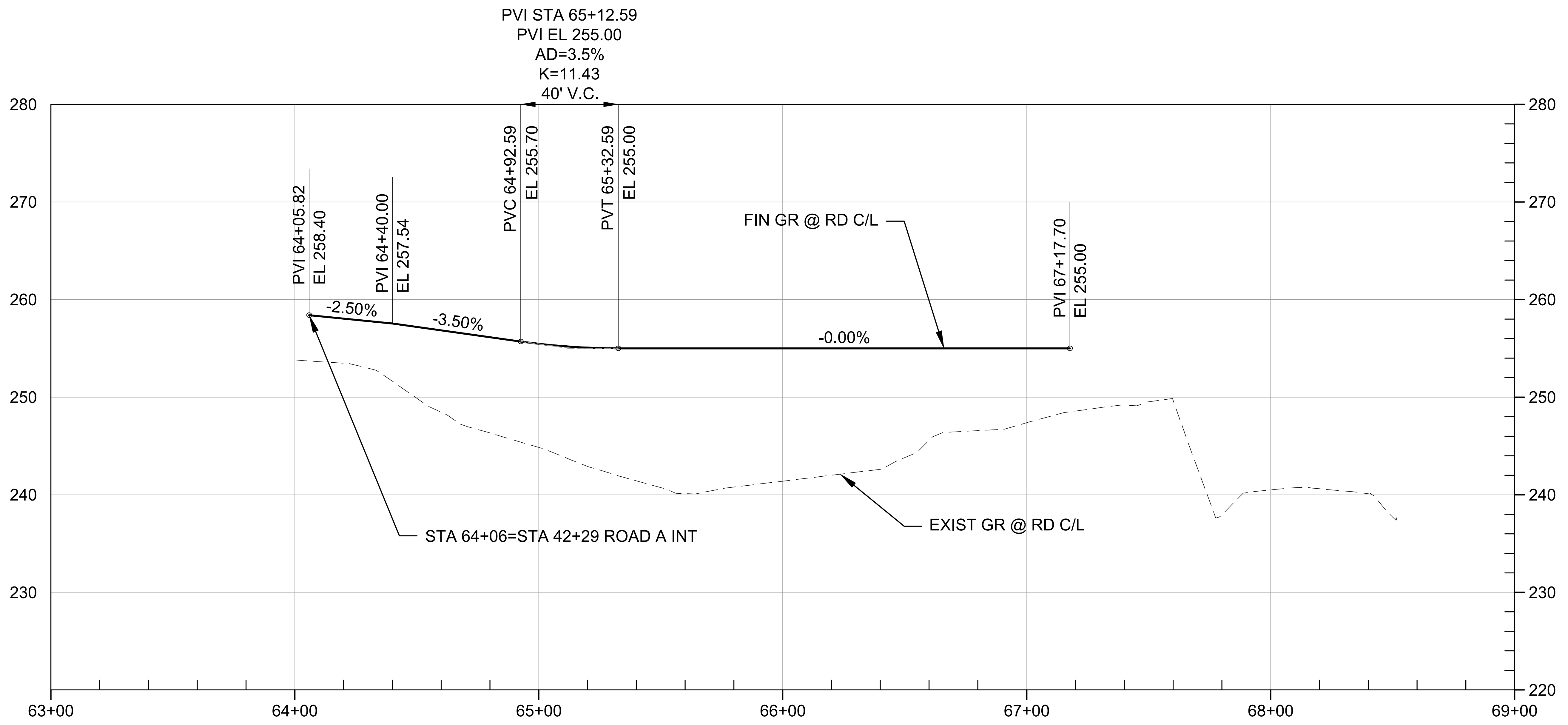

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SITE
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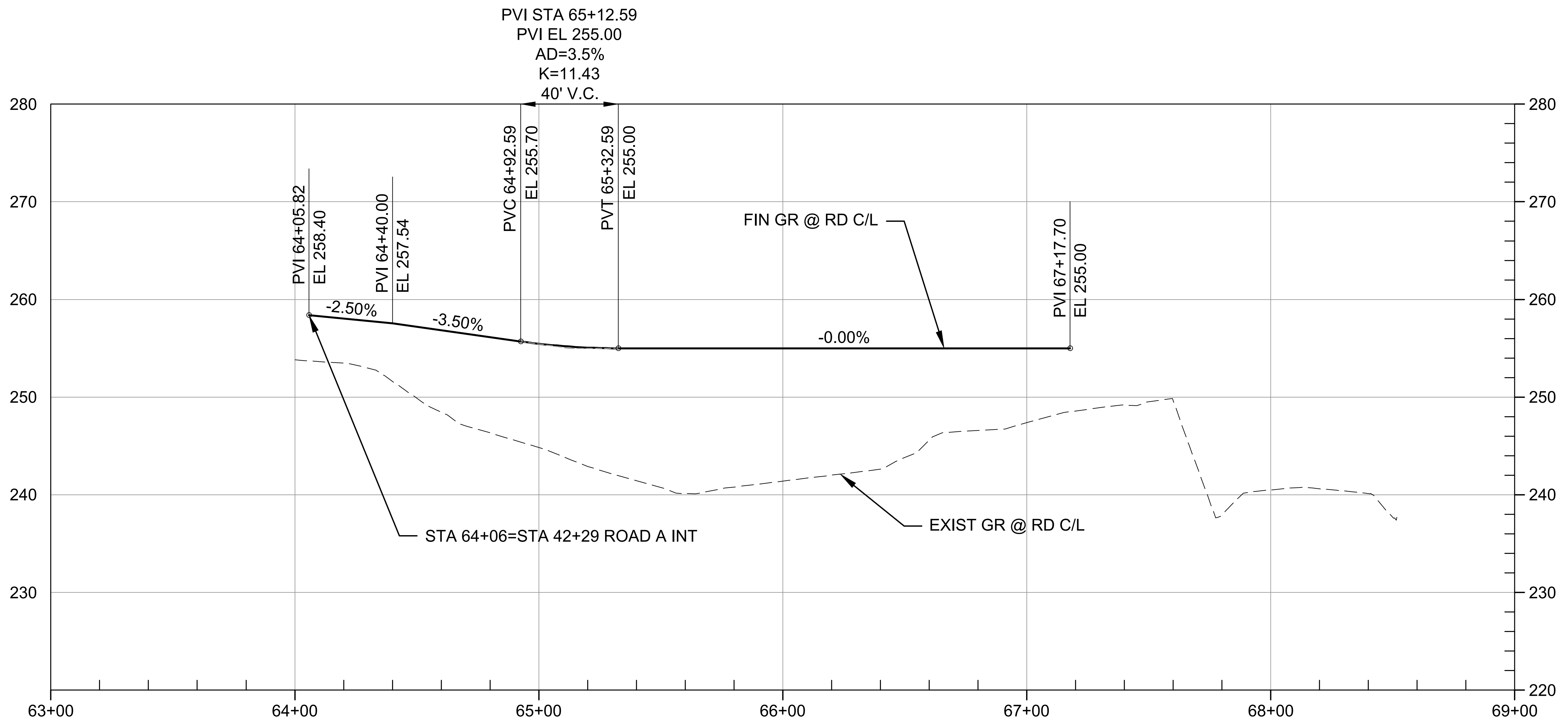

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