GEOTECHNICAL REPORT

Report of Geotechnical Engineering Services

Operations Center

West Linn, Oregon

April 3, 2025

Geotechnical ▪ Environmental ▪ Special Inspections

Columbia West Engineering, Inc

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April 3, 2025

City of West Linn
22500 Salamo Road
West Linn, OR 97068

Attn: Morgan Lovell

Re: Report of Geotechnical Engineering Services
Operations Center
Salamo Road and Greene Street
West Linn, Oregon
CWE Project: WestL-1-01-1

Columbia West Engineering, Inc. (Columbia West) is pleased to present this report of geotechnical engineering services for the proposed Operations Center located east of the Salamo Road and Greene Street intersection in West Linn, Oregon. Our services were conducted in accordance with the Geotechnical Consulting Services Agreement with the City of West Linn dated January 30, 2025.

We appreciate the opportunity to work on the project. Please contact us if you have any questions regarding this report.

Sincerely,

Jonathan A. Nasr, PE
Project Engineer

Najib A. Kalas, PE
Principal Engineer

cc: Erich Lais, City of West Linn
Jeff Dunn, Scott Edwards Architecture
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Trent Jorgensen, Scott Edwards Architecture

JAN:NAK:kat
Attachments
Document ID: WestL-1-01-1-040325-geor-DRAFT.docx

EXECUTIVE SUMMARY

This executive summary presents the primary geotechnical considerations associated with the proposed City of West Linn's Operations Center located in West Linn, Oregon. Our conclusions and recommendations are based on the subsurface information presented in the report and proposed development information provided by the design team. Detailed discussion of the geotechnical considerations summarized here is presented in respective sections of the report.

- Based on our understanding of the proposed development and the results of our explorations and engineering analyses, it is our opinion that the proposed development can be constructed at the site.

- Undocumented fill was encountered to depths between 1 foot and 31 feet BGS in 15 of our 18 explorations and to depths between 10.5 and 49.5 feet BGS in prior explorations by others. The composition and consistency of the fill are highly variable and generally include medium stiff to hard clay with varying proportions of sand and gravel or very loose to very dense gravel and sand with varying proportions of clay and silt. The fill also includes cobbles and boulders. Bark chip fill was encountered in seven of our explorations to depths between 6 and 24 inches. To prevent excessive differential settlement, foundations and floor slabs should not bear on the undocumented fill, unless it has been improved. This material should be evaluated during construction and removed from foundation and floor slab subgrade.

- Based on the assumed foundation loads, buildings in areas not underlain by undocumented fill or underlain by less than a few feet of undocumented fill can be supported on conventional spread footings, provided the undocumented fill is removed and replaced with structural fill and the buildings can safely withstand the predicted static foundation settlement.

- In lieu of removal of undocumented fill where the fill is more than a few feet thick, the buildings can be supported on deep foundation or ground improvement. Ground improvement is typically more economical than deep foundations. Therefore, this report focuses on ground improvement. If requested, our office can provide additional recommendations for deep foundations.

- The native, dense gravel, cobbles, and boulders encountered at shallow depths will result in difficult excavation with conventional equipment in both fill and decomposed/weathered basalt as well as the underlying intact basalt. In addition, because of the presence of cobbles and boulders in the near-surface fill and decomposed/weathered basalt, we expect difficult excavation at shallow depths. Utility trenches may result in slowed excavation and larger backfill volumes due to the presence of cobbles, boulders, and related caving.

- The project budget should include a contingency for removal of boulders.

- Static groundwater was measured at depths as shallow as 79.3 feet BGS in the VWPs installed at the site. Based on planned development grades, static groundwater is not a design consideration. Perched groundwater could be encountered at shallow depths following periods of heavy or prolonged rainfall.

- Based on soil and groundwater conditions at the site, liquefaction and lateral spreading are not design considerations at the site.

- Eleven of our 18 explorations encountered topsoil in the upper 6 to 12 inches, which includes an approximately 4- to 6-inch-thick root zone. In general, the topsoil is soft and contains organic material. This material exhibits low or highly variable strength and generally does not provide consistent subgrade support for foundations, floor slabs, or pavement. We recommend that the topsoil be removed, stabilized, or scarified and compacted as recommended for structural fill where it is present.

- Minor to severe caving was observed in 6 of our 13 test pits, starting at depths between 0 feet and 1 foot BGS. Caving was observed to occur within the fill.

- The on-site soil is suitable for use as structural fill, provided it is properly moisture conditioned and free of oversized materials. Moisture conditioning (drying) of the soil will likely be required depending on location and depth. It will be impossible to dry the soil during periods of extended wet weather.

- As an alternative to the use of thickened granular sections to support construction traffic, cement amendment can be used to obtain suitable support properties. However, depending on the grade and location, cobbles and boulders could be present in the subgrade. These oversized materials can damage cement tilling equipment when encountered and may require removal before cement amendment.

- Perimeter building foundation drains are recommended based on the preliminary grading plan and site surface and subsurface conditions. Columbia West should be contacted to review the final grading plans to verify our recommendations.

- The site is classified as Site Class C according to the 2022 SOSSC and ASCE 7-16 based on site-specific shear wave velocity measurements. Buildings founded directly on basalt can be designed using Site Class B.

- Due to the presence of steep slopes at the site and subsurface conditions, we do not recommend that stormwater drainage be infiltrated on the site. Stormwater should be routed to an appropriate stormwater system.

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ABBREVIATIONS AND ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ACP	asphalt concrete pavement
ADT	average daily traffic
ASCE	American Society of Civil Engineers
ASTM	ASTM International
BGS	below ground surface
CIP	cast-in-place
CLSM	controlled low strength material
CRBG	Columbia River Basalt Group
CSZ	Cascadia subduction zone
DEM	digital elevation model
DOGAMI	Oregon Department of Geology and Mineral Industries
ESAL	equivalent single-axle load
FHWA	Federal Highway Administration
fps	feet per second
g	gravitational acceleration (32.2 feet/second^2)
GEG	Geotechnical Engineering Group
GPS	global positioning system
H:V	horizontal to vertical
IBC	International Building Code
km	kilometer(s)
ksf	kips per square foot
LiDAR	light detection and ranging
MCE	maximum considered earthquake
mm	millimeter
mm/yr	millimeters per year
MSE	mechanically stabilized earth
M_W	moment magnitude
NA	not applicable
NAVD 88	North American Vertical Datum of 1988
ODOT	Oregon Department of Transportation
OSHA	Occupational Safety and Health Administration
OSHD	Oregon State Highway Department
OSSC	2024 Oregon Standard Specifications for Construction
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
PGA	peak ground acceleration
PGA_M	maximum considered earthquake geometric mean peak ground acceleration adjusted for site effects
psf	pounds per square foot
psi	pounds per square inch
RAP	rammed aggregate pier

ReMi	refraction microtremor
RI	rigid inclusion
RQD	rock quality designation
SOSSC	State of Oregon Structural Specialty Code
SPT	standard penetration test
USGS	U.S. Geological Survey
Vs_{100}	average shear wave velocity for the upper 100 feet of the soil profile
VWP	vibrating wire piezometer

REPORT OF GEOTECHNICAL ENGINEERING SERVICES
OPERATIONS CENTER
WEST LINN, OREGON

1.0 INTRODUCTION

Columbia West is pleased to submit this report of geotechnical engineering services for the proposed City of West Linn's Operations Center located east of the Salamo Road and Greene Street intersection in West Linn, Oregon.

The site is shown relative to surrounding physical features on Figure 1. The existing site conditions and proposed site layout are shown on Figures 2 and 3, respectively, as well as the approximate exploration locations. Logs of our explorations are presented in Appendix A. Results of our laboratory testing program are presented in Appendix B. Results of our seismic survey are presented in Appendix C. Results of our global slope stability analysis are presented in Appendix D. Our site-specific seismic hazard evaluation is presented in Appendix E. A photo log is presented in Appendix F. Pertinent information from prior geotechnical studies completed at the site are presented in Appendix G. Relevant schematic site plans are presented in Appendix H.

Abbreviations and acronyms used herein are defined immediately following the Table of Contents.

2.0 PROJECT UNDERSTANDING

The schematic site plans dated February 14, 2025, shows proposed improvements that include the following:

- An approximately 14,000-gross-square-foot, single-story administration building (Building C) with a finished floor elevation of 342 feet
- An approximately 14,000-square-foot, two-story storage building (Building B) with a finished floor elevation of 310 feet
- An approximately 13,500-gross-square-foot, single-story maintenance building (Building A) with a finished floor elevation of 310 feet
- A retaining wall with a maximum exposed height of approximately 32 feet north of the proposed maintenance and storage buildings
- Upper and lower access roads, via Salamo Road, and upper and lower parking lots
- Other improvements, including concrete pads, uncovered storage yards, covered and uncovered bulk storage, a manual car wash with a hot water boiler, a fueling station, and a deicer tank

All of the proposed developments will occur generally north and west of the existing wetland/wetland buffer zone that occupies the southeast portion of the site.

Foundation loads were not available at the time of this report; however, we have assumed maximum column loads will be less than 200 kips and maximum wall loads will be less than 5 kips per lineal foot. We estimate the distributed slab live load is less than 200 psf.

The schematic grading plans indicate that cuts and fills will be up to approximately 20 feet and 25 feet, respectively. The thickest fills are required for the proposed access road near the intersection with Salamo Road and the deepest cuts are required for the north side of the storage building.

The proposed administration building is classified as structural Risk Category IV per Table 1604.5 of the 2022 SOSSC as a designated operations center required for emergency response. Therefore, a site-specific seismic hazard evaluation is required for the building per SOSSC Section 1803.

We should be contacted to revise our recommendations if the assumptions stated above are incorrect.

3.0 BACKGROUND

The site is located within an area of mapped shallow landslide deposits (Burns et al. 2013) stemming from a slope failure that occurred in the 1960s during original construction of Interstate 205 (I-205). Details regarding the landslide and subsequent remediation efforts are discussed later in this report.

Prior geotechnical studies at the site and in the site vicinity have been conducted by GeoDesign, Inc. (2010, 2013), NV5 (2022, 2023), and CH2M Hill (1969). Subsurface conditions encountered in the field explorations for the above-referenced reports generally consist of topsoil, fill, or colluvium underlain by CRBG. The fill, up to approximately 50 feet thick, is described primarily as medium dense to very dense gravel with varying proportions of clay, silt, and sand. Varying proportions of basaltic gravel, cobbles, and boulders were observed in surface exposures of the fill. Fill comprised of reworked native colluvium was encountered in the southwest portion of the site. Medium stiff to stiff silt and medium dense to very dense silty gravel colluvium with varying proportions of sand and clay were encountered at the ground surface in the southwest portion of the site.

Basalt bedrock underlies the colluvium in the southwest portion of the site and the fill over the majority of the site. The top of the bedrock unit is weathered to a silty gravel soil with sand. Weathering and silt content generally decreases with depth. The silty gravel is very dense. Prior borings encountered practical refusal at depths between approximately 15 and 32 feet BGS within the ravine area in the southwest portion of the site. Prior test pits encountered refusal at depths as shallow as 2.5 feet BGS in the southwest portion of the site. Refusal depths likely represent the approximate depth of intact bedrock. The CRBG was encountered below the fill over the majority of the site at depths between approximately 9 and 49.5 feet BGS. In general, the basalt is medium hard to hard.

During prior field investigations by others, areas of standing water were observed at the site. Also, active running water was observed at the wetland feature in the east portion of the site.

4.0 PURPOSE AND SCOPE

The purpose of our services was to provide geotechnical engineering recommendations for use in design and construction of the proposed development. Specifically, we completed the following tasks:

- Reviewed published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Conducted a site reconnaissance and marked exploration locations prior to conducting the explorations.
- Coordinated and managed the field exploration program, which included locating public utilities, coordinating site access, and scheduling subcontractors and Columbia West field staff.
- Completed the following explorations at the site:
  - Drilled five borings to depths between 50 and 139 feet BGS using mud rotary and rock coring methods
  - Excavated 13 test pits to depths between 1.5 and 20.5 feet BGS
- Classified the soil and rock encountered in the explorations and maintained a detailed log of each exploration. Observed groundwater conditions in the explorations.
- Collected disturbed and relatively undisturbed soil samples from the explorations for laboratory testing.
- Completed a seismic survey using the ReMi technique to measure the shear wave velocity of the upper 100 feet of soil/rock at the site to support the site-specific seismic hazard evaluation.
- Installed three VWPs to measure the variation of groundwater levels beneath the site.
- Completed a laboratory testing program on select soil and rock samples, which included the following:
  - Twenty-four moisture content determinations in general accordance with ASTM D2216
  - Five particle-size analyses in general accordance with ASTM D1140
  - One particle-size analyses in general accordance with ASTM D6913
  - Three Atterberg limits tests in general accordance with ASTM D4318
  - One 3-point direct shear test in general accordance with ASTM D3080
  - Three dry density determinations in general accordance with ASTM D2937
  - Three unconfined compressive strength tests in general accordance with ASTM D7012-09
- Attended project meetings with the design and construction team.
- Conducted a two-dimensional limit equilibrium slope stability analysis.
- Conducted site-specific seismic hazard evaluation in accordance with the 2022 SOSSC and ASCE 7-16.
- Prepared this geotechnical report for use in design and construction of the site that includes the following:
  - Summary of soil, rock, and groundwater conditions at the site
  - Summary of laboratory testing results
  - Recommendations for foundation support, including preferred foundation type, allowable bearing capacity, and lateral resistance parameters
  - Recommendations for floor slab subgrade preparation

- Recommendations for use in design of retaining walls, including backfill and drainage requirements and lateral earth pressures
- Recommendations for site preparation, including grading, temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, subgrade preparation, and wet weather construction
- The results of our slope stability analysis and any relevant recommendations
- Recommendations for excavation and excavation support
- Recommendations for construction of AC pavement for on-site access roads and parking areas, including subbase, base course, and paving thickness
- Evaluation of groundwater conditions at the site and general recommendations for dewatering during construction and subsurface drainage
- Evaluation of liquefaction potential and associated settlement
- Seismic design parameters in accordance with the 2022 SOSSC and ASCE 7-16

5.0 SITE CONDITIONS

5.1 GEOLOGIC CONDITIONS

5.1.1 General

The site is located in the uplands, forming the boundary between the Portland Basin to the north and east, the Tualatin Basin to the west, and the Central Willamette Valley to the south, within the Willamette-Puget Sound Physiographic Province. Geologic mapping by Schlicker and Finlayson (1979) and Madin (2009) indicates that the site is situated on bedrock consisting of basalt flows of the middle Miocene age (16 million to 12 million years old) CRBG.

CRBG flows originated in southeast Washington and northeast Oregon approximately 15 million years ago. The flows traveled down the Columbia River during repeated eruptions and filled topographic lows within the Portland Basin, creating thick sequences of basalt flows locally, with associated unconformities and interbeds between flows created as a result of lulls between volcanic episodes. The basalt bedrock generally consists of dense, gray to black, fine-grained basalt that is locally deeply weathered, producing silty and clayey soil. Madin (2009) shows that the contact of the Wanapum and underlying Grande Ronde Basalts, two formations within the CRBG, is located in the vicinity of the site. Interbeds of saprolite are found between flows of the CRBG, and an extensive zone of thick saprolite is found at the contact of the Wanapum Basalt and Grande Ronde Basalt. This sedimentary interbed is referred to as the Vantage Horizon.

Bedding-plane landslides commonly occur within the Vantage Horizon where the interbed dips 10 degrees or more out of slope faces cut at steeper inclinations (Schlicker and Finlayson 1979). Outcrops of CRBG were exposed in the upper slope at an elevation of approximately 400 feet (NAVD 88) during prior site reconnaissance. We previously observed variable thicknesses of gravel, cobble, and boulder fill across the site associated with construction of the existing graded benches on the slopes above and beneath the roadbed of I-205.

5.1.2 Landslide History

5.1.2.1 Pre-November 1969

A large landslide occurred on the site during construction of I-205. According to Schlicker and Finlayson (1979), the landslide was activated by slope cuts that adversely exposed the Vantage Horizon during construction of I-205.

The headscarp of the landslide caused ground cracking at the Willamette Reservoir, which was part of the West Linn water supply system. The headscarp of this landslide as mapped in the CH2M Hill geotechnical report and mapped through the Willamette Reservoir is shown on Figure 2 from GeoDesign (2010) in Appendix G. The City of West Linn reported acceleration in the loss of water from the reservoir as a consequence of the landsliding. The reservoir was drained in the summer of 1969 and abandoned at a later date during the slide remediation.

While there is only limited documentation of this landslide and its remediation, CH2M Hill investigated the landslide and provided some details for the remediation efforts (CH2M Hill 1969). CH2M Hill reported that they drilled nine borings (SM-1 through SM-9; cross sections are presented in Appendix G, but boring logs are not available) within the landslide mass and installed slope inclinometers in each boring. These borings encountered unweathered, dense basalt overlain by weathered basalt and clay soil. A continuous, thin (1 foot to 2 feet thick) layer of micaceous silt was interbedded between the unweathered and weathered basalt units. This silt layer was identified as the Vantage Horizon.

Slippage measured using the slope inclinometers indicated that the failure surface of the landslide was located within the Vantage Horizon and was a translational landslide occurring along the bedding plane. CH2M Hill (1969) indicated that during the summer of 1969, OSHD, which would later become ODOT, started to flatten the cut slopes from 1.5H:1V to 2H:1V and widen the intervening benches. This slope flattening did little to stop the landslide movement. During the fall of 1969, OSHD began to install horizontal drains to intercept groundwater. The drains were not considered effective in intercepting groundwater (CH2M Hill 1969). The slope flattening and horizontal drains were inadequate in mitigating this landslide, and this was confirmed in a State of Oregon Geotechnical Engineering Group (GEG) interoffice memorandum (GEG 1991).

5.1.2.2 Post-November 1969

GeoDesign, Inc. (2010) previously prepared a map showing the difference in elevation in the vicinity of the landslide using a 10-meter DEM obtained from USGS (pre I-205 construction) and a DEM based on LiDAR surveying performed by the Puget Sound LiDAR Consortium (post I-205 construction). Based on the resulting elevation differences, GeoDesign determined that the ground surface elevation had been lowered by approximately 30 to 100 feet across much of the site.

Based on their study, GeoDesign, Inc. (2010) developed an interpretation of the actions undertaken by OSHD after November 1969. Their primary conclusions were as follows:

1. OSHD excavated the landslide mass down past the Vantage Horizon from the headscarp down to the I-205 alignment.
2. The area upslope of the headscarp was cut to a 1.5H:1V slope to facilitate removal of the landslide mass. The Vantage Horizon is still present in this zone.

3. Two large fill embankments were placed as toe buttresses after mass excavation of the slide mass. Records are generally incomplete or unavailable; therefore, specifics regarding the buttress size and extent are unknown.
4. The buttresses may be up to 50 feet thick and rock cuts, during mass excavation of the slide mass, were on the order of 140 feet deep.

The State of Oregon GEG interoffice memo (1991) states, "*In 1971 and 1972, a 20 to 40-foot wide and 2,400-foot long buttress was constructed near the middle of the landslide mass. Later a 400-foot section of the buttress was later enlarged to increase the stability.*

This buttress was designed for low/marginal stability. No indication of movement is evident. Existing construction records are incomplete and little is known of buttress size."

We are unaware of any documented slope movement since that memo was issued in 1991, and we did not observe any signs of slope instability during our site reconnaissance and field explorations.

5.2 SURFACE CONDITIONS

The approximately 26.3-acre site is bordered to the west by Salamo Road and a topographic hollow/ravine that might include an abandoned aggregate borrow pit (based on our prior site reconnaissance), to the south by I-205, to the east by an ODOT storage yard and vacant lots, and to the north by vacant lots and residential developments. The site is vegetated with grass, shrubs, poison oak, blackberry bushes, and Douglas fir and deciduous trees; some of the trees had been felled. Site grades range between approximately 480 and 230 feet (NAVD 88) from north to south.

The site is located on a south-facing slope that consists of two graded benches separating I-205 to the south and the top of a prominent ridge line to the north. The lower bench is covered with a thick layer of wood chips, generally 6 to 24 inches thick at the locations explored. The benches and adjacent slopes are large fill buttresses constructed as part of the landslide mitigation described in Section 5.1.2 (Landslide History). The fill slopes have an existing grade of approximately 1.5H:1V.

A wetland feature with active running water is present within the southeast portion of the site. A wetlands/water delineation report prepared for the site by Vigil-Agrimis, Inc. (2009) indicates that the wetland is spring fed.

5.3 SUBSURFACE CONDITIONS

5.3.1 General

Our field explorations included drilling 5 borings (B-1 through B-5) to depths between 50 and 139 feet BGS and excavating 13 test pits (TP-1 through TP-13) to depths between 1.5 and 20.5 feet BGS. Exploration locations are shown on Figures 2 and 3. A description of our field exploration program and the exploration logs are presented in Appendix A. A description of our laboratory testing program and the testing results are presented in Appendix B. In addition to the test pits and borings, we also completed a seismic survey using the ReMi technique to estimate the shear wave velocity of the subsurface soil and rock to a depth of 100 feet BGS. The location of the ReMi array is shown on Figures 2 and 3, and results of the seismic survey are presented in Appendix C.

A summary of the subsurface conditions is presented below.

5.3.2 Soil Conditions

5.3.2.1 Topsoil and Bark Chips

All of our explorations encountered either topsoil or bark chips at the ground surface.

The bark chips were encountered in borings B-3 and B-4 and test pits TP7, TP-8, TP-10, TP-12, and TP-13, all of which are located on the relatively flat bench where the proposed maintenance and storage buildings will be located. The bark chip layer is between 12 and 24 inches thick in all explorations where it was encountered, except for test pit TP-7 where it is 6 inches thick.

All other explorations (borings B-1, B-2, and B-5 and test pits TP-1 through TP-6, TP-9, and TP-11) encountered 6 to 12 inches thick of topsoil that includes a 4- to 6-inch-thick root zone.

5.3.2.2 Undocumented Fill

Fill was encountered beneath the topsoil or bark chips in borings B-2 and B-5 and test pits TP-3 through TP-7, TP-9, and TP-11. The fill generally extends to depths between 15 and 31 feet BGS, except in test pits TP-7, TP-9, and TP-11 where it is present to depths between 2.5 and 5.5 feet BGS. Test pits TP-3 through TP-6 were terminated in fill.

The fill is highly variable in nature and generally includes medium stiff to hard clay with varying proportions of sand and gravel or very loose to very dense gravel and sand with varying proportions of clay and silt. The fill also includes cobbles and boulders up to 48 inches in diameter.

A sloping contact between the fill and the underlying basalt was observed in test pits TP-9 and TP-11 at depths between 5.5 and 8.5 feet BGS and between 3 and 7.5 feet BGS, respectively.

Test pit TP-5 was terminated in fill at a depth of 19 feet BGS due to refusal on a large boulder.

Laboratory testing on select samples of the fill material indicates the moisture content varied from approximately 8 to 41 percent at the time of exploration. Atterberg limits testing and field classifications indicate that the clay fill exhibits low to medium plasticity. Fines content analysis of select sand and gravel fill samples indicates a fines content of between 10 and 15 percent.

Based on our understanding of historical grading at the site as a part of the landslide mitigation efforts, the encountered fill is associated with embankments constructed as part of the landslide mitigation. The explorations that encountered thicker fills were likely drilled or excavated at or near the middle portion of the buttresses.

5.3.2.3 Native Soil

In boring B-1, which was drilled in a relatively flat bench south of Barrington Drive (north of the proposed access road), the topsoil is underlain by sandy silt with varying proportions of gravel to a depth of 11 feet BGS. The sandy silt is medium stiff to stiff.

5.3.2.4 Decomposed Basalt

Beneath the native soil in boring B-1 and the topsoil, bark chips, or fill in borings B-2 through B-5 and test pit TP-2, we encountered what we interpret as decomposed basalt. The decomposed basalt consists of layers of hard silt or clay with varying proportions of sand and gravel; medium dense to very dense gravel with varying proportions of sand, silt, clay, and cobbles; or very dense sand with silt. The decomposed basalt was encountered to depths between 4 and 46 feet BGS. The bottom of this unit is deepest in borings B-2 and B-5. The thickness of the decomposed basalt generally ranges between 0.5 foot and 19 feet.

Laboratory testing on select samples of the decomposed basalt indicates the moisture content varied from approximately 16 to 55 percent at the time of exploration. Fines content analysis of a select silt (decomposed basalt) sample indicates a fines content of 59 percent.

5.3.2.5 Basalt

Beneath the fill, topsoil, bark chips, native soil, or decomposed basalt, all of our explorations encountered basalt, except test pits TP-3 through TP-6 that were terminated in the fill unit. In borings B-1 through B-5, this unit is present at depths between 4 and 46 feet BGS and extends to the maximum depths explored, which ranges from 50 to 139 feet BGS. The top of the basalt was deeper in the borings that encountered the thicker fill unit associated with the existing stability buttress.

In test pits TP-1, TP-2, and TP-7 through TP-13, this unit was encountered at depths between 0.5 foot and 5.5 feet BGS and was observed to the maximum depths explored, which range from 1.5 to 17 feet BGS. The majority of the test pits that encountered the basalt were terminated due to practical refusal after 0.5 foot to 3.5 feet of excavation into the basalt unit, but test pits TP-9 and TP-11 were excavated 11.5 and 7 feet below the top of the basalt, respectively, due to the level of weathering and close joint spacing of the rock in these two test pits. The strength classification of the basalt ranged from extremely weak (R0) to very strong (R5).

Laboratory testing on select core samples of the basalt material indicates a compressive strength of between 5,860 and 17,340 psi.

Thin clay beds, between 1 and 12 inches thick (generally 3 inches thick), were encountered in the basalt unit at variable depths between basalt flows. The clay beds were encountered in borings B-1, B-2, B-3, and B-5. Also, a 12-inch-thick layer interpreted as relict basalt or weathered sandy interflow was encountered in boring B-5 at a depth of 92 feet BGS.

Laboratory testing on a select clay bed sample indicates a moisture content of 50 percent at the time of exploration. Atterberg limits testing and field classifications indicate that the clay beds generally exhibit low plasticity.

5.3.3 Groundwater

Groundwater was not encountered in the test pits excavated to a maximum depth of 20.5 feet BGS, and groundwater could not be observed in the drilled borings due to the use of mud rotary drilling and rock coring methods.

Data collected from VWPs installed in borings B-2, B-3, and B-5 show groundwater levels varied from 79.3 to 94.3 feet BGS between February 26 and March 28, 2025, in borings B-3 and B-5. Groundwater is more than 98 feet BGS in boring B-2. The groundwater measurements from the VWPs are summarized in Table 1 and presented on Figure 4.

Table 1. Summary of VWP Data

Boring	Ground Surface Elevation (feet)	Installation Depth (feet BGS)	Depth to Groundwater (feet BGS / elevation in feet)	
			High	Low
B-2	345	98	>98 / <247	
B-3	315	97.5	94.3 / 220.7	91.2 / 223.8
B-5	332.5	98	83.8 / 248.7	79.3 / 253.2

Given the relatively shallow depth to basalt, the fill mantling the site, and prior site reconnaissance notes (by others), perched water may occur at shallow depths following extended periods of heavy rainfall. This possibility is further supported by the presence of a delineated spring-fed wetland on the east portion of the site (Vigil-Agrimis, Inc. 2009). Seasonal fluctuations in groundwater depth are anticipated due to factors such as prolonged rainfall, surface topography changes, and other influences not observed during this study.

5.3.4 Caving

Minor to severe caving was observed in six test pits (TP-3 through TP-6, TP-9, and TP-11) excavated through the gravel fill starting at depths between 0 feet and 1 foot BGS. Test pits TP-4 and TP-6 were terminated due to severe caving.

5.4 *SLOPE STABILITY ANALYSIS*

5.4.1 General

We completed limit equilibrium slope stability analyses of the slopes near the structures using the software SLOPE/W version 2024.1 by GeoStudio. Our analyses considered three critical cross sections that were selected based on slope geometry and proximity of proposed structures to the slopes. The locations of the cross sections are shown on Figures 2 and 3. Elevation data for existing conditions was gathered from publicly available LiDAR data from the Oregon LIDAR Consortium, and post-construction topography was developed from the schematic grading plan provided by Scott Edwards Architecture, which is shown on Figure 3.

5.4.2 Subsurface Model

Soil strength parameters were estimated based on soil classification, laboratory testing, measured SPT blow counts, and the analysis discussed in Section 5.4.3 (Quasi Back-Analysis). The basalt was modeled as a composite strength material in SLOPE/W, where the parent material was the basalt mass and the secondary material was used to represent joint pairs oriented between 10 and 30 degrees and between 60 and 80 degrees. The parent material was modeled using Hoek-Brown rock strength parameters and the secondary material was modeled using Barton shear failure criteria for rock joints, both as implemented in the SLOPE/W program.

Where we interpreted prominent flow contacts or soil-filled discontinuities in the basalt that were correlated between borings, we model them as discrete layers. Based on our interpretation, these layers, including the Vantage Horizon that is present within the basalt north of the site, have an apparent dip of approximately 3 degrees to the south. The strength parameters used in the slope stability model are presented in Appendix D.

Based on the results of our groundwater monitoring, the static groundwater level is expected to be within the basalt unit, well below the bottoms of the analyzed slopes. In addition, the use of impervious surfaces, drains in fill slopes (as discussed in Section 6.10 (Drainage)), diversion by grading, and stormwater collection systems will assist in control of perched groundwater within new and modified slopes. However, given the presence of the Vantage Horizon upslope of the proposed development and the existing fill that will remain in place beneath the site, we ultimately modeled a perched groundwater condition that could occur on a seasonal basis.

Given the depth to groundwater at the site, even with the perched groundwater condition considered in the slope stability modeling, the soil units are generally unsaturated, with the exception of the deeper existing fill (generally classified as silty or clayey gravel) and decomposed basalt. Neither of these units is considered susceptible to liquefaction or strain softening based on field and laboratory classifications. Therefore, drained strengths were used for all slope stability analyses.

5.4.3 Quasi Back-Analysis

As previously discussed, the site has experienced landsliding in the past as a result of man-made slope cuts during construction of I-205. Sufficient information on the geometry of the slope and subsurface was not available to conduct a traditional back-analysis of the prior slope failure. Although geotechnical analysis of the landslide and documentation of subsequent remediation efforts are sparse, we attempted to refine the soil properties used in the slope stability modeling by analyzing the existing conditions and targeting a marginal factor of safety against slope failure, consistent with discussion in the 1991 GEG memorandum. In a typical slope stability back-analysis, the goal is to work backward from a failure to establish model properties that result in a computed factor of safety equal to or close to 1.0. This is different from the quasi back-analysis we performed, which was focused on establishing reasonable lower end estimates of the strength parameters for fill and the Vantage Horizon. The soil properties shown in Appendix D reflect this analysis.

5.4.4 Stability Under Proposed Conditions

Fo each of the three cross sections we analyzed, we considered static and pseudo-static stability of slopes directly impacted by the planned grading, as well as global stability of the proposed retaining wall where it intersected the cross sections. When analyzing global wall stability, we assigned a minimum embedment for the toe of the wall and made the wall infinitely strong so that the potential failure surfaces were a true global failure; internal and external stability of the retaining wall should be considered by the wall designer once grading plans are finalized.

In the case of the pseudo-static analysis, we used a horizontal seismic coefficient (k_h) equal to one-half of the PGA_M. The analyses were carried out in SLOPE/W using the Morgenstern-Price method, which satisfies all conditions of equilibrium and considers over 10,000 potential non-circular slip surfaces. Table 2 summarizes the results of the slope stability analyses, and software output is presented in Appendix D.

Typically, factors of safety of 1.5 and 1.1 under static and pseudo-static conditions are considered minimum values at which point static slope instability and seismic displacements greater than 1 inch to 2 inches is unlikely. Computed factors of safety for cross sections 1 and 2 are greater than the minimum values for the various scenarios we analyzed, indicating that slope instability is unlikely.

For cross section 3, which includes a proposed fill slope with an approximate gradient of 2H:1V, results indicate that the slope is expected to be stable under static conditions but is only marginally stable under seismic loading. The results in Appendix D show that the critical failure surface with a factor of safety of 1.0 for seismic loading passes through the existing fill that comprises the bottom half of the slope. Based on the limited extents of the fill slope and the locations of critical and other nearly critical failure surfaces, large displacement of a deep failure mass is not likely, but enough slope movement could occur to impact trafficability of the proposed access road.

We conducted a supplementary analysis that considers an improved condition in which the proposed slope includes a benched-in sliver fill and shear key, both constructed of new structural fill. The conceptual geometry of the proposed slope is shown in Appendix D. Under this condition, the resulting pseudo-static factor of safety increase from 1.0 to 1.1, indicating that slope displacements greater than a few inches is unlikely (ODOT 2024a). This option for improving stability of the fill slope should be considered if maintaining unobstructed roadway access during a seismic event is critical.

Table 2. Slope Stability Results

Cross Section	Scenario	Factor of Safety	
		Static	Pseudo-Static
1	Upper slope	2.2	1.3
	Proposed roadway fill slope	2.8	1.5
	Proposed retaining wall	2.7	2.2
2	Upper slope	1.9	1.2
	Proposed retaining wall	1.8	1.4
	Slope south of proposed maintenance building	1.9	1.3
3	Proposed roadway fill slope	1.6	1.0
	Proposed roadway fill slope – improved	1.7	1.1

As previously discussed, our analysis assumes that the proposed development includes measures sufficient to mitigate the potential for perched water within new fill and behind retaining walls. Drainage recommendations are presented in Section 6.10 (Drainage) and Section 6.11 (Permanent Slopes).

In addition, we recommend careful planning and execution of tree removal to eliminate disturbance to tree stumps and roots, existing vegetation, and topsoil zones. Such disturbance and removal of existing vegetation on or near slopes may adversely impact the surficial stability.

5.5 SEISMIC HAZARDS

5.5.1 General

We completed a site-specific seismic hazard evaluation in accordance with the 2022 SOSSC and ASCE 7-16. A detailed discussion of seismic-induced geologic hazards is presented in Appendix E. The sections below highlight key considerations associated with seismic hazards at the site.

5.5.2 Liquefaction and Cyclic Softening

Based depth to groundwater as measured in the VWPs installed at the site and the soil conditions encountered in our explorations, liquefaction and cyclic softening are not considered design considerations for the project.

5.5.3 Seismic Settlement

Liquefaction and cyclic softening are not considered design considerations, and our explorations did not encounter thick deposits of very loose to loose, cohesionless soil beneath the site that is susceptible to seismic densification. Widespread seismic settlement is not a design consideration for this project.

Differential settlement between portions of the administration building bearing on undocumented fill and basalt is addressed in Section 6.3 (Shallow Foundations) and Section 6.4 (Ground Improvement).

5.5.4 Lateral Spreading

Materials that comprise the slopes at the site are not subject to significant strength loss during or after an earthquake; therefore, lateral spreading is not a design consideration for this project.

5.5.5 Surface Fault Rupture

The nearest Quaternary-aged fault, the Bolton fault, is more than 2 km northeast of the site. Accordingly, surface fault rupture is not a significant design consideration.

6.0 DESIGN

6.1 GENERAL

As discussed in Section 5.3 (Subsurface Conditions) and shown on Figures 2 and 3 and in Appendix A, our explorations encountered undocumented fill to depths between 1 foot and 31 feet BGS. Prior explorations encountered fill to depths between 10.5 and 49.5 feet BGS. The fill is highly variable and generally includes gravel and sand with varying proportions of silt and clay or clay with varying proportions of gravel and sand. The fill contains varying proportions of cobbles and boulders.

To minimize the risk of future foundation settlement due to the presence of undocumented fill, we recommend ground improvement systems be used to support the administration building foundations, including floor slabs in areas where thick undocumented fill is present. Based on our experience, RIs or the Geopier X1® system is a suitable option for foundation support.

Drilling through cobbles and boulders from prior grading operations will likely be challenging to the ground improvement contractors. We recommend the project budget include a contingency for removal of obstructions using excavation or specialty drilling equipment if encountered during ground improvement system installation.

Our explorations and the schematic grading plan indicate the subgrade at the base of the proposed maintenance and storage buildings will likely consist of native, dense gravel and sand (decomposed/weathered basalt) or intact basalt. Where undocumented fill is encountered, we recommend this material be removed from under building foundations and replaced with structural fill underlain by native, dense gravel and sand or bedrock. The structural fill under the maintenance and storage building footings should consist of granular pads. We recommend that foundation subgrade be evaluated at the time of construction to determine if undocumented fill is present.

We have provided additional recommendations for foundation support alternatives in the following sections.

6.2 AREAL SETTLEMENT CONSIDERATION

The schematic grading plan indicates cuts on the order of 5 feet may be required for the maintenance building and fills on the order of approximately 2 and 5 feet may be required for the administration and storage buildings, respectively. The administration building should be supported by a ground improvement system. Based on our explorations, we anticipate fill-induced settlement will be negligible under the buildings. Fills of up to approximately 25 and 5 feet will be required for the access road and east parking lot, respectively. The undocumented fill will be compressible under proposed fill-induced loads. Fill-induced settlement will vary based on fill thickness and subsurface soil conditions. Time-rate of settlement will also vary based on fill thickness; however, fill-induced areal settlement will generally be complete within 8 to 16 weeks of material placement. Paving is typically completed more than four months after placement of fill material, and settlement monitoring will likely not be required in areas of roadway or parking lot construction. However, settlement monitoring plates should be installed to monitor fill-induced settlement and evaluate the appropriate time to begin construction of settlement-sensitive project elements such as hardscape and underground utilities. We recommend Columbia West be allowed to review the final grading plans and verify our recommendations.

6.2.1 Areal Settlement Monitoring

Settlement monitoring plates should be used to monitor fill-induced settlement for fills greater than 5 feet, unless construction of settlement-sensitive features is conducted a minimum of

16 weeks after the completion of filling. Settlement monitoring plates should be installed and surveyed prior to fill placement. The casing and rod portions of the settlement plate are usually installed in 5-foot sections to facilitate handling by contractor personnel. As fill placement progresses, couplings are used to install additional rod sections. Continuity in the monitoring data is maintained by surveying the top of the measurement rod immediately prior to and following the addition of new rod sections. Special precautions must be taken by the contractor during fill placement to prevent bending or breaking the rods and settlement plates.

A typical settlement plate detail is presented on Figure 5. The settlement plates should be installed prior to site filling and immediately surveyed. Survey shots should be taken at each settlement plate at least twice per week during fill construction and for at least one month after fill construction, followed by once weekly thereafter. The settlement plates should be monitored using survey equipment with accuracy of at least 1/100th of a foot referenced to a stationary datum established at least 500 feet from the edge of the fill.

In addition to recording the elevation of the settlement plates during each survey event, a complete record of the preload history requires reading and recording the fill height at each settlement plate.

6.3 *SHALLOW FOUNDATIONS*

6.3.1 General

Based on our explorations and the schematic grading plan, the maintenance and storage buildings can be supported on conventional spread footings underlain by decomposed/weathered basalt or intact basalt or underlain by granular pads overlying decomposed/weathered basalt or intact basalt.

As discussed in Section 6.10 (Drainage), we recommend foundation drains be constructed for the buildings. Our recommendations assume that maximum column and wall loads will be less than 200 kips and 5 kips per lineal foot, respectively. The following sections provide our recommendation for use in foundation design and construction.

6.3.2 Granular Footing Pads

Granular footing pads should extend 6 inches beyond the margins of the footings for every foot excavated below the base grade of the footing. The gravel pads should consist of imported granular material compacted to not less than 95 percent of maximum dry density as determined by ASTM D1557 or until well keyed as determined by our geotechnical staff. We recommend that Columbia West observe all foundation subgrade to determine final footing pad depth.

6.3.3 Bearing Capacity

Continuous perimeter wall and isolated spread footings should have minimum widths of 18 and 24 inches, respectively. The bases of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bases of interior footings not exposed to frost should bear at least 12 inches below the base of the floor slab.

Footings bearing on dense, weathered bedrock subgrade or granular pads underlain by bedrock should be sized based on an allowable bearing pressure of 5,000 psf. The bearing pressure can be increased to 8,000 psf for footings established on hard, intact bedrock. As the allowable bearing pressure is a net bearing pressure, the weight of the footing and associated backfill may be ignored when calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be increased by 30 percent for transient (short-term) lateral forces such as seismic or wind forces.

6.3.4 Settlement

Settlement of shallow foundations founded on dense decomposed/weathered or hard intact bedrock should be negligible.

6.3.5 Resistance to Sliding

Lateral loads on building and retaining wall footings can be resisted by passive earth pressure on the sides of the structures and by friction on the bases of footings. Our analysis indicates that the available passive earth pressure for footings confined by the on-site soil or structural fill is 350 pcf, modeled as an equivalent fluid pressure, assuming that static groundwater remains below the base of the footing throughout the year. A passive resistance of 180 pcf should be used below groundwater. This will require that compacted structural fill be placed between the excavations and footings if passive pressure is relied on for lateral resistance. The recommended passive pressure can be increased to 500 pcf and 290 pcf for footings confined entirely by basalt bedrock above and below groundwater, respectively. Adjacent floor slabs, pavement, or the upper 12-inch depth of unpaved areas should not be considered when calculating passive resistance. In order to rely on passive resistance, a minimum of 10 feet of horizontal clearance must exist between the face of the footings and adjacent down slopes.

For footings in contact with basalt bedrock or granular pads, a coefficient of friction equal to 0.40 may be used when calculating resistance to sliding. The passive earth pressure and friction components may be combined, provided the passive component does not exceed two-thirds of the total.

6.3.6 Subgrade Observation and Preparation

Footing and floor subgrade soil should be evaluated by Columbia West prior to placing forms or reinforcing bar to verify subgrade support conditions are as anticipated in this report. Subgrade observation should confirm that all disturbed material, organic debris, unsuitable fill, remnant topsoil zones, and softened subgrade (if present) have been removed. Over excavation of footing subgrade soil may be required to remove deleterious material, particularly if footings are constructed during wet weather conditions.

6.4 *GROUND IMPROVEMENT*

Due to the presence of thick undocumented fill, the administration building can be supported on conventional spread footings that bear on improved ground. For minimum embedment and dimension requirements for the spread footings, refer to Section 6.3 (Shallow Foundations).

6.4.1 Spread Footings Bearing on RIs

The foundation system for the new administration building may consist of spread footings and floor slabs underlain by RIs. This method of ground improvement is typically design-build. The RIs are designed as a ground improvement system and rely on support both from the subgrade soil and RIs. The RI columns are installed using drilling techniques. Once the drill reaches the target depth, CLSM is pumped through the drill rods to the bottom of the hole. CLSM continues to be pumped as the drill rods are withdrawn from the hole. The completed RI columns consist of a continuous shaft of CLSM material.

The RI column is typically 24 to 36 inches in diameter and should be founded in basalt. The RI system could result in an allowable bearing pressure of up to 6,000 psf. We anticipate that this value can be increased by one-third when considering transient loads such as wind and seismic forces, but should be verified by the designer.

The contractor can use the information in this report and, if necessary, should conduct additional explorations if the geotechnical information is insufficient. The design-build contractor should complete a settlement analysis for the RIs. We recommend a load testing program be implemented for RIs to confirm design capacity.

Conventional spread foundations and floor slabs can be constructed over the completed RIs. The footings are typically supported on a 12- to 24-inch-thick granular pad between the footing and top of the RIs (full dimension of footing) to transfer the load.

6.4.2 Spread Footings Bearing on Geopier X1® System

The Geopier X1® system can be installed to provide support to a conventional spread footing system and floor slabs. The Geopier X1® piers are constructed using a track-mounted auger to drill out unsuitable fill soil and a specialty mandrel mounted to a mast rig to construct the RAP elements. The end result is a dense column of aggregate surrounded by stiffened matrix soil to control settlement and allow for higher bearing capacities typically on the order of 3,000 to 6,000 psf. The RAPs are typically 24 inches in diameter and should be founded in basalt. A one-third increase in allowable bearing pressure is also typical for such systems when resisting short-term loads such as wind and seismic forces. Geopier X1® is a proprietary design-build foundation system. Accordingly, if Geopier X1® is used, we recommend that the design-build contractor be contacted to provide the actual allowable bearing pressures and aggregate pier configurations. The contractor can use the information in this report and, if necessary, should conduct additional explorations if the geotechnical information is insufficient.

Installing RAPs will require drilling through soft and loose fill soil, which will be prone to raveling, sloughing, and running conditions if groundwater is encountered. Spoils that are generated during installation can be used as fill at the site or disposed of offsite. Also, the presence of cobbles and boulders in the fill from prior grading activities might result in caving/sloughing and hard drilling conditions or refusal. We recommend the project budget include a contingency for removal of obstructions using excavation or specialty drilling equipment if encountered during ground improvement system installation.

The design-build contractor should complete a settlement analysis for the RAPs. We recommend a load testing program be implemented for RAPs to confirm design capacity.

Conventional spread foundations can be constructed over the completed RAPs. The footing will typically be supported on a minimum 3-inch-thick granular pad that extends the full dimension (width and length) of the footing (on and between the piers).

6.4.3 Foundation Settlement

We anticipate the ground improvement system can be installed to meet the settlement criteria for the administration building. The design-build contractor should complete settlement analysis for the ground improvement system.

6.5 SEISMIC DESIGN CONSIDERATIONS

6.5.1 Seismic Design Criteria

We anticipate that seismic design will be performed in accordance with the 2022 SOSSC, which references ASCE 7-16. Based on shear wave velocity measurements made at the site and results of our field explorations, the seismic site class per ASCE 7-16 is Site Class C for structures founded within the existing fill embankment or Site Class B for structures founded on the basalt.

We understand the proposed development is designated as an operations center for emergency response. Per Table 1604.5 of the 2022 SOSSC, emergency response facilities are classified as Risk Category IV and require a site-specific seismic hazard evaluation. Details regarding our shear wave velocity measurements at the site are presented in Appendix C, and our site-specific seismic hazard evaluation, which includes seismic design parameters, is presented in Appendix E.

6.6 FLOOR SLABS

Satisfactory subgrade support for building floor slabs supporting floor loads of up to 200 psf can be obtained, provided the building pads are prepared as described in Section 7.0 (Construction). To help reduce moisture transmission and slab shifting, we recommend a minimum 6-inch-thick layer of floor slab base rock be placed and compacted over a subgrade that has been prepared in conformance with Section 7.1 (Site Preparation). The floor slab base rock should meet the requirements in Section 7.6.1 (Structural Fill) and be compacted to at least 95 percent of maximum dry density as determined by ASTM D1557. Load-bearing slabs on grade may be designed assuming a modulus of subgrade reaction of 150 pci, assuming the floor slab is underlain by native soil or structural fill overlying native soil.

Settlement of floor slabs supporting the anticipated design loads and constructed as recommended is not expected to exceed approximately ½ inch.

Due to the presence of thick undocumented fill, we recommend the administration building floor slabs be supported by a ground improvement system to mitigate excessive settlement as discussed in the preceding sections. The modulus of subgrade reaction and settlement estimate for floor slabs that are supported by a ground improvement system should be provided by the design-build ground improvement contractor.

If ground improvement is not implemented, disturbed soil, undocumented fill, and debris in proposed slab locations should be completely removed and replaced with structural fill to mitigate excessive settlement.

Some flooring manufacturers will not warrant their product unless a vapor barrier has been installed. Selection and design of an appropriate vapor barrier should be decided upon by all members of the design team.

All slab subgrade should be evaluated by a member of our staff to confirm suitable bearing conditions. Observations should also confirm that loose or soft material, organic material, unsuitable fill, prior topsoil zones, and softened subgrade have been removed and replaced with structural fill. In addition, contaminated base rock for the slabs should be removed and replaced prior to pouring the slab.

6.6.1 Radon Gas Mitigation

We understand the building code requires radon gas mitigation measures for projects located in Clackamas County. Where the below-slab passive radon system will be installed, we recommend a minimum 6-inch-thick layer of floor slab base rock be placed and compacted over the coarse (gas-permeable) aggregates for proper grading and compaction of the base rock and to protect the gas-permeable pipe from damage. We recommend the use of hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor) for base rock compaction for at least a 2-foot-wide zone around the gas-permeable pipe to protect the pipe from damage during compaction. We recommend subgrade geotextile fabric be installed to assist as a barrier between the subgrade and the coarse aggregates. We understand soil gas retarder will be installed between the floor slab base rock and the coarse aggregates and will assist as a barrier between the two materials. The coarse aggregates and the geotextile fabric should meet the specifications in Section 7.6 (Materials).

6.7 RETAINING STRUCTURES

6.7.1 Assumptions

Our retaining wall design recommendations are based on the following assumptions: (1) the walls are cantilevered walls, (2) the walls consist of CIP or MSE-type retaining walls, (3) the walls are less than 10 feet in height, (4) drainage is provided behind the walls, (5) the retained soil has a slope flatter than 4H:1V, and (6) the ground surface at the toes of the walls has an inclination flatter than 5H:1V. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

Lateral earth pressures should be considered during design of retaining walls and below-grade structures. Hydrostatic pressure and additional surcharge loading should also be considered. Wall foundation construction and bearing capacity should adhere to the specifications in Section 6.3 (Shallow Foundations) and Section 6.4 (Ground Improvement).

6.7.2 Retaining Wall Design Parameters

Permanent retaining walls that are not restrained from rotation should be designed for active earth pressures using an equivalent fluid pressure of 35 pcf. Walls that are restrained from rotation should be designed for an at-rest equivalent fluid pressure of 55 pcf. Lateral earth pressures induced by surcharge loads may be estimated using the criteria presented on Figure 6. Seismic lateral earth pressure for walls retaining level backfill may be calculated assuming an equivalent fluid weight of 13 pcf. The result of the triangular distribution increasing with depth should be applied at a height of 0.34H above the base of the wall, where H is the wall height. The lateral seismic earth pressure force should be superimposed with static active earth pressure.

The design equivalent fluid pressure should be increased for walls that retain sloping soil. We recommend the above lateral earth pressures be increased using the factors presented in Table 3 when designing walls that retain sloping soil. Where the total lateral earth pressure includes static and seismic components, the increase factors shown in Table 3 should be applied to static and seismic components separately and then combined.

Table 3. Lateral Earth Pressure Increase Factors for Sloping Soil

Slope of Retained Soil (degrees)	Lateral Earth Pressure Increase Factor	
	Static (active or at-rest)	Seismic
0	1.00	1.00
5	1.06	1.11
10	1.12	1.27
20	1.33	1.84
25	1.52	2.73
30	2.27	3.69

The above design parameters have been provided assuming adequate drainage is provided behind the walls to prevent hydrostatic pressures from developing. If a drainage system is not installed, our office should be contacted for revised design forces.

6.7.2.1 Wall Drainage and Backfill

A minimum 6-inch-diameter, perforated collector pipe should be placed at the base of the retaining wall. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock that is wrapped in a drainage geotextile fabric and extends up the back of the wall to within 1 foot of finished grade. The drain rock should meet the specifications in Section 7.6 (Materials). The perforated collector pipes should discharge at an appropriate location away from the base of the wall and any slopes. The discharge pipes should not be tied directly into stormwater drainage systems, unless measures are taken to prevent backflow into the drainage system of the wall.

Backfill material placed behind walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of select granular material placed and compacted as described in Section 7.6.1 (Structural Fill).

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be delayed at least four weeks after placement of wall backfill, unless survey data indicates that settlement is complete prior to that time.

6.7.3 MSE Wall Design Parameters

We anticipate MSE retaining walls may be required for the project. Detailed wall layouts and geometries were not available at the time of this report. We provided the following general information if the MSE walls become a contractor design-build product for the project.

We recommend the soil parameters presented in Table 4 be used for design of the MSE retaining walls. The material used to backfill behind the walls is discussed in Section 6.7.3.1 (Wall Backfill and Drains) and Section 7.6 (Materials).

Table 4. MSE Wall Design Parameters

Soil Type	Unit Weight (pcf)	Friction Angle, ϕ (degrees)	Cohesion (psf)
Reinforcing zone backfill: select granular wall backfill[1]	130	36	0
Retained soil: general fill	120	28	0
Foundation soil: native, dense gravel and sand[2]	130	36	0

1. Select granular wall backfill, as described in Section 7.6.1 (Structural Fill).
2. Foundation soil should be native decomposed/weathered basalt or structural fill soil prepared in conformance with Section 7.1 (Site Preparation).

6.7.3.1 Wall Backfill and Drains

Drainage should consist of a minimum 12-inch-wide zone of drain rock placed behind the retaining wall blocks and extending from the base of the wall to within 12 inches of finished grade. Perforated collector pipes should be embedded at the base of the drain rock. The drain rock should meet the requirements provided in Section 7.6 (Materials). The perforated collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe(s) should not be tied directly into stormwater drain systems, unless measures are taken to prevent backflow into the drainage system of the wall. The backfill material placed behind the walls should consist of retaining wall select backfill placed and compacted in conformance with Section 7.6 (Materials).

6.7.3.2 Surcharge Loads

Where traffic loads are located within a horizontal distance from the top of the wall equal to one-half the wall height, the lateral earth pressure shall be increased by a surcharge load equal to 2 feet of soil (assuming a soil density of 125 pcf). The traffic load should not be included for seismic analysis.

The design equivalent fluid pressures and lateral earth pressures (derived from Table 4) should be increased for walls that retain sloping soil. We recommend the lateral earth pressures be increased using the factors (presented in Table 3) when designing walls that retain sloping soil.

Seismic forces should be modeled based on the pseudo-static approach developed by the Mononobe-Okabe method. We recommend using a seismic coefficient of one-half of the PGA when analyzing internal stability.

The wall design should include all internal and external stability requirements, as well as global slope stability requirements.

6.7.3.3 Base Excavation

Excavation may be required to achieve internal and external stability, as well as global slope stability. All existing fill and topsoil should be removed from the wall alignment. This may locally require over excavation of the wall base. The excavation subgrade should expose competent native soil, extend in front of the wall face for at least 3 horizontal feet before the slope breaks, and allow for the minimum design embedment of the toe of the wall. A geotextile fabric should be placed over the excavation backslope prior to backfilling in order to keep on-site soil separate from the granular backfill and to prevent clay and silt migration.

All earthwork and excavation should be conducted in conformance with Section 7.6.1 (Structural Fill). If the wall is constructed during wet weather, measures should be taken to protect exposed subgrade from moisture. If sloughing of the backslope occurs during construction, granular wall backfill contaminated with silt or clay soil should be removed from the backfill zone.

6.7.3.4 Wall Settlement

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to retaining walls as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be postponed at least four weeks after backfilling of the wall, unless survey data indicates that settlement is complete prior to that time.

6.7.4 Soldier Pile Wall

6.7.4.1 General

An up to 32-foot-tall cut and fill wall will be required for the proposed storage building as well as pavement areas north of the maintenance building (see Figure 3). Support for the proposed cut and fill can be provided by installing soldier piles with timber lagging and tieback anchors. Typical wall facing may also include shotcrete or precast concrete panels for aesthetics. We understand the wall will be designed as a permanent installation. If a portion of the wall is designed as temporary shoring at the storage building, embedded walls constructed in contact with temporary shoring (if any) should be designed to resist the earth pressures developed by the shoring system, unless the shoring is designed as a permanent installation. Wall depth/design should take into consideration the excavation needed for the granular pads as recommended in Section 6.3 (Shallow Foundations). Tieback anchors will likely be required where the wall is tall. The height limit for cantilever walls is typically on the order of 10 feet. Cantilever shoring might be possible to support excavations of less than 10 feet if some settlement or horizontal movement can be tolerated. Settlement of the ground surface will be approximately 1 inch at the wall face and become negligible a horizontal distance of 10 feet from the wall. Cobbles and boulders will be encountered in the fill and decomposed/weathered basalt. The soldier pile contractor should be prepared to advance through or modify their procedures where cobbles and boulders and basalt are encountered.

6.7.4.2 Cantilever Wall

A soldier pile wall embedded into basalt can be designed using the values presented on Figure 7. These values do not include surcharged-induced lateral earth pressures. Figure 6 should be used to compute surcharge-induced lateral earth pressures. We recommend a vertical live load of 250 psf be applied at the surface of the retained soil where the wall shoring retains roadways. Dewatering below the base of the shoring will result in higher passive resistance as noted on Figure 7.

6.7.4.3 Anchored Wall

An anchored soldier pile wall embedded into basalt can be designed using the values presented on Figure 7. These values do not include surcharged-induced lateral earth pressures. Figure 6 should be used to compute surcharge-induced lateral earth pressures. We recommend a vertical live load of 250 psf be applied at the surface of the retained soil where the wall shoring retains roadways. Dewatering below the base of the shoring will result in higher passive resistance as noted on Figure 7.

Structural design of the soldier piles should consider the lateral earth pressures discussed above. In addition to lateral earth pressures, the soldier piles will be subject to compressive forces as a result of the downward component of the tieback anchor loads. We recommend the tips of soldier piles be embedded at least 10 feet below the base of the excavation or a minimum of 5 feet into basalt. An allowable bearing pressure of 4,000 psf may be used for the base of the soldier piles resting on the dense gravel and sand (decomposed/weathered basalt). Skin friction along the sides of the soldier piles will also be able to resist downward forces. An allowable skin friction of 3 ksf may be used for the gravel (decomposed/weathered basalt) and 1 ksf for the fill. Concrete backfill should be placed using tremie pipe methods, unless the bottom of the hole is dry.

The bonded zone for the tieback anchors should be maintained outside of the “unbonded zone for anchors” shown on Figure 7. We anticipate that the tieback anchors can achieve an ultimate adhesion of between 2 and 4 ksf in the fill and between 5 and 10 ksf in the native gravel and sand (decomposed/weathered basalt), depending on the method of construction. A variety of methods are available for construction of tieback anchors. Therefore, we recommend that the contractor be responsible for selecting the appropriate bonded length and installation methods to achieve the required anchor capacity. Tieback anchors should be locked off at 100 percent of the design load.

Prior to installing production anchors, we recommend that performance testing be conducted on a minimum of two anchors. The purpose of this testing is to verify the installation procedure selected by the contractor before a large number of anchors are installed. We recommend that proof testing be conducted on all production anchors. Performance and proof testing should be performed in accordance with the guidelines provided in *Recommendations for Prestressed Rock and Soil Anchors* (Post Tensioning Institute 2014).

We anticipate that wood lagging, shotcrete, or precast concrete panels will be used for the wall. To maintain the integrity of the excavation, prompt and careful installation of lagging, particularly in areas of seepage and loose soil, is recommended. All voids behind the lagging should be completely backfilled with grout slurry.

A back-of-wall drain should be installed behind the shoring system and extend to the base of the lagging. The drain may consist of a conventional drain rock and filter fabric system but will likely consist of prefabricated drainage panels; collector pipes may be used to discharge the water. We recommend a minimum coverage of 30 to 50 percent if drainage panels are used, increased to 100 percent in areas where water seepage is encountered during wall construction. We recommend that a collector pipe be installed at the lowest point of the wall and extend to an appropriate discharge.

6.7.4.4 Installation Considerations

Perched groundwater may be encountered in soldier pile foundation excavations. Sloughing, caving, and the potential for "running conditions" will likely occur if the excavation extends below the groundwater table or if the excavations are dry but left open for extended periods of time (more than a few hours). Depending on the foundation installation methods, dewatering and use of temporary casing, drilling slurries, or both will likely be required for soldier pile excavations. The use of open-hole drilling methods will likely not be feasible for soldier pile foundation excavations.

Construction of the soldier pile wall may require some temporary cuts. However, excavations for construction will need to be carefully planned and executed so as not to destabilize the existing slopes. In addition, water should not be allowed to flow over the face of temporary cuts. We recommend that temporary excavation slopes are protected using plastic sheeting. If seepage is observed from the cut, we should be contacted immediately to evaluate stabilization options.

6.8 GROUND ANCHORS

It may be necessary to install ground anchors to resist uplift forces. Uplift loads should be developed by the structural engineer for design of the anchors. Design and construction of the anchor system should be completed by specialty contractors who will be responsible for selecting the appropriate type and depth.

6.9 PAVEMENT

Pavement should be installed on native subgrade or new engineered fill prepared in conformance with the Section 7.1 (Site Preparation) and Section 7.6.1 (Structural Fill).

Undocumented fill was encountered in our explorations to depths of up to 31 feet BGS at the locations explored. Due to the variable composition of the undocumented fill and the unknown methods of placement and compaction, reliable strength properties for undocumented fill are extremely difficult to predict. There is a risk of differential settlement of the pavement between cut and fill areas and associated maintenance given the variable conditions (thickness and composition) of the undocumented fill. The design parameters provided below assume the pavement is underlain by post-stripping native soil, structural fill, or at least 18 inches of improved undocumented fill by scarification and compaction per Section 7.1 (Site Preparation) and

Section 7.7 (Fill Placement and Compaction). If encountered, boulders, large cobbles, and deleterious material should be removed before compaction. If this approach is used, it should be understood that the risk of pavement distress and associated maintenance is acceptable. If this risk is not acceptable, full removal of the undocumented fill and replacement with structural fill will be required.

Specific traffic information was not provided to us at the time of this report. Our pavement recommendations are based on the following assumptions:

- A resilient modulus of 20,000 psi was estimated for the aggregate base.
- Initial and terminal serviceability indices of 4.2 and 2.0, respectively.
- Reliability of 75 percent and standard deviation of 0.45.
- Structural coefficients of 0.42 and 0.10 for the AC and aggregate base, respectively.
- No growth.
- A resilient modulus of 3,500 psi for site subgrade assuming the surface 18 inches of subgrade is scarified and compacted to at least 92 percent of maximum dry density as defined by ASTM D1557.
- The pavement section of access roadways associated with the fire access road will support an imposed fire apparatus load of 75,000 pounds on an infrequent basis.
- Design life of 20 years.
- No growth.

If any of these assumptions are incorrect, our office should be contacted with the appropriate information so that the pavement designs can be revised.

Pavement design is strongly influenced by the type and frequency of truck traffic. The breakdown of truck traffic (relative to FHWA classification) used in our analysis is presented in Table 5. If any of these assumptions vary from project design values, our office should be contacted with the appropriate information so that the pavement designs can be revised.

Table 5. Truck Traffic Breakdown

FHWA Class Group	Description	Percent
4	Bus and fire truck	2
5	2-axle, single unit	50
6	3-axle, single unit	48
7	4-axle, single unit	0
8	Tractor/trailer 3- to 4-axle	0
9	Tractor/trailer 3- to 4-axle	0
10	Tractor/trailer 3- to 4-axle	0
11	5-axle, multi-trailer	0
12	6-axle, multi-trailer	0

Our pavement design recommendations assuming a truck ADT between 10 and 40 are presented in Table 6.

Table 6. Recommended Standard Pavement Sections

Traffic Levels	Trucks per Day[1]	ESALs	AC[2] (inches)	Aggregate Base[2] (inches)
Car parking stalls	0	10,000	3	8
Truck/drive aisles[1]	10	35,000	3	12
	20	70,000	3.5	12
	30	105,000	3.5	14
	40	139,000	4	14

1. See Table 5 for the assumed breakdown of the trucks.
2. All thicknesses are intended to be the minimum acceptable.

If the subgrade is cement amended to the thicknesses indicated below and the amended soil achieves a seven-day unconfined compressive strength of at least 100 psi, the pavement can be constructed as recommended in Table 7.

Table 7. Recommended Pavement Sections using Cement Amendment

Traffic Levels	Trucks per Day[1]	ESALs	AC[2] (inches)	Aggregate Base[2] (inches)	Cement Amendment[2,3] (inches)
Car parking stalls	0	10,000	3	4	12
Truck/drive aisles[1]	10	35,000	3	4	12
	20	70,000	3.5	4	12
	30	105,000	3.5	5	12
	40	139,000	4	6	12

1. See Table 5 for the assumed breakdown of the trucks.
2. All thicknesses are intended to be the minimum acceptable.
3. Assumes a minimum seven-day unconfined compressive strength of 100 psi.

To prevent strength loss during curing, cement-amended soil should be allowed to cure for at least four days prior to construction traffic or placing aggregate base. Lastly, the amended subgrade should be protected with a minimum of 4 inches of aggregate base prior to construction traffic access.

All thicknesses are intended to be the minimum acceptable. Design of the recommended pavement section is based on the assumption that construction will be completed during an extended period of dry weather. Wet weather construction could require an increased thickness of aggregate base. In addition, the pavement sections recommended above are for support of post-construction design traffic. The aggregate (with or without cement-amended subgrade) is not designed to support construction traffic. Increased aggregate thicknesses will likely be required to support construction traffic as discussed in Section 7.2 (Construction Traffic and Staging).

The AC, aggregate base, and cement amendment should meet the requirements outlined in Section 7.6 (Materials).

Construction traffic should be limited to non-building, unpaved portions of the site or haul roads. Construction traffic should not be allowed on new pavement. If construction traffic is to be allowed on newly constructed road sections, an allowance for this additional traffic will need to be made in the design pavement section.

6.10 DRAINAGE

6.10.1 Temporary

During work at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. During rough and finished grading of the site, the contractor should keep all pads and subgrade free of ponding water.

6.10.2 Surface Drains

Where possible, the finished ground surfaces around buildings should be sloped away from the structures at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts or roof scuppers should discharge into a storm drain system that carries the collected water to an appropriate stormwater system. Trapped planter areas should not be created adjacent to buildings without providing means for positive drainage (e.g., swales or catch basins).

6.10.3 Foundation Drains

We recommend that perimeter foundation drains be installed for the buildings, considering the layouts and elevations of the proposed buildings and site conditions. Also, the use of these drains should be considered in areas where landscaping planters are placed proximate to the foundations.

Foundation drains should be constructed at a minimum slope of approximately ½ percent and pumped or drained by gravity to a suitable discharge. The perforated drainpipe should not be tied to a stormwater drainage system without backflow provisions. Foundation drains should consist of 4-inch-diameter, perforated drainpipe embedded in a minimum 2-foot-wide zone of crushed drain rock that extends to the ground surface. The invert elevation of the drainpipe should be installed at least 18 inches below the elevation of the floor slab.

The drain rock and geotextile should meet the requirements specified in Section 7.6 (Materials). The drain rock and geotextile should extend up the sides of embedded walls to within a foot of the ground surface with geotextile wrapped over the top of the drain rock, as recommended in Section 6.7 (Retaining Structures). A typical perimeter footing drain detail is presented on Figure 8.

6.10.4 Underslab Drainage

In addition to the recommendations for foundation drains (see above), underslab drains may be necessary in cut areas. Floor slabs established below existing grades may encounter shallow perched groundwater conditions. Further details regarding permanent dewatering systems will need to be developed once grading plans have been finalized and/or site conditions are exposed. A typical underslab drain detail is presented on Figure 9.

6.10.5 Subdrains

Subdrains should be considered for portions of the site that are cut below surrounding grades. Shallow groundwater or seepage should be conveyed via a drainage channel or perforated pipe into an approved discharge. Recommendations for design and installation of perforated drainage pipe may be performed on a case-by-case basis by Columbia West during construction. Failure to provide adequate surface and subsurface drainage may result in soil slumping or unanticipated settlement of structures exceeding tolerable limits. A typical perforated drainpipe trench detail is presented on Figure 10.

6.10.6 Infiltration System

Due to the presence of steep slopes on the site and subsurface conditions, we do not recommend that stormwater drainage be infiltrated on the site. Stormwater should be routed to an appropriate stormwater system.

6.11 PERMANENT SLOPES

The schematic grading plan indicates fill slopes with gradients of approximately 2.2H:1V or flatter are proposed. All new permanent cut and fill slopes should be evaluated by a qualified geotechnical engineer. Access roads and pavement should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings. The slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope. Columbia West should be allowed to review final grading plans and building and pavement setbacks from cut and fill slopes.

Where seepage is encountered at the face of cut slopes, it will be necessary to install a subdrain to collect the water. The subdrain shall consist of a minimum 18-inch-deep trench excavated through the base of the seepage area and at least 5 feet beyond the limits of the seepage. The trench shall be filled with drain rock with a perforated collector pipe and should be lined with a geotextile fabric. The drain rock and fabric should meet specifications provided in Section 7.6 (Materials). The collector pipe shall discharge at the toe of the slope, preferably directly into the storm drain system. We also recommend that a 2-foot-deep "toe drain" be installed at the toes of cut and fill slopes to collect groundwater seepage and protect adjacent pavement subgrade. A Typical cut and fill slope detail is presented on Figure 11.

7.0 CONSTRUCTION

7.1 SITE PREPARATION

Site grading activities should be performed in accordance with the requirements specified in the 2021 IBC, Chapter 18 and Appendix J, with exceptions noted in this report. Site preparation should be observed and documented by Columbia West.

7.1.1 Demolition

Demolition includes complete removal of existing site improvements within 5 feet of areas to receive new pavement, buildings, retaining walls, or engineered fill. Underground vaults, tanks, manholes, and other subsurface structures should be removed from areas of new improvements. Utility lines should be completely removed or grouted full and left in place.

Voids resulting from removal of existing improvements should be backfilled with compacted structural fill, as discussed in Section 7.6.1 (Structural Fill). The bottoms of such excavations should be excavated to expose a firm subgrade before filling and their sides sloped at a minimum of 1H:1V to allow for more uniform compaction at the edges of the excavations.

Material generated during demolition should be transported offsite for disposal or stockpiled in areas designated by the owner. In general, these materials will not be suitable for reuse as engineered fill.

The project budget may need to include a contingency for removal of boulders or other buried elements from previous site grading activities.

7.1.2 Grubbing and Stripping

Trees and shrubs should be removed from fill areas. In addition, root balls should be grubbed out to the depth of the roots, which could exceed 3 feet BGS. Depending on the methods used to remove root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

The existing root zone should be stripped and removed from all fill areas. Based on our observations, we estimate the average depth of stripping will be approximately 5 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil. The stripping depth of the bark chips area in the south portion of the site is expected to range between approximately 6 and 24 inches. Greater stripping depths may be anticipated in areas with thicker vegetation and shrubs. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported offsite for disposal or used in landscaped areas.

We recommend careful planning and execution of tree removal to eliminate disturbance to tree stumps and roots, existing vegetation, and topsoil zones. Such a disturbance may adversely impact the stability of the surficial slope soil.

7.1.3 Topsoil

An approximately 6- to 12-inch-deep topsoil zone was observed at the ground surface in our explorations. Reliable strength properties are extremely difficult to predict for the topsoil material. There is a high risk for poor performance of floor slabs and pavement established directly over topsoil. In order to reduce the risk of settlement, we recommend the topsoil be improved during site preparation in areas where planned cuts do not extend to the bottom of the topsoil (up to

12 inches). Before fill placement and construction, the topsoil should be improved by removing and replacing it with structural fill or scarifying and re-compacting to structural fill requirements.

As discussed in Section 7.6.1 (Structural Fill), the on-site soil can be sensitive to small changes in moisture content and will be difficult, if not impossible, to compact adequately during wet weather. While scarification and compaction of the subgrade is the best option for subgrade improvement, it will likely only be possible during extended dry periods and following moisture conditioning of the soil. As discussed further on in this report, cement amendment is an option for conditioning the soil for use as structural fill during periods of wet weather or when drying the soil is not an option.

7.1.4 Undocumented Fill

7.1.4.1 General

Our explorations encountered undocumented fill to depths between 1 foot and 31 feet BGS. Prior explorations encountered fill to depths between 10.5 and 49.5 feet BGS. The fill is highly variable and generally includes gravel and sand with varying proportions of silt and clay or clay with varying proportions of gravel and sand. The fill contains varying proportions of cobbles and boulders. Due to the variable thickness, composition, and consistency of the fill, there is a risk of settlement of foundations, floor slabs, and pavement founded on undocumented fill. The undocumented fill should be evaluated at the time of construction.

7.1.4.2 Foundation and Floor Slab Areas

Undocumented fill should be removed from under new building foundations and floor slabs, and the footings and floor slabs should be supported on structural fill or by a ground improvement system if the fill is left in place as discussed in Section 6.3 (Shallow Foundations), Section 6.4 (Ground Improvement), and Section 6.6 (Floor Slabs).

7.1.4.3 Pavement Areas

There is a risk for settlement and poor performance of pavement established directly over the undocumented fill. Removal and replacement of the undocumented fill would eliminate all risk. However, provided the risk of settlement and distress is understood and accepted, there is an option to limit the subgrade stabilization to removal and replacement or scarifying and recompacting the upper 18 inches of the undocumented fill material within pavement areas as discussed in Section 6.9 (Pavement).

7.1.4.5 Subgrade Observations

Considerable soil processing, including moisture conditioning and the removal of deleterious material (organic material, cobbles, and boulders) from the existing fill, may be required during scarification and compaction or when using the excavated material as structural fill. We recommend that the exposed subgrade be closely evaluated by a geotechnical engineer during the process. Compaction should be performed as described in Section 7.6.1 (Structural Fill) and Section 7.7 (Fill Placement and Compaction).

7.1.5 Subgrade Evaluation

Upon completion of stripping and subgrade stabilization and before fill or pavement is placed, the exposed subgrade should be evaluated by proof rolling. The subgrade should be proof rolled with a fully loaded dump truck or similarly heavy, rubber-tired construction equipment to identify soft, loose, or unsuitable areas. A member of our geotechnical staff should observe proof rolling to evaluate yielding of the ground surface. If soft or yielding subgrade areas are identified during evaluation, we recommend the subgrade be over excavated and backfilled with compacted structural fill.

During wet weather, subgrade evaluation should be performed by probing with a foundation probe rather than proof rolling. Areas that appear soft or loose should be improved in accordance with subsequent sections of this report.

7.1.6 Compacting Test Pit Locations

The test pit excavations were backfilled using the relatively minimal compactive effort of the excavator bucket. Soft spots can be expected at these locations. We recommend this relatively uncompacted soil be removed from the test pits to a depth of 3 feet below finished subgrade. If a test pit is located within 10 feet of a footing, we recommend full-depth removal of the uncompacted soil. The resulting excavation should be brought back to grade with structural fill.

7.2 CONSTRUCTION TRAFFIC AND STAGING

Near-surface silt- and clay-rich soil will be easily disturbed during construction. If not carefully executed, site preparation, utility trench work, and roadway excavation can create extensive soft areas and significant repair costs can result. Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance.

If construction occurs during or extends into the wet season or if the moisture content of the surficial soil is more than a couple percentage points above optimum, site stripping and cutting may need to be accomplished using track-mounted equipment. Likewise, the use of granular haul roads and staging areas will be necessary for support of construction traffic during the rainy season or when the moisture content of the surficial soil is more than a few percentage points above optimum. The base rock thickness for pavement and slab areas is intended to support post-construction design traffic loads. This design base rock thickness will likely not support construction traffic or pavement construction. Moreover, if construction is planned for periods when the subgrade soil is wet, staging and haul roads with increased thicknesses of base rock will be required.

The amount of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor’s sequencing of a project and type/frequency of construction equipment and should, therefore, be the responsibility of the contractor. Based on our experience, between 12 and 18 inches of imported granular material are generally required in staging areas and between 18 and 24 inches in haul road areas. The contractor should also be responsible for selecting the type of material for construction of haul roads and staging areas. A geotextile fabric can be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic to help prevent fines migration into the base rock. Imported granular material, stabilization material, and geotextile fabric should meet the specifications in Section 7.6 (Materials).

As an alternative to thickened crushed rock sections, haul roads and utility work zones may be constructed using cement-amended subgrade overlain by a crushed rock wearing surface. If this approach is used, the thickness of granular material in staging areas and along haul roads can typically be reduced to between 6 and 9 inches. This recommendation is based on an assumed minimum unconfined compressive strength of 100 psi for subgrade amended to a depth of 12 to 16 inches. The actual thickness of the amended material and imported granular material will depend on the contractor's means and methods and should be the contractor's responsibility. Cement amendment is discussed in Section 7.6 (Materials).

7.3 EXCAVATION AND SHORING

7.3.1 Basalt Bedrock

The upper portions of the basalt bedrock are weathered and fractured and could generally be penetrated to varying degrees by the Kobelco SK140SR trackhoe used to excavate our test pits (see Appendix A). While excavation of weathered bedrock is expected to be difficult, we do not expect special excavation equipment to be necessary in the moderately weathered material. The test pit excavator could not penetrate the intact bedrock. Special excavation techniques (such as hydraulic breakers, rock trenchers, or blasting) will likely be required to excavate the intact basalt bedrock where our test pits encountered refusal. The depths to weathered or intact bedrock encountered in our explorations is presented Figures 2 and 3.

The contract bid documents should require that contractors list a unit cost for rock excavation using in-place (bank) volumes, which can be measured by surveying. The unit cost should not be provided in excavated (swelled) volumes, which are more difficult to accurately measure.

7.3.2 Rockfall Hazards

Grading and excavation should be conducted by the earthwork contractor in a sequence that will prevent overhanging rock material that might become dislodged. The contractor should also be responsible for providing protection for property and personnel. This might include sloping of cut areas, building protective berms, or providing catchment netting or equipment at the base of a slope excavation, depending on the final grading plan and construction sequence.

7.3.3 Trench Cuts and Shoring

In general, the fill soil at the site should be readily excavatable with conventional grading equipment. However, as mentioned previously, cobbles and boulders will be encountered in the existing fill and decomposed/weathered basalt. The native, dense gravel, cobbles, and boulders encountered at shallow depths will result in difficult excavation with conventional equipment in both fill and decomposed/weathered basalt. In addition, because of the presence of cobbles and boulders in the near-surface fill and decomposed/weathered basalt, we expect difficult excavation at shallow depths. Utility trenches may result in slowed excavation and larger backfill volumes due to the presence of cobbles, boulders, and related caving.

Temporary construction excavations adjacent to level ground (without adjacent structures) should stand vertical for short periods of time to a depth of approximately 4 feet BGS, provided groundwater seepage is not observed in the trench walls. However, where retaining walls, soft or loose fills, or sloping ground is located above the excavation, or the excavation is greater than 4 feet high or groundwater seepage is present, the cuts will need to be shored or trimmed back and should be carefully planned so as not to destabilize the excavation or nearby improvements. Open excavation techniques may be used to excavate trenches with depths between 4 and 8 feet BGS, provided the walls of the excavation are cut at a slope of 1.5:1V or flatter and groundwater seepage is not present. At this inclination, the slopes with soft silt and clay or loose gravel and sand, cobbles, and boulders may ravel and require some ongoing repair. Excavations should be flattened to 2H:1V if excessive sloughing or raveling occurs. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. Use of approved temporary shoring is recommended where the slopes cannot be cut back, within the influence area of structural elements, and for cuts below the water table. The influence area can be defined as a 1H:1V slope extending down from a 5-foot setback from the edge of a foundation element. A wide variety of shoring and dewatering systems are available. Consequently, we recommend that the contractor be responsible for selecting the appropriate shoring and dewatering systems.

If box shoring is used, it should be understood that box shoring is a safety feature used to protect workers and does not prevent caving. If excavations are left open for extended periods of time, caving of the sidewalls may occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The contractor should be prepared to fill voids between the box shoring and the sidewalls of the trenches with sand or gravel before caving occurs.

If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation.

7.3.4 Open Excavations

Open excavations may be necessary for building cuts. Cuts in slightly weathered to intact bedrock will likely stand vertical and do not require shoring or sloping. We recommend that cuts in the overlying gravel and sand (decomposed/weathered basalt) layers be sloped in accordance with OSHA regulations. Temporary shoring may be possible to support cuts in soil, but may be difficult to install due to the underlying bedrock. If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation.

7.3.5 Safety

All excavations should be made in accordance with applicable OSHA requirements and regulations of the state, county, and local jurisdiction. While this report describes certain approaches to excavation and dewatering, the contract documents should specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety, and providing shoring (as required) to protect personnel and adjacent structural elements.

7.4 CONSTRUCTION DEWATERING

7.4.1 General

Groundwater was measured in the VWPs installed at the site at depths as shallow as 79.3 feet BGS. Based on the proposed site grading, we do not expect that static groundwater will impact the proposed development.

Areas of perched water may be encountered, particularly within the fill or on top of the decomposed/weathered basalt. We expect that perched groundwater may be encountered in trench excavations and possibly during mass grading. Temporary and permanent dewatering systems will likely be required.

7.4.2 Construction Dewatering

The contractor should be made responsible for temporary drainage of surface water, perched water, and groundwater as necessary to prevent standing water and/or erosion at the working surface.

Because of the instability of saturated, low plasticity silt, sand, and gravel, sloughing and "running" conditions can occur if the excavation extends below groundwater seepage levels. Accordingly, positive control of groundwater seepage will be required to maintain stable trench sides and base. The proposed dewatering plan should be capable of maintaining groundwater levels at least 2 feet below the base of the trench excavation (including the depth required for trench bedding and stabilization material). In addition to safety considerations, running soil, caving, or other loss of ground will increase backfill volumes and can result in damage to adjacent structures or utilities. Flow rates for dewatering are likely to vary depending on location, soil type, and the season in which the excavation occurs. The dewatering systems should be capable of adapting to variable flows.

Trench dewatering will be required to maintain dry working conditions if the invert elevations of the proposed utilities encounter groundwater. If perched water is encountered, pumping from a sump located within trenches may be effective in dewatering localized groundwater seepage. Given the silt and sand present, pumping from sumps located within the trench may result in excessive sloughing, caving, or running conditions, and dewatering by well points may be required.

If groundwater is present at the base of utility excavations, we recommend placing 1 foot to 2 feet of stabilization material at the base of the excavation. The use of a subgrade geotextile fabric may reduce the amount of stabilization material required. The actual thickness should be based on field observations during construction.

Trench stabilization material and the subgrade geotextile fabric should meet the requirements described in Section 7.6 (Materials). Trench stabilization material should be placed in one lift and compacted until well keyed.

While we have described certain approaches to the excavation dewatering, it is the contractor's responsibility to select the dewatering methods.

7.4.3 Permanent Dewatering

Permanent dewatering systems may be required along the bases of cut and fill slopes, in areas where upwelling water conditions are observed, or at other susceptible areas identified by our office in the field. If required, we anticipate the systems may typically consist of a series of gravel blanket drains beneath paved areas in cut conditions, sub-drains at the toes of the cut and fill slopes, and retaining wall drains.

Further details regarding permanent dewatering systems will need to be developed once development plans have been finalized and/or site conditions are exposed. Also, refer to Section 6.10 (Drainage) for additional discussion regarding site drainage.

7.5 TEMPORARY DRAINAGE

The contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water or erosion at the working surface. During rough and finished grading of the building site, the contractor should keep all footing excavations and building pads free of water.

7.6 MATERIALS

7.6.1 Structural Fill

7.6.1.1 General

Areas proposed for fill placement should be appropriately prepared as described in Section 7.1 (Site Preparation). Engineered fill placement should be observed by Columbia West. Compaction of engineered structural fill should be verified by nuclear gauge field compaction testing performed in accordance with ASTM D6938. Field compaction testing should be performed for each vertical foot of engineered fill placed.

Various materials may be acceptable for use as structural fill. Structural fill should be free of organic material or other unsuitable material and should meet the specifications provided in the following sections. Representative samples of proposed engineered structural fill should be submitted for laboratory testing and approval by Columbia West prior to placement.

7.6.1.2 On-Site Soil

The material at the site should be suitable for use as general structural fill, provided it is properly moisture conditioned and free of debris, organic material, and particles over 6 inches in diameter. Occasional cobbles greater than 6 inches in diameter may be acceptable if they can be properly mixed into the fill matrix to reduce void creation and are spaced sufficiently to allow compaction of the soil. Material greater than 10 inches in diameter shall not be placed in fills. Fine grading of soil may result in segregating cobbles or coarse gravel from the soil matrix, resulting in unsatisfactory (poorly graded or "bony") fill. Fill material should be maintained as well graded with gravel, sand, and silt or clay material for proper compaction during fill placement and mass grading. A qualified geotechnical engineer should observe fill material prior to placement.

During our explorations, we observed the on-site soil was wet of optimum. Consequently, we anticipate that the near-surface soil will require significant moisture conditioning (drying) prior to use as on-site soil for structural fill. Accordingly, extended dry weather will be required to adequately condition and place the soil as structural fill. It will be difficult, if not impossible, to adequately compact on-site soil during the rainy season or during prolonged periods of rainfall.

When used as structural fill, on-site soil should be placed in lifts with a maximum uncompacted thickness of 8 to 10 inches and compacted to not less than 92 percent of maximum dry density as determined by ASTM D1557.

7.6.1.3 Imported Granular Materials

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The imported granular material should also be durable, angular, and fairly well graded between coarse and fine material; should have less than 5 percent fines by dry weight; and should have at least two mechanically fractured faces.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of maximum dry density as determined by ASTM D1557. During the wet season or when wet subgrade conditions exist, the initial lift should be approximately 18 inches in uncompacted thickness and should be compacted by rolling with a smooth drum roller without using vibratory action.

7.6.1.4 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of durable, well-graded granular material with a maximum particle size of 1½ inches, should have less than 7 percent fines by dry weight, and should have at least two mechanically fractured faces. The pipe zone backfill should be compacted to at least 90 percent of maximum dry density as determined by ASTM D1557 or as required by the pipe manufacturer or local building department.

Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of durable, well-graded granular material with a maximum particle size of 2½ inches, should have less than 7 percent fines by dry weight, and should have at least two mechanically fractured faces. This material should be compacted to at least 92 percent of maximum dry density as determined by ASTM D1557 or as required by the pipe manufacturer or local building department. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of maximum dry density as determined by ASTM D1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads), trench backfill placed above the pipe zone may consist of general fill material that is free of organic material and material over 6 inches in diameter. This general trench backfill should be compacted to at least 90 percent of maximum dry density as determined by ASTM D1557 or as required by the pipe manufacturer or local building department.

7.6.1.5 Stabilization Material

Stabilization material used in staging or haul road areas or in trenches should consist of 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The material should have a maximum particle size of 6 inches, should have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and should have at least two mechanically fractured faces. The material should be free of organic material and other deleterious material. Stabilization material should be placed in lifts between 12 and 24 inches thick and compacted to a firm condition.

7.6.1.6 Drain Rock

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches. The material should be free of roots, organic material, and other unsuitable material; should have less than 2 percent fines by dry weight; and should have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

7.6.1.7 Retaining Wall Backfill

Backfill placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of imported granular material as described above and should have less than 7 percent fines by dry weight. We recommend the wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

Backfill should be placed and compacted as recommended for structural fill, with the exception of backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for generation of excessive pressure on the walls. Backfill located within a horizontal distance of 3 feet from retaining walls should be compacted to approximately 90 percent of maximum dry density as determined by ASTM D1557. Backfill placed within 3 feet of a wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). If flatwork (slabs, sidewalk, or pavement) will be placed adjacent to a wall, we recommend that the upper 2 feet of fill be compacted to 95 percent of maximum dry density as determined by ASTM D1557.

7.6.1.8 Retaining Wall Leveling Pad

Imported granular material placed at the bases of retaining wall footings should consist of select granular material. The granular material should be ¾- to 1½-inch-minus aggregate size and should have at least two mechanically fractured faces. The leveling pad material should be placed in a 6- to 12-inch-thick lift and compacted to not less than 95 percent of maximum dry density as determined by ASTM D1557.

7.6.1.9 Floor Slab and Pavement Aggregate Base

Imported granular material used as base rock for building floor slabs and pavement should consist of ¾- or 1½-inch-minus material (depending on the application). In addition, the aggregate should have less than 5 percent fines by dry weight. The aggregate base should be compacted to not less than 95 percent of maximum dry density as determined by ASTM D1557.

7.6.2 Soil Amendment with Cement

7.6.2.1 General

As an alternative to the use of imported granular material for wet weather structural fill, an experienced contractor may be able to amend the on-site soil with portland cement to obtain suitable support properties. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content, and amendment quantities. The amount of cement used during amendment should be based on an assumed soil dry unit weight of 100 pcf.

In addition, the new Oregon Department of Environmental Quality requirements, under 1200C permits, include additional requirements for routing and testing runoff from sites where cement amendment is used.

Depending on the grade and location, cobbles and boulders could be present in the subgrade. Cobbles and boulders can damage cement tilling equipment and when encountered may require removal before cement amendment.

7.6.2.2 Subbase Stabilization

Specific recommendations based on exposed site conditions for soil amendment can be provided if necessary. However, for preliminary design purposes, we recommend a target strength for cement-amended subgrade for building and pavement subbase (below aggregate base) of 100 psi. The amount of cement used to achieve this target generally varies with moisture content and soil type. It is difficult to predict field performance of soil to cement amendment due to variability in soil response, and we recommend laboratory testing to confirm expectations. In general, 6 percent cement by weight of dry soil can be used when the soil moisture content does not exceed approximately 20 percent. If the soil moisture content is in the range of 25 to 35 percent, 7 to 8 percent by weight of dry soil is recommended. The amount of cement added to the soil may need to be adjusted based on field observations and performance. Moreover, depending on the time of year and moisture content levels during amendment, water may need to be applied during tilling to appropriately condition the soil moisture content.

We recommend assuming a minimum cement ratio of 6 percent by dry weight, with higher rates as discussed above. Because of the higher organic content and moisture, we recommend using a higher cement ratio when stabilizing topsoil zone material, likely a minimum of 7 percent.

We recommend cement amendment equipment be equipped with balloon tires to reduce rutting and disturbance of the fine-grained soil. A sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds should be used for initial compaction of the fine-grained soil without the use of vibratory action. A smooth drum roller with a minimum applied linear force of 700 pounds per inch should be used for final compaction. The amended soil should be compacted to at least 92 percent of the achievable dry density at the moisture content of the material as defined in ASTM D1557.

A minimum curing time of four days is required between amendment and construction traffic access. Construction traffic should not be allowed on unprotected, cement-amended subgrade. To protect the cement-amended surfaces from abrasion or damage, the finished surface should be covered with 4 to 6 inches of imported granular material.

Amendment depths for subgrade beneath buildings and pavement, haul roads, and staging areas are typically approximately 12, 16, and 12 inches, respectively. The crushed rock typically becomes contaminated with soil during construction. Contaminated base rock should be removed and replaced with clean rock in pavement areas. The actual thickness of the amended material and imported granular material for haul roads and staging areas will depend on the anticipated traffic as well as the contractor's means and methods and should be the contractor's responsibility.

Cement amendment should not be attempted when air temperature is below 40 degrees Fahrenheit or during moderate to heavy precipitation. Cement should not be placed when the ground surface is saturated or standing water exists.

7.6.2.3 Cement-Amended Structural Fill

On-site soil that would not otherwise be suitable for structural fill may be amended and placed as fill over a subgrade prepared in conformance with Section 7.1 (Site Preparation). The cement ratio for general cement-amended fill can generally be reduced by 1 percent (by dry weight). Typically, a minimum curing time of four days is required between amendment and construction traffic access. Consecutive lifts of fill may be amended immediately after the previous lift has been amended and compacted (e.g., the four-day wait period does not apply). However, where the final lift of fill is a building or roadway subgrade, the four-day wait period is in effect.

7.6.2.4 Other Considerations

Portland cement-amended soil is hard and has low permeability. This soil does not drain well and it is not suitable for planting. Future planted areas should not be cement amended, if practical, or accommodations should be made for drainage and planting. Moreover, cement amendment of soil within building areas must be done carefully to avoid trapping water under floor slabs. We should be contacted if this approach is considered. Cement amendment should not be used if runoff during construction cannot be directed away from adjacent wetlands. In general, cement amendment is not recommended during cold weather (temperatures less than 40 degrees Fahrenheit) or during steady rainfall.

7.6.3 Pavement

7.6.3.1 AC

The AC should be Level 2, ½-inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement) and compacted to 92 percent of the theoretical maximum density of the mix as determined by AASHTO T 209 or ASTM D2041. The minimum and maximum lift thicknesses are 2 and 3 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22 or better. The binder grade should be adjusted depending on the aggregate gradation and amount of recycled asphalt pavement and/or recycled asphalt shingles in the contractor’s mix design submittal.

7.6.3.2 Cold Weather Paving Considerations

In general, AC paving is not recommended during cold weather (temperatures less than 40 degrees Fahrenheit). Compacting under these conditions can result in low compaction and premature pavement distress. Each AC mix design has a recommended compaction temperature range that is specific for the particular AC binder used. In colder temperatures, it is more difficult to maintain the temperature of the AC mix, as it can lose heat while stored in the delivery truck, as it is placed, and in the time between placement and compaction. In Oregon, the AC surface temperature during paving should be at least 40 degrees Fahrenheit for lift thicknesses greater than 2.5 inches and at least 50 degrees Fahrenheit for lift thicknesses between 2 and 2.5 inches.

If AC paving must take place during cold weather construction as defined in this section, the contractor and design team should discuss options for minimizing risk to pavement serviceability.

7.6.4 Geotextile Fabric

7.6.4.1 Subgrade Geotextile

Subgrade geotextile should conform to OSSC Table 02320-4 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles. All drainage aggregate and stabilization material should be underlain by a subgrade geotextile.

7.6.4.2 Drainage Geotextile

Drainage geotextile should conform to Type 2 material of OSSC Table 02320-1 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

7.7 FILL PLACEMENT AND COMPACTION

Fill soil should be compacted at a moisture content that is near optimum. The maximum allowable moisture content varies with the soil gradation and should be evaluated during construction.

Fill and backfill material should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The maximum lift thickness will vary depending on the material and compaction equipment used but should generally not exceed the loose thicknesses provided in Table 8. Fill material should be compacted in accordance with the compaction criteria provided in Table 9.

Table 8. Recommended Uncompacted Lift Thickness

Compaction Equipment	Recommended Uncompacted Lift Thickness (inches)		
	On-Site Silty, Clayey, and Sandy Soil	Granular and Crushed Rock Maximum Particle Size ≤ 1½ Inches	Crushed Rock Maximum Particle Size > 1½ Inches
Hand tools: Plate compactor and jumping jack	4 to 8	4 to 8	Not recommended
Rubber-tired equipment	6 to 8	10 to 12	6 to 8
Light roller	8 to 10	10 to 12	8 to 10
Heavy roller	10 to 12	12 to 18	12 to 16
Hoe pack equipment	12 to 16	18 to 24	12 to 16

The table above is based on our experience and is intended to serve only as a guideline. The information provided in this table should not be included in the project specifications.

Table 9. Compaction Criteria

Fill Location	Compaction Requirements in Structural Zones Percent Maximum Dry Density Determined by ASTM D1557		
	0 to 2 Feet Below Subgrade (percent)	> 2 Feet Below Subgrade (percent)	Pipe Zone (percent)
Beneath footings and columns	95[1]	95	--
Beneath interior slabs	95	95	--
Beneath pavement	95	95	--
Utility trenches	95[1]	92	90[2]
Sidewalks	95	92	--
Retaining wall backfill	95[1,3]	92[3]	--
Area fills	95[1]	92	--
Non-structural (landscape) areas	85	80	--

1. May be reduced to 92 percent if fine-grained soil is used.
2. Or as recommended by the pipe manufacturer.
3. Should be reduced to 90 percent within a horizontal distance of 3 feet from the retaining wall.

7.8 EROSION CONTROL

Soil at this site is susceptible to erosion by wind and water; therefore, erosion control measures should be carefully planned and installed before construction begins. Surface water runoff should be collected and directed away from sloped areas to prevent water from running down the slope face. Measures that can be employed to reduce erosion include the use of silt fences, hay bales, buffer zones of natural growth, sedimentation ponds, and granular haul roads. All erosion control methods should be in accordance with local jurisdiction standards.

8.0 OBSERVATION OF CONSTRUCTION

Satisfactory pavement, earthwork, and foundation performance depends to a large degree on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Columbia West should be retained to observe subgrade preparation, fill placement, foundation excavations, ground improvement installation, drainage system installation, and pavement placement and to review laboratory compaction and field moisture-density information.

Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

9.0 LIMITATIONS

We have prepared this report for use by the addressee and members of the design and construction team for the proposed project. This report is subject to the limitations expressed in Appendix I.

◆ ◆ ◆

We appreciate the opportunity to be of service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

Jonathan A. Nasr, PE
Project Engineer

Najib A. Kalas, PE
Principal Engineer

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FIGURES

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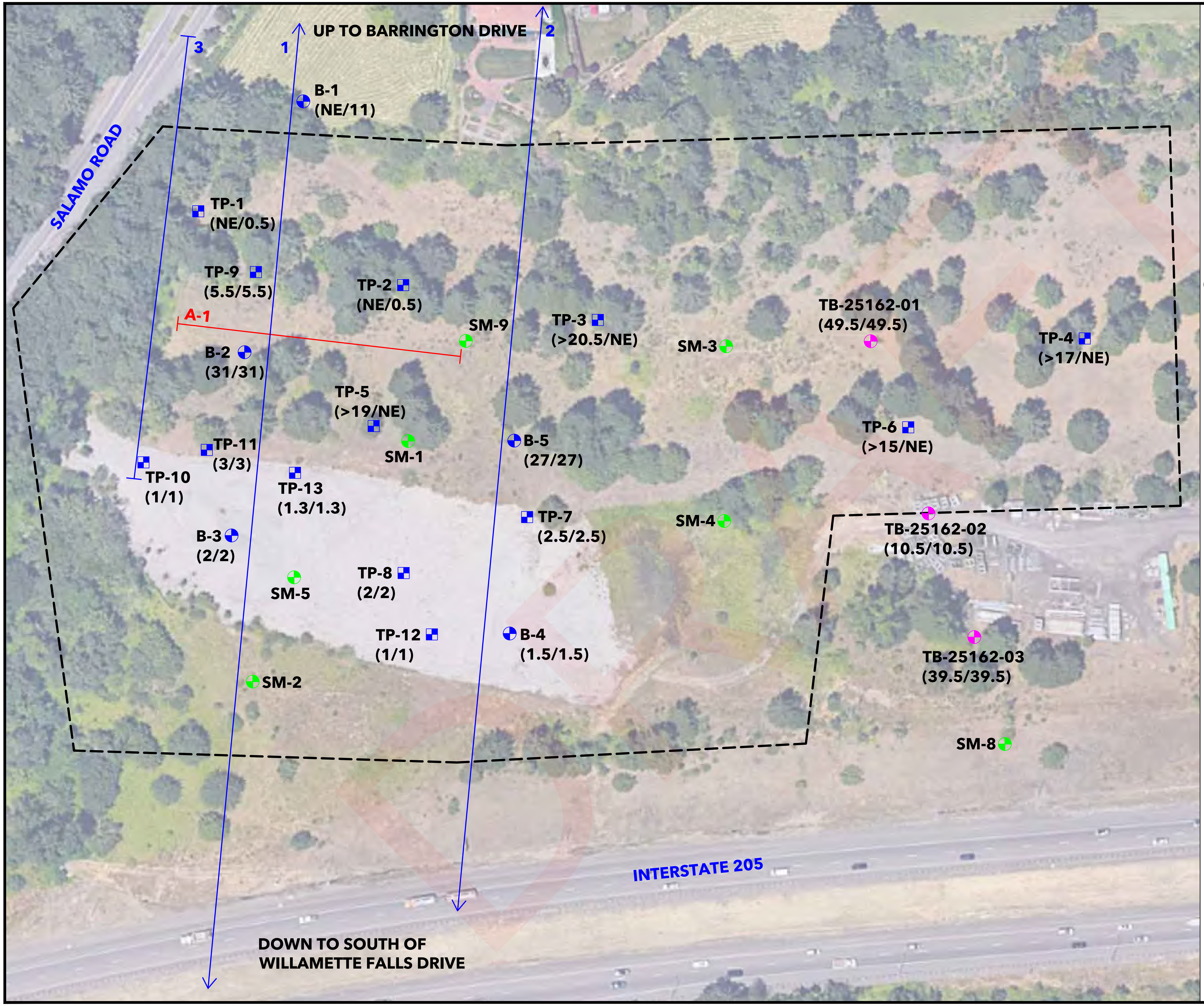
PROJECT NO.:
WESTL-1-01-1

APRIL 2025

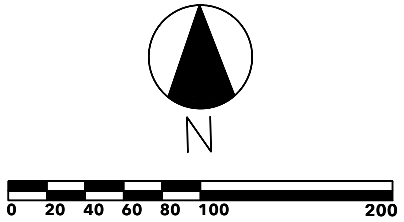
VICINITY MAP
OPERATIONS CENTER
WEST LINN, OREGON

FIGURE
1



LEGEND

⬚	APPROXIMATE SITE BOUNDARY
◑	BORING (COLUMBIA WEST 2025)
▚	TEST PIT (COLUMBIA WEST 2025)
◑	BORING (GEODESIGN 2010)
◑	BORING (CH2M HILL 1969 - SEE NOTE 4)
⊢⊣	APPROXIMATE REMI ARRAY
⊢⊣	SLOPE STABILITY CROSS SECTION
(NE/11)	DEPTH OF FILL/DEPTH TO BASALT, WHERE ENCOUNTERED (FEET BGS)
NE	NOT ENCOUNTERED



NOTES:
1. AERIAL PHOTO SOURCED FROM GOOGLE EARTH.
2. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
3. REFER TO REPORT TEXT FOR EXPLORATION DESCRIPTIONS.
4. BORING LOGS NOT AVAILABLE.

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OPERATIONS CENTER
WEST LINN, OREGON
SALAMO ROAD AND GREEN STREET

SITE PLAN - EXISTING CONDITIONS

PROJECT NO:
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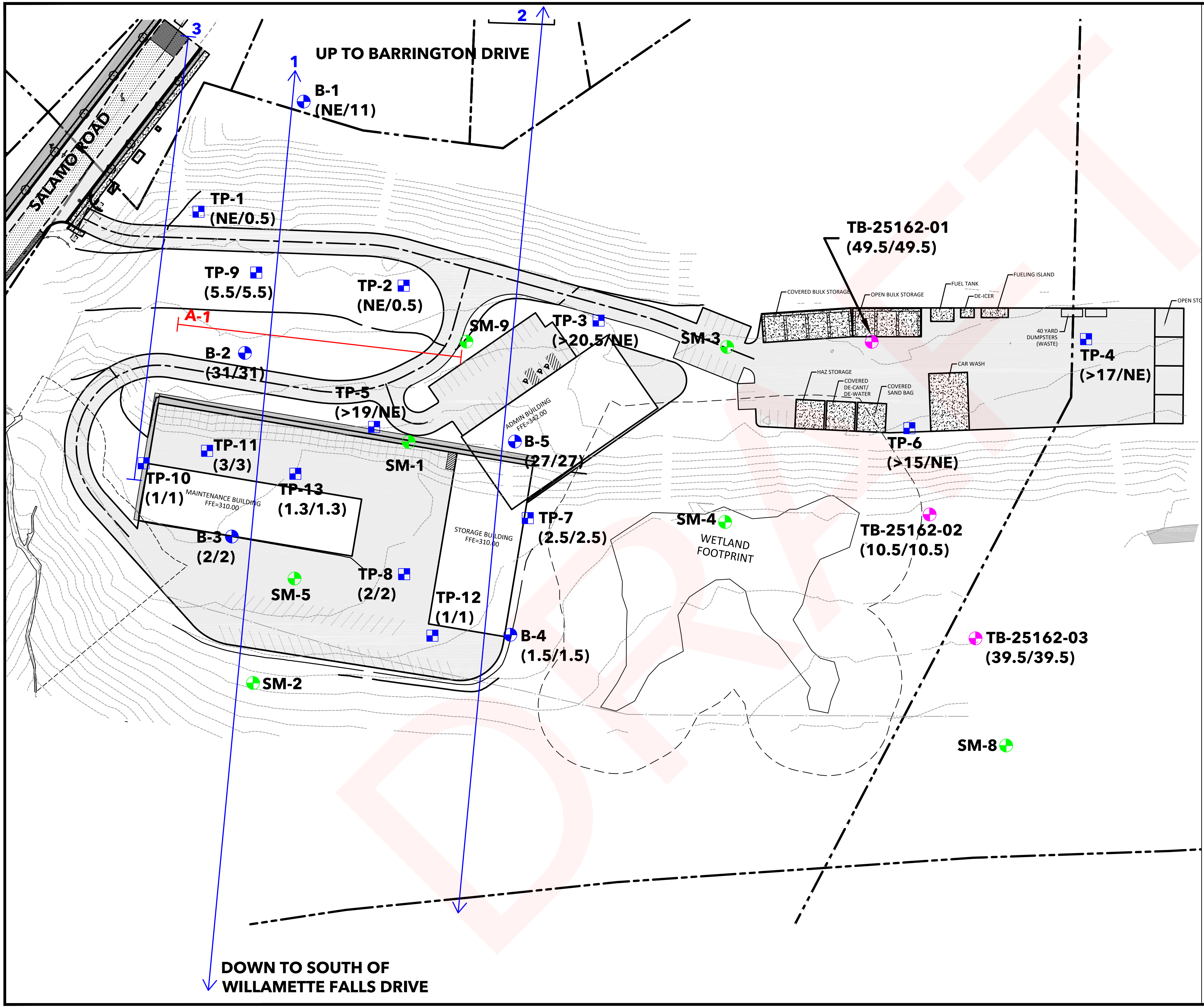
APRIL 2025

FIGURE
2



LEGEND

●	BORING (COLUMBIA WEST 2025)
▨	TEST PIT (COLUMBIA WEST 2025)
●	BORING (GEODESIGN 2010)
●	BORING (CH2M HILL 1969 - SEE NOTE 4)
⊢⊣	APPROXIMATE REMI ARRAY
⊢⊣	SLOPE STABILITY CROSS SECTION
(NE/11)	DEPTH OF FILL/DEPTH TO BASALT, WHERE ENCOUNTERED (FEET BGS)
NE	NOT ENCOUNTERED

N

0 20 40 60 80 100 200

NOTES:
1. PROPOSED DEVELOPMENT PLAN PROVIDED BY SCOTT EDWARDS ARCHITECTURE.
2. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
3. REFER TO REPORT TEXT FOR EXPLORATION DESCRIPTIONS.
4. BORING LOGS NOT AVAILABLE.

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OPERATIONS CENTER
WEST LINN, OREGON
SALAMO ROAD AND GREEN STREET

SITE PLAN - PROPOSED CONDITIONS

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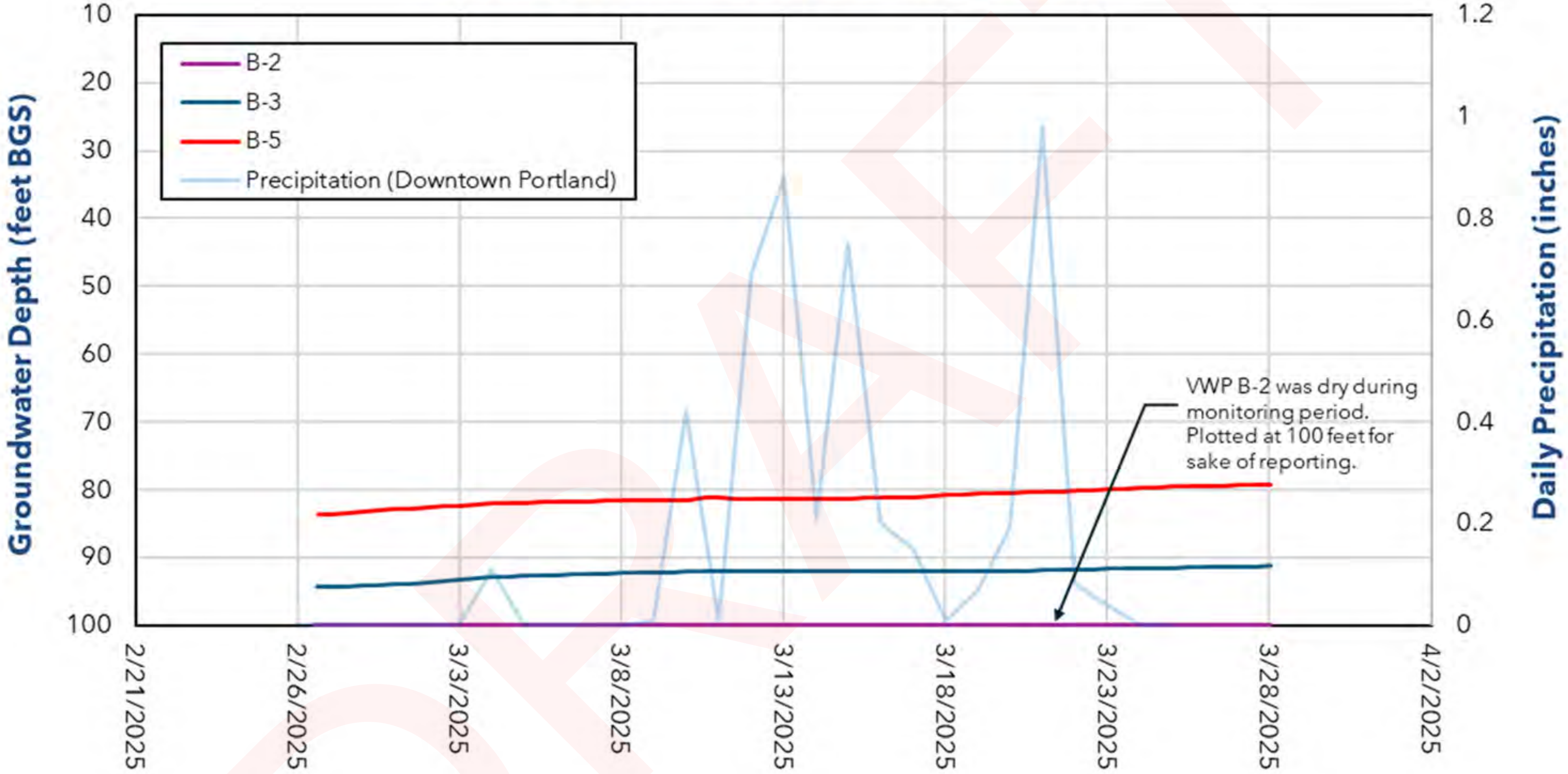
APRIL 2025

FIGURE
3

Results of Groundwater Monitoring in VWPs



VWP B-2 was dry during monitoring period. Plotted at 100 feet for sake of reporting.

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GROUNDWATER LEVEL MONITORING
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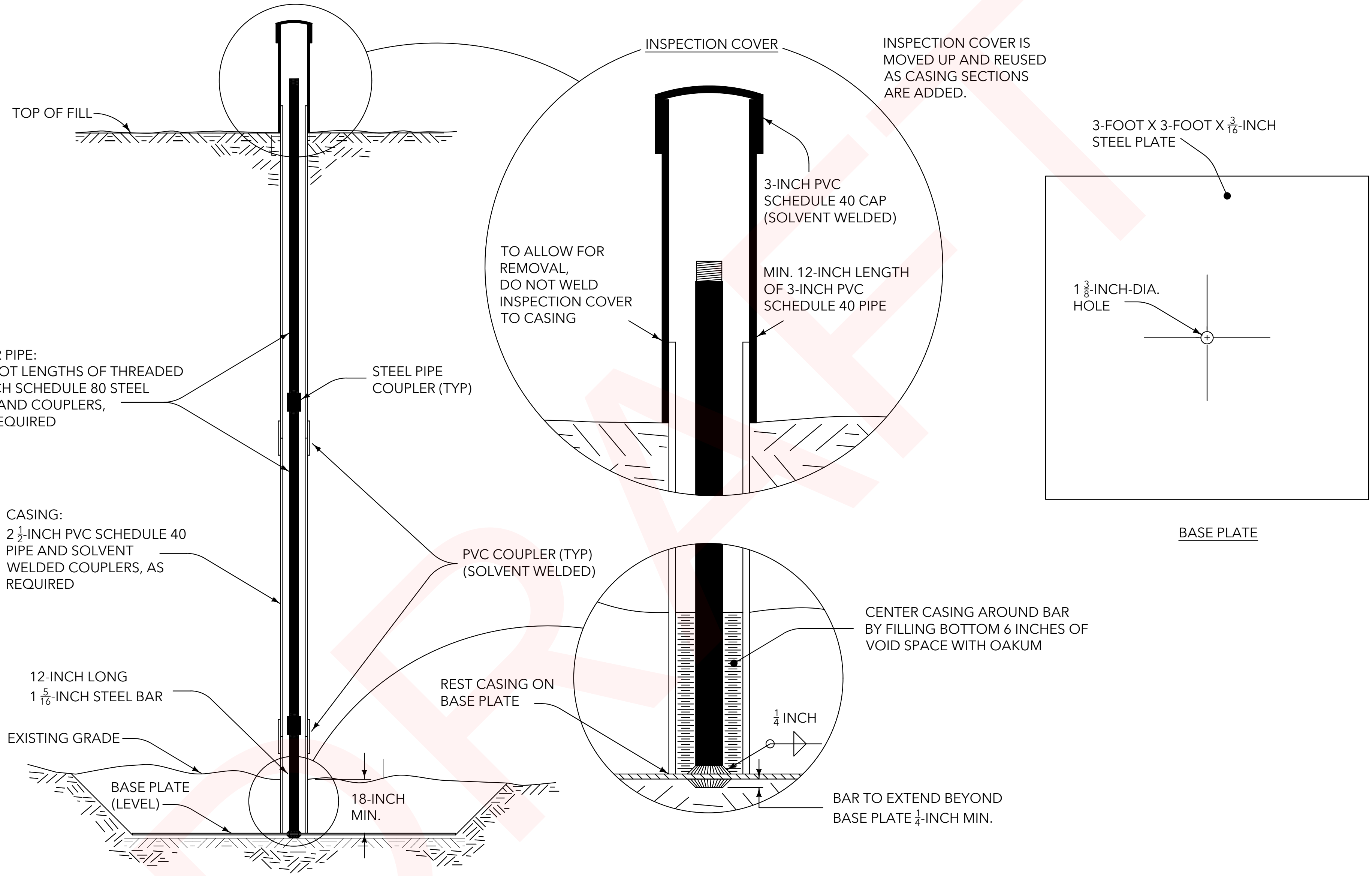
WEST LINN, OREGON

FIGURE
4



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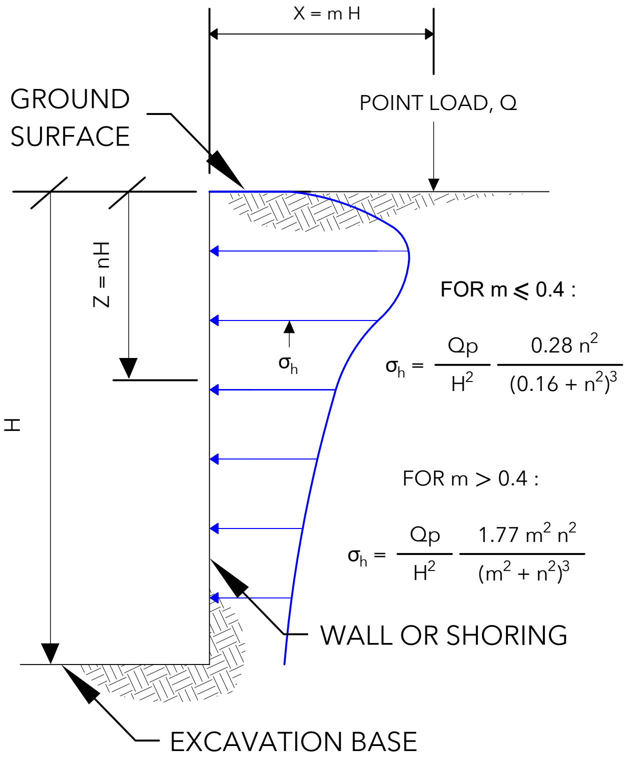
TYPICAL SETTLEMENT PLATE DETAIL

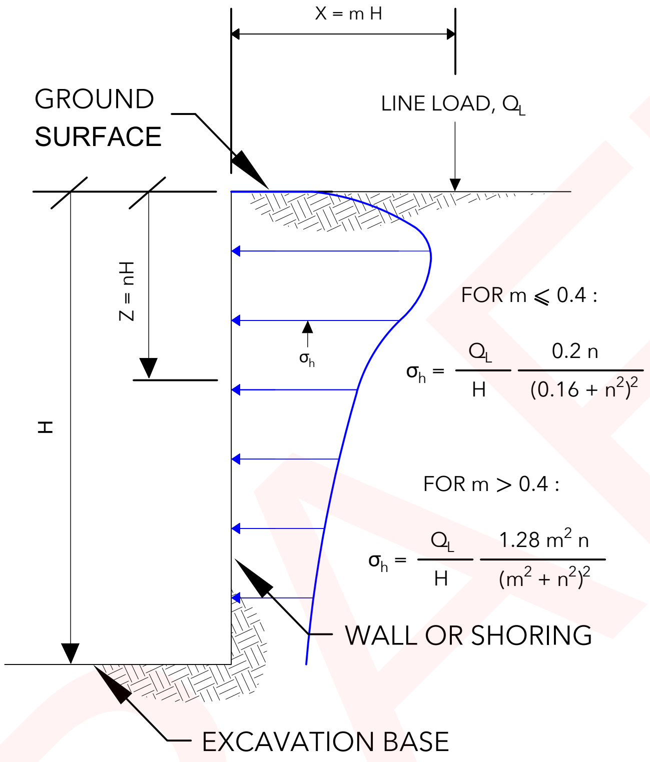
NOTES:
1. DRAWING IS NOT TO SCALE.

FIGURE
5

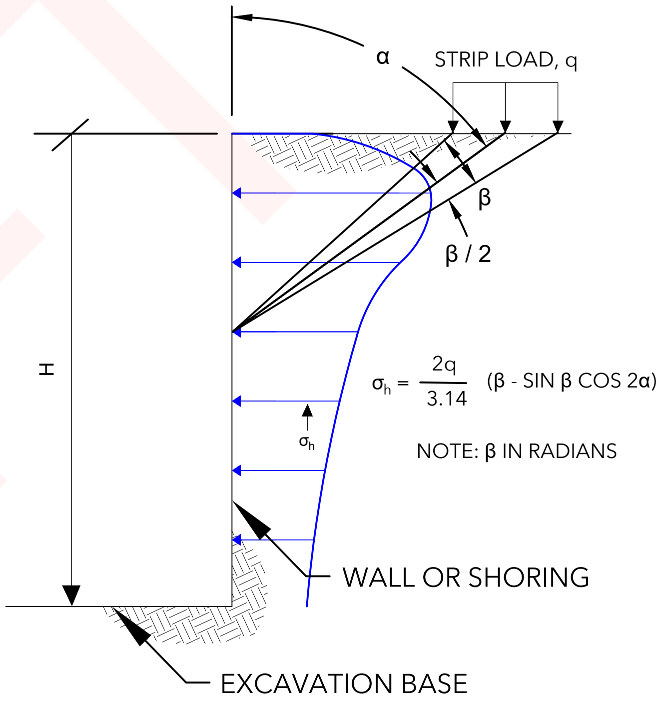
VERTICAL POINT LOAD



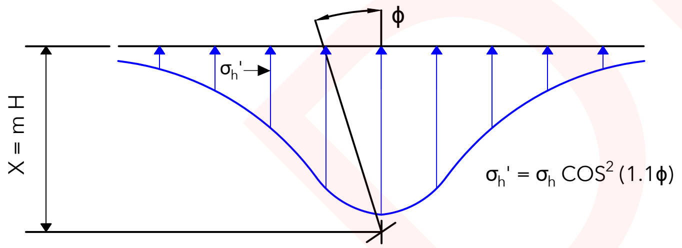
LINE LOAD PARALLEL TO WALL



STRIP LOAD PARALLEL TO WALL



VERTICAL POINT LOAD
HORIZONTAL PRESSURE DISTRIBUTION



NOTES:
1. FIGURE SHOULD BE USED JOINTLY WITH RECOMMENDATIONS PRESENTED IN THE REPORT TEXT.
2. LATERAL EARTH PRESSURES ASSUME RIGID WALLS WITH BACKFILL MATERIALS HAVING A POISSON'S RATIO OF 0.5.
3. TOTAL LATERAL EARTH PRESSURES RESULTING FROM COMBINED LOADS MAY BE CALCULATED USING SUPERPOSITION.
4. DRAWING IS NOT TO SCALE.

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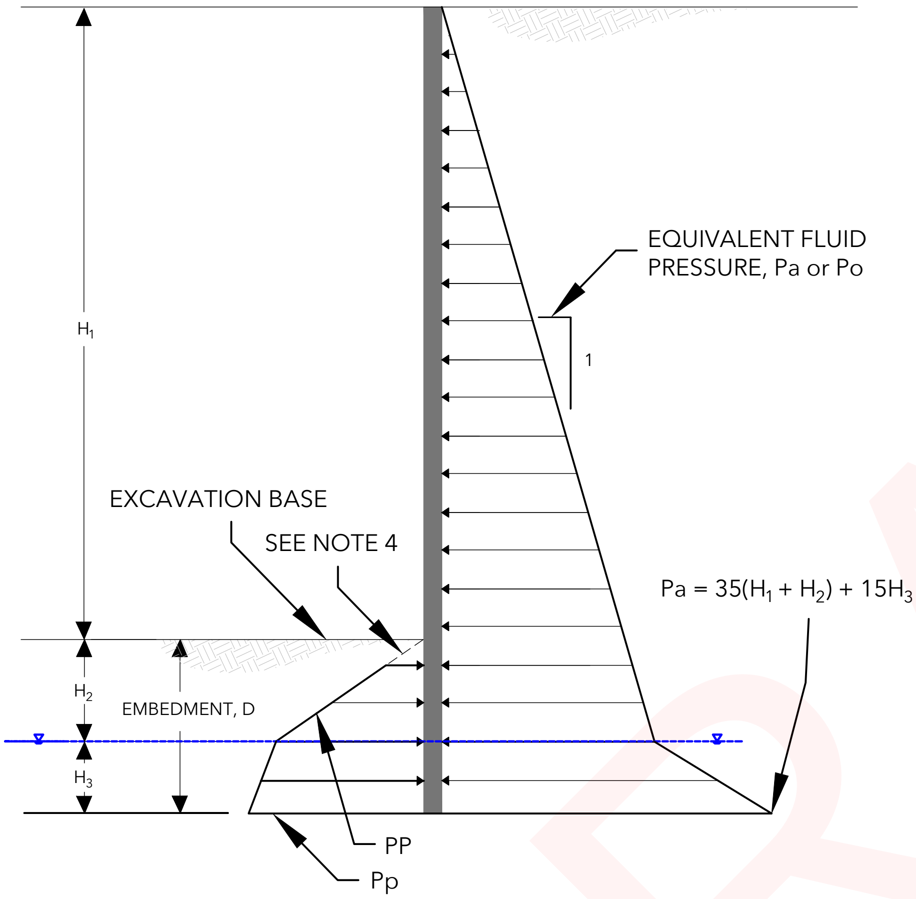
SURCHARGE-INDUCED LATERAL EARTH PRESSURES

FIGURE
6

RECOMMENDED DESIGN CRITERIA FOR CANTILEVERED WALLS

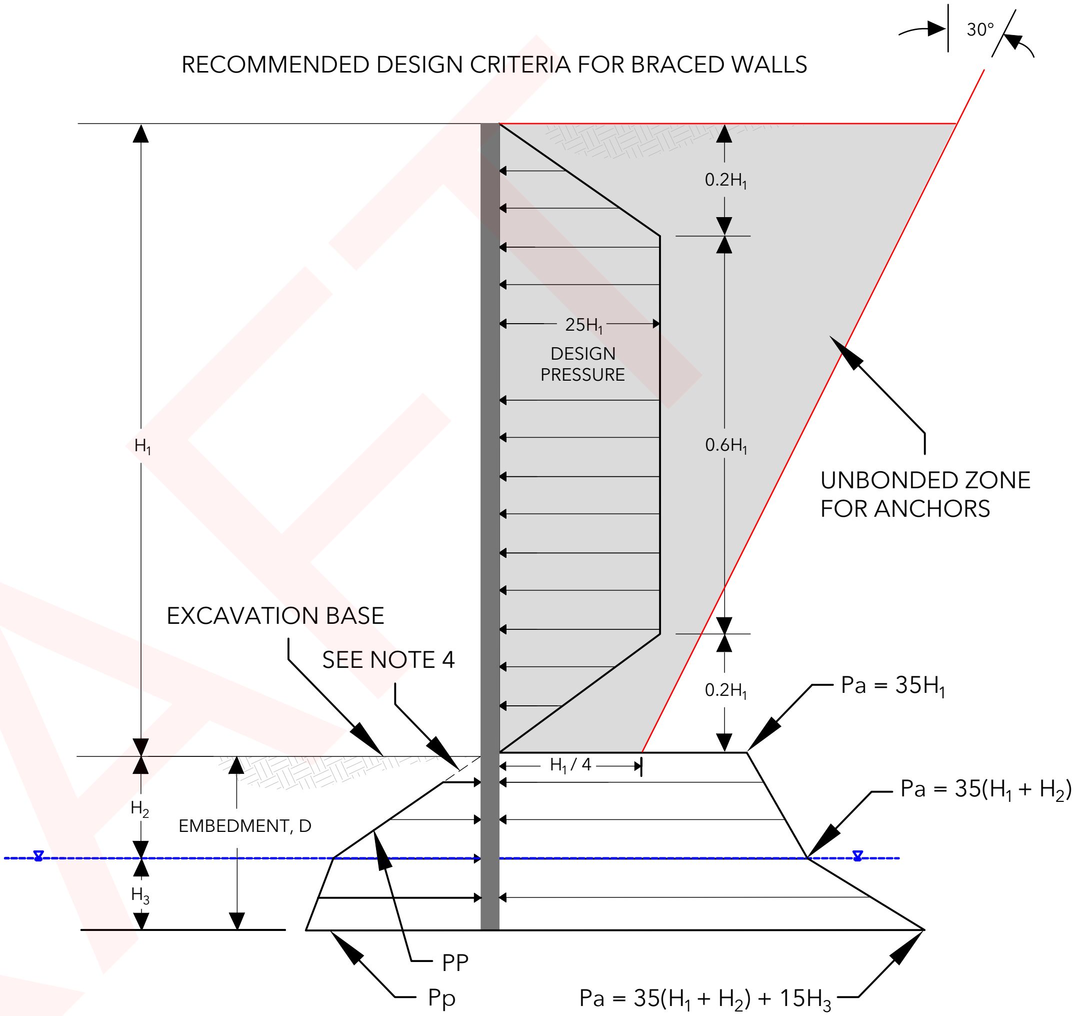


Pa = ACTIVE EQUIVALENT FLUID PRESSURE = 35 PCF (ASSUMES WALL IS FREE TO ROTATE)
Po = AT-REST EQUIVALENT FLUID PRESSURE = 55 PCF (ASSUMES WALL IS RESTRAINED FROM ROTATION)
PP = PASSIVE EQUIVALENT FLUID PRESSURE ABOVE GROUNDWATER TABLE = 500 PCF
Pp = PASSIVE EQUIVALENT FLUID PRESSURE BELOW GROUNDWATER TABLE = 290 PCF
H1 = EXPOSED HEIGHT OF SOLIDER PILE WALL
D = EMBEDMENT DEPTH IN FEET
PASSIVE PRESSURE RESISTANCE ACTS OVER 3X THE PILE WIDTH.
ACTIVE AND AT-REST EARTH PRESSURES ACT OVER 1X THE PILE WIDTH BELOW THE EXCAVATION BASE.

NOTES:

1. PRESSURE DISTRIBUTIONS DO NOT INCLUDE SEISMIC OR SURCHARGE LOADING.
2. LATERAL EARTH PRESSURES ASSUME THAT GROUNDWATER TABLE IS BELOW THE BASE OF THE EXCAVATION.
3. LATERAL EARTH PRESSURES ARE PRESENTED WITHOUT A FACTOR OF SAFETY.
4. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 24 INCHES BELOW THE EXCAVATION BASE.
5. DRAWING IS NOT TO SCALE.
6. REFER TO GEOTECHNICAL REPORT FOR APPROPRIATE GROUNDWATER ELEVATION.

RECOMMENDED DESIGN CRITERIA FOR BRACED WALLS



PP = PASSIVE EQUIVALENT FLUID PRESSURE ABOVE GROUNDWATER TABLE = 500 PCF
Pp = PASSIVE EQUIVALENT FLUID PRESSURE BELOW GROUNDWATER TABLE = 290 PCF
H1 = EXPOSED HEIGHT OF SOLIDER PILE WALL
D = EMBEDMENT DEPTH IN FEET
PASSIVE PRESSURE RESISTANCE ACTS OVER 3X THE PILE WIDTH.
ACTIVE AND AT-REST EARTH PRESSURES ACT OVER 1X THE PILE WIDTH BELOW THE EXCAVATION BASE.

NOTES:

1. PRESSURE DISTRIBUTIONS DO NOT INCLUDE SEISMIC OR SURCHARGE LOADING.
2. LATERAL EARTH PRESSURES ASSUME THAT GROUNDWATER TABLE IS BELOW THE BASE OF THE EXCAVATION.
3. LATERAL EARTH PRESSURES ARE PRESENTED WITHOUT A FACTOR OF SAFETY.
4. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 24 INCHES BELOW THE EXCAVATION BASE.
5. DRAWING IS NOT TO SCALE.
6. REFER TO GEOTECHNICAL REPORT FOR APPROPRIATE GROUNDWATER ELEVATION.

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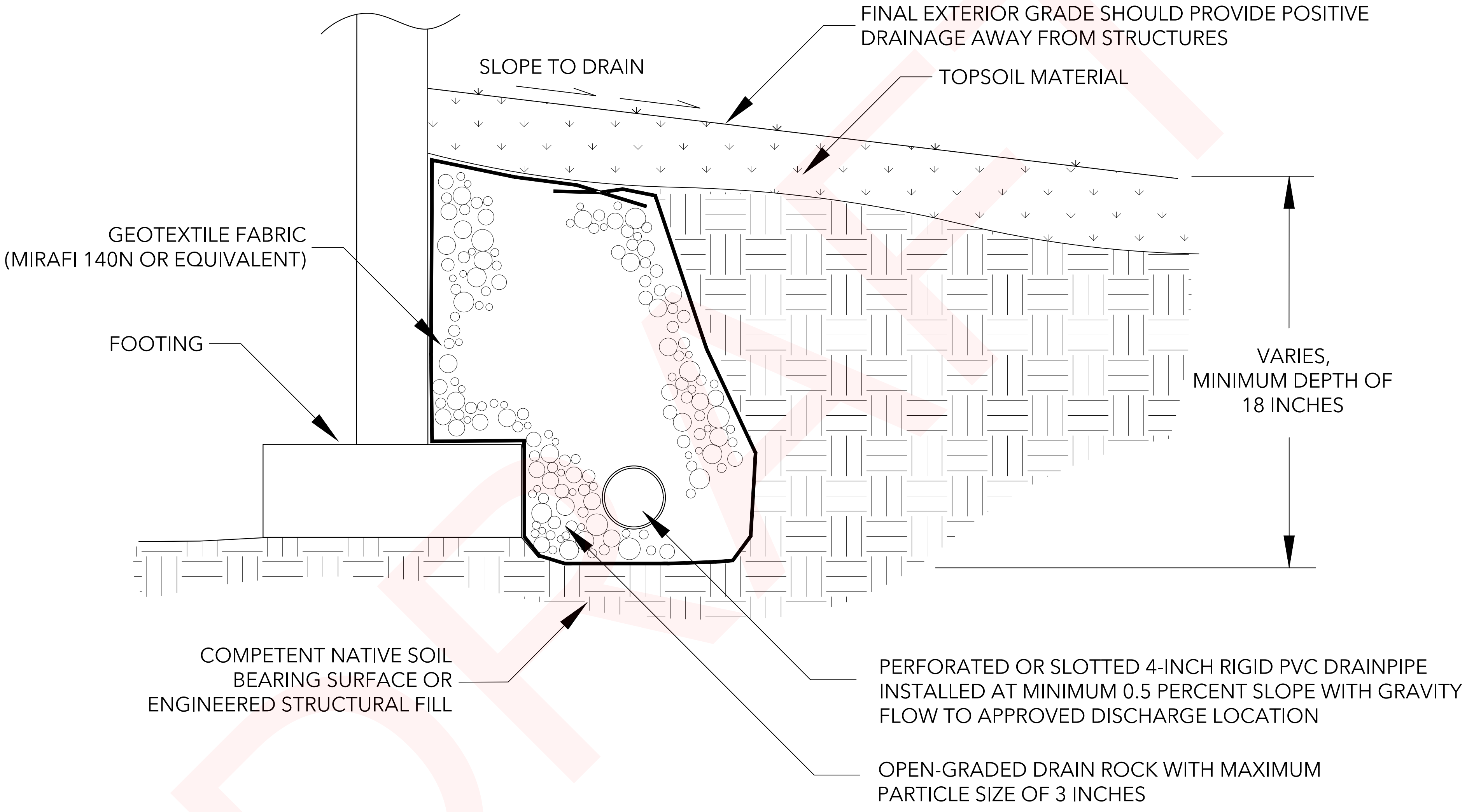
CANTILEVERED AND BRACED WALL DESIGN CRITERIA

FIGURE
7



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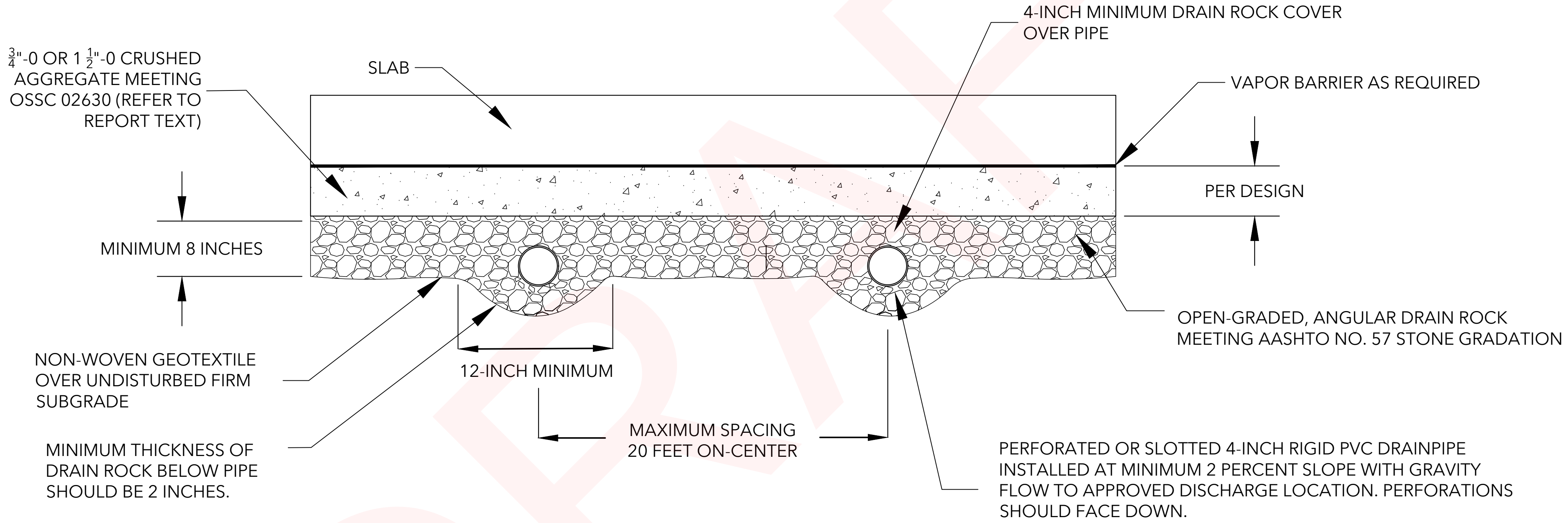
TYPICAL PERIMETER FOOTING DRAIN DETAIL

NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL PERIMETER FOOTING DRAIN DETAIL AND MAY NOT BE SITE-SPECIFIC.

FIGURE 8



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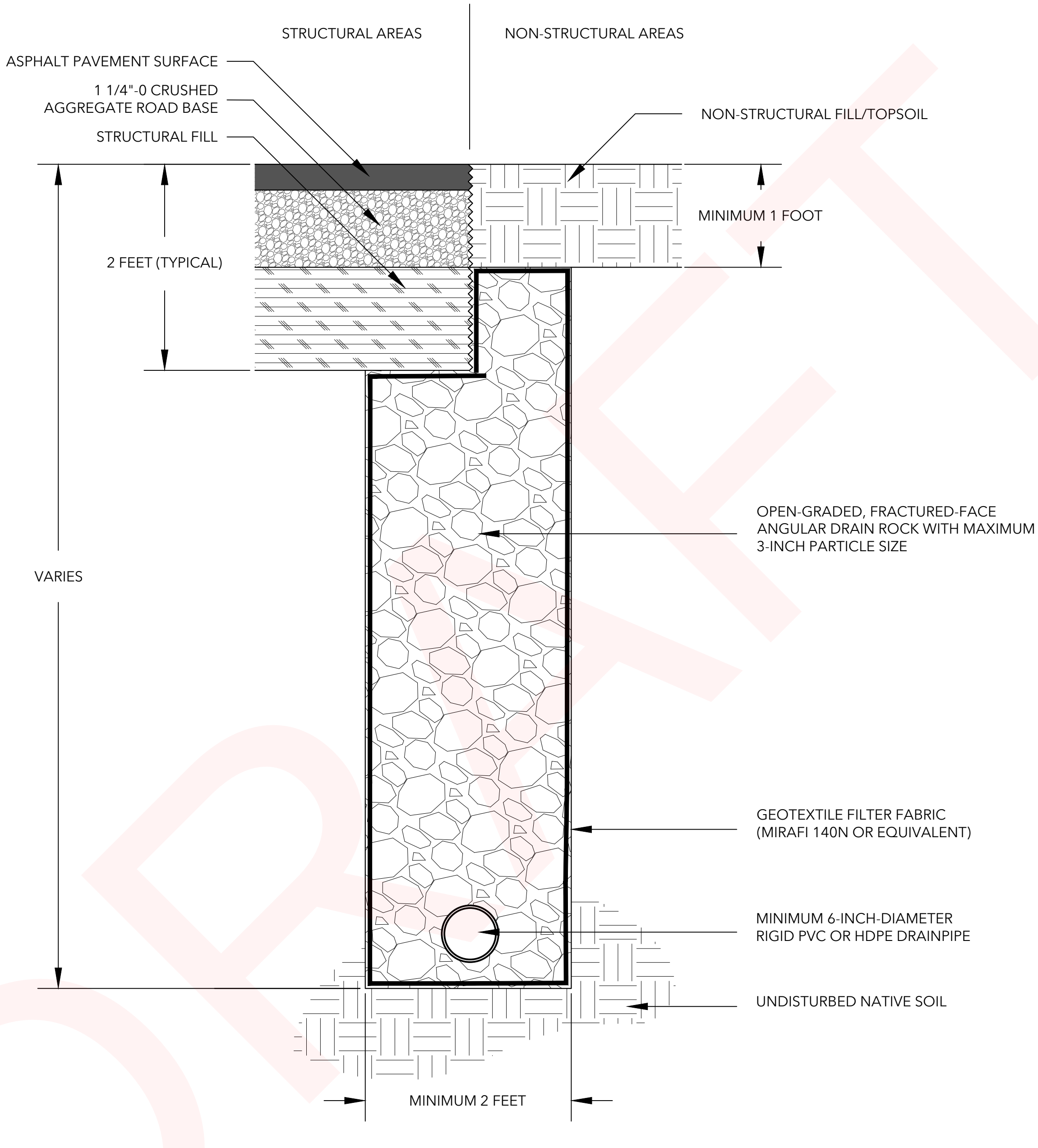
TYPICAL UNDERSLAB DRAIN DETAIL

NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL UNDERSLAB DRAIN DETAIL AND MAY NOT BE SITE-SPECIFIC.

FIGURE 9



NOTE: LOCATION, INVERT ELEVATION, DEPTH OF TRENCH, AND EXTENT OF PERFORATED PIPE REQUIRED MAY BE MODIFIED BY THE SITE GEOTECHNICAL ENGINEER DURING CONSTRUCTION BASED UPON FIELD OBSERVATION AND SITE-SPECIFIC SOIL CONDITIONS.

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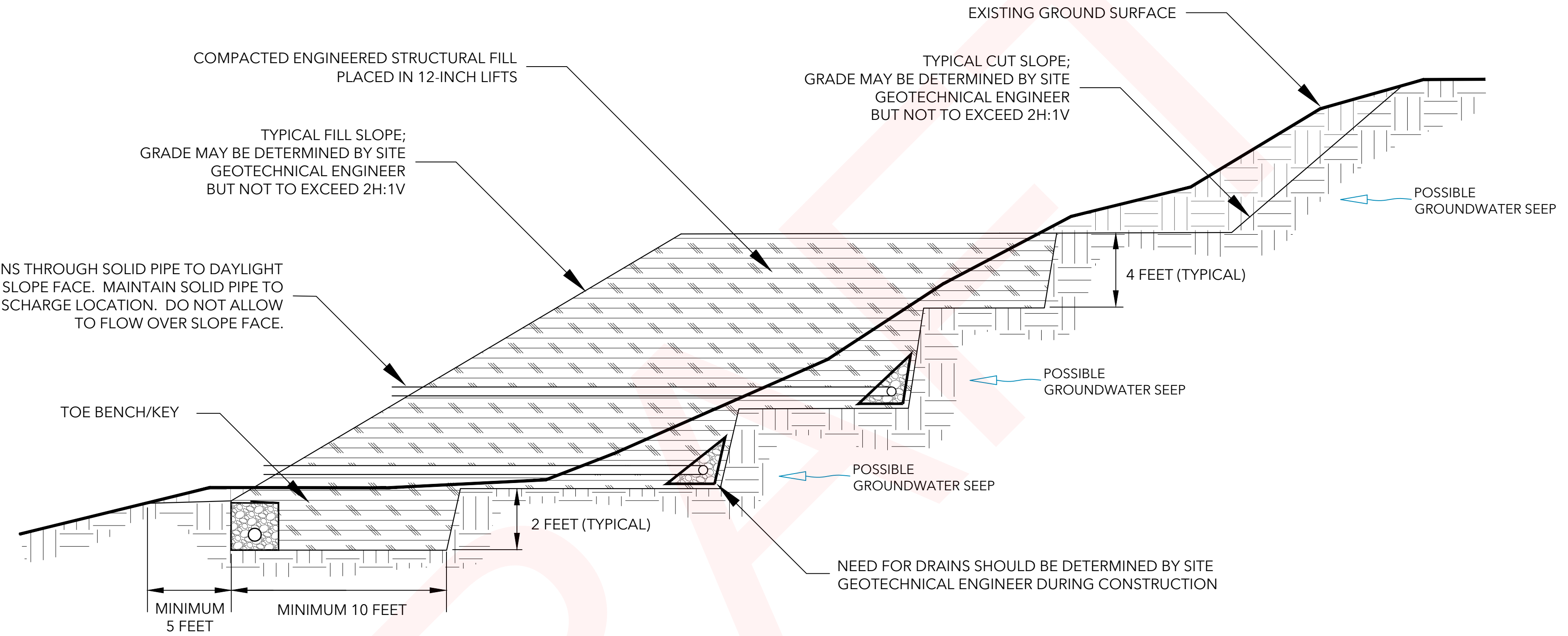
TYPICAL PERFORATED
DRAINPIPE TRENCH DETAIL

NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL PERFORATED DRAINPIPE TRENCH DETAIL AND MAY NOT BE SITE-SPECIFIC.

FIGURE
10

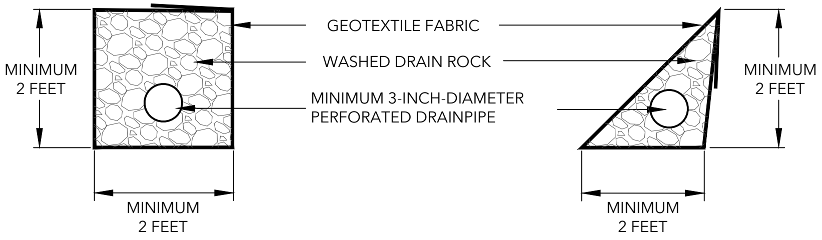


DRAIN SPECIFICATIONS

GEOTEXTILE FABRIC SHALL CONSIST OF MIRAFI 140N OR APPROVED EQUIVALENT WITH AOS BETWEEN U.S. STANDARD NO. 70 AND NO. 100 SIEVES.

WASHED DRAIN ROCK SHALL BE OPEN-GRADED ANGULAR DRAIN ROCK WITH LESS THAN 2 PERCENT PASSING THE U.S. STANDARD NO. 200 SIEVE AND A MAXIMUM PARTICLE SIZE OF 3 INCHES.

TYPICAL DRAIN SECTION DETAIL



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TYPICAL CUT AND FILL
SLOPE CROSS SECTION

NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL CUT AND FILL SLOPE CROSS SECTION AND MAY NOT BE SITE-SPECIFIC.

FIGURE
11

APPENDIX A

APPENDIX A
FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by drilling 5 borings (B-1 through B-5) to depths between 50 and 139 feet BGS and excavating 13 test pits (TP-1 through TP-13) to depths between 1.5 and 20.5 feet BGS. Drilling services were provided by Western States Soil Conservation, Inc., of Hubbard, Oregon, using a track-mounted drill rig and a combination of mud rotary and HQ wire line rock coring methods. Drilling was conducted between January 27 and February 7, 2025. The test pits were excavated by Dan J. Fischer Excavating, Inc., of Forest Grove, Oregon, using a Kobelco SK140SR trackhoe on January 23 and 24, 2025. The explorations were conducted under the supervision of Columbia West personnel. The exploration logs are presented in this appendix.

The exploration locations are shown on Figures 2 and 3. The explorations were located in the field by pacing from existing site features. This information should be considered accurate only to the degree implied by the methods used. A member of our staff observed the explorations.

SOIL AND ROCK SAMPLING

Soil samples were collected from the borings using a split-barrel SPT sampler in general accordance with ASTM D1586, Dames & Moore sampler in general accordance with ASTM D3550, or modified California sampler in general accordance with ASTM D3550. The samplers were driven a total distance of 18 inches with a 140-pound hammer free falling 30 inches. The number of blows required to drive the samplers the final 12 inches, or as otherwise indicated, into the soil is shown adjacent to the sample symbols on the boring log. Disturbed samples were collected from the samplers or subsequent classification and index testing. The average efficiency of the automatic SPT hammer used by Western States Soil Conservation, Inc. was 82.8 percent. The calibration testing results are presented at the end of this appendix. Sampling methods and intervals are shown on the exploration logs.

Rock cores were collected from the borings. Rock was continuously cored using HQ wire line rock coring methods in general accordance with ASTM D2113-99. Percent core recovery and RQD are noted on the boring logs. RQD is determined by summing the length of intact pieces of core longer than 100 mm (4 inches) and dividing by the length of the core advance.

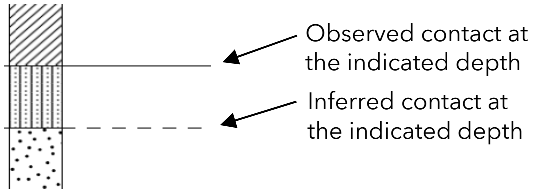
Representative disturbed samples of soil observed in the test pit explorations were collected from the test pit walls and base using the excavator's bucket. Sampling methods and intervals are shown on the exploration logs.

SOIL AND ROCK CLASSIFICATION

The soil and rock samples were classified in the field in accordance with the "Exploration Legend," "Soil Classification System," and "Rock Classification System," which are presented in this appendix. The exploration logs indicate the depths at which the soil and rock characteristics change, although the change could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

EXPLORATION LEGEND

SAMPLER TYPE	DESCRIPTION
SPT	Sample collected from the indicated depth in general accordance with ASTM D1586, *Standard Test Method Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils,* using an SPT sampler and 140-pound hammer
SH	Sample collected from the indicated depth in general accordance with ASTM D1587, *Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes,* using a thin-walled Shelby tube
D&M	Sample collected from the indicated depth in general accordance with ASTM D3550, *Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils,* using a Dames & Moore sampler and 140-pound hammer or pushed
CSS	Sample collected from the indicated depth in general accordance with ASTM D3550, *Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils,* using a 3-inch-outside diameter California split-spoon sampler and 140-pound hammer
DP	Sample collected from the indicated depth in general accordance with ASTM D6282, *Standard Guide for Direct Push Soil Sampling for Environmental Site Characterizations,* using a direct push soil sampler
GRAB	Grab sample collected from the indicated depth
CORE	Pavement or rock core interval at the indicated depth



GEOTECHNICAL ABBREVIATIONS

ATT	Atterberg limits	PP	Pocket penetrometer
CBR	California bearing ratio	P200	Percent passing No. 200 sieve
CON	Consolidation test	RES	Resilient modulus
DD	Dry density	SIEV	Sieve analysis
DS	Direct shear	TS	Torvane shear
HYD	Hydrometer	tsf	Tons per square foot
MC	Moisture content	UC	Unconfined compressive strength
MD	Moisture-density relationship	UU	Unconsolidated undrained triaxial test
NP	Non-plastic	VS	Vane shear
OC	Organic content	WD	Wet density

ENVIRONMENTAL ABBREVIATIONS

CA	Sample submitted for chemical analysis	ND	Not detected
		NS	No sheen
PID	Photoionization detector headspace analysis	SS	Slight sheen
		MS	Moderate sheen
ppm	Parts per million	HS	Heavy sheen

SOIL CLASSIFICATION SYSTEM

PARTICLE-SIZE CLASSIFICATION

COMPONENT	ASTM / USCS		AASHTO	
	Size Range	**Sieve Size Range**	**Size Range**	**Sieve Size Range**
Boulders	Greater than 300 mm	Greater than 12 inches	--	--
Cobbles	75 mm to 300 mm	3 inches to 12 inches	Greater than 75 mm	Greater than 3 inches
Gravel	75 mm to 4.75 mm	3 inches to No. 4 sieve	75 mm to 2.00 mm	3 inches to No. 10 sieve
Coarse	75 mm to 19.0 mm	3 inches to 3/4-inch sieve	--	--
Fine	19.0 mm to 4.75 mm	3/4-inch to No. 4 sieve	--	--
Sand	4.75 mm to 0.075 mm	No. 4 to No. 200 sieve	2.00 mm to 0.075 mm	No. 10 to No. 200 sieve
Coarse	4.75 mm to 2.00 mm	No. 4 to No. 10 sieve	2.00 mm to 0.425 mm	No. 10 to No. 40 sieve
Medium	2.00 mm to 0.425 mm	No. 10 to No. 40 sieve	--	--
Fine	0.425 mm to 0.075 mm	No. 40 to No. 200 sieve	0.425 mm to 0.075 mm	No. 40 to No. 200 sieve
Fines (Silt and Clay)	Less than 0.075 mm	Passing No. 200 sieve	Less than 0.075 mm	Passing No. 200 sieve

CONSISTENCY FOR COHESIVE SOIL

CONSISTENCY	SPT N-VALUE (blows per foot)	D&M N-VALUE (blows per foot)	POCKET PENETROMETER (unconfined compressive strength [tsf])
Very soft	0 to 2	0 to 3	Less than 0.25
Soft	2 to 4	3 to 6	0.25 to 0.5
Medium stiff	4 to 8	6 to 12	0.5 to 1.0
Stiff	8 to 15	12 to 25	1.0 to 2.0
Very stiff	15 to 30	25 to 65	2.0 to 4.0
Hard	Greater than 30	Greater than 30	Greater than 4.0

RELATIVE DENSITY FOR GRANULAR SOIL

RELATIVE DENSITY	SPT N-VALUE (blows per foot)	D&M N-VALUE (blows per foot)
Very loose	0 to 4	0 to 11
Loose	4 to 10	11 to 26
Medium dense	10 to 30	26 to 74
Dense	30 to 50	74 to 120
Very dense	Greater than 50	Greater than 120

MOISTURE DESIGNATIONS

TERM	FIELD IDENTIFICATION
Dry	Very low moisture, dry to touch
Moist	Damp, color appears darkened, without visible moisture, cohesive soil will clump, sand will bulk
Wet	Visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Percent	SILT AND CLAY IN		Percent	SAND AND GRAVEL IN		Percent	SECONDARY MATERIAL
	Fine-Grained Soil	**Coarse-Grained Soil**		**Fine-Grained Soil**	**Coarse-Grained Soil**		**Organics and Man-Made Debris**
< 5	trace	trace	< 5	trace	trace	< 4	trace
5 – 12	minor	with	5 – 15	minor	minor	4 – 12	some
> 12	some	silty/clayey	15 – 30	with	with		
			> 30	sandy/gravelly	with		

ROCK CLASSIFICATION SYSTEM

STRENGTH	DESCRIPTION	UNCONFINED COMPRESSIVE STRENGTH (psi)
Extremely weak (R0)	Easily indented by thumbnail	35 to 150
Very weak (R1)	Scratched with fingernail, peeled by knife, indented by rock pick	150 to 725
Weak (R2)	Peeled by knife, indented by rock pick	725 to 3,500
Medium strong (R3)	Cannot be peeled or scraped with a knife	3,500 to 7,250
Strong (R4)	Requires more than one blow with a rock hammer to fracture it	7,250 to 14,500
Very strong (R5)	Requires many blows with a rock hammer to fracture it	14,500 to 36,250
Extremely strong (R6)	Can only be chipped with a rock hammer	Greater than 36,250

WEATHERING	DESCRIPTION
Decomposed	A soil formed in place with original texture of rock destroyed
Completely weathered	Rock wholly weathered but rock texture preserved
Highly weathered	Rock weakened so that large pieces can be broken by hand
Moderately weathered	Rock mass is decomposed locally
Slightly weathered	Discoloration along discontinuities
Fresh	No visible signs of weathering or discoloring

JOINT SPACING	DESCRIPTION
Very close	Less than 0.2 foot
Close	0.2 foot to 1 foot
Moderately 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, fragments, with scattered short core lengths
Intensely fractured	0.1 foot to 0.3 foot with scattered fragments
Moderately fractured	0.3 foot to 1 foot
Slightly fractured	1 foot to 3 feet
Very slightly fractured	Greater than 3 feet
Unfractured	No fractures observed

HEALING	DESCRIPTION
Not healed	Discontinued surface, fractured zone, sheared material, filling is not cemented
Partly healed	Less than 50% of fractures or sheared zone bonding
Moderately healed	Greater than 50% fractures or sheared zone bonding
Totally healed	All fragments are bonded

QUALITY	RQD (percent)
Very poor	Less than 25
Poor	25 to 50
Fair	51 to 75
Good	76 to 90

Rock quality designation (RQD) is a measure of quality of rock core taken from a borehole. The length of core pieces is measured along the center line of the pieces. All pieces of intact rock core equal to or greater than 100 millimeters (4 inches) long are summed and divided by the total length of the core run to obtain RQD value

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BORING NUMBER: B - 1

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 02/05/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 4:13 PM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 02/07/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 5:00 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
		▩	Soft, brown sandy SILT, organics, moist (12 inches of topsoil, 4-inch-thick root zone). 1.0	430							
	ML	▤	Medium stiff, brown-orange sandy SILT, moist, low plasticity, sand is fine to medium.		SPT	12	B1.1		2-3-3 (6)		
5			Stiff, red-brown-orange with white and black mottles at 5 feet.	425	SPT	18	B1.2		5-7-6 (13)		
			Brown-orange-gray, trace gravel, gravel is fractured at 7.5 feet.		SPT	18	B1.3		3-5-9 (14)		Drill chatter at 9 feet.
10			Hard, with gravel, gravel is fine and angular (decomposed basalt) at 11 feet. 11.5	420	SPT	9	B1.4		18-12-50/1"		
					SPT	1	B1.5		50/1"		
		⬡	Weak to medium strong (R2-R3), brown-gray BASALT, highly to moderately weathered, very instensely to intensely fractured, not healed, moist. 12.0								Switched to rock coring at 12 feet. Brown return water from 13.5 to 42 feet.
15			Strong (R4), gray BASALT, slightly weathered, intensely to moderately fractured (boulder!). 13.5	415	Core	48	BC1.1	23			
			Weak to medium strong (R2-R3), brown-gray BASALT, highly to moderately weathered, very instensely to intensely fractured, not healed, moist.		Core	30	BC1.2	0			

Water Levels

▽ Unable to observe

▼ -

BORING NUMBER: B - 1

PROJECT NAME	Operations Center	**CLIENT**	City of West Linn
PROJECT NUMBER	WestL-1-01-1	**PROJECT LOCATION**	West Linn, Oregon
DRILLING CONTRACTOR	Western States Soil Conservation, Inc.	**DATE STARTED**	02/05/2025
DRILLING METHOD	Mud Rotary and Rock Coring	**TIME STARTED**	4:13 PM
EQUIPMENT	CME-850, Rig 7	**DATE COMPLETED**	02/07/2025
BORING DIAMETER	4.875 inches	**TIME COMPLETED**	5:00 PM
HAMMER EFFICIENCY	82.8%	**LOGGED BY**	E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page)								
25				405	Core	30	BC1.2	0			
30				400	Core	48	BC1.4	0			
35				395	Core	48	BC1.5	0			
					Core	46	BC1.6	19			

Water Levels

▽ Unable to observe

▼ -

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Columbia West
Engineering, Inc

BORING NUMBER: B - 1

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 02/05/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 4:13 PM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 02/07/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 5:00 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page)								
45				385	Core	58	BC1.7	26			
			48.8								
			Strong (R4), gray BASALT, slightly weathered, slightly fractured. 50.0		Core	57	BC1.8	33			
50			Weak to medium strong (R2-R3), brown-gray BASALT, highly to moderately weathered, very instensely to intensely fractured, not healed, moist. 52.0	380							
			Very weak to weak (R1-R2), brown-gray BASALT, decomposed to moderately weathered, very instensely to intensely fractured, vesicular.		Core	36	BC1.9	0			Lost circulation at 54 feet.
55				375							
			Weak to medium strong (R2-R3), intensely fractured, vesicular at 57 feet.		Core	59	BC1.10	39			

Water Levels

▽ Unable to observe

▼ -

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BORING NUMBER: B - 1

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 02/05/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 4:13 PM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 02/07/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 5:00 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page)								
			Extremely weak to weak (R0-R2), decomposed to moderately weathered, very intensely to intensely fractured, less vesicular at 62 feet.		Core	18	BC1.11	20			
65				365							
70			Extremely weak to medium strong (R0-R3), brown-gray, decomposed to moderately fractured at 69 feet.	360	Core	48	BC1.12	28			Actual end of sample C1.11 at 69 feet. Driller commented he might have over drilled first couple of cores.
75			Extremely weak to weak (R0-R2), brown, very intensely to intensely fractured at 74 feet. CLAY bed (3 inches thick) at 75.5 feet.	355	Core	48	BC1.13	0			
			CLAY bed (3 inches thick) at 77.7 feet.		Core	54	BC1.14	0			

Water Levels

▽ Unable to observe

▼ -

BORING NUMBER: B - 1

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 02/05/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 4:13 PM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 02/07/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 5:00 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			CLAY bed (3 inches thick) at 80.2 feet.								Clay has parallel staining, native soil structure, and uniform grain size, so likely not a product of grinding rock during coring.
85			Very weak to medium strong (R1-R3), moderately weathered at 84 feet.	345	Core	50	BC1.15	20			
90			Weak to strong (R2-R4), moderately to slightly weathered, very intensely to slightly fractured at 89 feet.	340	Core	60	BC1.16	27			
95			Medium strong to strong (R3-R4), very intensely to slightly fractured at 94 feet.	335	Core	61	BC1.17	36			
			Strong (R4), gray, intensely to moderately fractured at 99 feet.		Core	60	BC1.18	63			

Water Levels

∇ Unable to observe

▼ -

BORING NUMBER: B - 1

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 02/05/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 4:13 PM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 02/07/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 5:00 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page)								
			104.0								
105			Extremely weak to weak (R0-R2), red-brown to gray-brown BASALT, decomposed to moderately weathered, intensely to moderately fractured, vesicular. CLAY layer (paleosol) from 104.4 to 105.4 feet.	325	Core	59	BC1.19	58			
			106.5								
110			Medium strong (R3), brown-gray BASALT, moderately to slightly weathered, moderately fractured, vesicular. No vesicles at 111 feet.	320	Core	60	BC1.20	70			
115			Medium strong to strong (R3-R4), gray, slightly weathered at 115 feet.	315	Core	60	BC1.21	59			
					Core	60	BC1.22	38			

Water Levels

⊻ Unable to observe

▼ -

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BORING NUMBER: B - 1

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 02/05/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 4:13 PM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 02/07/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 5:00 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page)								
			123.5								
			Weak to medium strong (R2-R3), red to gray BASALT, completely to moderately weathered, intensely fractured, vesicular.								Yellow microfracture where basalt is moist, red, and weathered
125			125.0	305							
			Medium strong (R3), gray to brown-gray BASALT, moderately weathered, moderately to slightly fractured, vesicular.		Core	60	BC1.23	19			
130				300							
			CLAY layer (1 inch thick) at 131.2 feet.		Core	61	BC1.24	43			
			Medium strong to strong (R3-R4), large vesicles and vugs, oriented transverse of vesicles at 5 to 20 degrees at 132 feet.								
135				295	Core	60	BC1.25	63			
			139.0								

Boring completed at 139 feet.

Water Levels

▽ Unable to observe

▼ -

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BORING NUMBER: B - 2

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/31/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 11:35 AM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 12/05/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 12:45 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
		▤	Soft, brown sandy SILT, trace organics, moist (12 inches of topsoil, 6-inch-thick root zone) - **FILL**. 1.0	345							
		▨	Loose, brown-gray-orange silty GRAVEL with sand, moist, gravel is fine to coarse, sand is fine to coarse - **FILL**.		SPT	6	B2.1		4-4-5 (9)	41	
5		▨	Medium dense at 5 feet.	340	SPT	8	B2.2		9-7-7 (14)		
		▨			SPT	3	B2.3		6-6-6 (12)		
10		▨	Very loose at 10 feet.	335	D&M	6	B2.4		4-5-4 (9)		
15		▨	Medium dense at 15 feet.	330	CSS	13	B2.5		10-24-22 (46)	18	Driller Comment: Lost circulation at 15 feet. Lost 130 gallons from 15 to 20 feet.
		▨									Drill grinding at 19.5 feet.

Water Levels

▽ Unable to observe

▼ -

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

BORING NUMBER: B - 2

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/31/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 11:35 AM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 12/05/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 12:45 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
		▩	Very dense, gray-brown at 20 feet.		SPT	0.5	B2.6		50/3"		Slow drilling from 20 to 23 feet. Possible nested cobbles and boulders. Driller Comment: Lost circulation at 22 feet. Lost 130 gallons of mud from 20 to 22 feet. Drill chatter at 24 feet.
25		▩	25.0 Medium dense, brown-gray-orange SAND with silt and gravel, moist, sand is fine to coarse, gravel is fine to coarse - **FILL**.	320	SPT	10	B2.7		9-12-15 (27)	24	
30		▩	29.0 Medium dense, gray-brown GRAVEL with sand, trace silt, moist, gravel is fine to coarse, sand is fine to coarse - **FILL**.	315	D&M	14	B2.8		27-30-120 (150)	43	
	ML	⋮	31.0 Hard, brown-orange-gray SILT with sand, moist, low plasticity, sand is fine to coarse (decomposed basalt).								Native relict soil; looks like interflow volcanic breccia at 31 feet.
35	CL/CH	▨	35.0 Hard, orange-brown sandy CLAY, moist, medium plasticity, sand is fine to medium (decomposed basalt).	310	SPT	8	B2.9		41-50/2"	55	Lost 65 gallons of mud from 35 to 40 feet.

Water Levels

⊻ Unable to observe

▼ -

BORING NUMBER: B - 2

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/31/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 11:35 AM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 12/05/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 12:45 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
	SP-SC		Very dense, brown-orange SAND with clay, moist, sand is fine to medium (decomposed basalt). 44.0		SPT	4	B2.10		50/4"		
45			Medium strong to strong (R3-R4), gray-brown BASALT, moderately to slightly weathered, intensely to moderately fractured, not healed, vesicular, moist.	300	SPT	0	B2.11		50/1"		Grinding/drill chatter at 44 feet. Switched to rock coring at 46 feet.
					Core	23	BC2.1	34			
50			Strong (R4), gray, slightly weathered, moderately fractured at 48 feet. Large vesicles and vugs, oriented transverse of vesicles at 10 to 40 degrees at 50 feet.	295	Core	62	BC2.2	66			
55			Intensely to moderately fractured at 53 feet. No vesicles at 56.5 feet. High-angle joints healed and not healed from 57 to 61 feet.	290	Core	60	BC2.3	55			
					Core	59	BC2.4	46			

Water Levels

▽ Unable to observe

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BORING NUMBER: B - 2

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/31/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 11:35 AM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 12/05/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 12:45 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page)								
65				280	Core	62	BC2.5	33			
			Vesicular injection with non-vesicular basalt at 75 degrees from 66 to 68 feet.								
			Moderate to slightly fractured, no vesicles at 68 feet.								
70				275	Core	60	BC2.6	66			
			Slightly fractured at 73 feet.								
75				270	Core	60	BC2.7	100			
			Moderately to slightly weathered, moderately to slightly fractured at 78 feet.		Core	59	BC2.8	66			

Water Levels

⊻ Unable to observe

▼ -

BORING NUMBER: B - 2

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/31/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 11:35 AM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 12/05/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 12:45 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page) Intensely fractured at 82 feet.								
			82.5								
			Weak (R2), gray-brown to red-brown BASALT, moderately to slightly weathered, intensely to moderately fractured, vesicular.								
85				260	Core	61	BC2.9	52			
			Gray, moderately to slightly fractured at 88 feet.								
90				255	Core	61	BC2.10	80			
			Medium strong to strong (R3-R4), [other than clay layer], gray-brown, moderately to slightly weathered at 93 feet. CLAY layer, low plasticity (0.4-foot thick) at 93.3 feet.								
95				250	Core	60	BC2.11	68			Yellow return water at 96 feet.
					Core	60	BC2.12	7			Vibrating wire piezometer 2312663 installed at 98 feet.

Water Levels

▽ Unable to observe

▼ -

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/31/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 11:35 AM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 12/05/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 12:45 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page)								
105			Medium strong (R3), brown-gray to yellow-brown, vesicular (weathered bottom of flow) at 103.5 feet. 104.7 Weak to medium strong (R2-R3), brown-gray BASALT, moderately weathered, intensely to moderately fractured, vesicular with filled in vesicles (clay mineralization).	240	Core	55	BC2.13	50			Yellow return water from 104.5 to 105.5 feet.
110			Medium strong to strong (R3-R4), gray, slightly weathered, moderately fractured, vesicular with open vesicles at 108 feet.	235	Core	60	BC2.14	19			
115			No vesicles at 116 feet.	230	Core	60	BC2.15	53			
			Medium strong to strong (R3-R4), moderately to slightly weathered, moderately jointed at 118 feet.		Core	60	BC2.16	45			

Water Levels

Unable to observe

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BORING NUMBER: B - 2

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/31/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 11:35 AM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 12/05/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 12:45 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
		⬡	(continued from previous page) 123.0								

Boring completed at 123 feet.

Water Levels

⊻ Unable to observe

▼ -

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

BORING NUMBER: B - 3

Page 1 of 6

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/27/2025
DRILLING METHOD Hollow-Stem Auger and Rock Coring
TIME STARTED 9:17 AM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 01/29/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 4:32 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			Bark Chips (approximatley 24 inches) - **FILL**. 2.0	315							
	GM		Very dense, brown-gray silty GRAVEL with sand, moist, gravel is fine to coarse, sand is fine to coarse (decomposed basalt). 4.0		SPT	5	B3.1		50/5"	21	
			Very weak to medium strong (R1-R3), brown-gray BASALT, completely to highly weathered, very instensely to intensely fractured, not healed, moist.		SPT	1	B3.2		50/1"		Switched to rock coring at 4 feet.
5				310							
					Core	48	BC3.1	23			
			Gray at 9 feet.								
10				305	Core	24	BC3.2	0			
					Core	56	BC3.3	0			
15				300							
			Weak to medium strong (R2-R3), gray-brown, slightly to moderately weathered, very intensely fractured at 16 feet.		Core	60	BC3.4	0			Intensity of fracturing due to sub-vertical columnar of joints.

Water Levels

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PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/27/2025
DRILLING METHOD Hollow-Stem Auger and Rock Coring
TIME STARTED 9:17 AM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 01/29/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 4:32 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page)								
			Medium strong (R3) at 22 feet.		Core	60	BC3.5	17			
25				290							
			Very strong (R5), gray, moderately fractured, 5 to 20 degrees, alteration around fractures at 26 feet.		Core	59.1	BC3.6	73			UC = 16,150 psi
30				285							
			Very weak to weak (R1-R2), red-brown-gray-orange, completely to highly weathered, very intensely to intensely, vesicular at 31 feet.		Core	50	BC3.7	22			Rock loss at 31 feet due to increased weathering.
35				280							
			Weak (R2), red-brown, highly weathered, intensely to moderately fractured at 36 feet.		Core	59	BC3.8	28			

Water Levels

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BORING NUMBER: B-3

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/27/2025
DRILLING METHOD Hollow-Stem Auger and Rock Coring
TIME STARTED 9:17 AM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 01/29/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 4:32 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page)								
45			Medium strong (R3), gray, moderately to slightly weathered, moderately to slightly fractured, 15 to 20 degrees, mineralized surfaces, 80 percent clay infill (partially healed), columnar joint, sub-vertical (70 to 80 degrees) at 41 feet.	270	Core	60	BC3.9	72			
50			Medium strong to strong (R3-R4), moderately fractured, not healed at 46 feet.	265	Core	60	BC3.10	39			
55			No vesicles at 50.5 feet. Slightly weathered, moderately fractured at 51 feet. Columnar joint from 51.9 to 53.8 feet.	260	Core	57	BC3.11	89			
			Moderately jointed at 56 feet.		Core	63	BC3.12	27			

Water Levels

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BORING NUMBER: B - 3

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/27/2025
DRILLING METHOD Hollow-Stem Auger and Rock Coring
TIME STARTED 9:17 AM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 01/29/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 4:32 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page) 61.0								
65			Weak to medium strong (R2-R3), brown to brown-gray BASALT, intensely to slightly weathered, moderately to slightly fractured, vesicular.	250	Core	60	BC3.13	42			
70			Medium strong to strong (R3-R4), gray to orange, moderately to slightly weathered, intensely to moderately fractured at 67 feet. Medium strong (R3) at 69 feet. 71.0	245	Core	60	BC3.14	18			
75			Weak to medium strong (R2-R3), brown to brown-gray BASALT, highly to moderately weathered, very instensely to intensely fractured, vesicular, jointed (60 to 80 degrees). No vesicles, several joints with mineralized fill, not healed, (60 to 90 degrees), columnar at 73 feet.	240	Core	60	BC3.15	22			
			Gray at 76 feet.		Core	60	BC3.16	55			

Water Levels

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Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

BORING NUMBER: B - 3

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/27/2025
DRILLING METHOD Hollow-Stem Auger and Rock Coring
TIME STARTED 9:17 AM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 01/29/2025
BORING DIAMETER 4.875 inches
TIME COMPLETED 4:32 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
85			Medium strong (R3), brown BASALT, moderately weathered, very instensely to intensely fractured, vesicular. 85.5	230	Core	57	BC3.17	32			
90			Extremely weak to very weak (R0-R1), gray to blue-gray and yellow BASALT, decomposed to completely weathered, very intensely fractured, vesicular (where not decomposed). CLAY filling from 86.5 to 87 feet. Weak (R2), brown-gray, highly to moderately weathered, intensely fractured at 88 feet. Medium strong (R3), gray, moderately to slightly weathered, intensely to moderately fractured at 89 feet. No vesicles at 91 feet.	225	Core	60	BC3.18	7		50	
95				220	Core	60	BC3.19	48			
			Red-brown, moderately weathered, vesicular at 98.5 feet.		Core	36	BC3.20	0			Vibrating wire piezometer 2449284 installed at 97.5 feet.

Water Levels

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Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West Engineering, Inc

BORING NUMBER: B - 3

PROJECT NAME Operations Center

CLIENT City of West Linn

PROJECT NUMBER WestL-1-01-1

PROJECT LOCATION West Linn, Oregon

DRILLING CONTRACTOR Western States Soil Conservation, Inc.

DATE STARTED 01/27/2025

DRILLING METHOD Hollow-Stem Auger and Rock Coring

TIME STARTED 9:17 AM

EQUIPMENT CME-850, Rig 7

DATE COMPLETED 01/29/2025

BORING DIAMETER 4.875 inches

TIME COMPLETED 4:32 PM

HAMMER EFFICIENCY 82.8%

LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
		⬡	Gray-brown, slightly weathered at 100 feet.		Core	9	BC3.21	0			

101.0

Boring terminated at 101 feet due to drill shake.

Water Levels

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BORING NUMBER: B-4

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/29/2025
DRILLING METHOD Hollow-Stem Auger and Rock Coring
TIME STARTED 4:08 PM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 01/29/2025
BORING DIAMETER 4.25 inches
TIME COMPLETED 1:30 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft) 303.7	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
		▩	Bark Chips (approximatley 18 inches) - **FILL**. 1.5								
	SP-SM	▒	Very dense, brown-gray SAND with silt, moist, sand is fine to medium (decomposed basalt). 5.0		SPT	7	B4.1		40-50/1"		
				300	SPT	5	B4.2		50/5"		
5		⬡	Weak to strong (R2-R4), gray-brown BASALT, moderately weathered, intensely to moderately fractured, not healed, moist.	295	Core	53	BC4.1	45			Switched to rock coring at 5 feet.
10		⬡	Strong to very strong (R4-R5), gray, intensely fractured at 10 feet.	290	Core	22	BC4.2	9			Washout at 11 feet. UC = 17,340 psi Washout at 12.5 feet. Washout at 14 feet.
15		⬡		285	Core	44	BC4.3	0			

Water Levels

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BORING NUMBER: B-4

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/29/2025
DRILLING METHOD Hollow-Stem Auger and Rock Coring
TIME STARTED 4:08 PM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 01/29/2025
BORING DIAMETER 4.25 inches
TIME COMPLETED 1:30 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page) Red-gray, intensely to moderately fractured, vesicular with yellow mineralization at 20.4 feet.	280	Core	48	BC4.4	23			
25			Medium strong (R3), gray, moderately to slightly weathered, moderately to slightly fractured, vesicular at 25 feet.	275	Core	61	BC4.5	59			
30			Medium strong to strong (R3-R4), moderately fractured, not vesicular at 30 feet.	270	Core	59	BC4.6	88			
35			Slightly weathered, slightly fractured at 35 feet. Vesicular at 39 feet.	265	Core	60	BC4.7	80			

Water Levels

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Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West Engineering, Inc

BORING NUMBER: B-4

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/29/2025
DRILLING METHOD Hollow-Stem Auger and Rock Coring
TIME STARTED 4:08 PM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 01/29/2025
BORING DIAMETER 4.25 inches
TIME COMPLETED 1:30 PM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			Medium strong to strong (R3-R4), moderately to slightly weathered, intensely fractured at 40 feet.	260	Core	61	BC4.8	23			
45			Strong (R4), moderately fractured, fewer vesicles at 45 feet.	255	Core	60	BC4.9	37			
50			50.0								

Boring completed at 50 feet.

Water Levels

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Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West Engineering, Inc

BORING NUMBER: B - 5

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PROJECT NAME Operations Center

CLIENT City of West Linn

PROJECT NUMBER WestL-1-01-1

PROJECT LOCATION West Linn, Oregon

DRILLING CONTRACTOR Western States Soil Conservation, Inc.

DATE STARTED 01/29/2025

DRILLING METHOD Mud Rotary and Rock Coring

TIME STARTED 2:30 PM

EQUIPMENT CME-850, Rig 7

DATE COMPLETED 01/31/2025

BORING DIAMETER 4.25 inches

TIME COMPLETED 11:25 AM

HAMMER EFFICIENCY 82.8%

LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
				332.5							
			Soft, brown sandy SILT, trace organics, moist (12 inches of topsoil, 6-inch-thick root zone) - **FILL**. 1.0								
			Medium dense, brown clayey GRAVEL with sand, moist, gravel is fine to coarse, sand is fine to coarse - **FILL**.	330	SPT	5	B5.1		6-7-12 (19)	20	
5					SPT	12	B5.2		9-10-7 (17)		
			7.5								
			Medium stiff, brown sandy CLAY with gravel, moist, medium plasticity, sand is fine to coarse, gravel is fine - **FILL**.	325	SPT	9	B5.3		3-3-3 (6)	23	
10					CSS	8	B5.4		4-4-4 (8)		
				320							Driller Comment: Lost circulation at 12.5 feet. Lost 80 gallons of mud. Lost 230 gallons of mud at 15 feet. Possible nested cobbles and boulders at 15 feet.
15					SPT	0	B5.5		5-1-6 (7)		
			Hard, gravel is fractured at 18 feet.	315	SPT	1	B5.6		50/2"		

Water Levels

▽ Unable to observe

▼ -

BORING NUMBER: B - 5

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PROJECT NAME Operations Center

CLIENT City of West Linn

PROJECT NUMBER WestL-1-01-1

PROJECT LOCATION West Linn, Oregon

DRILLING CONTRACTOR Western States Soil Conservation, Inc.

DATE STARTED 01/29/2025

DRILLING METHOD Mud Rotary and Rock Coring

TIME STARTED 2:30 PM

EQUIPMENT CME-850, Rig 7

DATE COMPLETED 01/31/2025

BORING DIAMETER 4.25 inches

TIME COMPLETED 11:25 AM

HAMMER EFFICIENCY 82.8%

LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			Medium dense, brown-gray SAND with clay and gravel, moist, sand is fine to coarse, gravel is fine to coarse - **FILL**.		SPT	1	B5.7		12-10-8 (18)		
				310							
25					D&M	12	B5.8		9-9-27 (36)	27	
			27.0								
	ML		Hard, pink-brown with orange mottled sandy SILT, moist, sand is fine to medium (decomposed basalt).	305							
30					SPT	18	B5.9		23-35-47 (82)	26	
				300							
35			35.0								
	GC/GP-GC		Very dense, gray-brown-orange clayey GRAVEL with sand, moist, to GRAVEL with clay and sand, moist, gravel is fine to coarse and fractured, sand is fine to coarse (decomposed basalt).		SPT	6	B5.10		40-50/3"		
				295							

Water Levels

▽ Unable to observe

▼ -

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BORING NUMBER: B - 5

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PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/29/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 2:30 PM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 01/31/2025
BORING DIAMETER 4.25 inches
TIME COMPLETED 11:25 AM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
45	GC/GP-GC		(continued from previous page) 46.0	290	D&M	5	B5.11		120/5" (100)		
			Strong (R4), gray BASALT, slightly weathered, moderately fractured, vesicular, not healed, moist. Medium strong to strong (R3-R4) at 47 feet. Intensely to moderately fractured at 48 feet.	285	Core	9	BC5.1	58			Switched to rock coring at 46 feet. UC = 5,860 psi
50					Core	63	BC5.2	68			
55			Moderately fractured at 52 feet. Vesicular and vuggy, trunks of vesicles oriented 5 to 20 degrees at 53 feet.	280	Core	40	BC5.3	46			
			Strong (R4) at 55.3 feet. No vesicles at 57 feet.	275	Core	60	BC5.4	95			

Water Levels

⊻ Unable to observe

▼ -

BORING NUMBER: B - 5

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/29/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 2:30 PM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 01/31/2025
BORING DIAMETER 4.25 inches
TIME COMPLETED 11:25 AM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			Slightly weathered, intensely fractured at 60 feet.		Core	20	BC5.5	40			Intense fracturing due to coring through sub-vertical joints (columnar) 60 degrees.
65			Slightly to moderately weathered, moderately fractured at 62 feet.	270	Core	55	BC5.6	33			
70			Slightly weathered, intensely to slightly fractured, vesicular at 67 feet.	265	Core	52	BC5.7	38			
75			Moderately to slightly weathered, intensely to moderately fractured at 72 feet. Intensely to moderately weathered, intensely fractured, vesicular with yellow staining at 74.5 feet. Larger vesicles have blue-gray mineral coatings at 75 feet.	260	Core	62	BC5.8	53			
			Gray-blue, slightly weathered, moderately fractured at 77 feet.	255	Core	62	BC5.9	70			

Water Levels

∇ Unable to observe

▼ -

PROJECT NAME Operations Center

CLIENT City of West Linn

PROJECT NUMBER WestL-1-01-1

PROJECT LOCATION West Linn, Oregon

DRILLING CONTRACTOR Western States Soil Conservation, Inc.

DATE STARTED 01/29/2025

DRILLING METHOD Mud Rotary and Rock Coring

TIME STARTED 2:30 PM

EQUIPMENT CME-850, Rig 7

DATE COMPLETED 01/31/2025

BORING DIAMETER 4.25 inches

TIME COMPLETED 11:25 AM

HAMMER EFFICIENCY 82.8%

LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page)								
85			Last vesicles oriented 10 degrees at 82 feet. No vesicles below 82 feet.	250	Core	54	BC5.10	35			
90			Weak to strong (R2-R4), gray-orange-red, completely to slightly weathered at 87 feet. Highly weathered zones from 91 to 92 feet. 92.0	245	Core	36	BC5.11	41			
95			Extremely weak to very weak (R0-R1), gray-red and yellow-white BASALT, very intensely fractured (relict basalt or weathered sandy interflow). 93.0 Medium strong (R3), gray BASALT, moderately to slightly weathered, moderately fractured with columnar joints, vesicular, moist. Highly weathered zone from 93 to 94 feet.	240	Core	60	BC5.12	40			
			Strong (R4), gray, slightly weathered, moderately fractured at 97 feet.	235	Core	62	BC5.13	42			Vibrating wire piezometer 2449541 installed at 98 feet.

Water Levels

▽ Unable to observe

▼ -

BORING NUMBER: B - 5

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NUMBER WestL-1-01-1
PROJECT LOCATION West Linn, Oregon
DRILLING CONTRACTOR Western States Soil Conservation, Inc.
DATE STARTED 01/29/2025
DRILLING METHOD Mud Rotary and Rock Coring
TIME STARTED 2:30 PM
EQUIPMENT CME-850, Rig 7
DATE COMPLETED 01/31/2025
BORING DIAMETER 4.25 inches
TIME COMPLETED 11:25 AM
HAMMER EFFICIENCY 82.8%
LOGGED BY E. Uren

DEPTH (ft)	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION AND NOTES	ELEVATION (ft)	SAMPLE TYPE	RECOVERY (in)	SAMPLE ID	% RQD	BLOW COUNTS	MOISTURE CONTENT (%)	Remarks
			(continued from previous page)								
105			Extremely weak (R0), white-gray-orange, completely to intensely weathered, very intensely fractured at 104 feet. 106.0	230	Core	48	BC5.14	17		57	
110			Medium strong to strong (R3-R4), gray BASALT, slightly weathered, moderately to slightly fractured, some vesicles. Vesicles at 107 feet. Fewer vesicles at 109 feet.	225	Core	60	BC5.15	60			
115			CLAY with basalt fragments interflow from 112.3 to 112.6 feet. Medium strong to strong (R3-R4) at 112.6 feet. 117.0	220	Core	60	BC5.16	63			

Boring completed at 117 feet.

Water Levels

▽ Unable to observe

▼ -

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TEST PIT NUMBER: TP-1

Page 1 of 1

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NO. WestL-1-01-1 **LOGGED BY** E. Uren
PROJECT LOCATION West Linn, Oregon
CONTRACTOR Dan J. Fischer Excavating, Inc.
EQUIPMENT Kobelco SK140SR
CAVING Not observed
DATE COMPLETED 01/23/2025
GROUNDWATER Not observed
TIME STARTED 10:59 AM **TIME COMPLETED** 11:15 AM

DEPTH (ft)	SAMPLE TYPE	SAMPLE ID	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft)	POCKET PEN (tsf)	MOISTURE CONTENT (%)	REMARKS
						367.5			
					Soft, brown sandy SILT, trace organics, moist (6 inches of topsoil). 0.5				
	Grab	TP1.1			Extremely weak to weak (R0-R2), brown-gray BASALT, completely to highly weathered, very close to close jointed, moist. 4.0	365		13	
					Test pit terminated at 4 feet on basalt.				
5									
10									
15									
20									

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TEST PIT NUMBER: TP-2

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PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NO. WestL-1-01-1 **LOGGED BY** E. Uren
PROJECT LOCATION West Linn, Oregon
CONTRACTOR Dan J. Fischer Excavating, Inc.
EQUIPMENT Kobelco SK140SR
CAVING Not observed
DATE COMPLETED 01/23/2025
GROUNDWATER Not observed
TIME STARTED 9:56 AM **TIME COMPLETED** 10:23 AM

DEPTH (ft)	SAMPLE TYPE	SAMPLE ID	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft) 350.7	POCKET PEN (tsf)	MOISTURE CONTENT (%)	REMARKS
				≈	Soft, brown sandy SILT, trace organics, moist (6 inches of topsoil, 4-inch-thick root zone). 0.5	350			
	Grab	TP2.1	GM	▨	Medium dense, brown-gray silty GRAVEL with sand and cobbles, moist, gravel is fine to coarse, sand is fine to coarse, cobbles are subangular and approximately 5 to 15% (decomposed basalt). Boulder (subangular and 24 inches in diameter) at 3 feet. 4.0			16	
5				⬡	Very weak to weak (R1-R2), brown-gray BASALT, highly to moderately weathered, close to moderately close jointed, moist. 6.0	345			Decomposed pockets.
					Test pit terminated at 6 feet on basalt.				
10									
15									
20									

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TEST PIT NUMBER: TP-3

Page 1 of 1

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NO. WestL-1-01-1 **LOGGED BY** E. Uren
PROJECT LOCATION West Linn, Oregon
CONTRACTOR Dan J. Fischer Excavating, Inc.
EQUIPMENT Kobelco SK140SR
CAVING Minor from 1 to 3 feet, severe from 4 to 15 feet
DATE COMPLETED 01/23/2025
GROUNDWATER Not observed
TIME STARTED 9:05 AM **TIME COMPLETED** 9:55 AM

DEPTH (ft)	SAMPLE TYPE	SAMPLE ID	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft)	POCKET PEN (tsf)	MOISTURE CONTENT (%)	REMARKS
						345			
					Soft, brown sandy SILT, trace organics, moist (6 inches of topsoil, 4-inch-thick root zone) - **FILL**. 0.5				
	Grab	TP3.1			Medium dense, brown clayey GRAVEL with sand, cobbles, and boulders, moist, gravel is fine to coasre and fractured, sand is fine to coarse, cobbles are subangular to angular and approximately 5 to 15%, boulders are subangular to angular, 12 to 24 inches in diameter, and approximately 5 to 10% - **FILL**.			17	
5					5.0	340			
	Grab	TP3.2			Medium dense, brown GRAVEL with clay, sand, cobbles, and boulders, moist, gravel is fine to coasre and fractured, sand is fine to coarse, cobbles are subangular to angular and approximately 5 to 10%, boulders are subangular to angular, 12 to 24 inches in diameter, and approximately 5 to 15% - **FILL**.				
10						335			
	Grab	TP3.3						10	
					Loose to medium dense, moist to wet at 12 feet.				
					Boulder (subrounded and 36 inches in diameter) at 13 feet.				
15						330			
	Grab	TP3.4			Loose at 17 feet.				
					Boulder (subrounded and 18 inches in diameter) at 18 feet.				
20	Grab	TP3.5			20.5	325			

Test pit completed at 20.5 feet.

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TEST PIT NUMBER: TP-4

Page 1 of 1

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NO. WestL-1-01-1 **LOGGED BY** E. Uren
PROJECT LOCATION West Linn, Oregon
CONTRACTOR Dan J. Fischer Excavating, Inc.
EQUIPMENT Kobelco SK140SR
CAVING Severe from 0 to 14 feet
DATE COMPLETED 01/23/2025
GROUNDWATER Not observed
TIME STARTED 12:45 PM **TIME COMPLETED** 1:40 PM

DEPTH (ft)	SAMPLE TYPE	SAMPLE ID	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft)	POCKET PEN (tsf)	MOISTURE CONTENT (%)	REMARKS
				▤	Soft, brown sandy SILT, trace organics, moist (12 inches of topsoil, 6-inch-thick root zone) - **FILL**. 1.0	342.3			
				▩	Medium dense, brown-gray clayey GRAVEL, moist, gravel is fine to coarse (blasted basalt) - **FILL**.	340			
	Grab	TP4.1		▩	Boulders (four; subangular and 36 to 48 inches in diameter) at 4 feet.			8	
5				▩		335			
				▩	9.5				
10	Grab	TP4.2		▩	Medium dense, brown clayey GRAVEL with sand and cobbles, moist, gravel is fine to coarse, sand is fine to coarse, cobbles are subangular and approximately 10 to 20% - **FILL**.	330			
	Grab	TP4.3		▩					
15				▩	17.0				

Test pit terminated at 17 feet due to caving.

20

TEST PIT NUMBER: TP-5

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NO. WestL-1-01-1 **LOGGED BY** E. Uren
PROJECT LOCATION West Linn, Oregon
CONTRACTOR Dan J. Fischer Excavating, Inc.
EQUIPMENT Kobelco SK140SR
CAVING Minor from 1 foot to 6 feet, severe from 12 to 17 feet
DATE COMPLETED 01/23/2025
GROUNDWATER Not observed
TIME STARTED 11:44 AM **TIME COMPLETED** 12:12 PM

DEPTH (ft)	SAMPLE TYPE	SAMPLE ID	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft)	POCKET PEN (tsf)	MOISTURE CONTENT (%)	FINES (%)	REMARKS
				▨	Soft, brown sandy SILT, trace organics, moist (6 inches of topsoil, 6-inch-thick root zone) - **FILL**. 0.5	336.7				
				▨	Loose, brown silty GRAVEL with sand and cobbles, moist, gravel is fine to coarse and fractured, sand is fine to coarse, cobbles are subangular and approximately 0 to 10% - **FILL**.	335				
	Grab	TP5.1		▨				19		
5				▨		330				
	Grab	TP5.2		▨	With boulders, boulders are subangular, 12 to 24 inches in diameter, and approximately 5 to 10% at 7 feet.					
10				▨	11.0					
	Grab	TP5.3		▨	Loose, brown clayey GRAVEL with sand and cobbles, wet, medium plasticity, gravel is fine to coarse, sand is fine to coarse, cobbles are subangular and approximately 0 to 10% - **FILL**.	325		20	15	
15				▨		320				
				▨	18.0					
	Grab	TP5.4		▨	Medium dense to dense, brown GRAVEL with silt, clay, sand, and cobbles, wet, gravel is fine to coarse, sand is fine to coarse, cobbles are subangular and approximately 0 to 10% - **FILL**. 19.0			24		
20					Test pit terminated at 19 feet due to refusal on large boulder.					

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TEST PIT NUMBER: TP-6

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PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NO. WestL-1-01-1 **LOGGED BY** E. Uren
PROJECT LOCATION West Linn, Oregon
CONTRACTOR Dan J. Fischer Excavating, Inc.
EQUIPMENT Kobelco SK140SR
CAVING Severe from 0 to 15 feet
DATE COMPLETED 01/23/2025
GROUNDWATER Not observed
TIME STARTED 1:51 PM **TIME COMPLETED** 2:20 PM

DEPTH (ft)	SAMPLE TYPE	SAMPLE ID	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft)	POCKET PEN (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS (LL-PL-PI)	REMARKS
						337				
					Soft, brown sandy SILT, trace organics, moist (6 inches of topsoil, 6-inch-thick root zone) - **FILL**. 0.5					
					Loose, brown clayey GRAVEL with sand, cobbles, and boulders, moist, medium plasticity, gravel is fine to coarse, sand is fine to coarse, cobbles are subangular and approximately 10 to 20%, boulders are subangular, and 12 to 24 inches in diameter, and approximately 0 to 5% - **FILL**.	335				
	Grab	TP6.1							48-26-22	
5										
						330				
	Grab	TP6.2						13		
10										
						325				
15					15.0					

Test pit terminated at 15 feet due to caving.

20

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TEST PIT NUMBER: TP-7

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PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NO. WestL-1-01-1 **LOGGED BY** E. Uren
PROJECT LOCATION West Linn, Oregon
CONTRACTOR Dan J. Fischer Excavating, Inc.
EQUIPMENT Kobelco SK140SR
CAVING Not observed
DATE COMPLETED 01/23/2025
GROUNDWATER Not observed
TIME STARTED 2:38 PM **TIME COMPLETED** 3:00 PM

DEPTH (ft)	SAMPLE TYPE	SAMPLE ID	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft) 305.5	POCKET PEN (tsf)	MOISTURE CONTENT (%)	REMARKS
				▩	Bark Chips (6 inches) - **FILL**. 0.5	305			
	Grab	TP7.1		▩	Medium dense, orange-brown clayey SAND with gravel, moist, sand is fine to coarse, gravel is fine to coarse - **FILL**. 2.5			36	
				⬡	Extremely weak to very weak (R0-R1), brown-gray-orange BASALT, decomposed to highly weathered, very close to close jointed, moist. 4.0				

Test pit terminated at 4 feet on basalt.

5
10
15
20

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TEST PIT NUMBER: TP-8

Page 1 of 1

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NO. WestL-1-01-1 **LOGGED BY** E. Uren
PROJECT LOCATION West Linn, Oregon
CONTRACTOR Dan J. Fischer Excavating, Inc.
EQUIPMENT Kobelco SK140SR
CAVING Not observed
DATE COMPLETED 01/24/2025
GROUNDWATER Not observed
TIME STARTED 8:06 AM **TIME COMPLETED** 8:20 AM

DEPTH (ft)	SAMPLE TYPE	SAMPLE ID	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft) 310	POCKET PEN (tsf)	MOISTURE CONTENT (%)	REMARKS
				▨	Bark Chips (approximately 24 inches) - **FILL**. 2.0				
	Grab	TP8.1		⬡	Very weak to weak (R1-R2), gray BASALT, highly to moderately weathered, very close jointed, moist. 2.5				
					Test pit terminated at 2.5 feet on basalt.				
5									
10									
15									
20									

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TEST PIT NUMBER: TP-9

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PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NO. WestL-1-01-1 **LOGGED BY** E. Uren
PROJECT LOCATION West Linn, Oregon
CONTRACTOR Dan J. Fischer Excavating, Inc.
EQUIPMENT Kobelco SK140SR
CAVING Minor from 0 to 4 feet
DATE COMPLETED 01/23/2025
GROUNDWATER Not observed
TIME STARTED 10:25 AM **TIME COMPLETED** 10:58 AM

DEPTH (ft)	SAMPLE TYPE	SAMPLE ID	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft)	POCKET PEN (tsf)	MOISTURE CONTENT (%)	REMARKS
					Soft, brown sandy SILT, trace organics, moist (6 inches of topsoil, 4-inch-thick root zone) - **FILL**. 0.5	350			
	Grab	TP9.1			Loose, gray-brown silty GRAVEL with sand and cobbles, moist, gravel is fine to coarse, sand is fine to coarse, cobbles are subangular and 5 to 15% - **FILL**.				
5					5.5	345			
	Grab	TP9.2			Extremely weak to weak (R0-R2), brown-gray-orange BASALT, decomposed to highly weathered, very close to close jointed, moist.				Native rock along north wall of test pit at 5.5 feet.
									Native rock along south wall of test pit at 8.5 feet.
10						340			
15	Grab	TP9.3				335		14	
									Harder digging at 15.5 feet.
					17.0				

Test pit terminated at 17 feet on basalt.

20

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TEST PIT NUMBER: TP-10

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PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NO. WestL-1-01-1 **LOGGED BY** E. Uren
PROJECT LOCATION West Linn, Oregon
CONTRACTOR Dan J. Fischer Excavating, Inc.
EQUIPMENT Kobelco SK140SR
CAVING Not observed
DATE COMPLETED 01/24/2025
GROUNDWATER Not observed
TIME STARTED 8:21 AM **TIME COMPLETED** 8:30 AM

DEPTH (ft)	SAMPLE TYPE	SAMPLE ID	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft) 316.5	POCKET PEN (tsf)	MOISTURE CONTENT (%)	REMARKS
				▩	Bark Chips (approximately 12 inches) - **FILL**. 1.0				
	Grab	TP10.1		⬡	Very weak to weak (R1-R2), gray-black BASALT, highly to moderately weathered, very close jointed, moist, moist. 1.5	315			
5									
10									
15									
20									

Test pit terminated at 1.5 feet on basalt.

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TEST PIT NUMBER: TP-11

Page 1 of 1

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NO. WestL-1-01-1 **LOGGED BY** E. Uren
PROJECT LOCATION West Linn, Oregon
CONTRACTOR Dan J. Fischer Excavating, Inc.
EQUIPMENT Kobelco SK140SR
CAVING Severe from 0 to 7 feet
DATE COMPLETED 01/24/2025
GROUNDWATER Not observed
TIME STARTED 8:31 AM **TIME COMPLETED** 9:17 AM

DEPTH (ft)	SAMPLE TYPE	SAMPLE ID	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft) 319.5	POCKET PEN (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS (LL-PL-PI)	REMARKS
				▒	Soft, brown sandy SILT, trace organics, moist (12 inches of topsoil, 4-inch-thick root zone) - **FILL**. 1.0					
	Grab	TP11.1		▩	Loose to medium dense, gray-brown GRAVEL with silt, sand, cobbles, and boulders, moist, gravel is fine to coarse, silt has medium to high plasticity, sand is fine to coarse, cobbles are subangular, fractured, and approximately 20 to 25%, boulders are subangular to angular, 12 to 24 inches in diameter, and 10 to 15% - **FILL**. 3.0				52-36-16	Native rock along south wall of test pit at 3 feet.
5	Grab	TP-11.2		⬡	Extremely weak to very weak (R0-R1), brown-black-orange BASALT, completely to highly weathered, very close to close jointed, moist.	315		31		Native rock along north wall of test pit at 7.5 feet.
10				⬡	Very weak to weak (R1-R2), gray, highly to moderately weathered, very close jointed at 9.5 feet. 10.0	310				

Test pit terminated at 10 feet on basalt.

15

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TEST PIT NUMBER: TP-12

PROJECT NAME Operations Center

CLIENT City of West Linn

PROJECT NO. WestL-1-01-1 **LOGGED BY** E. Uren

PROJECT LOCATION West Linn, Oregon

CONTRACTOR Dan J. Fischer Excavating, Inc.

EQUIPMENT Kobelco SK140SR

CAVING Not observed

DATE COMPLETED 01/24/2025

GROUNDWATER Not observed

TIME STARTED 9:19 AM **TIME COMPLETED** 9:31 AM

DEPTH (ft)	SAMPLE TYPE	SAMPLE ID	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft) 306.5	POCKET PEN (tsf)	MOISTURE CONTENT (%)	REMARKS
				▩	Bark Chips (approximately 12 inches) - **FILL**. 1.0				
	Grab	TP12.1		⬡	Very weak to weak (R1-R2), gray-black BASALT, highly to moderately weathered, very close jointed, moist. 1.5	305			
					Test pit terminated at 1.5 feet on basalt.				
5									
10									
15									
20									

TEST PIT NUMBER: TP-13

PROJECT NAME Operations Center
CLIENT City of West Linn
PROJECT NO. WestL-1-01-1 **LOGGED BY** E. Uren
PROJECT LOCATION West Linn, Oregon
CONTRACTOR Dan J. Fischer Excavating, Inc.
EQUIPMENT Kobelco SK140SR
CAVING Not observed
DATE COMPLETED 01/24/2025
GROUNDWATER Not observed
TIME STARTED 9:34 AM **TIME COMPLETED** 9:55 AM

DEPTH (ft)	SAMPLE TYPE	SAMPLE ID	USCS	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft) 315	POCKET PEN (tsf)	MOISTURE CONTENT (%)	REMARKS
				▨	Bark Chips (approximately 15 inches) - **FILL**. 1.3				
	Grab	TP13.1		⬡	Extremely weak to weak (R0-R2), gray-black BASALT, highly to moderately weathered, very close to close jointed, moist. 3.0				

Test pit terminated at 3 feet on basalt.

5

10

15

20

Summary of SPT Test Results

Project: rig 7, Test Date: 12/31/2024

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute
EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
57.50	**8-13-8**	**21**	**28**	42	16.5	48.2	282	80.6
60.00	**1-5-9**	**14**	**19**	45	14.9	49.2	289	82.6
62.50	**4-7-8**	**15**	**20**	46	15.0	50.7	298	85.3
67.50	**15-13-12**	**25**	**34**	44	16.8	51.6	292	83.4
			Overall Average Values:	44	16.0	50.0	290	82.8
			Standard Deviation:	3	0.9	1.4	7	1.9
			Overall Maximum Value:	61	17.2	51.7	304	86.8
			Overall Minimum Value:	41	14.0	48.1	277	79.2

APPENDIX B

APPENDIX B
LABORATORY TESTING

GENERAL

Laboratory testing was conducted on select soil samples to confirm field classifications and determine the index engineering properties and strength characteristics. The laboratory classifications are shown on the exploration logs if those classifications differed from the field classifications. The locations of the tested samples are shown on the exploration logs. Descriptions of the tests are presented below, and results of the testing are presented in this appendix.

PARTICLE-SIZE ANALYSIS

Particle-size analysis was completed on select soil samples in general accordance with ASTM D1140 (P200). This test is a quantitative determination of the percent passing the U.S. Standard No. 200 sieve by dry weight. In addition, particle-size analysis was completed on a select soil sample in general accordance with ASTM D6913. This test is a quantitative determination of the soil particle size distribution expressed as a percentage of dry soil weight.

MOISTURE CONTENT

The natural moisture content of select soil samples was determined in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to dry soil in a test sample and is expressed as a percentage.

ATTERBERG LIMITS TESTING

Atterberg limits (plastic and liquid limits) testing was performed on select soil samples in general accordance with ASTM D4318. The plastic limit is defined as the moisture content where the soil becomes brittle. The liquid limit is defined as the moisture content where the soil begins to act similar to a liquid. The plasticity index is the difference between the liquid and plastic limits.

DIRECT SHEAR

Direct shear testing was completed on a select soil sample in general accordance with ASTM D3080. The test is performed by deforming a specimen at a controlled strain rate on or near a single shear plane. In general, three or more specimens are tested, each under a different normal load, to determine the Mohr strength envelope.

DRY DENSITY TESTING

Dry density testing was completed on relatively undisturbed soil samples in general accordance with ASTM D2937. The dry density is the dry unit weight of a soil sample in units of pcf.

UNCONFINED COMPRESSIVE STRENGTH

Unconfined compressive strength testing was completed on select rock samples in general accordance with ASTM D7012. The recorded value is reported as the peak compressive strength of the specimen prior to failure.

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MOISTURE CONTENT, PERCENT PASSING NO. 200 SIEVE BY WASHING

PROJECT
Operations Center
Salamo Road and Greene Street
West Linn, Oregon

CLIENT
City of West Linn
22500 Salamo Road
West Linn, OR 97068

PROJECT NO. WestL-1-01-1

ISSUE DATE 03/06/25

PAGE 1 of 2

DATE SAMPLED 01/23/25 - 02/06/25

SAMPLED BY E. Uren

LABORATORY TEST DATA

TEST PROCEDURE
ASTM D2216 - Method A, ASTM D1140

LAB ID	CONTAINER MASS (g)	MOIST MASS + CONTAINER (g)	DRY MASS + CONTAINER (g)	AFTER WASH DRY MASS + CONTAINER (g)	FIELD ID	SAMPLE DEPTH (ft)	PERCENT MOISTURE CONTENT	PERCENT PASSING NO. 200 SIEVE
S25-0305	771.52	3,692.50	3,350.55	-	TP1.1	2	13%	-
S25-0306	849.49	3,597.99	3,217.31	-	TP2.1	1	16%	-
S25-0307	780.05	3,952.42	3,499.22	-	TP3.1	3	17%	-
S25-0308	853.22	4,186.71	3,878.31	-	TP3.3	11	10%	-
S25-0309	776.96	3,086.82	2,916.80	-	TP4.1	4	8%	-
S25-0310	752.71	3,978.70	3,474.92	-	TP5.1	3	19%	-
S25-0311	784.19	3,493.96	3,034.75	2,695.74	TP5.3	11	20%	15%
S25-0312	866.51	4,091.88	3,477.85	-	TP5.4	18	24%	-
S25-0314	776.40	3,631.15	3,292.24	-	TP6.2	8	13%	-
S25-0315	867.38	3,566.51	2,857.86	-	TP7.1	2	36%	-
S25-0316	1,536.73	4,829.89	4,432.75	-	TP9.3	14.5	14%	-
S25-0318	1,554.19	3,459.56	3,005.00	-	TP11.2	7.5	31%	-
S25-0319	548.10	979.92	854.68	-	B2.1	2.5	41%	-
S25-0320	539.89	2,638.41	2,319.25	-	B2.5	15	18%	-
S25-0321	548.47	1,118.21	1,006.92	961.89	B2.7	25	24%	10%
S25-0389	87.16	236.86	192.16	-	B2.8	30	43%	-

NOTES:
Sample weights received for Lab ID: S25-0305, 0306, 0307, 0308, 0309, 0310, 3011, 0312, 0314, 0315, 0316, 0318, 0319, 0320, 0321, 0324, 0325, 0326 and 0395 did not meet the minimum size requirements; entire sample used for analysis.

DATE TESTED 02/17/25, 02/28/25

TESTED BY B. Taylor, M. Scherette

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MOISTURE CONTENT, PERCENT PASSING NO. 200 SIEVE BY WASHING

PROJECT
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Salamo Road and Greene Street
West Linn, Oregon

CLIENT
City of West Linn
22500 Salamo Road
West Linn, OR 97068

PROJECT NO.: WestL-1-01-1

ISSUE DATE: 03/06/25

PAGE: 2 of 2

DATE SAMPLED: 01/23/25 - 02/06/25

SAMPLED BY: E. Uren

LABORATORY TEST DATA

TEST PROCEDURE
ASTM D2216 - Method A, ASTM D1140

LAB ID	CONTAINER MASS (g)	MOIST MASS + CONTAINER (g)	DRY MASS + CONTAINER (g)	AFTER WASH DRY MASS + CONTAINER (g)	FIELD ID	SAMPLE DEPTH (ft)	PERCENT MOISTURE CONTENT	PERCENT PASSING NO. 200 SIEVE
S25-0322	542.54	924.22	789.24	-	B2.9	35	55%	-
S25-0323	556.41	869.53	815.53	-	B3.1	2.5	21%	-
S25-0392	384.26	721.64	608.69	-	BC3.18	86	50%	-
S25-0324	574.84	1,002.15	931.57	-	B5.1	2.5	20%	-
S25-0325	537.40	978.10	896.65	-	B5.3	7.5	23%	-
S25-0326	760.40	1,560.87	1,389.03	1,311.66	B5.8	25	27%	12%
S25-0327	556.11	840.41	782.05	648.87	B5.9	30	26%	59%
S25-0395	787.39	2,605.01	1,946.74	sieved sample	BC5.14	102	57%	20%

NOTES:
Sample weights received for Lab ID: S25-0305, 0306, 0307, 0308, 0309, 0310, 3011, 0312, 0314, 0315, 0316, 0318, 0319, 0320, 0321, 0324, 0325, 0326 and 0395 did not meet the minimum size requirements; entire sample used for analysis.

DATE TESTED: 02/17/25, 02/28/25

TESTED BY: B. Taylor, M. Scherette

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DIRECT SHEAR REPORT

PROJECT Operations Center Salamo Road and Greene Street West Linn, Oregon	CLIENT City of West Linn 22500 Salamo Road West Linn, OR 97068	PROJECT NO. WestL-1-01-1	
		ISSUE DATE 03/06/25	PAGE 1 of 1
		LAB ID S25-0389	FIELD ID B2.8
		DATE SAMPLED 01/31/25	SAMPLED BY E. Uren

MATERIAL DATA

MATERIAL SAMPLED SILT with Sand Dames & Moore sampler	MATERIAL SOURCE Boring B-2 depth = 30 feet	USCS SOIL TYPE no data provided

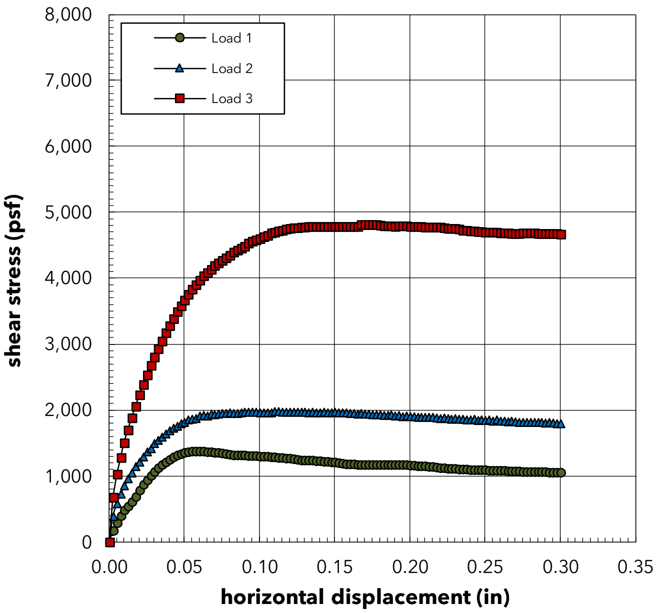
LABORATORY TEST DATA

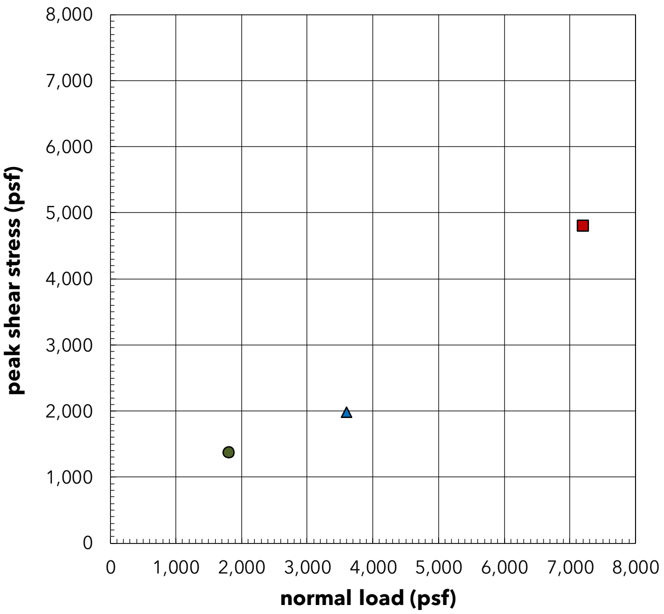
LABORATORY EQUIPMENT Humboldt Automated Direct/Residual Shear Apparatus HM-5760.3F	TEST PROCEDURE ASTM D3080

SAMPLE INFORMATION

	PARAMETER	LOAD 1	LOAD 2	LOAD 3
initial	initial dry density, pcf	77.7	74.9	76.9
	initial moisture content, %	42.6	42.6	42.6
final	final dry density, pcf	80.4	79.2	81.1
	final moisture content, %	44.6	49.4	45.8

HORIZONTAL DISPLACEMENT RATE 0.001 in/min
SAMPLE TYPE relatively undisturbed
MOISTURE CONDITION inundated





load	applied normal load (psf)	shear stess at failure (psf)
1	1,800	1,379
2	3,600	1,977
3	7,200	4,809

DATE TESTED 02/24/25 to 03/03/25	TESTED BY M. Scherette

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ATTERBERG LIMITS REPORT

PROJECT
Operations Center
Salamo Road and Greene Street
West Linn, Oregon

CLIENT
City of West Linn
22500 Salamo Road
West Linn, OR 97068

PROJECT NO.
WestL-1-01-1

ISSUE DATE 03/06/25	PAGE 1 of 1
LAB ID S25-0390	FIELD ID BC2.11
DATE SAMPLED 02/03/25	SAMPLED BY E. Uren

MATERIAL DATA

MATERIAL SAMPLED
CLAY

MATERIAL SOURCE
Boring B-2
depth = 93.3 feet

USCS SOIL TYPE
no data provided

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Liquid Limit Machine, Hand Rolled

TEST PROCEDURE
ASTM D4318 - Method A

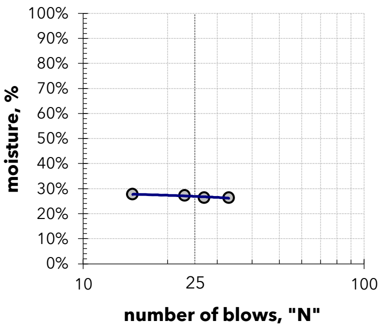
ATTERBERG LIMITS

liquid limit = 27
plastic limit = 18
plasticity index = 9

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	37.34	38.13	39.74	37.50
dry soil + pan weight, g =	33.92	34.55	35.73	33.87
pan weight, g =	20.97	20.95	21.01	20.81
N (blows) =	33	27	23	15
moisture, % =	26.4 %	26.3 %	27.2 %	27.8 %

LIQUID LIMIT



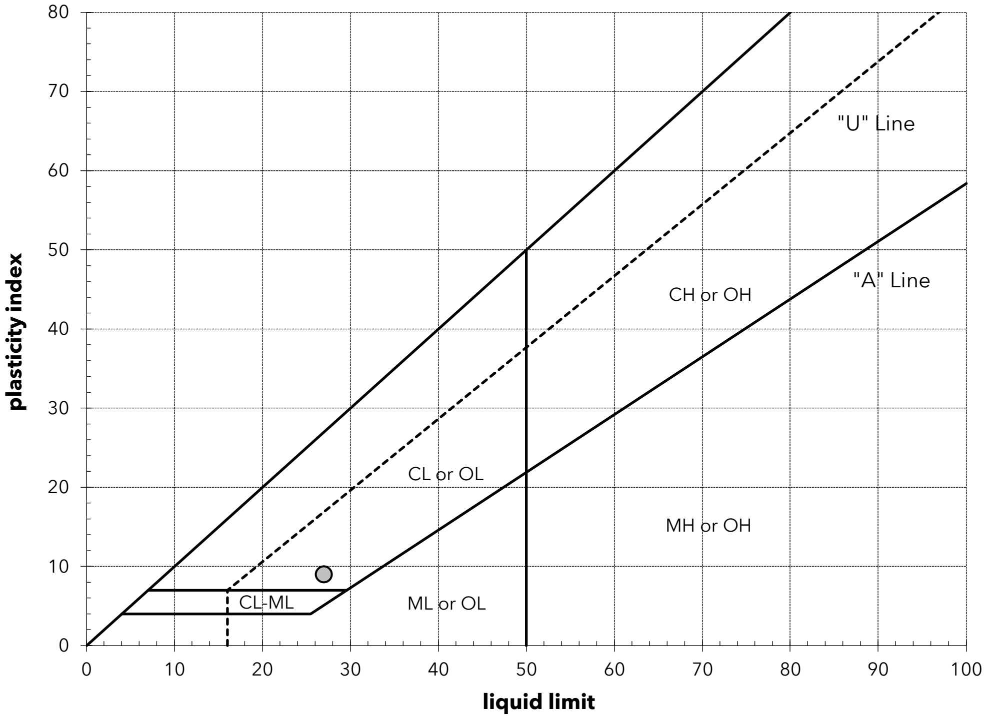
SHRINKAGE

shrinkage limit = n/a
shrinkage ratio = n/a

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.49	29.92		
dry soil + pan weight, g =	27.33	28.50		
pan weight, g =	20.97	20.91		
moisture, % =	18.2 %	18.7 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = n/a
% sand = n/a
% silt and clay = n/a
% silt = n/a
% clay = n/a
moisture content = n/a

DATE TESTED 02/27/25	TESTED BY B. Taylor

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT
Operations Center
Salamo Road and Greene Street
West Linn, Oregon

CLIENT
City of West Linn
22500 Salamo Road
West Linn, OR 97068

PROJECT NO. WestL-1-01-1	
ISSUE DATE 03/06/25	PAGE 1 of 1
LAB ID S25-0395	FIELD ID BC5.14
DATE SAMPLED 01/30/25	SAMPLED BY E. Uren

MATERIAL DATA

MATERIAL SAMPLED
Clayey GRAVEL with Sand
(weathered basalt)

MATERIAL SOURCE
Boring B-5
depth = 102 feet

USCS SOIL TYPE
no data provided

SPECIFICATIONS
none

AASHTO CLASSIFICATION
no data provided

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Rainhart "Mary Ann" Sifter, moist prep, hand washed, 12" single sieve-set

TEST PROCEDURE
ASTM D6913, Method A

ADDITIONAL DATA

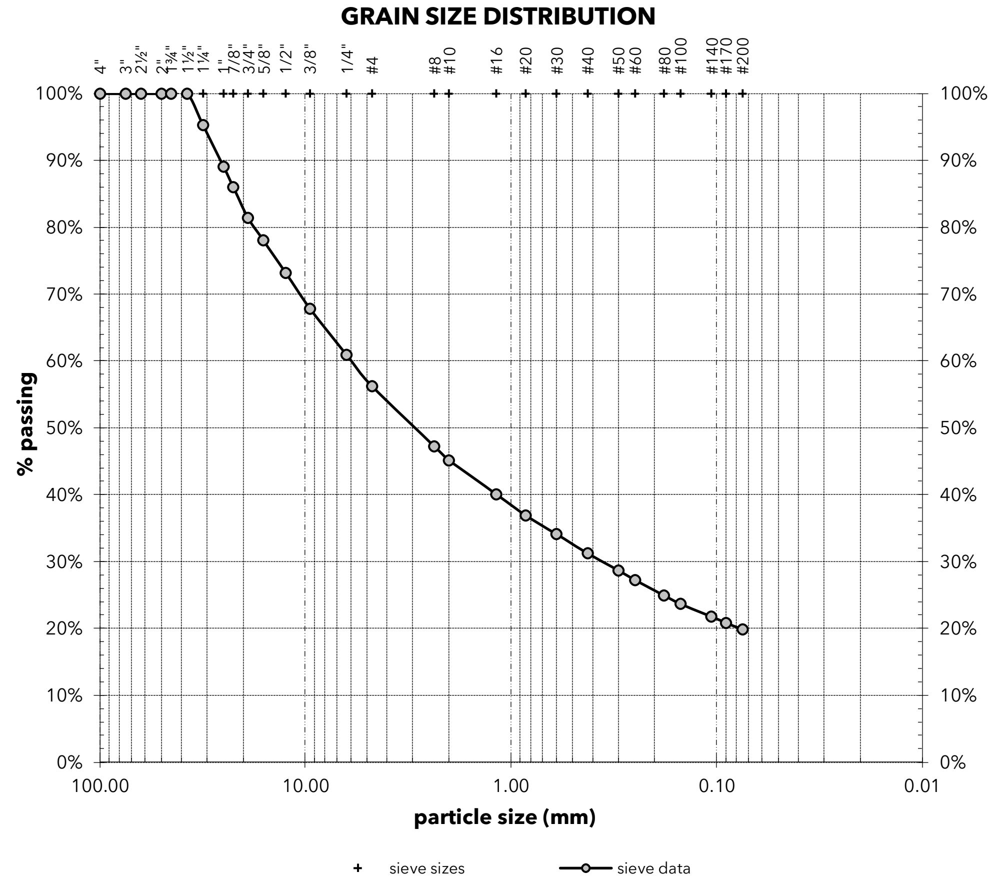
initial dry mass (g) = 1159.35
as-received moisture content = 57%
liquid limit = n/a
plastic limit = n/a
plasticity index = n/a
fineness modulus = n/a
coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = 0.359 mm
$D_{(60)}$ = 5.950 mm

NOTE: Entire sample used for analysis; did not meet minimum size required.

SIEVE DATA

% gravel = 43.8%
% sand = 36.3%
% silt and clay = 19.9%

	SIEVE SIZE		PERCENT PASSING			
			SIEVE		SPECS	
	US	mm	act.	interp.	max	min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5	100%			
	1.25"	31.5		95%		
	1.00"	25.0	89%			
	7/8"	22.4		86%		
	3/4"	19.0	81%			
	5/8"	16.0		78%		
	1/2"	12.5		73%		
	3/8"	9.50	68%			
	1/4"	6.30		61%		
	#4	4.75	56%			
SAND	#8	2.36		47%		
	#10	2.00	45%			
	#16	1.18		40%		
	#20	0.850	37%			
	#30	0.600		34%		
	#40	0.425	31%			
	#50	0.300		29%		
	#60	0.250	27%			
	#80	0.180		25%		
	#100	0.150	24%			
	#140	0.106		22%		
	#170	0.090		21%		
	#200	0.075	20%			



DATE TESTED 02/27/25	TESTED BY M. Scherette

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March 6, 2025

City of West Linn
22500 Salamo Road
West Linn, OR 97068

Re: **Basalt Core Compressive Strength**
Operations Center
Salamo Road and Greene Street
West Linn, Oregon
CWE Project: WestL-1-01-1

Columbia West Engineering, Inc. (Columbia West) is pleased to submit test results for basalt rock cores sampled during the recent subsurface investigation at the above-referenced site. Specimen core ends were saw-cut wet and capped per *ASTM C617* seven days prior to testing. Compression testing was conducted in general accordance with *ASTM D7012*. A seating load of approximately 180 lbf was applied to each specimen. Results for ultimate compressive strength conducted on March 4, 2025, are presented in Table 1.

Table 1. Compressive strength results.

LAB ID	TECH	FIELD ID	CORE SAMPLE DEPTH (ft)	TRIMMED SPECIMEN WEIGHT (lbm)	TRIMMED SPECIMEN LENGTH (in)	AVERAGE SPECIMEN DIAMETER (in)	CROSS-SECTIONAL AREA (in²)	COMPRESSIVE STRENGTH		FRACTURE TYPE
								MAXIMUM LOAD (lbf)	MAXIMUM PRESSURE (psi)	
S25-0391	MJR	BC3.6	27	2.170	4.97	2.38	4.45	71,876	16,150	cone/shear
S25-0393	MJR	BC4.2	11	2.021	4.76	2.37	4.41	76,488	17,340	cone/shear
S25-0394	MJR	BC5.2	47	1.744	4.82	2.35	4.34	25,427	5,860	cone/shear

Results apply only to the samples analyzed. Columbia West appreciates the opportunity to provide materials testing services. Please call me at 360-823-2900 if you have any questions or need additional information.

Sincerely,

Jared J. Comastro, CET
Laboratory Manager

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ATTERBERG LIMITS REPORT

PROJECT Operations Center Salamo Road and Greene Street West Linn, Oregon	CLIENT City of West Linn 22500 Salamo Road West Linn, OR 97068	PROJECT NO. WestL-1-01-1	
		ISSUE DATE 02/20/25	PAGE 1 of 1
		LAB ID S25-0313	FIELD ID TP6.1
		DATE SAMPLED 01/23/25	SAMPLED BY E. Uren

MATERIAL DATA

MATERIAL SAMPLED Clayey GRAVEL with Sand and Cobbles	MATERIAL SOURCE Test Pit TP-6 depth = 3 feet	USCS SOIL TYPE no data provided

LABORATORY TEST DATA

LABORATORY EQUIPMENT Liquid Limit Machine, Hand Rolled	TEST PROCEDURE ASTM D4318 - Method A

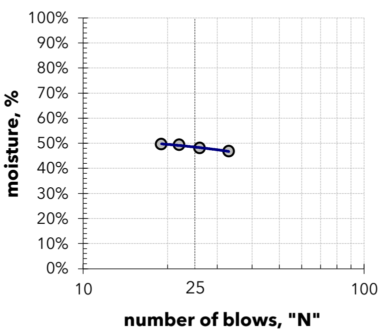
ATTERBERG LIMITS

liquid limit = 48
plastic limit = 26
plasticity index = 22

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	32.65	34.23	33.06	34.61
dry soil + pan weight, g =	28.93	29.92	29.08	30.03
pan weight, g =	20.98	20.96	21.01	20.81
N (blows) =	33	26	22	19
moisture, % =	46.8 %	48.1 %	49.3 %	49.7 %

LIQUID LIMIT



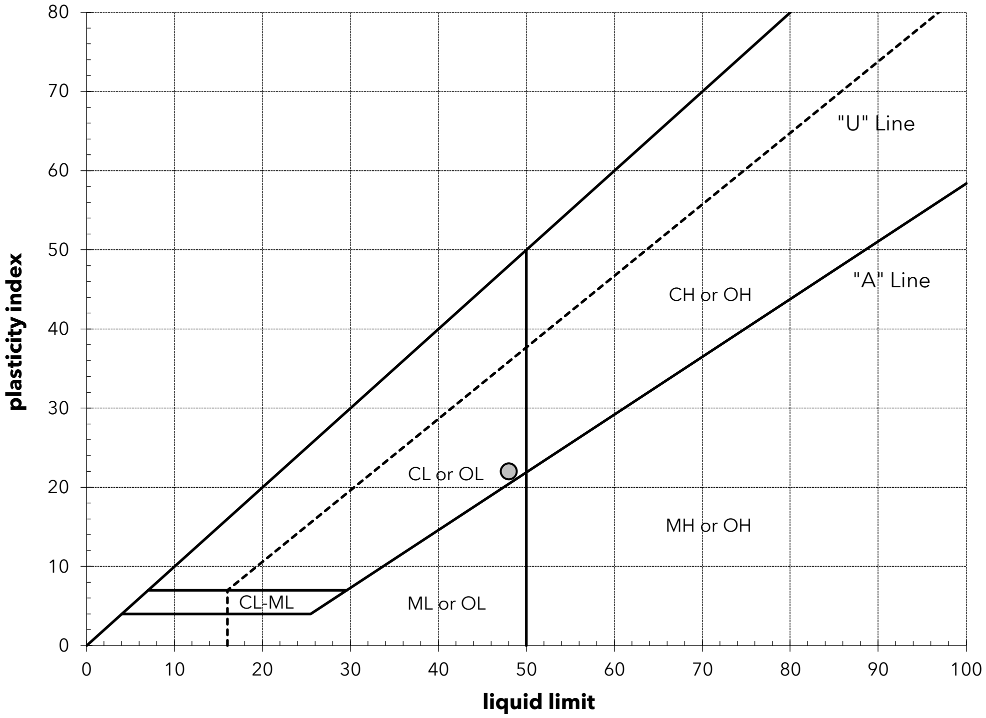
SHRINKAGE

shrinkage limit = n/a
shrinkage ratio = n/a

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.11	27.70		
dry soil + pan weight, g =	25.83	26.30		
pan weight, g =	20.98	20.91		
moisture, % =	26.4 %	26.0 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = n/a
% sand = n/a
% silt and clay = n/a
% silt = n/a
% clay = n/a
moisture content = n/a

DATE TESTED 02/17/25	TESTED BY K. Summers

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ATTERBERG LIMITS REPORT

PROJECT Operations Center Salamo Road and Greene Street West Linn, Oregon	CLIENT City of West Linn 22500 Salamo Road West Linn, OR 97068	PROJECT NO. WestL-1-01-1	
		ISSUE DATE 02/20/25	PAGE 1 of 1
		LAB ID S25-0317	FIELD ID TP11.1
		DATE SAMPLED 01/24/25	SAMPLED BY E. Uren

MATERIAL DATA

MATERIAL SAMPLED GRAVEL with Silt and Sand and Cobbles	MATERIAL SOURCE Test Pit TP-11 depth = 3 feet	USCS SOIL TYPE no data provided

LABORATORY TEST DATA

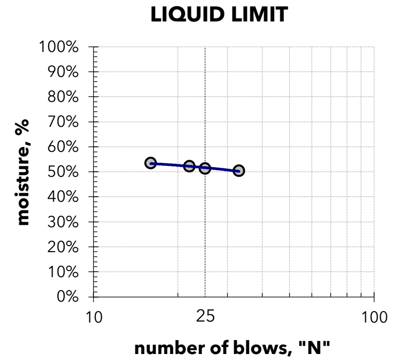
LABORATORY EQUIPMENT Liquid Limit Machine, Hand Rolled	TEST PROCEDURE ASTM D4318 - Method A

ATTERBERG LIMITS

liquid limit = 52
plastic limit = 36
plasticity index = 16

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	33.54	34.33	33.49	32.98
dry soil + pan weight, g =	29.15	29.62	29.19	28.65
pan weight, g =	20.44	20.43	20.95	20.56
N (blows) =	33	25	22	16
moisture, % =	50.4 %	51.3 %	52.2 %	53.5 %



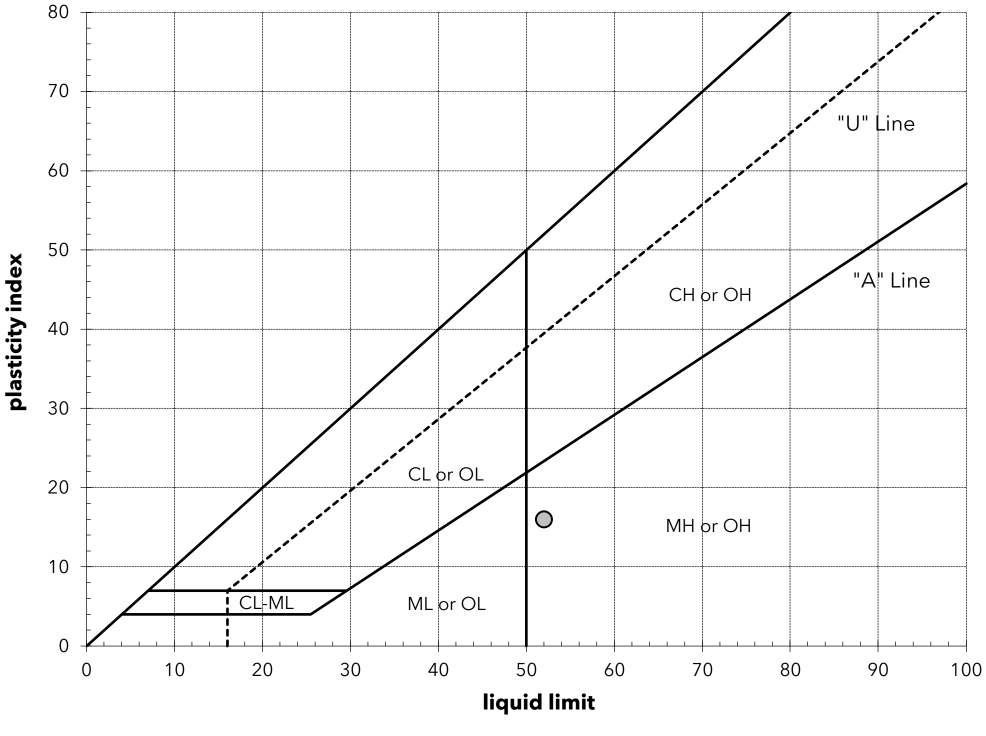
SHRINKAGE

shrinkage limit = n/a
shrinkage ratio = n/a

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	28.26	27.74		
dry soil + pan weight, g =	26.31	25.94		
pan weight, g =	20.93	20.98		
moisture, % =	36.3 %	36.3 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = n/a
% sand = n/a
% silt and clay = n/a
% silt = n/a
% clay = n/a
moisture content = n/a

DATE TESTED 02/18/25	TESTED BY K. Summers

COLUMBIA WEST ENGINEERING, INC. authorized signature

APPENDIX C

APPENDIX C
SEISMIC SURVEY

INTRODUCTION

This appendix summarizes the results of our seismic survey for the Operations Center project in West Linn, Oregon. The objective of the survey was to determine the shear wave velocity profile of approximately 100 feet of the subsurface soil. Fieldwork for the survey was completed on January 23, 2025.

Detailed discussion of the surface and subsurface conditions at the site is presented in the main report.

SCOPE OF SERVICES

Our scope of services included the following:

- Completed one linear ReMi array consisting of 12 geophones.
- Generated additional seismic noise as needed via hammer strikes or other active sources.
- Acquired at least twenty-five 30-second-long data records.
- Pre-processed and analyzed the data to extract a dispersion curve.
- Inverted the dispersion curve and iteratively adjusted the interpreted shear wave velocity profile to align theoretical and modeled dispersion curves.
- Prepared this appendix that includes an estimate of the Vs_{100} and a corresponding seismic site class in accordance with ASCE 7-16.

METHODOLOGY

The seismic survey technique known as ReMi was used to estimate the Vs_{100} for a linear array of geophones deployed at the site. ReMi is a non-invasive surface wave method used to estimate subsurface shear wave velocity profiles using ambient seismic noise to capture surface wave velocities through a linear array of geophones. The technique was developed in 2001 by Dr. John Louie as a method to accurately and efficiently capture the one-dimensional Vs_{100} in a code-compliant manner (Louie 2001). The technique is well suited for areas with pronounced background noise such as urban settings or high-traffic areas. In areas lacking passive noise, active sources such as sledgehammer strikes or pedestrian foot traffic can be used to enhance the seismic signal.

Given the topography and locations of various features at the site (e.g., sloping ground, vegetation, etc.), we were able to deploy an approximately 289-foot-long array in the northwest portion of the site. The array was set generally perpendicular to the nearest arterial road (Salamo Road). The linear array consisted of 12 HG-6 4.5-hertz geophones, with an approximate center-to-center geophone spacing of 26 feet. Approximate GPS coordinates for the first and last geophones are provided in Table C-1, and the approximate array location is shown on Figures C-1 and C-2.

Table C-1. ReMi Array Coordinates

Array	Orientation	Approximate GPS Coordinates	
		Geophone 1	Geophone 12
A-1	W-E	45.350501, -122.643577	45.350306, -122.642430

Geophones equipped with spikes were driven into the ground. Each geophone installation was checked for plumb (within +/-5 degrees) and adequate contact with the surrounding soil. The surface wave data were acquired using a ReMiDAQ 4-12 channel seismograph for approximately 90 minutes. Noise sources included 10-pound sledgehammer blows to a plastic strike plate on both ends and at the midpoint of the array, a hydraulic excavator operating within 100 feet of the array, and ambient noise generated by traffic on nearby roads.

Pre- and post-processing was completed using Terēan's proprietary VsSurf ReMi 1dS software, which indicated that the data collected along the array were sufficient in quality to characterize shear wave velocity to a minimum depth of 100 feet BGS. A minimum of 10 individual records were "stacked" and inverted to facilitate dispersion curve selection along the lowest velocity envelope. The dispersion curve was then imported into the VsSurf ReMi 1dS Disper module, which was used to perform forward modeling of the interpreted shear wave velocity profile. The shear wave velocity profile was adjusted based on our understanding of the site geology and subsurface conditions until the theoretical dispersion curve fit the selected dispersion points within an acceptable margin.

RESULTS

Table C-2 summarizes the interpreted one-dimensional Vs_{100} for the array. The interpreted value should be thought of as an average representation along the length of the array. Figure C-3 shows the interpreted shear wave velocity profile from the ground surface to a depth of 100 feet BGS. Note that the velocity structure shown in Table C-2 and on Figure C-3 represents a non-unique interpretation of the geophysical data and other interpretations are possible; however, the resulting Vs_{100} value is generally considered to be reliable for the purpose of seismic design in accordance with ASCE 7-16.

Table C-2. Average Shear Wave Velocity in Upper 100 Feet BGS

Array	Depth (feet BGS)	Shear Wave Velocity (fps)	Vs_{100} (fps)	ASCE 7-16 Site Class
A-1	0 – 7	727	2,053	C
	7 – 11	870		
	11 – 27	1,112		
	27 – 100	3,661		

REFERENCES

ASCE 2016. *Minimum Design Loads and Associated Criteria for Buildings and Other Structures.* ASCE Standard ASCE/SEI 7-16.

Louie, John N. 2001. "Faster, Better: Shear-Wave Velocity to 100 Meters Depth from Refraction Microtremor Arrays." *Bulletin of the Seismological Society of America*, vol. 91, no. 2, 1 April, pp. 347–364, https://doi.org/10.1785/0120000098.

VsSurf ReMi 1dS. Version 2.1.7. Terēan. https://www.terean.com/applications.

LEGEND

⛶ APPROXIMATE SITE BOUNDARY

⊢⊣ APPROXIMATE REMI ARRAY

N

0 20 40 60 80 100 200

NOTES:
1. AERIAL PHOTO SOURCED FROM GOOGLE EARTH.
2. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
3. REFER TO REPORT TEXT FOR EXPLORATION DESCRIPTIONS.

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SALAMO ROAD AND GREEN STREET

ARRAY LOCATION
PLAN

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FIGURE
C-1

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ARRAY PHOTOGRAPH
OPERATIONS CENTER
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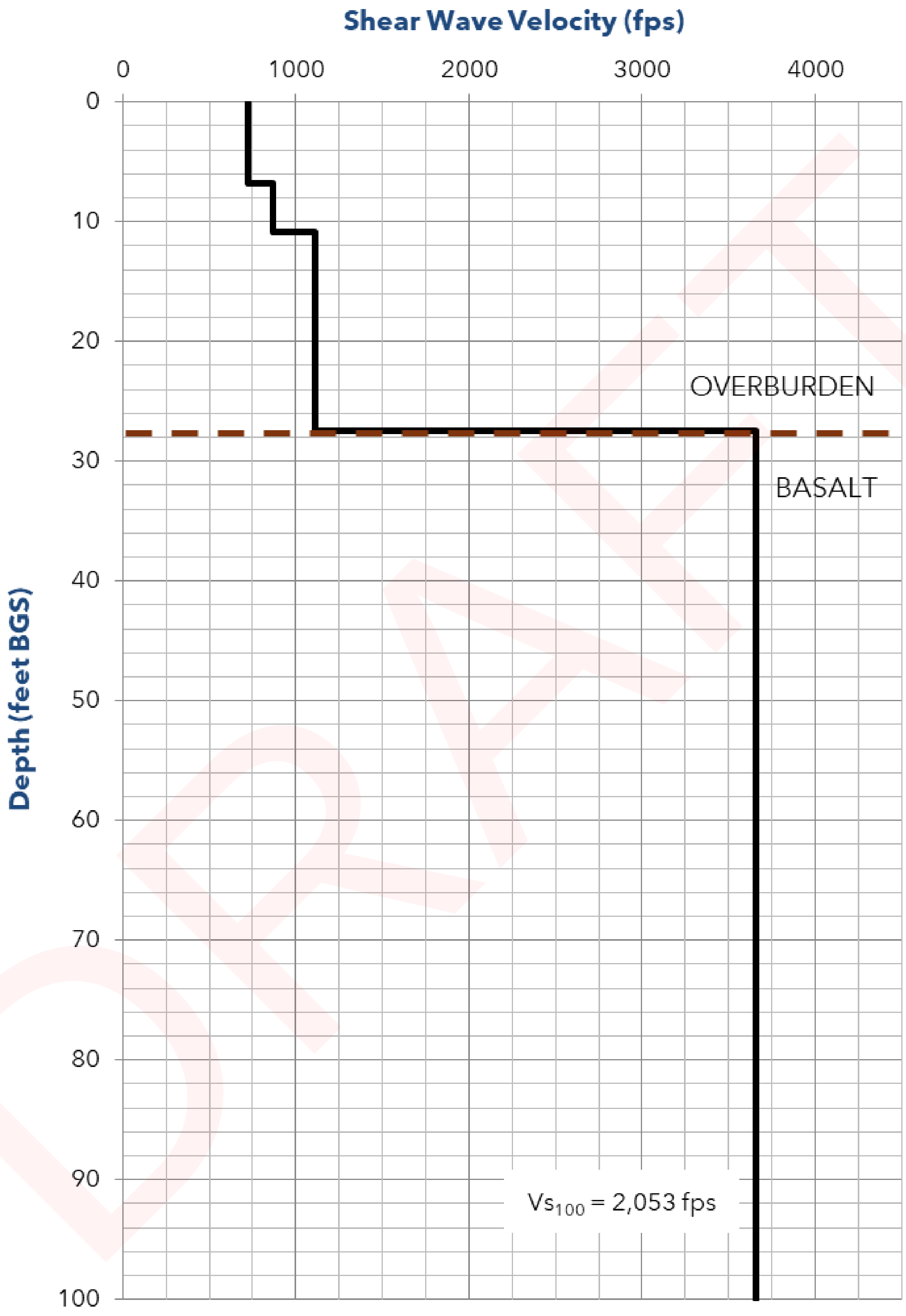
FIGURE
C-2



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WESTL-1-01-1

APRIL 2025

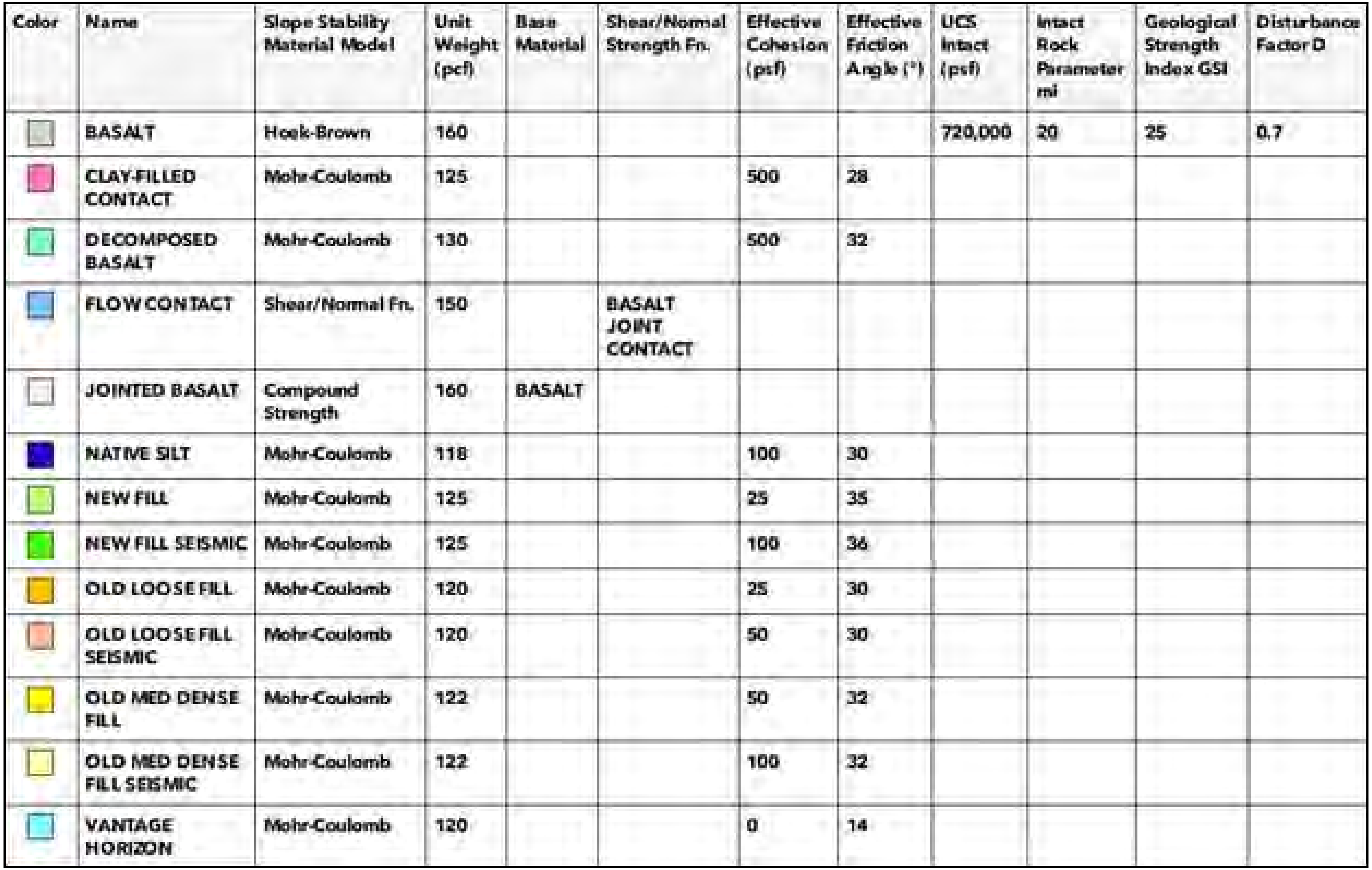
SHEAR WAVE VELOCITY PROFILE
OPERATIONS CENTER
WEST LINN, OREGON

FIGURE
C-3

APPENDIX D

APPENDIX D
SLOPE STABILITY RESULTS

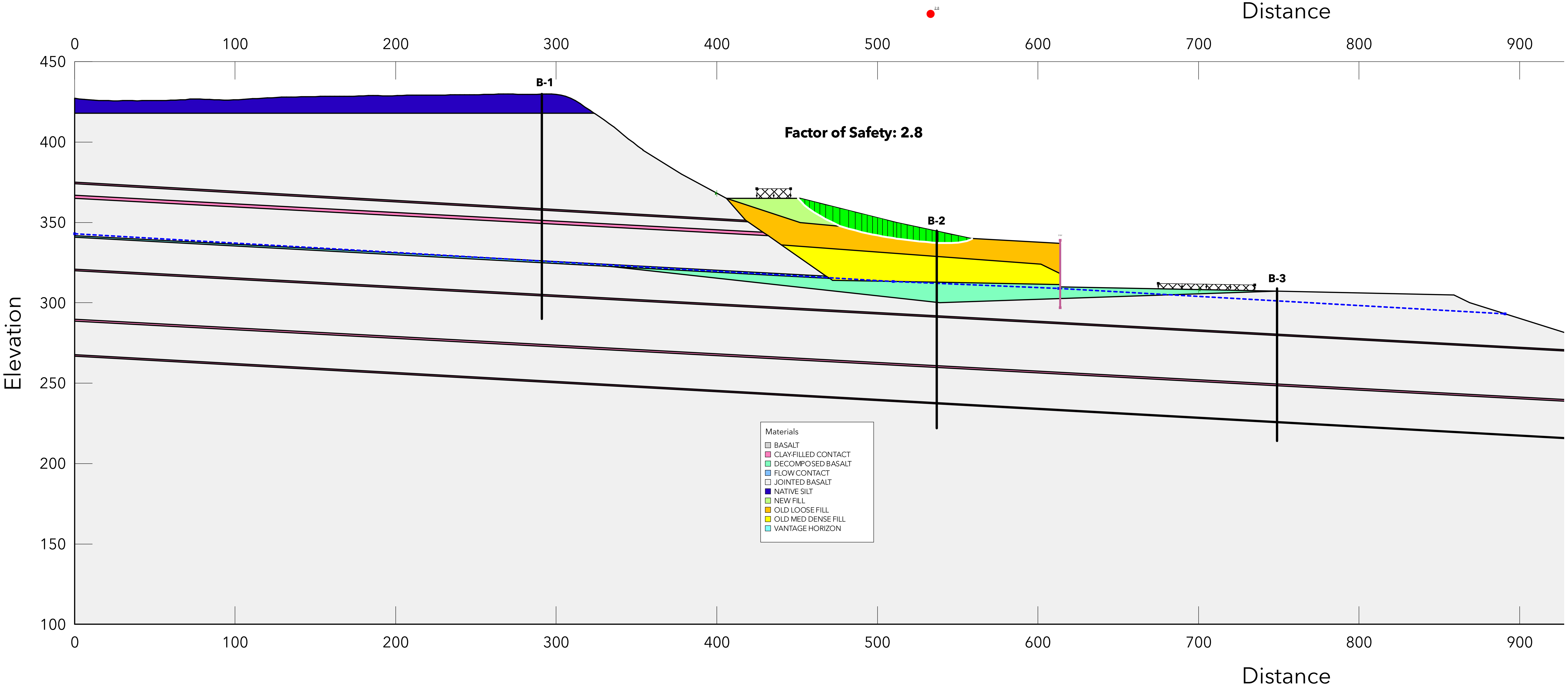
Material properties, model geometry, and critical failure surfaces from the slope stability analysis are presented in this appendix.



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Base Material	Shear/Normal Strength Fn.	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D
⬜	BASALT	Hoek-Brown	160					720,000	20	25	0.7
🟥	CLAY-FILLED CONTACT	Mohr-Coulomb	125			500	28				
🟩	DECOMPOSED BASALT	Mohr-Coulomb	130			500	32				
🟦	FLOW CONTACT	Shear/Normal Fn.	150		BASALT JOINT CONTACT						
⬜	JOINTED BASALT	Compound Strength	160	BASALT							
🟦	NATIVE SILT	Mohr-Coulomb	118			100	30				
🟩	NEW FILL	Mohr-Coulomb	125			25	35				
🟩	NEW FILL SEISMIC	Mohr-Coulomb	125			100	36				
🟧	OLD LOOSE FILL	Mohr-Coulomb	120			25	30				
🟧	OLD LOOSE FILL SEISMIC	Mohr-Coulomb	120			50	30				
🟨	OLD MED DENSE FILL	Mohr-Coulomb	122			50	32				
🟨	OLD MED DENSE FILL SEISMIC	Mohr-Coulomb	122			100	32				
🟦	VANTAGE HORIZON	Mohr-Coulomb	120			0	14				

MATERIAL PROPERTIES USED IN SLOPE STABILITY MODELS

CROSS-SECTION 1: PROPOSED ROADWAY FILL SLOPE STATIC LOADING



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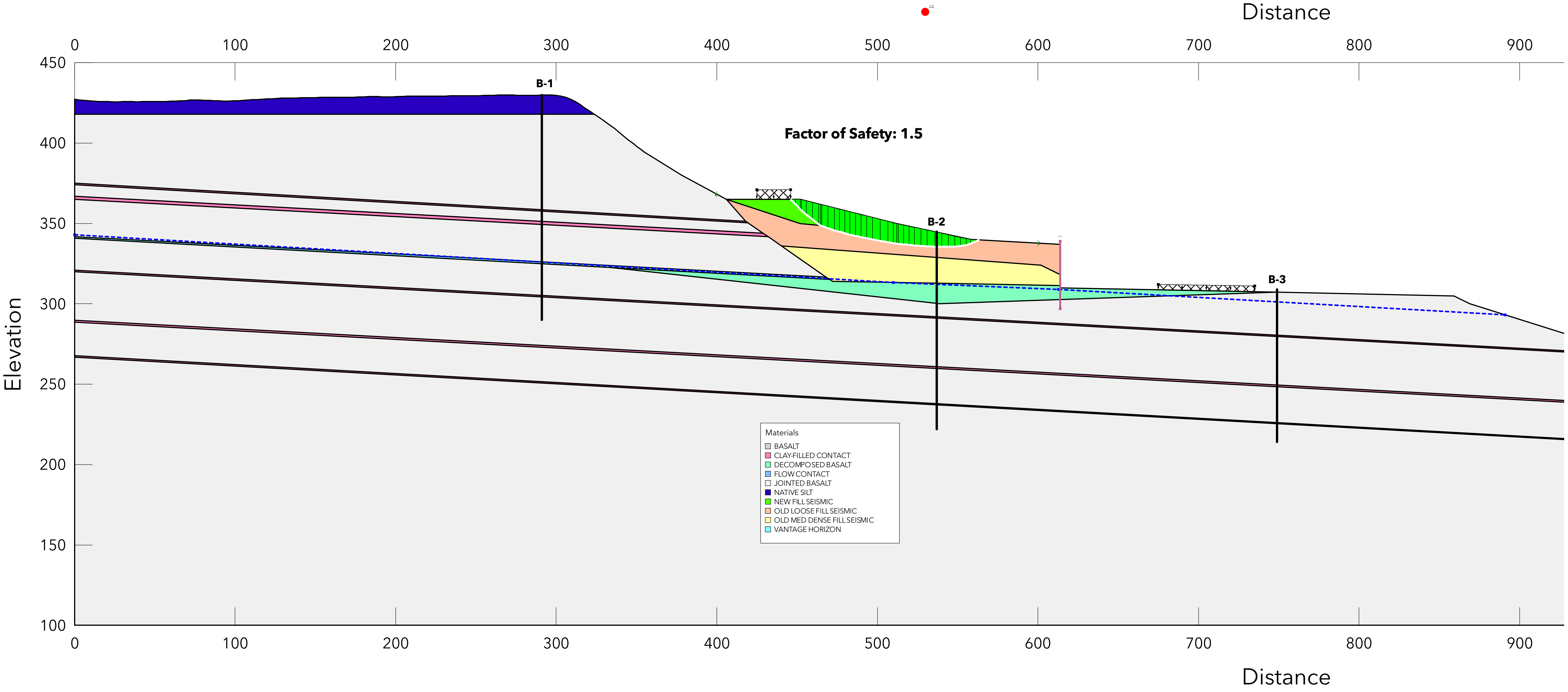
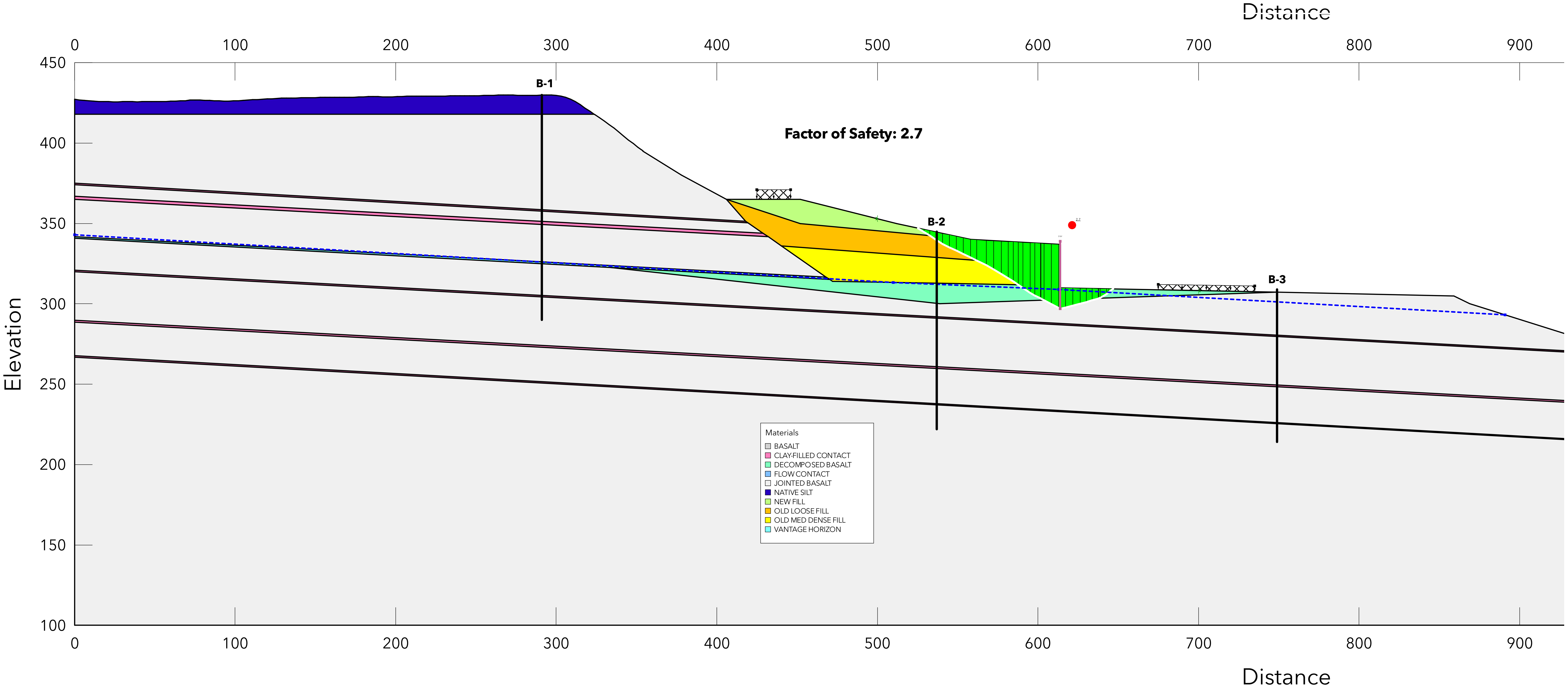


CROSS-SECTION 1: GLOBAL WALL STABILITY STATIC LOADING



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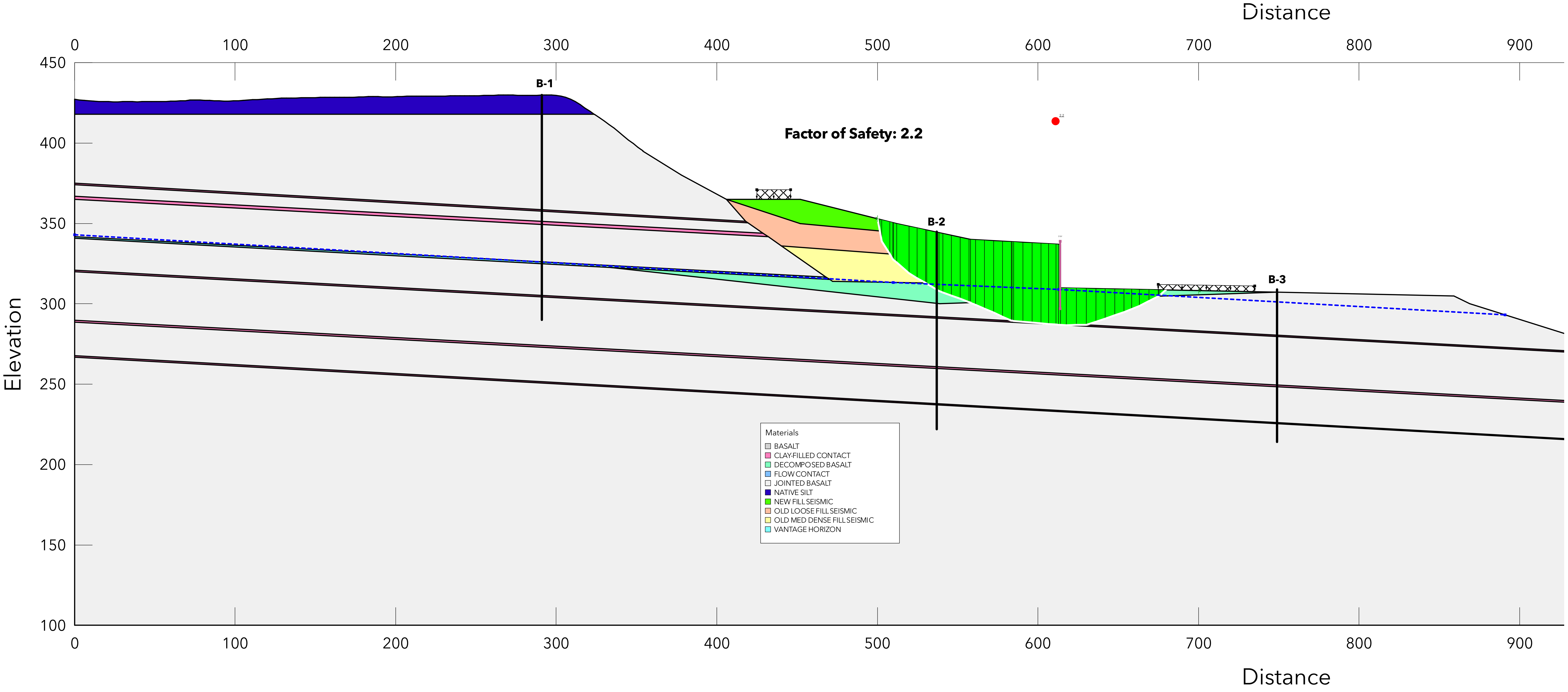
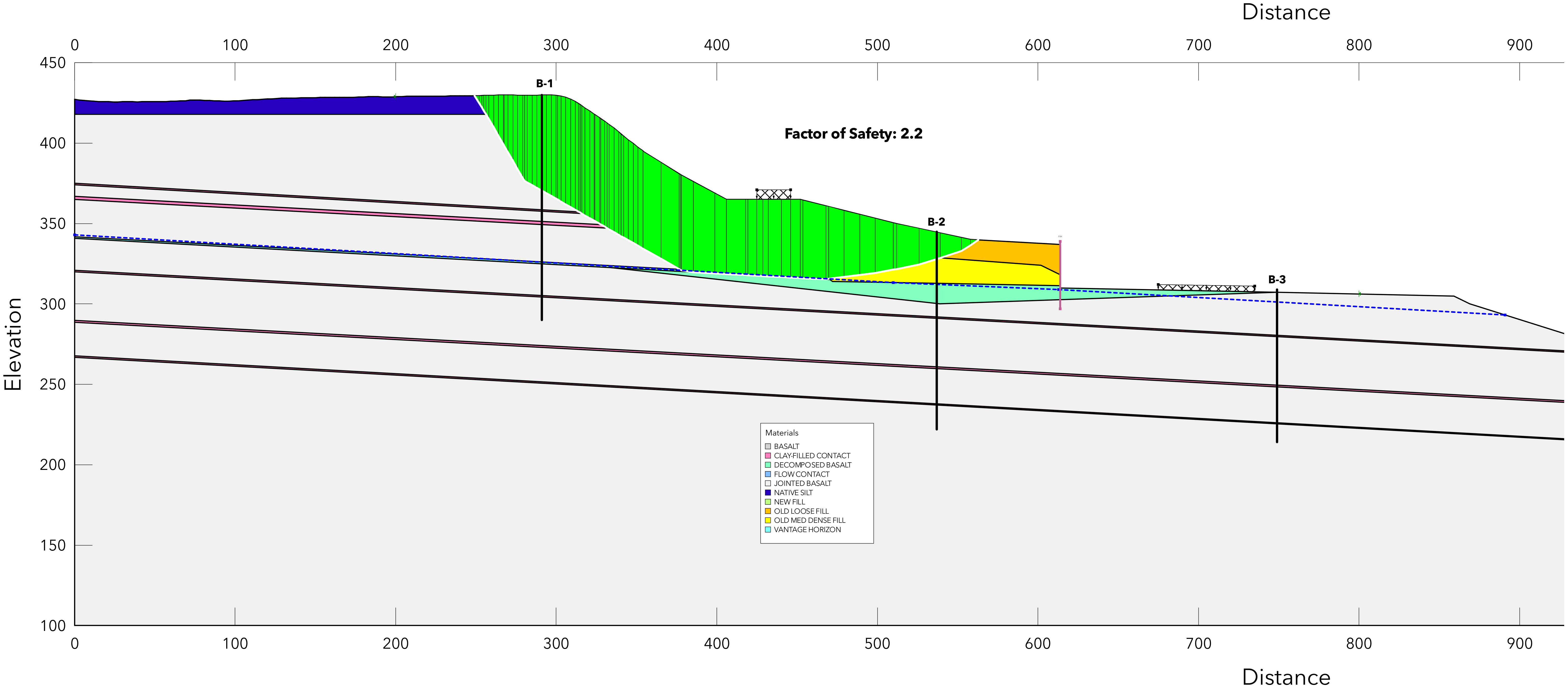
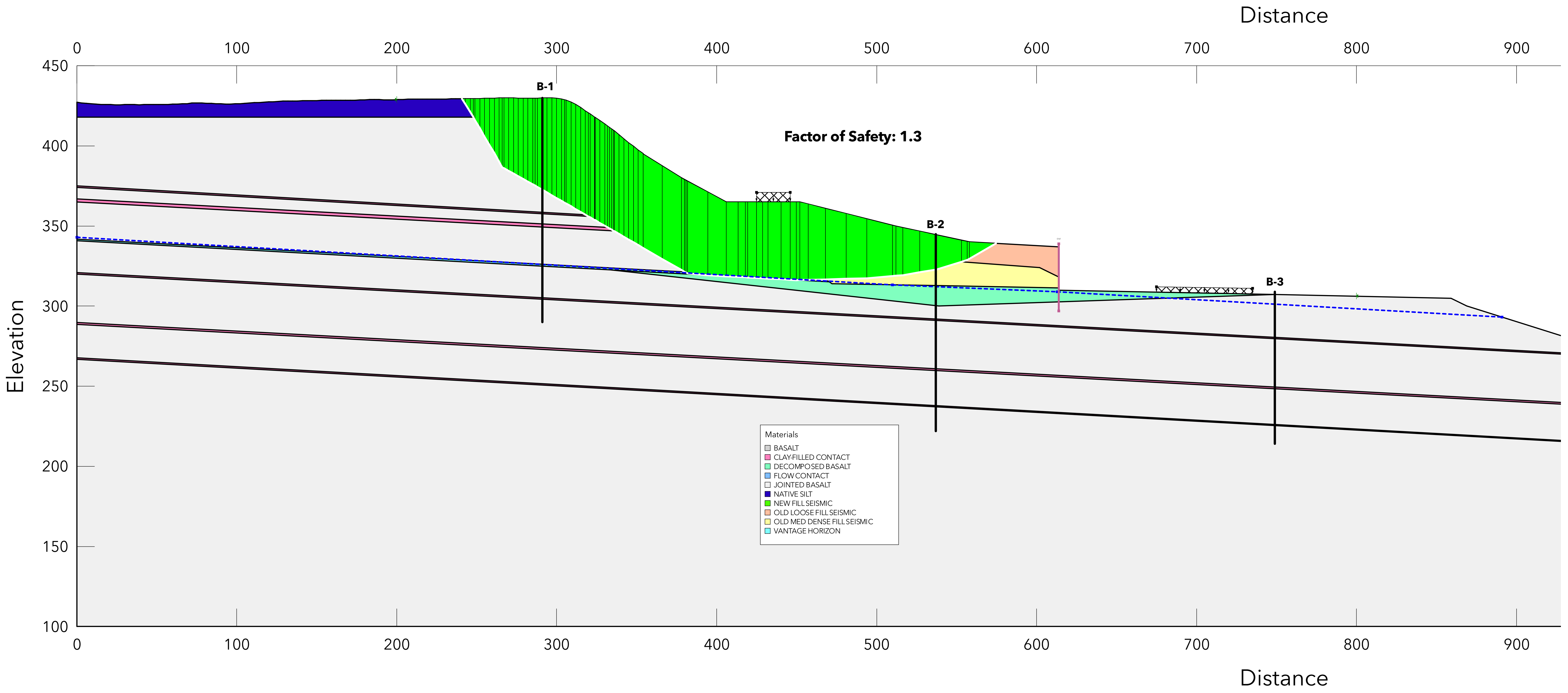


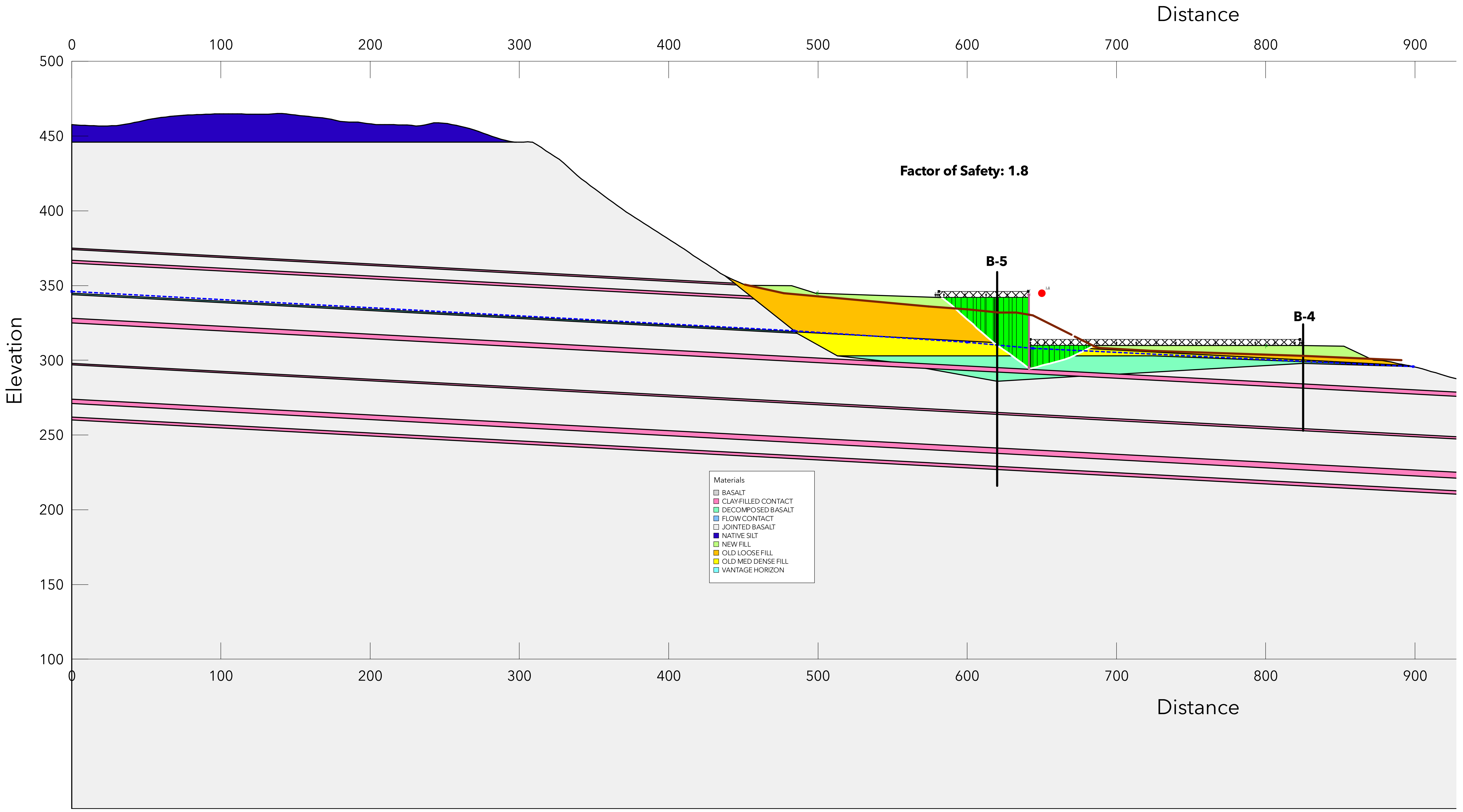
CROSS-SECTION 1: UPPER SLOPE STATIC LOADING



CROSS-SECTION 1: UPPER SLOPE PSEUDOSTATIC LOADING



CROSS-SECTION 2: GLOBAL WALL STABILITY STATIC LOADING



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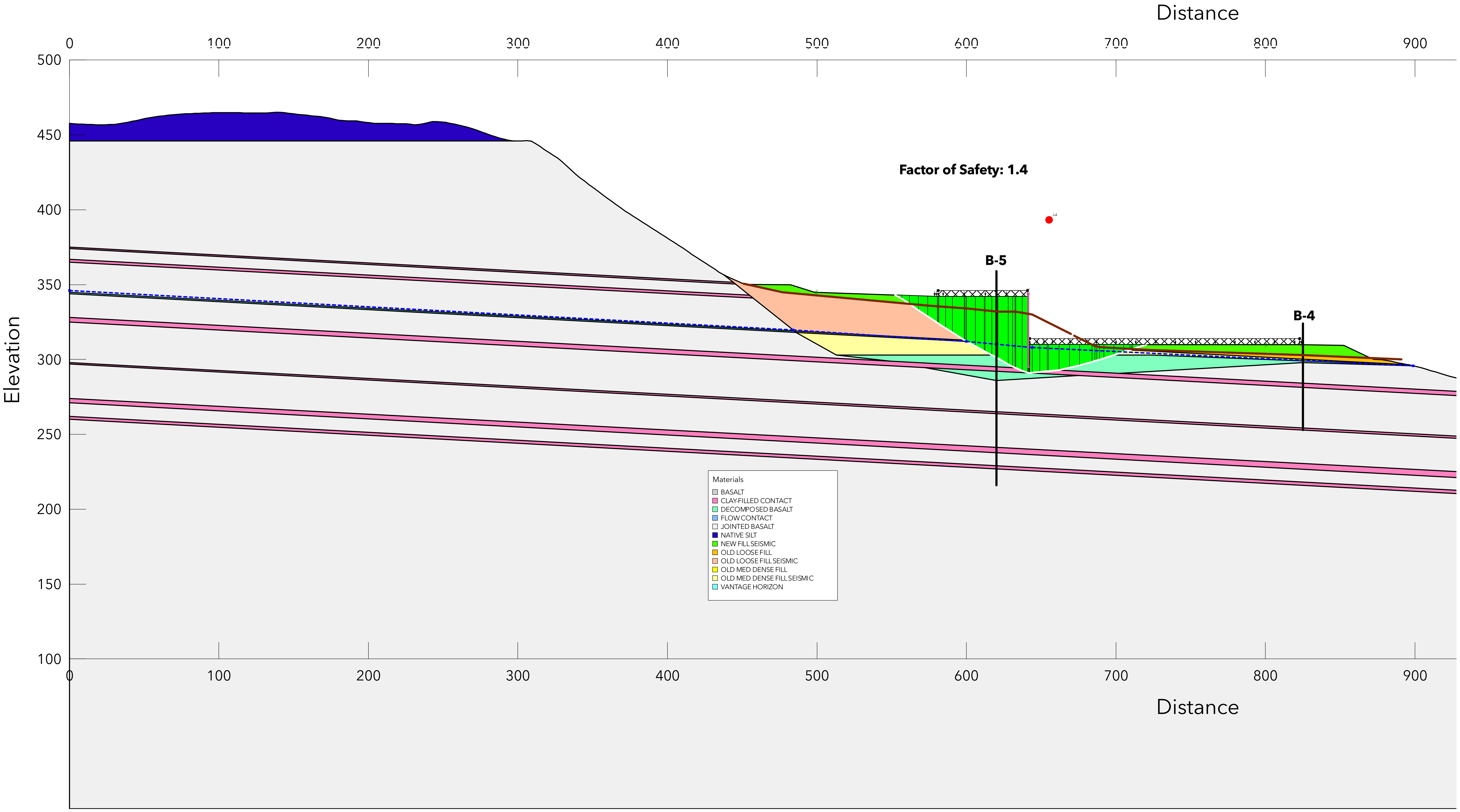
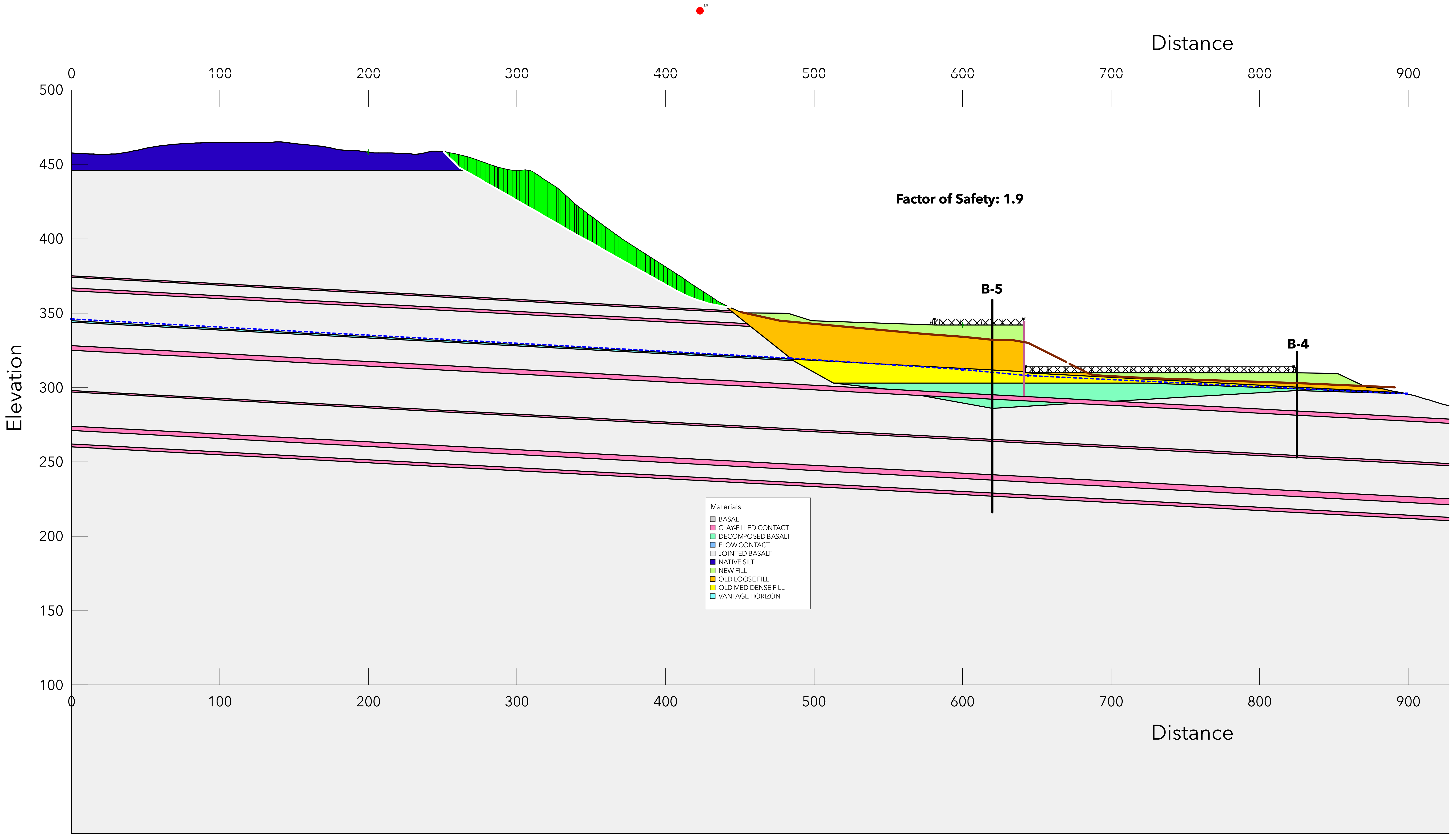


CROSS-SECTION 2: UPPER SLOPE STATIC LOADING



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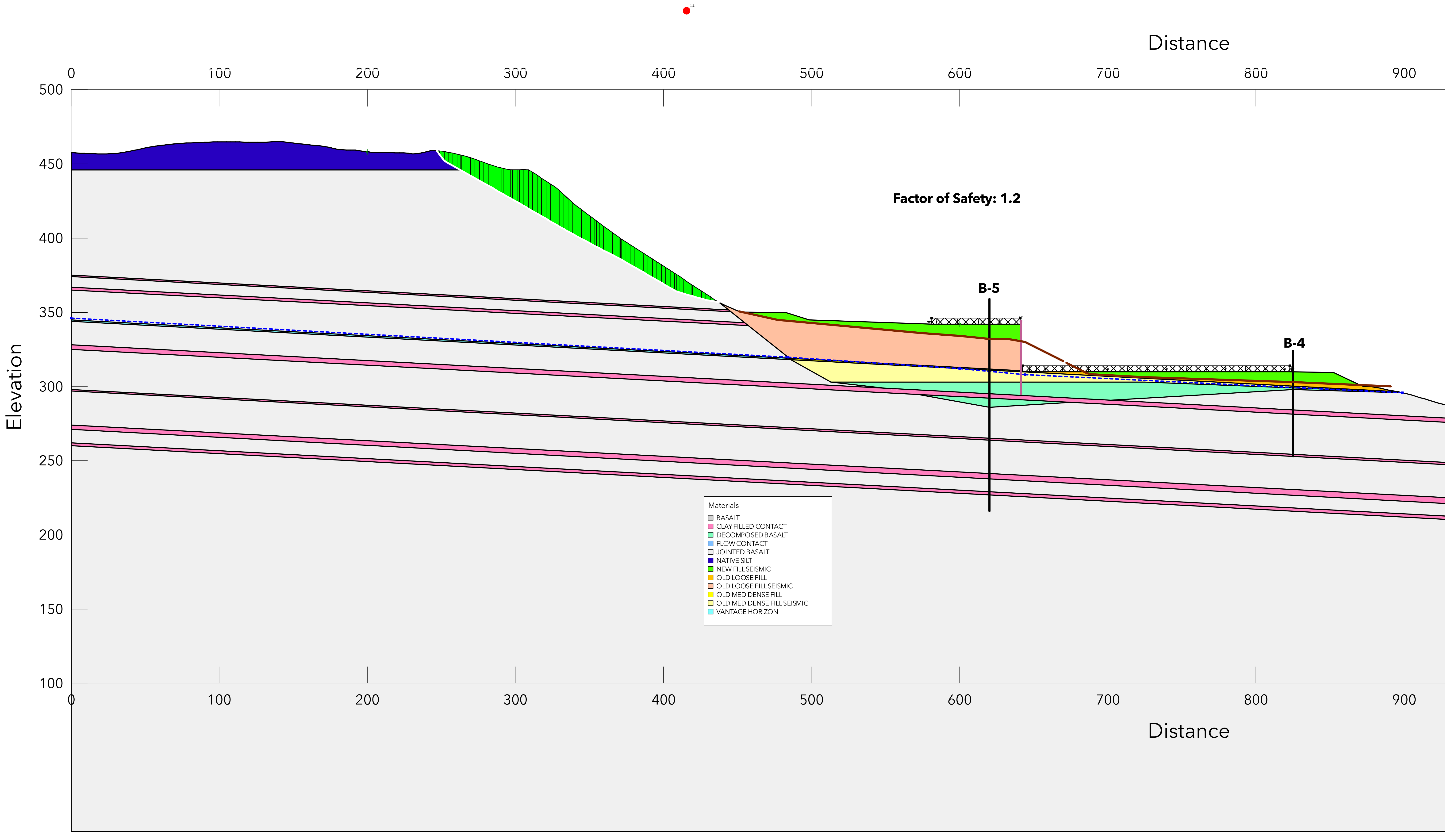
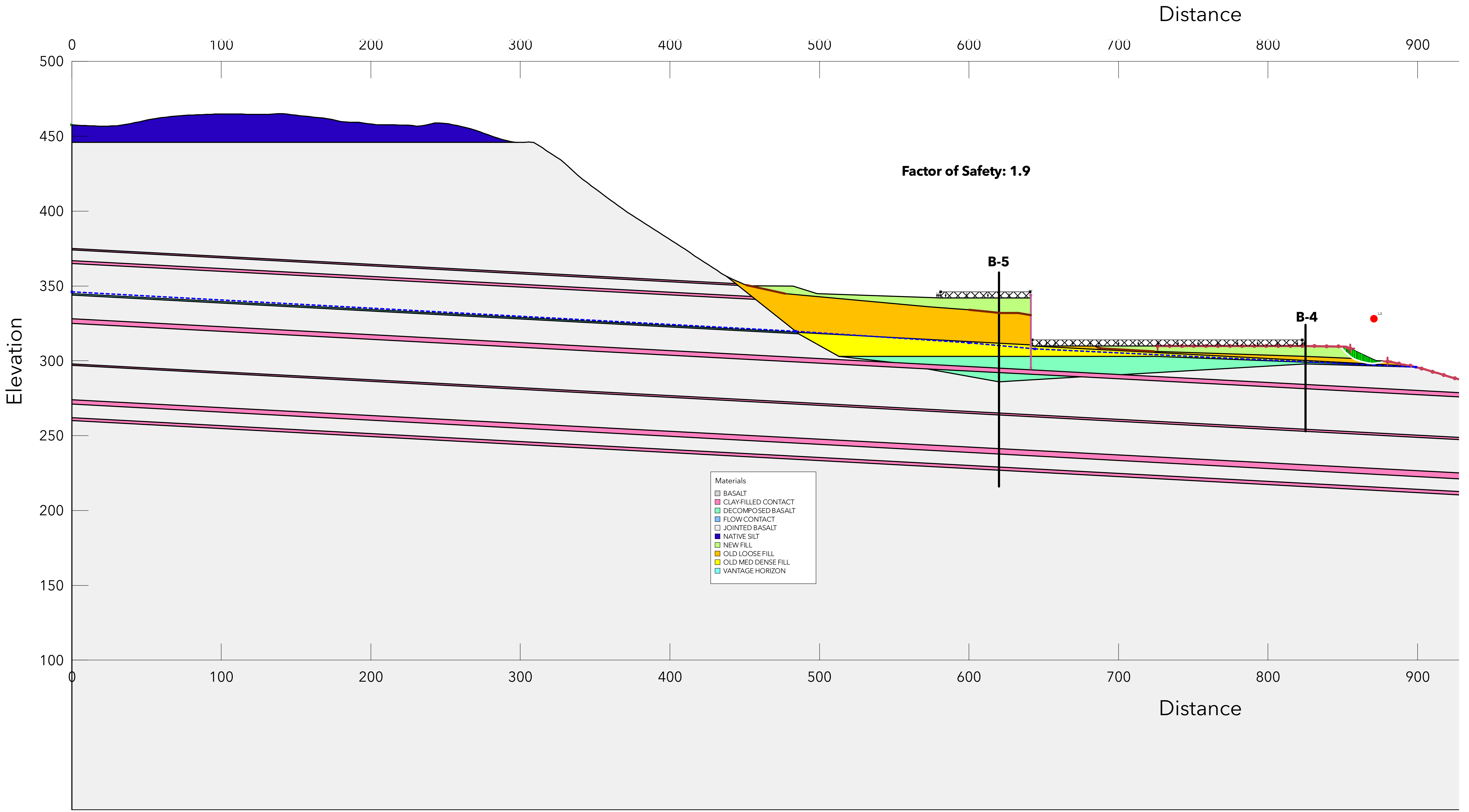
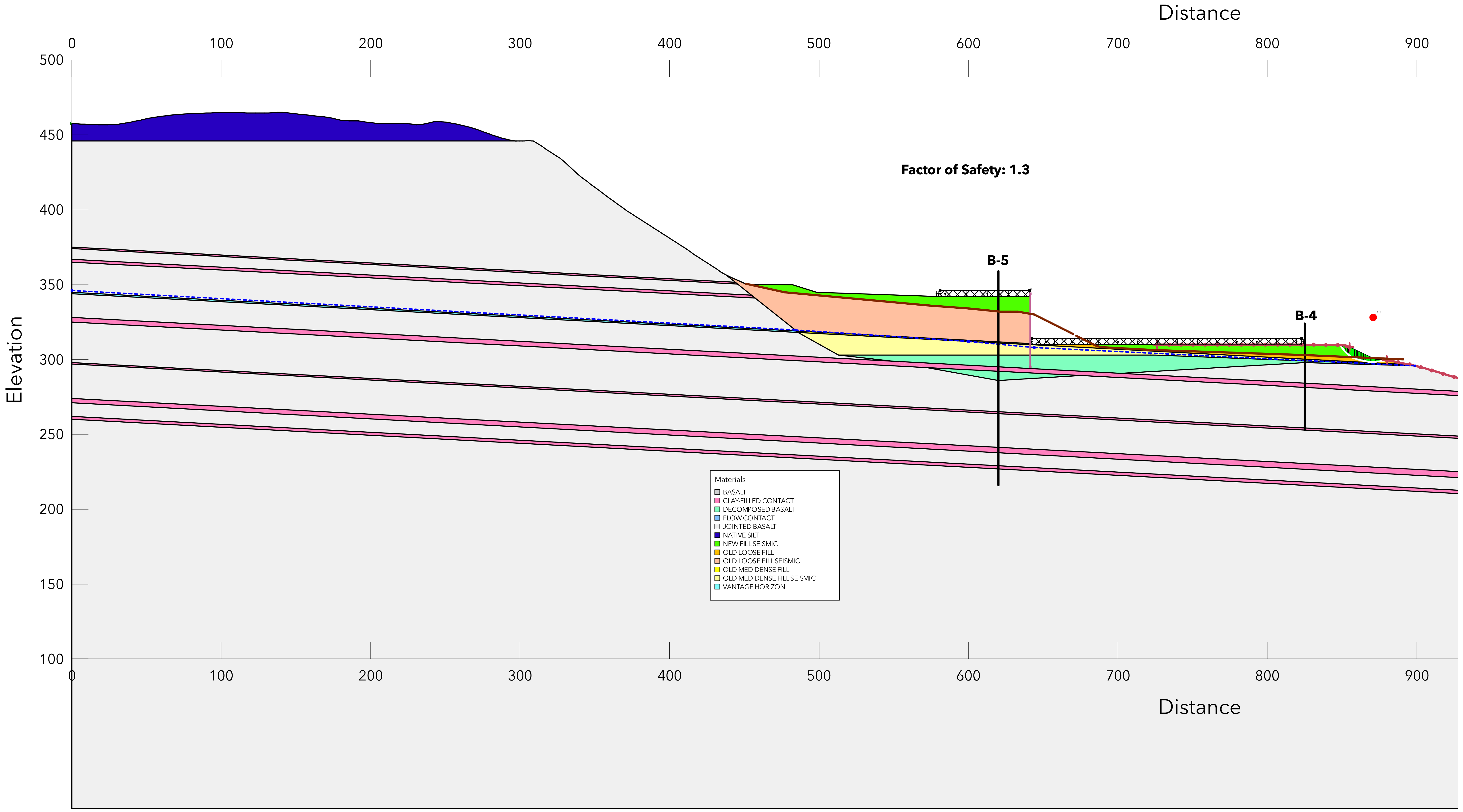


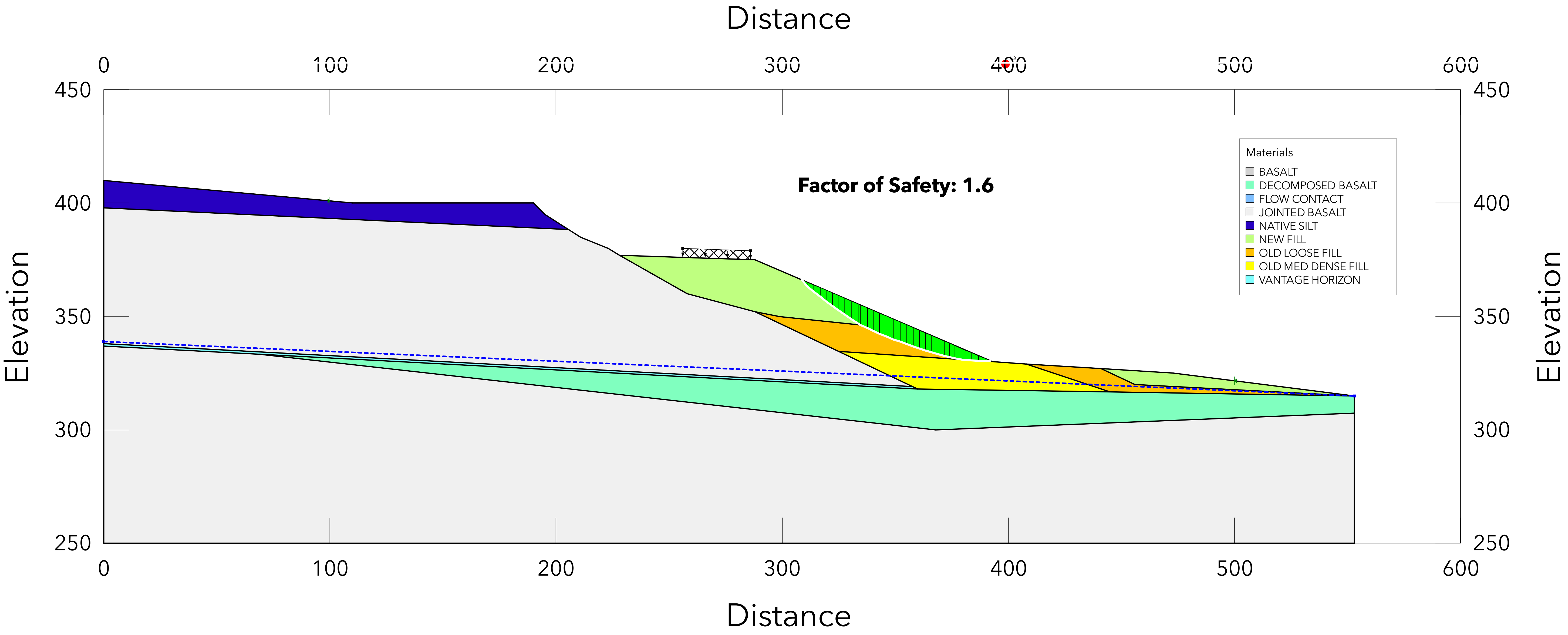
CROSS-SECTION 2: SLOPE SOUTH OF PROPOSED MAINTENANCE BUILDING STATIC LOADING



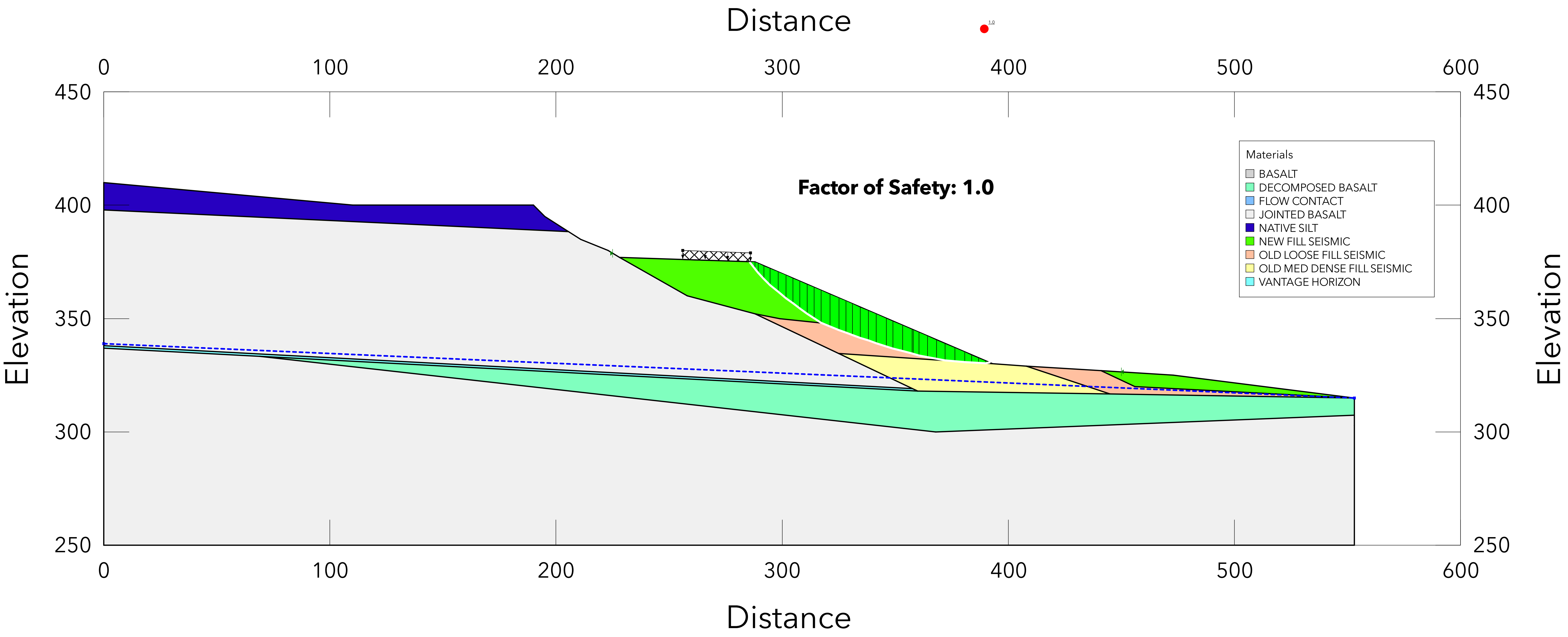
CROSS-SECTION 2: SLOPE SOUTH OF PROPOSED MAINTENANCE BUILDING PSEUDOSTATIC LOADING



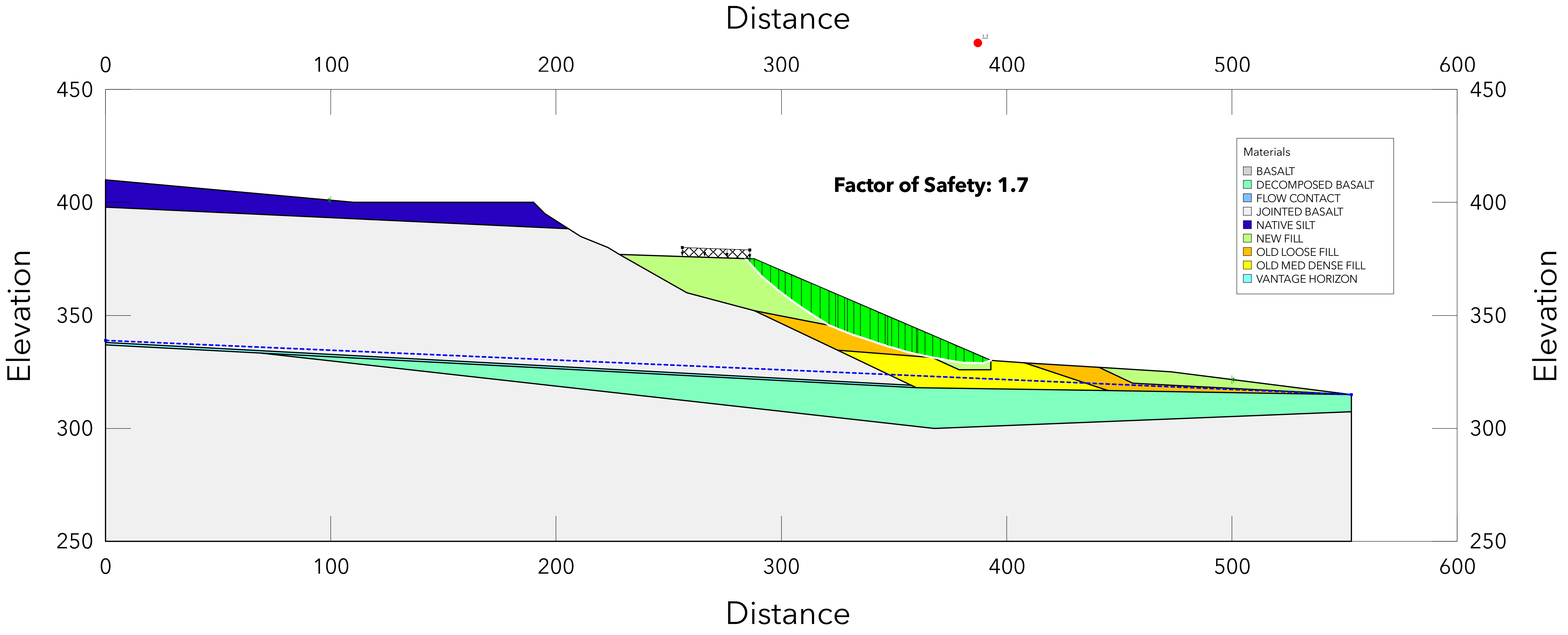
CROSS-SECTION 3: PROPOSED ROADWAY FILL SLOPE STATIC LOADING



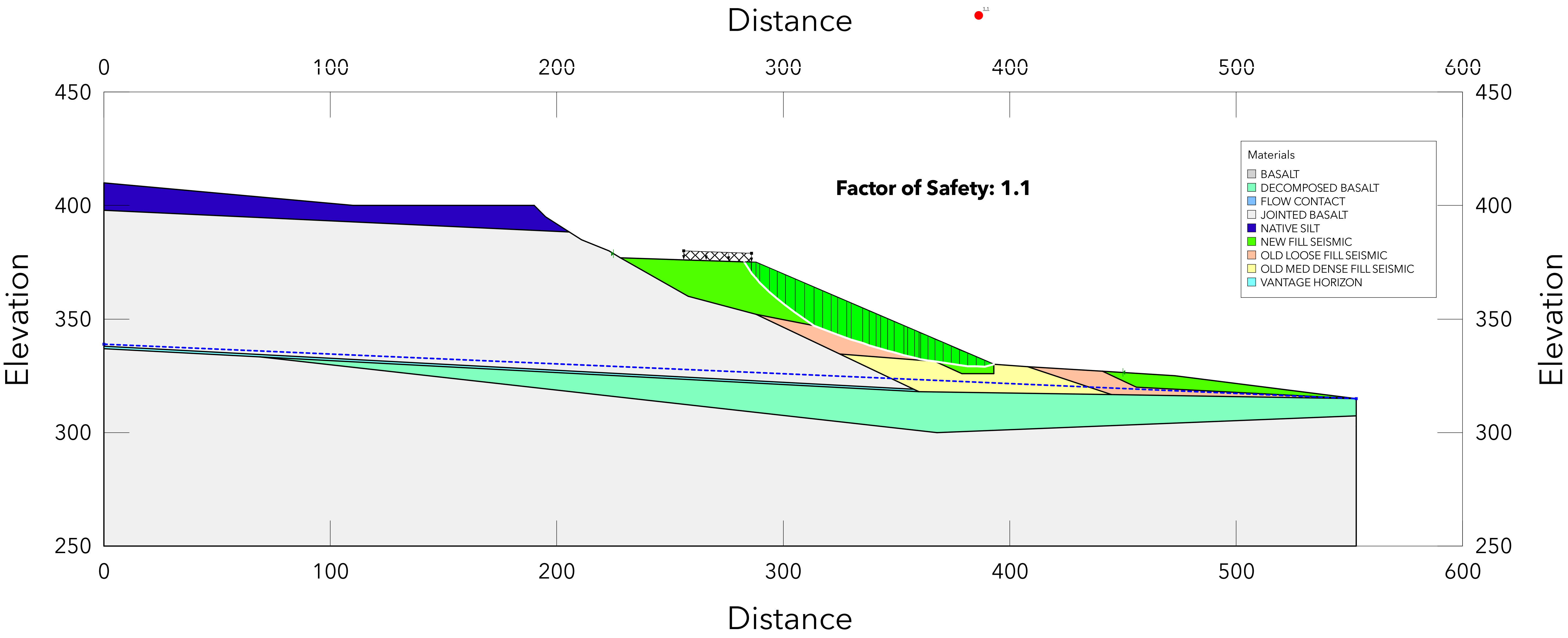
CROSS-SECTION 3: PROPOSED ROADWAY FILL SLOPE PSEUDOSTATIC LOADING



CROSS-SECTION 3: IMPROVED PROPOSED ROADWAY FILL SLOPE STATIC LOADING



CROSS-SECTION 3: IMPROVED PROPOSED ROADWAY FILL SLOPE PSEUDOSTATIC LOADING



APPENDIX E

APPENDIX E
SITE-SPECIFIC SEISMIC HAZARD EVALUATION

INTRODUCTION

This appendix summarizes the results of a site-specific seismic hazard evaluation for the proposed Operations Center in West Linn, Oregon. This seismic hazard evaluation was performed in accordance with the requirements of the 2022 SOSSC and ASCE 7-16.

SITE CONDITIONS

REGIONAL GEOLOGY AND SUBSURFACE CONDITIONS

The regional geology and subsurface conditions at the site are presented in the main report.

SEISMIC SETTING

Earthquake Source Zones

Three scenario earthquakes were considered for this study consistent with the local seismic setting. Two of the possible earthquake sources are associated with the CSZ, and the third event is a shallow, local crustal earthquake that could occur in the North American Plate. The three earthquake scenarios are discussed below.

Regional Events

The CSZ is the region where the Juan de Fuca Plate is being subducted beneath the North American Plate. This subduction is occurring in the coastal region between Vancouver Island and northern California. Evidence has accumulated suggesting that this subduction zone has generated eight great earthquakes in the last 4,000 years, with the most recent event occurring approximately 300 years ago (Weaver and Shedlock 1991). The fault trace is mapped approximately 50 to 120 km off the Oregon Coast.

Two types of possible subduction zone earthquakes were considered in this study:

1. An interface event earthquake on the seismogenic part of the interface between the Juan de Fuca Plate and the North American Plate on the CSZ. This source is capable of generating earthquakes with a M_W of 9.0+.
2. A deep intraplate earthquake on the seismogenic part of the subducting Juan de Fuca Plate. These events typically occur at depths of between 30 and 60 km. This source is capable of generating an event with a M_W of up to 8.0.

Local Events

In addition to a regional subduction earthquake, an earthquake could occur on a local crustal fault. Figure E-1 shows the locations of faults with potential Quaternary movement near the site (USGS 2025b). Figure E-2 shows the interpreted locations of historical seismic events with an M_W of 3 or larger (PNSN 2025). Table E-1 provides information on local faults within a 20-km radius of the site. The faults within 15 km of the site have a slip rates less than 2 mm/yr; therefore, the site is not considered a "near-fault site" according to Section 11.4 of ASCE 7-16.

Table E-1. Nearest Mapped Crustal Faults[2]

Source	Closest Mapped Distance[1] (km)	Mapped Length (km)	Slip Rate (mm/yr)
Bolton fault	2.8	9	Less than 0.2
Canby-Molalla fault	4.0	50	Less than 0.2
Oatfield fault	6.2	29	Less than 0.2
Portland Hills fault	7.4	49	Less than 0.2
Damascus-Tickle Creek fault zone	9.9	10 and 16	Less than 0.2
East Bank fault	18.1	29	Less than 0.2
Beaverton fault zone	19.2	15	Less than 0.2

1. Rounded to nearest 0.5 km
2. As reported by USGS (2025b)

DESIGN EARTHQUAKE

Based on deaggregation of spectral acceleration at two spectral periods using the USGS Earthquake Hazards Toolbox (USGS 2025a), the seismic hazard at the site is dominated by the CSZ interface source at both periods. A summary of the seismic hazard contributions is presented in Table E-2.

Table E-2. Seismic Hazard Deaggregation

Source	Percent Contribution to Overall Hazard[1]	
	PGA	T = 0.5 seconds[2]
CSZ – interface	48	56
CSZ – intraslab	15	8
Local faults	21	22
Gridded seismicity	16	14

1. Individual contributions rounded to the nearest percent; therefore, the sum may be greater than 100.
2. Where T is the structural period.

The "mean" earthquakes associated with the CSZ interface and local fault sources were M_w of 9.0 with a rupture distance of 102 km and M_w of 6.6 with a rupture distance of 7.2 km. The mean earthquake scenario associated with local faults is generally representative of a Portland Hills fault rupture. These two mean earthquake scenarios were largely insensitive to the structural period used in the deaggregation.

SEISMIC DESIGN

SITE CLASS

Based on the shear wave velocity measurements acquired from the ReMi analysis described in Appendix C, the proposed structures founded on existing fill should be designed according to

Site Class C per Chapter 20 of ASCE 7-16. The shear wave velocity profile and V_{s100} calculations are summarized in Table E-3. For structures founded directly on basalt, Site Class B is appropriate for seismic design.

Table E-3. Seismic Site Class Determination for Array A-1

Array	Depth[1] (feet BGS)	Thickness (feet)	Shear Wave Velocity[1] (fps)	Thickness/Shear Wave Velocity (second)
A-1	0 – 7	7	727	0.009629
	7 – 11	4	870	0.004598
	11 – 27	16	1,112	0.014388
	27 – 100	73	3,661	0.019940
Sum		100	NA	0.048555
Average Shear Wave Velocity in the Upper 100 Feet BGS, V_{S100} (fps)		2,053		
ASCE 7-16 Site Class		C		

1. Values rounded to nearest whole number for sake of reporting

SEISMIC DESIGN PARAMETERS

Based on the subsurface conditions encountered at the site, it is our opinion that amplification factors prescribed in ASCE 7-16 for Site Class C are appropriate for design of structures founded on or within the fill buttress and Site Class B is appropriate for structures founded on basalt; therefore, a site-response analysis is not required. The parameters in Table E-4 can be used for seismic design of the proposed structures that are considered Site Class C and Table E-5 can be used for structures that are considered Site Class B.

Table E-4. Recommend ASCE 7-16 Site Class C Seismic Design Parameters[1]

Parameter	Site Class C	
	Short Period (T_s)	1-Second Period (T_1)
MCE spectral response acceleration, S	$S_s = 0.835$ g	$S_1 = 0.376$ g
Site coefficient, F	$F_a = 1.2$	$F_v = 1.5$
Adjusted spectral response acceleration, S_M	$S_{MS} = 1.003$	$S_{M1} = 0.565$
Design spectral response acceleration, S_D	$S_{DS} = 0.668$	$S_{D1} = 0.376$
PGA	0. 376 g	
PGA_M	0.452 g	

1. Parameters obtained using the ASCE 7 hazard tool

Table E-5. Recommend ASCE 7-16 Site Class B Seismic Design Parameters[1]

Parameter	Site Class B	
	Short Period (T_s)	1-Second Period (T_1)
MCE spectral response acceleration, S	S_s = 0.836 g	S_1 = 0.377 g
Site coefficient, F	F_a = 0.9	F_v = 0.8
Adjusted spectral response acceleration, S_M	S_{MS} = 0.752	S_{M1} = 0.301
Design spectral response acceleration, S_D	S_{DS} = 0.501	S_{D1} = 0.201
PGA	0. 376 g	
PGA_M	0.339 g	

1. Parameters obtained using the ASCE 7 hazard tool

SEISMIC-INDUCED HAZARDS

LIQUEFACTION AND CYCLIC STRAIN SOFTENING

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles. It is most commonly triggered by dynamic loading associated with earthquake shaking and can dramatically reduce the shear strength and stiffness of the soil until excess pore pressures can dissipate. In general, loose, saturated sandy soil with low silt and clay content is the most susceptible to liquefaction.

Conversely, fine-grained, plastic soil may be susceptible to a phenomenon known as cyclic softening, which is characterized by a decrease in strength and stiffness that is gradual relative to liquefaction. Soft or sensitive silt and clay subjected to strong shaking are generally most susceptible to significant cyclic softening.

The VWPs installed to depths between 97.5 and 98 feet BGS indicate that the depth to static groundwater is greater than 75 feet BGS and the mapped depth to groundwater at the site is greater than 100 feet BGS (Snyder 2009). Given the depth to the water table and the relatively shallow basalt encountered in our explorations, liquefaction and/or cyclic strain softening are not design considerations for this project.

LATERAL SPREADING

Lateral spreading is a liquefaction-related seismic hazard and occurs on gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank. Liquefied soil adjacent to an open face can flow toward the open face, resulting in lateral ground displacement. The site is gently to moderately sloped, but liquefaction and/or cyclic strain softening are not likely; therefore, lateral spreading is not an important design consideration for this project.

FAULT SURFACE RUPTURE

Active faults are not mapped directly beneath the site. The nearest fault, the Bolton fault, is approximately 2.4 km away; therefore, it is our opinion that the risk of fault surface rupture at the site is low.

GROUND MOTION AMPLIFICATION

Soil capable of significantly amplifying ground motions beyond the levels determined using code-based site coefficients was not encountered during the field explorations. The main report provides a detailed description of the subsurface conditions encountered. We conclude that the level of amplification implicit in the code-based site class recommendation is appropriate for the project.

LANDSLIDE

Earthquake-induced landsliding generally occurs on steeper slopes comprised of relatively weak soil and rock deposits. As discussed in the main report, the site is located within a zone of mapped landslide deposits, in an area considered to have "high" susceptibility to shallow landslides (DOGAMI 2025).

We considered the impact of an earthquake on the stability of the on-site slopes by completing a pseudo-static slope stability analysis. Details of that analysis are provided in the main report. Results of the pseudo-static slope stability analysis indicate that all but one of the slopes at the site that we analyzed have factors of safety of greater than 1.1 when subjected to a pseudo-static load equal to one-half of the PGA$_M$ value presented in Table E-4. Slopes are typically considered to be generally stable (deformation less than 2 inches) when the calculated factor of safety is greater than 1.1 under pseudo-static loading (ODOT 2024).

The only slope in our analysis that did not have a factor of safety of 1.1 under pseudo-static loading was a marginally stable (factor of safety equal to 1.0) fill slope considered in cross section 3. The fill slope will support a portion of the proposed access road near Salamo Road. Based on the topography in the vicinity of the cross section and results of our analysis, slope displacement under seismic loading is unlikely to involve a considerable volume of soil or impact any existing or proposed structures; however, it is possible that under seismic loading the slope could displace enough to adversely impact trafficability of the access road. We performed supplementary analysis considering an "improved" fill slope condition and found that the pseudo-static factor of safety could be increased to 1.1 by replacing some of the exiting fill at the bottom of the proposed slope with new fill and constructing a shear key. Additional details regarding the slope stability analysis are presented in the main report and a conceptual representation of the improved slope is shown in the slope stability output presented in Appendix D.

SEISMIC SETTLEMENT

Vertical settlements due to earthquakes typically occur during and after ground shaking. Settlement of loose, dry sandy soil, known as seismic compression, is the most common source of ground surface settlement during shaking. Settlement of saturated soil, known as reconsolidation, typically occurs after the shaking stops. In general, the sum of these types of settlement comprises the total free-field seismic settlement.

Seismic compression is most prevalent in relatively deep deposits of dry, clean sand. Liquefied granular soil or cyclically softened, fine-grained soil is most susceptible to reconsolidation settlement. Typically, reconsolidation settlement of cyclically softened soil is less than for liquefied soil because matrix effects tend to reduce volumetric strain.

There are no thick deposits of loose, cohesionless soil beneath the site and the risk of liquefaction and/or cyclic softening at the site is low. We estimate a total seismic settlement, primarily seismic compression of non-plastic soil, of 1 inch and a differential settlement of 0.5 inch of over a span of 50 feet. This settlement is in addition to the static settlement estimate provided in the main report.

LURCHING

Lurching is a phenomenon generally associated with very high levels of ground shaking, which causes localized failures and distortion of the soil. The anticipated ground accelerations are below the threshold required to induce lurching of the site soil.

SUBSIDENCE/UPLIFT

Subduction zone earthquakes can cause vertical tectonic movements. The movements reflect coseismic strain release accumulation associated with interplate coupling in the subduction zone. Based on our review of the literature, the locked zone of the CSZ is located in excess of 60 miles from the site. Consequently, we do not anticipate that subsidence or uplift is a significant design concern.

TSUNAMI AND SEICHE

The site located more than 60 miles away from the Oregon coast and is not located near any standing bodies of water; therefore, tsunami and seiche inundation are not design considerations for this project.

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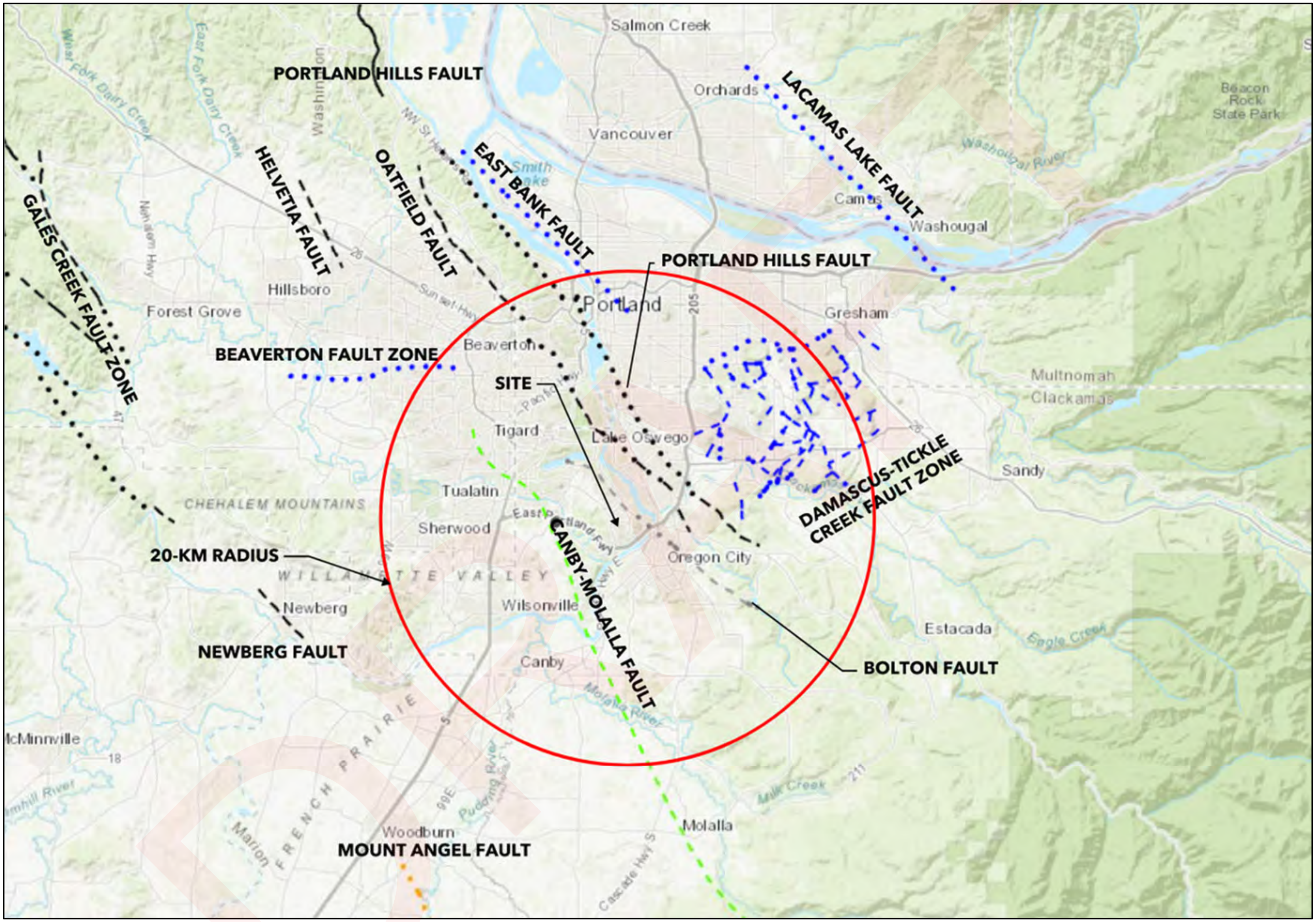

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QUARTERNARY FAULT MAP
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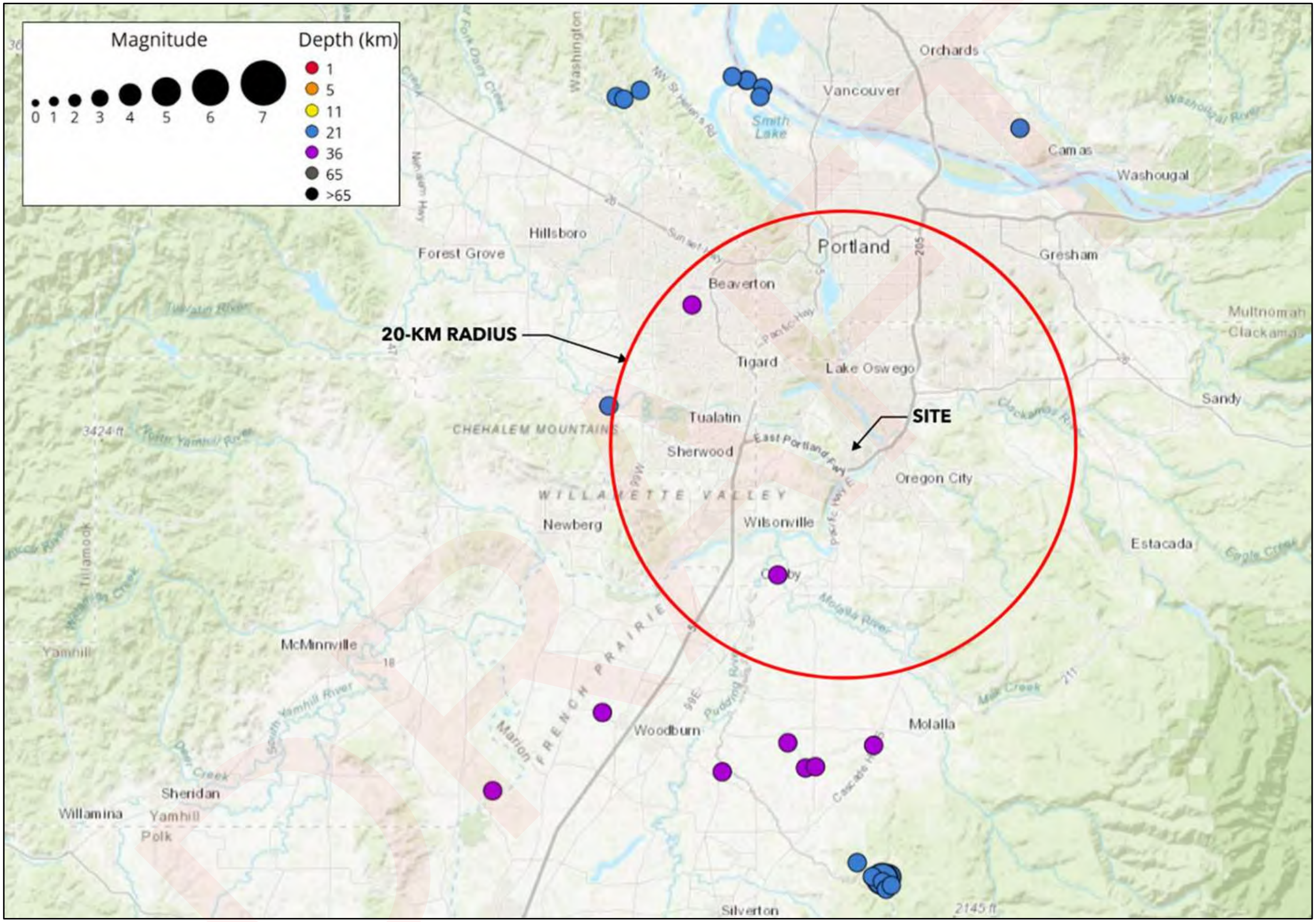
FIGURE
E-1



Geotechnical ▪ Environmental ▪ Special Inspections

Columbia West

Engineering, Inc

PROJECT NO.:
WESTL-1-01-1

APRIL 2025

HISTORICAL SEISMICITY MAP
OPERATIONS CENTER
WEST LINN, OREGON

FIGURE
E-2

APPENDIX F

APPENDIX F
PHOTO LOG

Photographs taken during our field explorations are presented in this appendix.

Boring B-1 – runs 1 through 4 from 12 to 28 feet BGS

Boring B-1– runs 4 through 6 from 28 to 39 feet BGS

Boring B-1 – runs 6 through 8 from 39 to 48 feet BGS

Boring B-1 – runs 8 through 10 from 48 to 59 feet BGS

Boring B-1 – runs 10 through 12 from 59 to 74 feet BGS

Boring B-1– runs 13 through 15 from 74 to 84 feet BGS

Boring B-1– runs 15 and 16 from 84 to 94 feet BGS

Boring B-1 – runs 17 and 18 from 94 to 102 feet BGS

Boring B-1– runs 18 through 20 from 102 to 111 feet BGS

Boring B-1– runs 20 and 21 from 111 to 119 feet BGS

Boring B-1– runs 22 and 23 from 119 to 128.5 feet BGS

Boring B-1 – runs 23 through 25 from 128.5 to 135.5 feet BGS

Boring B-2 – runs 1 through 3 from 46 to 54 feet BGS

Boring B-2 – runs 3 and 4 from 54 to 63 feet BGS

Boring B-2 – runs 5 and 6 from 63 to 71 feet BGS

Boring B-2 – runs 6 through 8 from 71 to 80 feet BGS

Boring B-2 – runs 8 and 9 from 80 to 88 feet BGS

Boring B-2 – runs 10 and 11 from 88 to 96 feet BGS

Boring B-2 – runs 11 through 13 from 96 to 104.3 feet BGS

Boring B-2 – runs 13 through 15 from 104.3 to 113.5 feet BGS

Boring B-2 – runs 15 and 16 from 113.5 to 121 feet BGS

Boring B-2 – run 16 from 121 to 123 feet BGS

Boring B-3 – runs 1 through 3 from 5 to 16 feet BGS

Boring B-3 – runs 3 through 5 from 16 to 22.5 feet BGS

Boring B-3 – runs 5 and 6 from 22.5 to 30.8 feet BGS

Boring B-3 – runs 6 through 8 from 30.3 to 40.3 feet BGS

Boring B-3 – runs 8 through10 from 40.3 to 48.8 feet BGS

Boring B-3 – runs 10 through 12 from 48.8 to 57.3 feet BGS

Boring B-3 – runs 12 and 13 from 57.3 to 65.6 feet BGS

Boring B-3 – runs 13 through 15 from 65.6 to 73.6 feet BGS

Boring B-3 – runs 15 through 17 from 73.6 to 82.3 feet BGS

Boring B-3 – runs 17 and 18 from 82.3 to 90.8 feet BGS

Boring B-3 – runs 18 through 20 from 90.8 to 100 feet BGS

Boring B-4 – runs 1 through 3 from 5 to 19 feet BGS

Boring B-4 – runs 3 through 5 from 19 to 27.5 feet BGS

Boring B-4 – runs 5 and 6 from 27.5 to 35 feet BGS

Boring B-4 – runs 7 and 8 from 35 to 43.8 feet BGS

Boring B-4 – run 8 from 43.8 to 50 feet BGS

Boring B-5 – runs 1 through 3 from 46 to 53 feet BGS

Boring B-5 – runs 3 and 4 from 53 to 59.3 feet BGS

Boring B-5 – runs 4 through 7 from 59.3 to 69 feet BGS

Boring B-5 – runs 7 and 8 from 69 to 77 feet BGS

Boring B-5 – runs 9 and 10 from 77 to 86.5 feet BGS

Boring B-5 – runs 10 through 12 from 86.5 to 97 feet BGS

Boring B-5 – runs 13 and 14 from 97 to 105 feet BGS

Boring B-5 – runs 14 through 16 from 105 to 113.5 feet BGS

Boring B-5 – run 16 from 113.5 to 117 feet BGS

Test pit TP-1

Test pit TP-2

Test pit TP-3

Test pit TP-5

Test pit TP-8

Test pit TP-9

Test pit TP-10

Test pit TP-11

Middle portion of site, looking east

Middle portion of site, looking north

Middle portion of site, looking west

North boundary of site, looking northeast

North boundary of site, looking northwest

North boundary of site, looking south

APPENDIX G

APPENDIX G
BACKGROUND INFORMATION

Site plans, exploration logs, and laboratory testing results from prior geotechnical studies at the site and in the site vicinity are presented in this appendix.

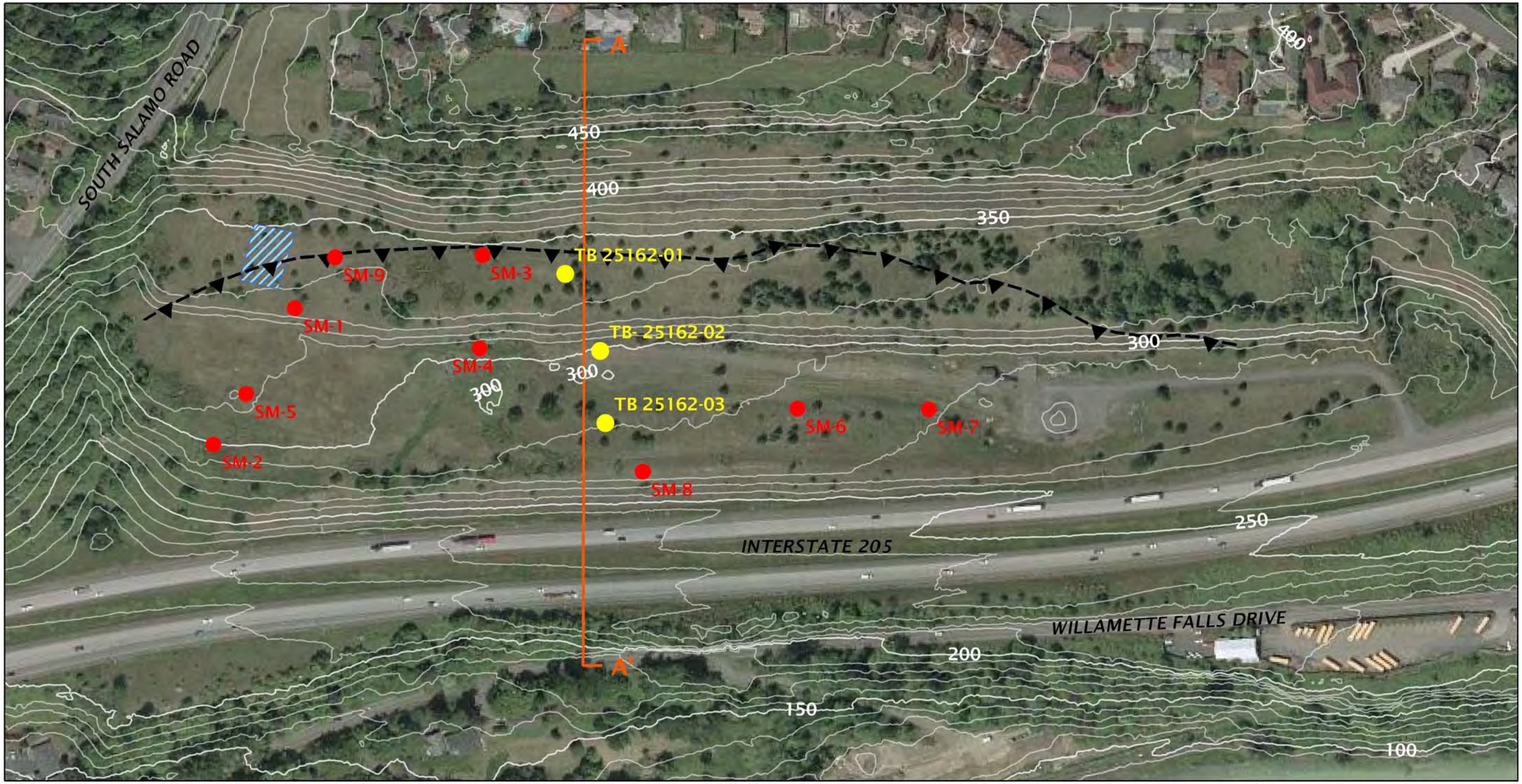
SLOPE STABILITY EVALUATION

Oregon Solar Highway – West Linn Project Site
West Linn, Oregon

For
Oregon Department of Transportation
February 4, 2010

GeoDesign Project: ODOT-51-01

Engineers | Geologists | Environmental Consultants



EXPLANATION

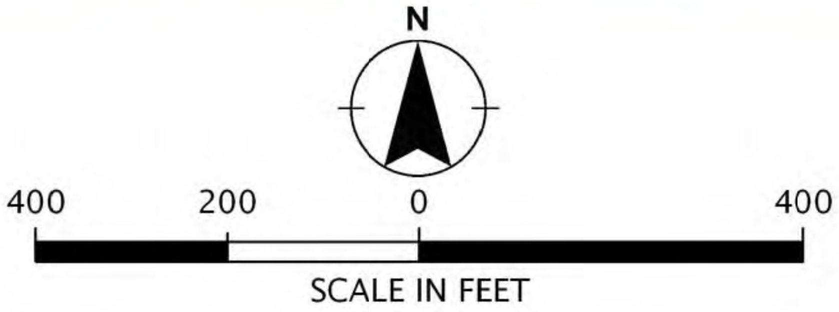
● PROJECT BORINGS

● CH2M BORINGS

–▲– LANDSLIDE HEADSCARP

▨ WILLAMETTE RESERVOIR

A A' CROSS SECTION



LIDAR DATA OBTAINED FROM PUGET SOUND LIDAR CONSORTIUM
ORTHOPHOTOGRAPH OBTAINED FROM USGS TERRASERVER

FIGURE 2

SITE PLAN

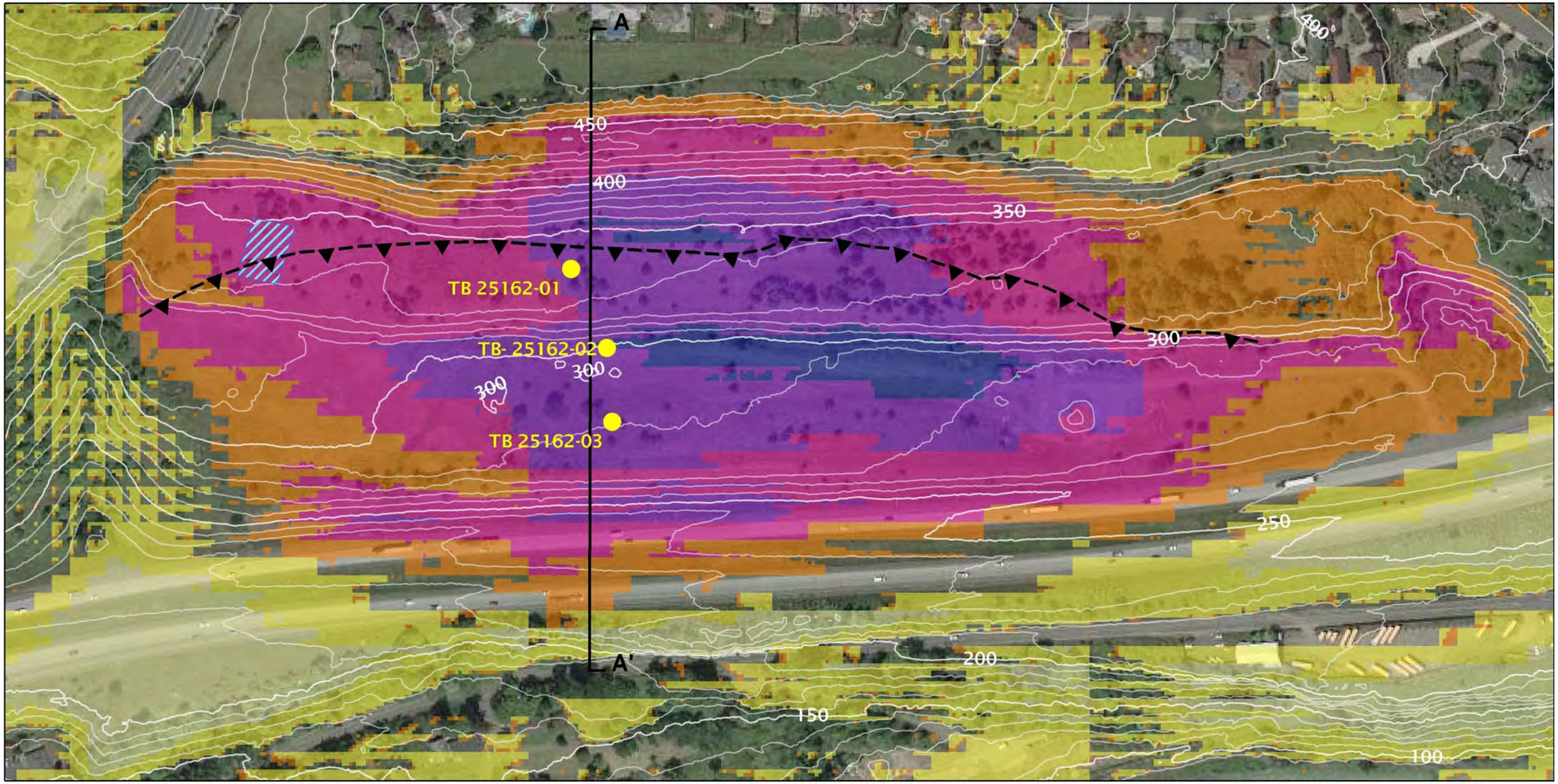
OREGON SOLAR HIGHWAY – WEST LINN PROJECT
WEST LINN, OR

ODOT-51-01

FEBRUARY 2010

GEODESIGN INC
1201 SE Tech Center Drive - Suite 160
Vancouver WA 98683
Off 360.693.8416 Fax 360.693.8426

ODOT-51-01-F2.4 Print Date: 2/4/10



EXPLANATION

● PROJECT BORINGS

▲ LANDSLIDE HEADSCARP

▨ WILLAMETTE RESERVOIR

A A' CROSS SECTION

ELEVATION CHANGE
(FEET)

▨	+60 TO +30
🟨	+30 TO +10
🟧	-10 TO -30
🟥	-30 TO -60
🟪	-60 TO -90
🟦	-90 TO -112

N

400 200 0 400
SCALE IN FEET

LIDAR DATA OBTAINED FROM PUGET SOUND LIDAR CONSORTIUM
ORTHOPHOTOGRAPH OBTAINED FROM USGS TERRASERVER

GROUND SURFACE ELEVATION DIFFERENCE

OREGON SOLAR HIGHWAY - WEST LINN PROJECT
WEST LINN, OR

FIGURE 4

ODOT-51-01

FEBRUARY 2010

GEODESIGN INC
1201 SE Tech Center Drive - Suite 160
Vancouver WA 98683
Off 360.693.8416 Fax 360.693.8426

ODOT-51-01-F2.4 Print Date: 2/4/10

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We explored the subsurface conditions at the site by drilling and coring three geotechnical borings (TB 25162-01 through TB 25162-03) between August 26 and September 1, 2009. Western States Soil Conservation Service, Inc. drilled and cored the borings using mud-rotary and HQ coring techniques. The approximate locations of the explorations were determined in the field using a hand-held GPS and are shown on Figure 2.

Members of our engineering geology staff observed the explorations. Soil and rock sampling procedures used in our boring explorations are summarized in the following section. Classifications and sampling intervals are presented on the exploration logs included in this appendix and followed the procedures presented in ODOT Soil and Rock Classification Manual (ODOT, 1987).

SOIL AND ROCK SAMPLING

Soil and rock samples were obtained from borings TB 25162-01 through TB 25162-03 using the following methods.

During mud rotary drilling, SPTs were performed in general conformance with ASTM D 1586 in soil and decomposed rock. The sampler was driven with a 140-pound hammer free-falling 30 inches. The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soils is shown adjacent to the sample symbols on the exploration log. Disturbed samples were obtained from the split barrel for subsequent classification and index testing.

HQ rock coring was performed in borings TB 25162-01 through TB 25162-03 in general accordance with ASTM D 2113 to recover intact samples of rock. Core samples were photographed, preserved, and transported in general accordance with ASTM D 5079.

SOIL AND ROCK CLASSIFICATION

The soil and rock samples were classified in accordance with the ODOT Soil and Rock Classification Manual (ODOT, 1987). The exploration logs indicate the depths at which the soil or rock characteristics change, although the change actually could be gradual. Classifications and sampling intervals are presented on the exploration logs included in this appendix.

CORE PHOTOGRAPHS

We photographed each core box used for archiving the rock core collected in borings TB 25162-01 through TB 25162-03. These core photographs are presented on Figures A-1 through A-9 included in this appendix.

DRILL LOG

OREGON DEPARTMENT OF TRANSPORTATION

		Hole No. **TB-25162-01**
Project **Oregon Solar Highway - West Linn Project**	Purpose **Slope Stability**	E.A. No.
Highway **I-205**	County **Clackamas**	Key No.
Hole Location Northing: **621,117.02** Easting: **7,651,841.35**		Start Card No.
Equipment **mud-rotary**	Driller **Western States Soil Conservation**	Bridge No.
Project Geologist **Palmer/Woodcock/Heidgerken**	Recorder **Woodcock**	Ground Elev. **345.63 ft**
Start Date **August 27, 2009** End Date **August 31, 2009**	Total Depth **82.50 ft**	Tube Height

Test Type

"A" - Auger Core
"X" - Auger
"C" - Core, Barrel Type
"N" - Standard Penetration
"U" - Undisturbed Sample
"T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Lab Acronyms

CON - Consolidation Test
DS - Direct Shear
P200 - P200 Test
HYD - Hydrometer Gradation
SIEVE - Sieve Gradation
DD - Dry Density
ATT - Atterberg Limits Test

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance	Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0							**0.00 - 49.50** **Sandy GRAVEL with Silt, Clay, Cobbles, and Boulders; Lenses of Silty SAND with Gravel to Sandy SILT with Gravel; ML, SM, GP, GM, GW; (Fill)**	▨	Advance auger. Occasional drill chatter from 0 to 7.0 feet (possible cobbles and boulders).		
5	N1	94	19-31-40		21	N- 1 (5.00-6.50) Silty SAND with gravel; SM; Brown with orange mottles; Low plasticity; Moist; Very dense. Fill		▨			
								▨	Increased drill chatter from 7.0 to 10.0 feet.		
10	N2	50	5-7-6		25	N- 2 (10.00-11.50) Sandy GRAVEL with silt and clay; GP/GM; Brown-gray with orange mottled; Moist; Medium dense; Angular to subangular; Lens of red-brown clay with sand. Fill		▨			
15	N3	0	50/3"			N- 3 (15.00-15.30) No recovery		▨	Possible cobbles and boulders at 15.0 feet.		
20								▨			

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance / Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
20	N4	72	16-7-6	37	N- 4 (20.00-21.50) Sandy GRAVEL with silt and clay; GP/GM; Brown-gray; Medium plasticity; Moist; Medium dense; Angular to subangular; Lenses of red-brown sandy clay. Fill		▨			
25	N5	94	7-9-9	29	N- 5 (25.00-26.50) Sandy SILT; ML; Gray-brown with orange and red-brown mottles; Low plasticity; Moist; Lenses of silty sand and some gravel to red-brown clay. Very stiff. Fill		▨			
30	N6	72	12-31-45	17	N- 6 (30.00-31.50) Silty SAND with gravel, trace clay; SM; Gray-brown with orange mottles; Low plasticity; Moist; Very dense. Fill		▨			
35	N7	72	12-25-21	19	N- 7 (35.00-36.50) Sandy GRAVEL with silt; GP/GM; Gray with orange mottles; Moist; Dense, Angular to subangular. Fill		▨			
40	N8	100	44-50/4"	33	N- 8 (40.00-40.80) Silty SAND with gravel; SM; Gray-brown with orange mottles; Low plasticity; Moist; Very dense. Fill		▨			
45	N9	100	50/3"	27	N- 9 (45.00-45.20) Sandy GRAVEL with silt; GP/GM; Gray with orange mottles; Moist; Very dense, Angular to subangular. Fill		▨			
50	N10	0	50/0"		N- 10 (49.50-50.00) No recovery	**49.50 - 82.50**		Drilled to 50.0 feet, then		

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance	Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
50	C1	95		RQD = 25		C- 1 (50.00-53.00) BASALT; Gray; Fresh to slightly weathered; Medium hard (R-3); Jointed 10° to 20°, 80º; Moderately close to close spacing; Iron oxide staining on joints; some vesicles. Columbia River Basalt	**BASALT (Columbia River Basalt)**		switched to rock coring.		
	C2	103		RQD = 60		C- 2 (53.00-58.00) BASALT; Gray; Fresh to slightly weathered; Medium hard (R-3); Jointed 10° to 20°, 80° to 90º;Close spacing; Iron oxide staining on joints; Some vesicles. Columbia River Basalt					
55											
	C3	100		RQD = 92		C- 3 (58.00-63.00) BASALT; Gray; Fresh; Hard (R-4); Jointed 10° to 20°, 50º; Moderately close spacing; Some vesicles. Columbia River Basalt					
60											
	C4	100		RQD = 94		C- 4 (63.00-68.00) BASALT; Gray; Fresh; Hard (R-4); Jointed 10° to 20°; Moderately close to close spacing; Some vesicles. Columbia River Basalt					
65											
	C5	100		RQD = 36		C- 5 (68.00-73.00) BASALT; Gray; Fresh; Hard (R-4); Jointed 10°, 70º; Moderately close to close spacing; Some vesicles. Columbia River Basalt					
						(69.50) BASALT; Gray-brown; Slightly weathered, Medium hard (R-3); Very close to close spacing, Iron oxide joint staining; Some vesicles. Columbia River Basalt					
70						(71.00) BASALT; Red-brown; Moderately Weathered; Medium hard (R-3); Very close to close spacing; Jointed 10°; Yellow zeolite/clay joint infilling; Some vesicles. Columbia River Basalt					
	C6	100		RQD = 33		C- 6 (73.00-78.00) BASALT; Red-brown to gray-brown; Moderately weathered; Medium hard (R-3); Jointed 10° to 20°, 70º to 80°; Very close to close spacing; Yellow-orange zeolite/clay joint infilling; Some to highly vesicular. Columbia River Basalt					
75											
	C7	100		RQD = 19		C- 7 (78.00-82.50) BASALT; Gray; Slightly weathered; Medium hard (R-3); Jointed 10° to 30°, 90º; Very close spacing; Yellow patchy zeolite/clay joint infilling; Iron oxide joint staining; Some vesicles. Columbia River Basalt					
80											

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance	Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
80											
									BOH @ 82.5 feet.		
85											
90											
95											
100											
105											
110											

DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

Page **1** of **4**

		Hole No. **TB-25162-02**
Project **Oregon Solar Highway - West Linn Project**	Purpose **Slope Stability**	E.A. No.
Highway **I-205**	County **Clackamas**	Key No.
Hole Location Northing: **620,989.32** Easting: **7,651,908.05**		Start Card No.
Equipment **mud-rotary**	Driller **Western States Soil Conservation**	Bridge No.
Project Geologist **Palmer/Woodcock/Heidgerken**	Recorder **Woodcock**	Ground Elev. **299.08 ft**
Start Date **August 31, 2009** End Date **September 1, 2009**	Total Depth **87.00 ft**	Tube Height

Test Type

- "A" - Auger Core
- "X" - Auger
- "C" - Core, Barrel Type
- "N" - Standard Penetration
- "U" - Undisturbed Sample
- "T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Lab Acronyms

- CON - Consolidation Test
- DS - Direct Shear
- P200 - P200 Test
- HYD - Hydrometer Gradation
- SIEVE - Sieve Gradation
- DD - Dry Density
- ATT - Atterberg Limits Test

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance	Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0							**0.00 - 10.50** **Silty SAND; SM; (Fill)**		Advance auger.		
5	N1	100	48-50/4"		35	N- 1 (5.00-5.80) Silty SAND with lenses of Sand SILT; SM; Gray with orange and dark brown mottles; Low plasticity; Moist; Blocky texture; Moderately indurated. Fill					
									Very hard drilling at 9.0 feet.		
10	N2		50/1"			N- 2 (10.00-10.10) As in N-1					
	N3	87	50/1"			N- 3 (10.50-10.60) Sandy GRAVEL, trace silt; GP; Gray; Moist; Angular to subangular. Columbia River Basalt	**10.50 - 87.00** **BASALT (Columbia River Basalt)**		Switched to rock coring at 10.5 feet.		
	C1			RQD = 60		C- 1 (10.60-12.00) BASALT; Gray; Fresh; Hard (R-4); Jointed 10° to 30°; Close spacing; Some vesicles. Columbia River Basalt					
	C2	92		RQD = 79		C- 2 (12.00-17.00) BASALT; Gray; Fresh; Hard (R-4); Jointed 10°, 30°, and 70° to 80°; Moderately close spacing; some vesicles. Columbia River Basalt					
15											
	C3	88		RQD = 60		C- 3 (17.00-22.00) BASALT; Gray; Fresh; Hard (R-4); Jointed 60° to 70°; Close spacing; Patchy yellow zeolite/clay joint infilling; Some vesicles. Columbia River Basalt					
20											

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance	Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
20											
	C4	100		RQD = 84		C- 4 (22.00-27.00) BASALT; Gray; Fresh; Hard (R-4); Jointed 10° to 30°, 50°; Moderately close to close spacing; Dark gray clayey silt joint infilling; Some vesicles. Columbia River Basalt					
25											
	C5	100		RQD = 58		C- 5 (27.00-32.00) BASALT; Gray; Fresh; Hard (R-4); Jointed 10° to 30º; Close to moderately close spacing; Light gray silt joint infilling; Some vesicles. Columbia River Basalt					
30						(30.00) BASALT, Red-brown; Moderately weathered, Soft to medium hard (R-2 to R-3); Jointed 10° to 30°; Close to very close spacing; Highly vesicular.					
	C6	100				C- 6 (32.00-37.00) BASALT; Red-brown; Slightly weathered; Medium hard (R-3); Jointed 10º, 40° to 55º, 90º; Close to very close spacing; Patchy yellow zeolite/clay joint infilling; Highly vesicular. Columbia River Basalt					
35						(36.00) BASALT; Red-brown to brown-gray; Slightly weathered; Medium hard (R-3); Jointed 10° to 90º; Orange clayey silt joint infilling; Highly vesicular.					
	C7	100		RQD = 67		C- 7 (37.00-42.00) BASALT; Gray to gray-brown; Slightly weathered; Medium hard to hard (R-3 to R-4); Jointed 10° to 20°, 60° to 70°; Moderately close to close spacing; Patchy joint infilling with orange clayey sand and yellow zeolite/clay; Some vesicles. Columbia River Basalt					
40											
	C8	100		RQD = 52		C- 8 (42.00-47.00) BASALT; Gray; Slightly weathered; Hard (R-4); Jointed 20° to 30°, 60º to 90º; Close to very close spacing; Patchy infilling with orange-brown sandy clay; Some vesicles. Columbia River Basalt					
45											
	C9	100		RQD = 38		C- 9 (47.00-52.00) BASALT; Gray to red-brown; Slightly weathered; Medium hard (R-3); Jointed 10º to 30°, 60º to 80º; Close to very close spacing; Patchy brown sandy clay and yellow zeolite/clay oint infilling; Highly vesicular. Columbia River Basalt					
50											

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance	Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
50											
	C10	100		RQD = 85		C- 10 (52.00-57.00) BASALT; Gray; Fresh to slightly weathered; Hard (R-4); Jointed 10° to 30°, 70° to 80º; Close spacing; Joint infilling with yellow-brown clay; Highly vesicular. Columbia River Basalt					
55											
	C11	100		RQD = 92		C- 11 (57.00-62.00) BASALT; Gray; Fresh to slightly weathered; Hard (R-4); Jointed 10° to 50º, 80° to 90°; Close to moderately close spacing; Patchy joint infilling with orange-brown clay with sand; Highly vesicular. Columbia River Basalt					
60											
	C12	100		RQD = 72		C- 12 (62.00-67.00) BASALT; Gray; Fresh to slightly weathered; Hard (R-4); Jointed 10º to 40º, 70º to 80°; Close to very close spacing; Patchy orange clayey sand and yellow zeolite/clay joint infilling; Some to highly vesicular. Columbia River Basalt					
65											
	C13	100		RQD = 86		C- 13 (67.00-72.00) BASALT; Gray; Fresh to slightly weathered; Hard (R-4); Jointed 10°, 80º; Close spacing; Some vesicles. Columbia River Basalt					
70											
	C14	100		RQD = 55		C- 14 (72.00-77.00) BASALT; Gray; Fresh to slightly weathered; Hard (R-4); Jointed 10º to 30º, 80° to 90º; Moderately close to close spacing; Some vesicles. Columbia River Basalt					
75											
	C15	100		RQD = 88		C- 15 (77.00-82.00) BASALT; Gray; Fresh; Hard (R-4); Jointed 20º, 50º, 70° to 90º; Moderately close to close spacing; Patchy yellow zeolite joint coating; Some vesicles. Columbia River Basalt					
80											

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
80										
	C16	100	RQD = 60		C- 16 (82.00-87.00) BASALT; Gray; Fresh to slightly weathered; Hard (R-4); Jointed 10° to 20°, 60° to 80°; Moderately close to very close spacing; Patchy orange clayey sand infilling and yellow zeolite joint coating; Highly vesicular. Columbia River Basalt					
85										
								BOH @ 87.0 feet.		
90										
95										
100										
105										
110										

DRILL LOG

OREGON DEPARTMENT OF TRANSPORTATION

Page **1** of **3**

Hole No. **TB-25162-03**

Project **Oregon Solar Highway - West Linn Project**	Purpose **Slope Stability**	E.A. No.
Highway **I-205**	County **Clackamas**	Key No.
Hole Location Northing: **620,831.85** Easting: **7,651,918.56**		Start Card No.
Equipment **HSA/mud-rotary**	Driller **Western States Soil Conservation**	Bridge No.
Project Geologist **Palmer/Woodcock/Heidgerken**	Recorder **Heiderken/Woodcock**	Ground Elev. **290.97 ft**
Start Date **August 26, 2009** End Date **August 27, 2009**	Total Depth **72.00 ft**	Tube Height

Test Type

- "A" - Auger Core
- "X" - Auger
- "C" - Core, Barrel Type
- "N" - Standard Penetration
- "U" - Undisturbed Sample
- "T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Lab Acronyms

- CON - Consolidation Test
- DS - Direct Shear
- P200 - P200 Test
- HYD - Hydrometer Gradation
- SIEVE - Sieve Gradation
- DD - Dry Density
- ATT - Atterberg Limits Test

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance	Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0							**0.00 - 39.50** **Silty, clayey GRAVEL to Sandy GRAVEL with cobbles and boulders; GM, GC, GW; (Fill)**	▨	Hard, slow drilling with minor silty gravel in cuttings at surface.		
5	N1	61	10-10-10		15	N- 1 (5.00-6.50) Clayey GRAVEL with silt, sand, and cobbles; GC; Brown-gray; Nonplastic to low plasticity; Moist; Medium dense; Angular to subangular. Fill		▨	Switched to mud-rotary drilling (4½ TriCone) at 5.0 feet.		
	N2	44	3-3-3		22	N- 2 (7.50-9.00) Sandy GRAVEL with silt and clay to GRAVEL with sand, silt, and clay; GW/GM; Brown; Nonplastic; Moist to wet; Loose. Fill		▨	Rapid drill rate from 7.5 to 12.5 feet.		
10								▨			
	N3	72	4-4-9		18	N- 3 (12.50-14.00) Sandy GRAVEL with silt and clay to GRAVEL with sand, silt, and clay; GW; Brown; Nonplastic; Moist to wet; Medium dense. Fill		▨			
15								▨	Driller comments: cobbles and boulders at 15.0 feet. Losing mud at approximately 20 gallons.		
	N4	6	5-6-4			N- 4 (17.50-19.00) Sandy GRAVEL with clay to Silty GRAVEL with clay; GC/GM; Brown; Nonplastic; Moist to wet; Loose to medium dense. Fill		▨	Rapid drill rate from 17.5 to 22.5 feet.		
20								▨			

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance	Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
20											
	N5	39	7-5-10		18	N- 5 (22.50-24.00) Sandy GRAVEL with silt, clay, cobbles, and boulders; GW; Brown-gray; Nonplastic; Moist to wet; Medium dense; Angular to subangular. Fill					
25											
	N6	17	7-5-3		22	N- 6 (27.50-29.00) Sandy GRAVEL with silt, clay, cobbles, and boulders; GW; Brown-gray; Nonplastic; Moist to wet; Loose; Angular to subangular. Fill			Driller comments: still losing mud to 27.5 feet. Fairly rapid drill rate from 27.5 to 32.5 feet.		
30											
	N7	39	7-5-10		24	N- 7 (32.50-34.00) Silty GRAVEL with sand, cobbles, and boulders; GM; Gray-brown; Nonplastic; Moist to wet; Medium dense; Angular to subangular. Fill					
35											
									Losing mud circulation and drill chatter scattered at 36.0 feet.		
	N8	44	45-29-15		18	N- 8 (37.50-39.00) GRAVEL with sand, silt, cobbles, and boulders, trace clay; GM/GW; Gray; Nonplastic; Moist to wet; Dense; Angular to subangular. Fill					
40	C1	100		RQD = 48		C- 1 (39.50-42.00) BASALT; Gray; Fresh; Hard (R-4); Jointed 30° to 40°, 75°; Close spacing; Iron oxide joint staining; Some vesicles. Columbia River Basalt	**39.50 - 72.00 BASALT (Columbia River Basalt)**		Switched to rock coring at 39.5 feet.		
	C2	100		RQD = 90		C- 2 (42.00-47.00) BASALT; Gray; Fresh; Hard (R-4); Jointed 10° to 20°, 80°; Close to moderately close spacing; Some vesicles. Columbia River Basalt					
45											
	C3	100		RQD = 34		C- 3 (47.00-52.00) BASALT; Gray; Fresh; Hard (R-4); Jointed 0° to 10°, 40° to 50°; Close to very close spacing; Iron oxide joint staining; Some vesicles. Columbia River Basalt					
50											

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance	Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
50						(50.00) BASALT, Brown-gray, slightly weathered; Hard (R-4); Iron oxide joint staining; Some vesicles.					
	C4	100		RQD = 45		C- 4 (52.00-57.00) BASALT; Red-brown; Moderately weathered; Medium hard (R-3); Jointed 20°, 60°; Close to very close spacing; Yellow-brown zeolite/clay joint infilling; Highly vesicular. Columbia River Basalt					
55											
	C5	100		RQD = 51		C- 5 (57.00-62.00) BASALT; Red-brown; Moderately weathered; Medium hard (R-3); Jointed 0° to 10°, 30° to 40°, 60° to 80°; Close spacing; Yellow-brown zeolite/clay joint infilling; Highly vesicular. Columbia River Basalt.					
						(59.70) BASALT; Gray; Fresh to slightly weathered; Hard (R-4); Jointed 10° to 30°; Moderately close to close spacing; Highly vesicular.					
60											
	C6	100		RQD = 52		C- 6 (62.00-67.00) BASALT; Gray; Fresh to slightly weathered; Hard (R-4); Jointed 10° to 20°, 60° to 70°; Close spacing; Some to highly vesicular. Columbia River Basalt					
						(64.00) BASALT; Brown-gray; Moderately weathered; Medium hard (R-3); Jointed 10° to 80°; Close to very close spacing; Yellow zeolite/clay joint infilling; Highly vesicular.					
65											
						(65.80) BASALT; Gray; Fresh to moderately weathered; Hard (R-4); Jointed 60° to 80°; Moderately close spacing; Iron staining on joints; Some vesicles.					
	C7	100		RQD = 76		C- 7 (67.00-72.00) BASALT; Gray; Fresh to slightly weathered; Hard (R-4); Jointed 40° to 55º, 75° to 80º; Close spacing; Patchy infilling with yellow-brown zeolite/clay. Columbia River Basalt					
70											
									BOH @ 72.0 feet.		
75											
80											

ODOT-51-01-FA1-9.doc Print Date: 2/4/10

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Vancouver WA 98683
Off 360.693.8416 Fax 360.693.8426

ODOT-51-01

FEBRUARY 2010

CORE PHOTOGRAPHS, TB25162-01 BOXES 1 AND 2

OREGON SOLAR HIGHWAY – WEST LINN PROJECT
WEST LINN, OR

FIGURE A-1

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CORE PHOTOGRAPHS, TB25162-01 BOXES 3 AND 4

OREGON SOLAR HIGHWAY – WEST LINN PROJECT
WEST LINN, OR

FIGURE A-2

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CORE PHOTOGRAPHS, TB25162-02 BOXES 1 AND 2

OREGON SOLAR HIGHWAY – WEST LINN PROJECT
WEST LINN, OR

FIGURE A-3

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ODOT-51-01	**CORE PHOTOGRAPHS, TB25162-02 BOXES 3 AND 4**		
FEBRUARY 2010	OREGON SOLAR HIGHWAY – WEST LINN PROJECT WEST LINN, OR	**FIGURE A-4**	

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CORE PHOTOGRAPHS, TB25162-02 BOXES 5 AND 6

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OREGON SOLAR HIGHWAY – WEST LINN PROJECT
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FIGURE A-5

GEODESIGN INC 1201 SE Tech Center Drive - Suite 160 Vancouver WA 98683 Off 360.693.8416 Fax 360.693.8426	ODOT-51-01	**CORE PHOTOGRAPHS, TB25162-02 BOXES 7 AND 8**
	FEBRUARY 2010	OREGON SOLAR HIGHWAY – WEST LINN PROJECT WEST LINN, OR
		FIGURE A-6

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CORE PHOTOGRAPHS, TB25162-02 BOX 9

OREGON SOLAR HIGHWAY – WEST LINN PROJECT
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FIGURE A-7

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CORE PHOTOGRAPHS, TB25162-03 BOXES 1 AND 2

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OREGON SOLAR HIGHWAY – WEST LINN PROJECT
WEST LINN, OR

FIGURE A-8

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CORE PHOTOGRAPHS, TB25162-01 BOXES 3 AND 4

OREGON SOLAR HIGHWAY – WEST LINN PROJECT
WEST LINN, OR

FIGURE A-9

APPENDIX B

APPENDIX B

LABORATORY TESTING

We visually examined soil samples obtained from the explorations to confirm field classifications. We also performed the following testing on samples obtained from the borings.

MOISTURE CONTENT

We tested the natural moisture content of 16 soil samples in general accordance with ASTM D 2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The moisture content values are presented on the exploration logs included in Appendix A and summarized in this appendix.

GRAIN-SIZE TESTING

Grain-size testing was completed on three samples that consisted of percent fines determinations in general accordance with guidelines presented in ASTM C 136 and ASTM D 1140. The results of the percent fines testing are included in this appendix.

ROCK STRENGTH TESTING

Unconfined uniaxial compressive strength tests were conducted by ACS Testing, Inc. of Tigard, Oregon, on core samples from borings TB 25162-01, TB 25162-02, and TB 25162-03. The tests were performed in general accordance with ASTM D 2938, and the results of the unconfined uniaxial compressive strength tests are presented in this appendix.

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT (PERCENT)	PLASTIC LIMIT (PERCENT)	PLASTICITY INDEX (PERCENT)
TB-25162-1	5.0		21				29			
TB-25162-1	10.0		25							
TB-25162-1	20.0		37							
TB-25162-1	25.0		29							
TB-25162-1	30.0		17				17			
TB-25162-1	35.0		19							
TB-25162-1	40.0		33							
TB-25162-1	45.0		27							
TB-25162-2	5.0		35							
TB-25162-3	5.0		15							
TB-25162-3	7.5		22							
TB-25162-3	12.5		18				9			
TB-25162-3	22.5		18							
TB-25162-3	27.5		22							
TB-25162-3	32.5		24							
TB-25162-3	37.5		18							

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ODOT-51-01

FEBRUARY 2010

SUMMARY OF LABORATORY DATA

OREGON SOLAR HIGHWAY - WEST LINN PROJECT
WEST LINN, OR

FIGURE B-1

LAB SUMMARY ODOT-51-01-LAB.GPJ GEODESIGN.GDT PRINT DATE: 2/4/10:KT

Asphalt Concrete Soils Testing, Inc.

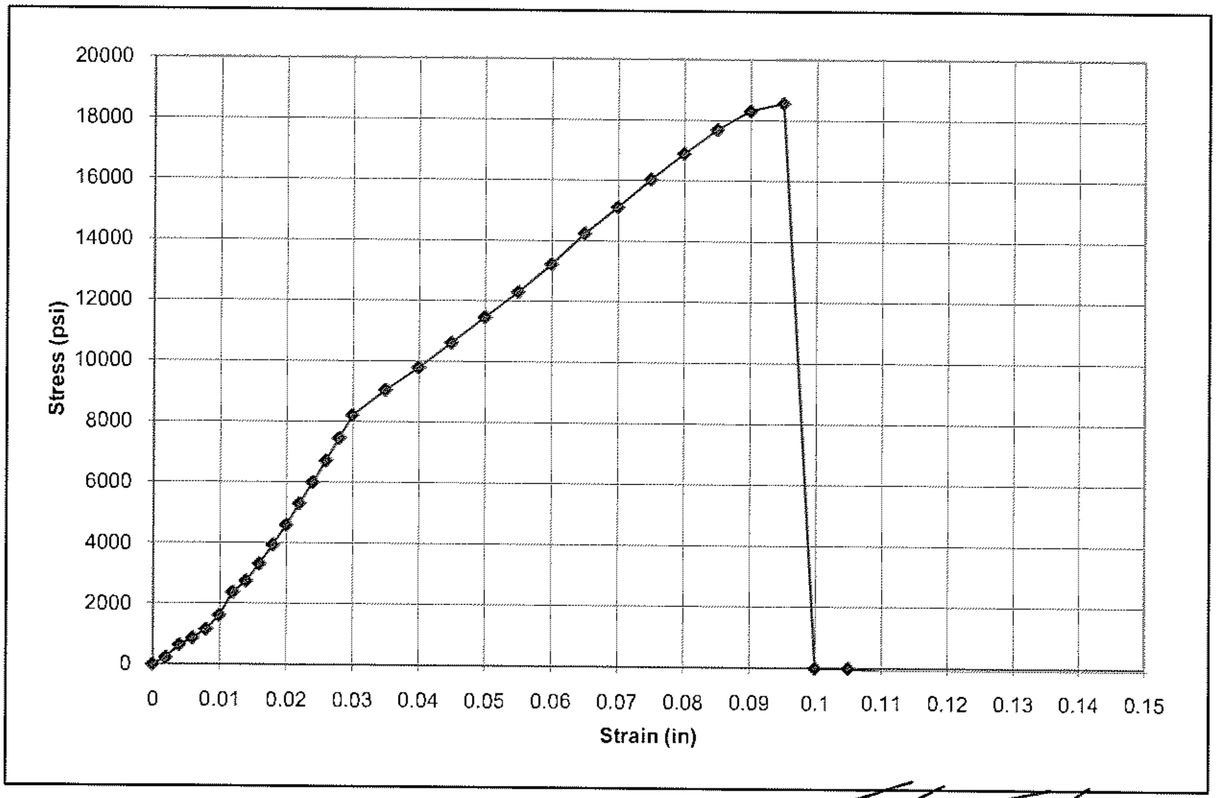
ACS Testing, Inc
7409 SW Tech Center Dr Ste 145
Tigard, OR 97223
PH: 503-443-3799 F: 503-620-2748

Geo Design
15575 SW Sequoia Parkway#100
Portland OR 97224

PROJECT:	Lab Services	**JOB NO:**	09-2307
LOCATION:	Oregon Solar Hwy	**WORK ORDER NO:**	ODOT-51-01
MATERIAL:	Rock Cores	**LAB NO:**	4745-3
SAMPLE SOURCE:	66.8-67.7 TB 25162-01	**DATE SAMPLED:**	09/16/09
SAMPLE PREP:			

UNCONFINED COMPRESSION STRENGTH OF Rock Cores
APPLICABLE PORTIONS OF (ASTM D2938)

DIAMETER:	2.39 in	MAXIMUM STRESS:	18,563 psi
HEIGHT:	5.97 in	AT STRAIN:	1.59%
STRAIN RATE:	.006 inches/min.		
DRY DENSITY:	112.1 lb/cu.ft		
MOISTURE:	2.5%		



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ASPHALT
CONCRETE
SOILS
Testing, Inc.

ACS Testing, Inc
7409 SW Tech Center Dr Ste 145
Tigard, OR 97223
PH: 503-443-3799 F: 503-620-2748

Geo Design
15575 SW Sequoia Parkway#100
Portland OR 97224

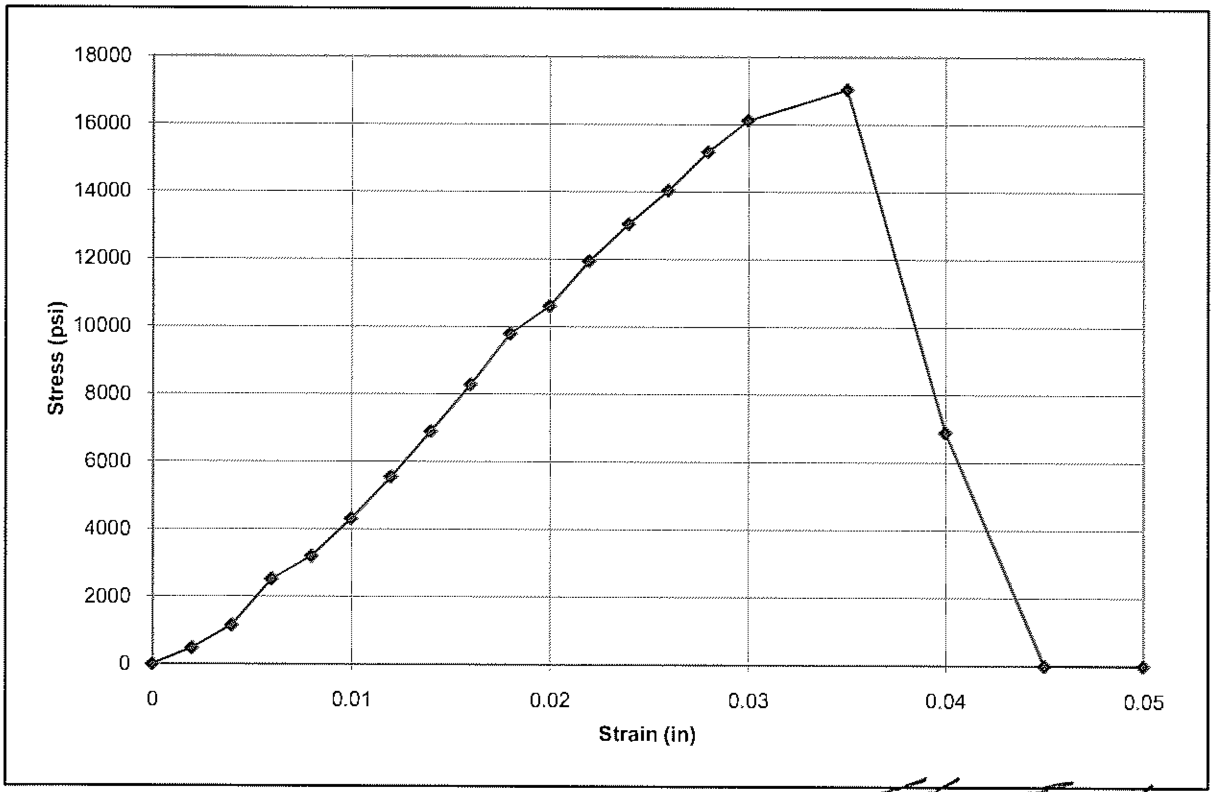
PROJECT: Lab Services
LOCATION: Oregon Solar Hwy
MATERIAL: Rock Cores
SAMPLE SOURCE: 45.1-45.7 TB 25162-03
SAMPLE PREP:

JOB NO: 09-2307
WORK ORDER NO: ODOT-51-01
LAB NO: 4745-2
DATE SAMPLED: 09/16/09

UNCONFINED COMPRESSION STRENGTH OF Rock Cores
APPLICABLE PORTIONS OF (ASTM D2938)

DIAMETER: 2.39 in
HEIGHT: 5.98 in
STRAIN RATE: .006 inches/min.
DRY DENSITY: 38.1 lb/cu.ft
MOISTURE: 2.9%

MAXIMUM STRESS: 17,040 psi
AT STRAIN: 0.59%



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ACS Testing, Inc
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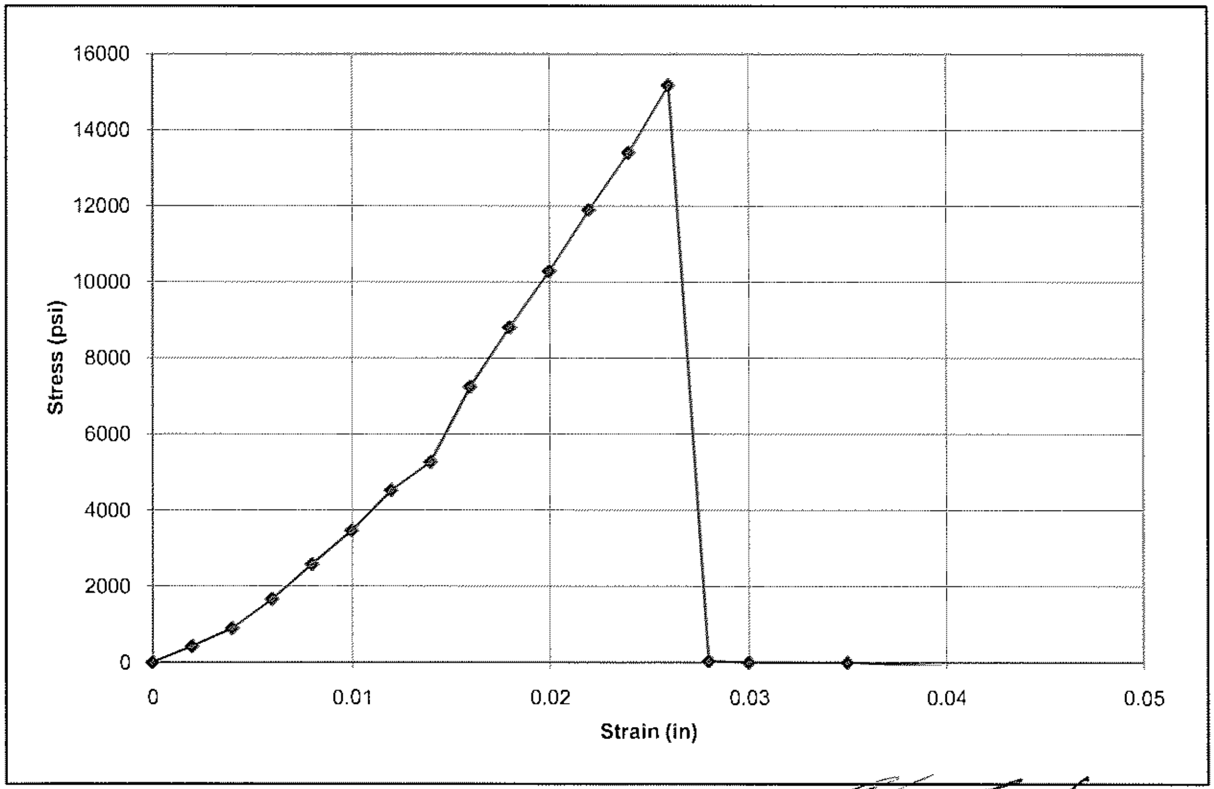
Tigard, OR 97223
PH: 503-443-3799 F: 503-620-2748

Geo Design
15575 SW Sequoia Parkway#100
Portland OR 97224

PROJECT:	Lab Services	**JOB NO:**	09-2307
LOCATION:	Oregon Solar Hwy	**WORK ORDER NO:**	ODOT-51-01
MATERIAL:	Rock Cores	**LAB NO:**	4745-1
SAMPLE SOURCE:	17.0-17.7 TB 25162-02	**DATE SAMPLED:**	09/16/09
SAMPLE PREP:			

UNCONFINED COMPRESSION STRENGTH OF Rock Cores
APPLICABLE PORTIONS OF (ASTM D2938)

DIAMETER:	2.39 in	MAXIMUM STRESS:	15,180 psi
HEIGHT:	5.96 in	AT STRAIN:	0.44%
STRAIN RATE:	.006 inches/min.		
DRY DENSITY:	63.8 lb/cu.ft		
MOISTURE:	2.9%		



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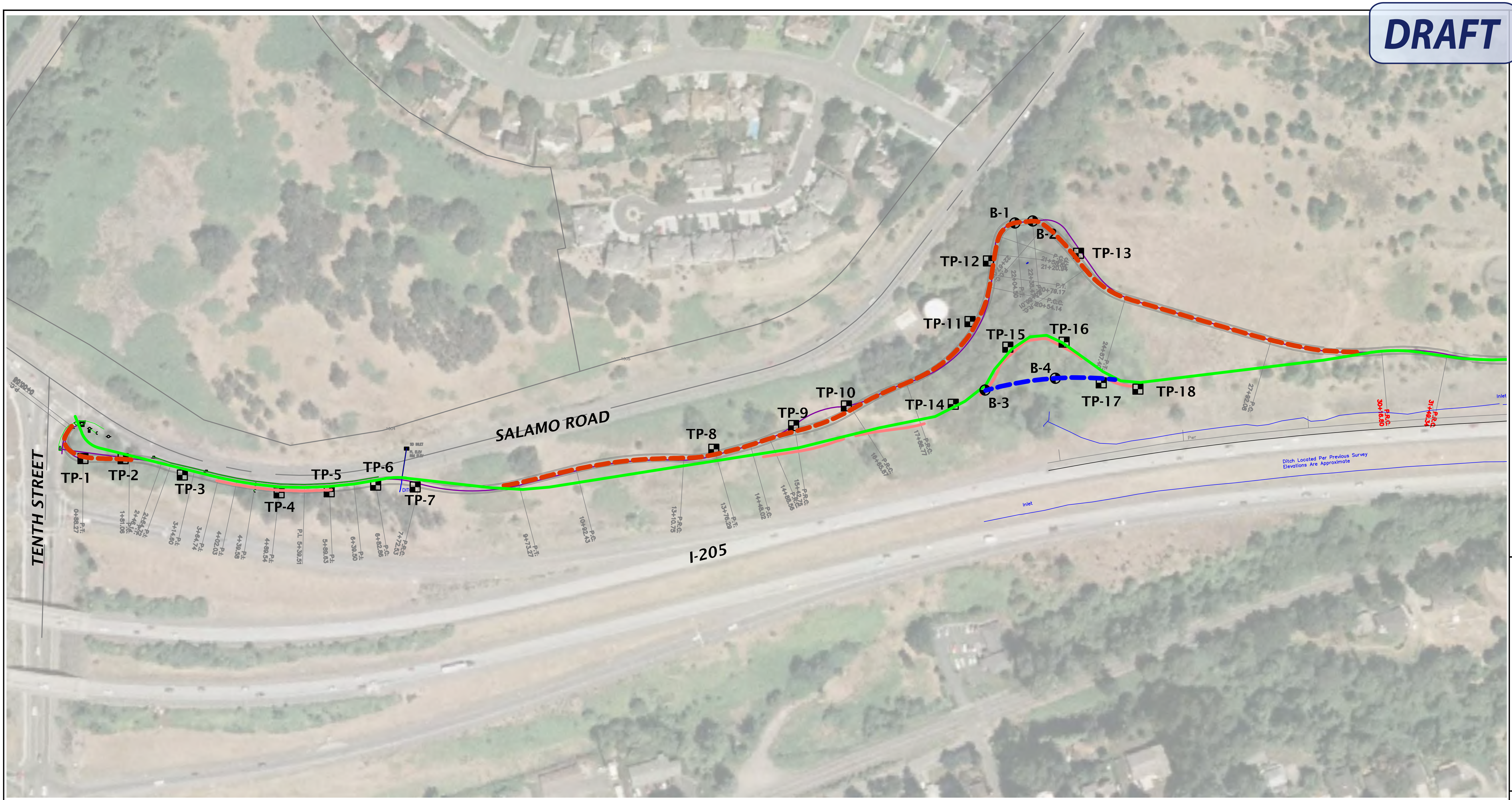
DRAFT

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

West Linn Trail Bike/Pedestrian Path
West Linn, Oregon

For
Otak, Inc.
March 7, 2013

GeoDesign Project: ODOT-71-01



LEGEND:

B-1⊕ BORING

TP-1▣ TEST PIT

━━━ PROPOSED TRAIL ALIGNMENT

╍╍╍ PREVIOUS PROPOSED TRAIL ALIGNMENT 1

╍╍╍ PREVIOUS PROPOSED TRAIL ALIGNMENT 2

━━━ PROPOSED RETAINING WALL LOCATION

N

0 200 400

(SCALE IN FEET)

SITE PLAN BASED ON DRAWING PROVIDED BY
OTAK, NOVEMBER 26, 2012

FIGURE 3

EXPLORATION LOCATIONS

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

ODOT-71-01

MARCH 2013

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

Printed By: aday | Print Date: 3/7/2013 2:12:03 PM
File Name: C:\Users\aday\appdata\local\temp\AcPublish_3520\ODOT-71-01-SP03.dwg | Layout: FIGURE 3

APPENDIX

FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions along the trail alignment by advancing four borings (B-1 through B-4) to depths ranging between approximately 15 and 32 feet BGS, excavating 18 test pits (TP-1 through TP-18) to depths ranging between 2.5 and 12 feet BGS, and by shallow hand excavations with a shovel. Figure 3 shows the approximate exploration locations. Drilling and test pit excavation services were provided by Western States Drilling on May 26 and 27, 2011 and October 8, 2012 as well as by Dan J. Fischer Excavating, Inc. on January 15, 2013. The drilling was completed using a track-mounted drill rig using mud rotary methods. The test pits were completed using a Hitachi 135 equipped with a rock-toothed bucket and a Kamatsu PC60 trackhoe. The explorations were observed by a member of our geologic staff. Hand excavations were performed by a member of geologic staff. We obtained representative samples of the various soils encountered in the explorations for geotechnical laboratory testing. Classifications and sampling depths are presented on the exploration logs included in this appendix.

Exploration locations were chosen based on the preliminary site plans provided to our office by Otak, Inc. The locations of the explorations were determined in the field by pacing from site features. This information should be considered accurate to the degree implied by the methods used.

SOIL SAMPLING

A member of our geologic staff observed the explorations. We obtained representative samples of the various soils encountered in the explorations for geotechnical laboratory testing. Samples were obtained from the borings using 1½-inch-inside diameter, split-spoon sampler (SPT sampler) and/or Shelby tubes. The split-spoon sampling was conducted in general accordance with ASTM D 1586. The 1½-inch-inside diameter, split-spoon samplers were driven into the soil with a 140-pound hammer free-falling 30 inches. The samplers were driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the exploration logs, unless otherwise noted. Representative grab samples of the soil from the test pit explorations were obtained from the walls and/or base of the test pits using the trackhoe bucket.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the “Exploration Key” (Table A-1) and “Soil Classification System” (Table A-2), which are included in this appendix. The exploration logs indicate the depths at which the soils or their characteristics change, although the change could be gradual. A horizontal line between soil types indicates an observed (visual or drill action) change. If the change occurred between sample locations and was not observed or obvious, the depth was interpreted and the change is indicated using a dashed line. Classifications and sampling intervals are presented on the exploration logs included in this appendix.

LABORATORY TESTING

CLASSIFICATION

The soil samples were classified in the laboratory to confirm field classifications. The laboratory classifications are included on the exploration logs if those classifications differed from the field classifications.

MOISTURE CONTENT

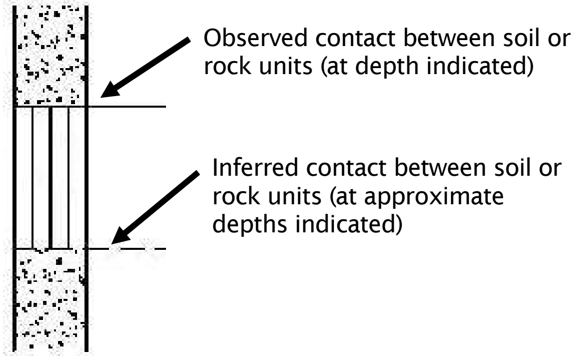
We determined the natural moisture content of selected soil samples in general accordance with ASTM D 2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented on the exploration logs included in this appendix.

GRAIN-SIZE TESTING

We completed grain-size testing on a select soil sample in order to determine the fines content (percent passing the U.S. Standard No. 200 Sieve) in general accordance with ASTM D 1140. The fines content test results are presented on the appropriate exploration log.

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



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

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EXPLORATION KEY

TABLE A-1

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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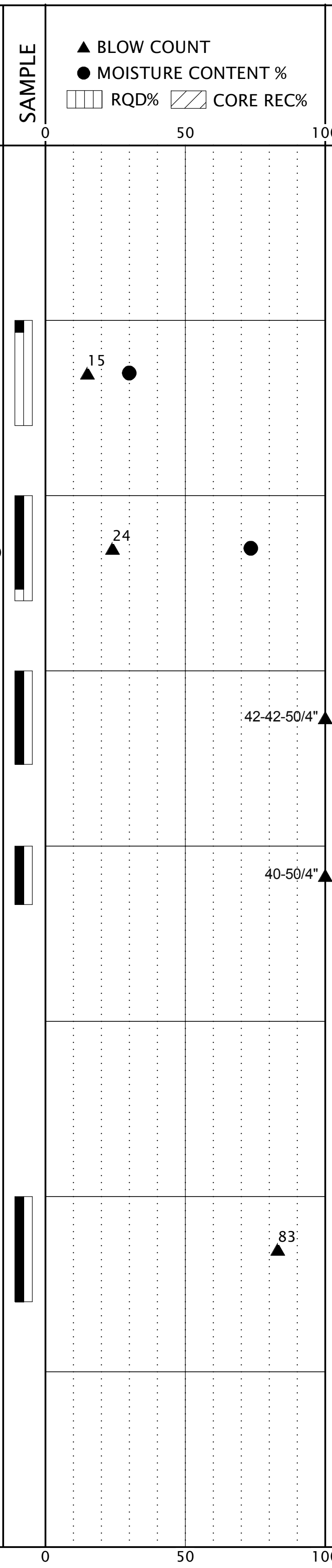
SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Medium dense, gray, silty GRAVEL with sand (GM); moist, angular (colluvium).			
	Very stiff, orange mottled brown, silty SAND (SM), minor gravel; moist, gravel is fine (colluvium).	4.5	P200	P200 = 30%
	Very dense, red-brown, fine to coarse SAND with silt (SM); moist, angular (weathered bedrock).	7.0		
	Very dense, brown, silty GRAVEL with sand (GM); wet, angular, sand is fine to coarse (weathered bedrock).	9.5		
20.0				



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: CR
COMPLETED: 05/26/11

BORING METHOD: mud rotary (see report text)
BORING BIT DIAMETER: 3 7/8-inch

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BORING B-1

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-1

DRAFT

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

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	BLOW COUNT (0 – 50 – 100)	INSTALLATION AND COMMENTS
20.0	grades with sand at 20.0 feet			■	50/3" ▲	
	Very dense, gray GRAVEL (GP), minor sand; moist, slightly weathered, sand is fine to coarse.	24.0				
25.0				■	6-50/3" ▲	
30.0				■	50/3" ▲	
				▭	50/1" ▲	Drilling refusal at 32.0 feet Surface elevation was not measured at the time of exploration.
	Exploration terminated due to practical refusal on bedrock at a depth of 32.1 feet.	32.1				
32.5						
35.0						
37.5						
40.0						

DRILLED BY: Western States Soil Conservation, Inc.
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COMPLETED: 05/26/11

BORING METHOD: mud rotary (see report text)
BORING BIT DIAMETER: 3 7/8-inch

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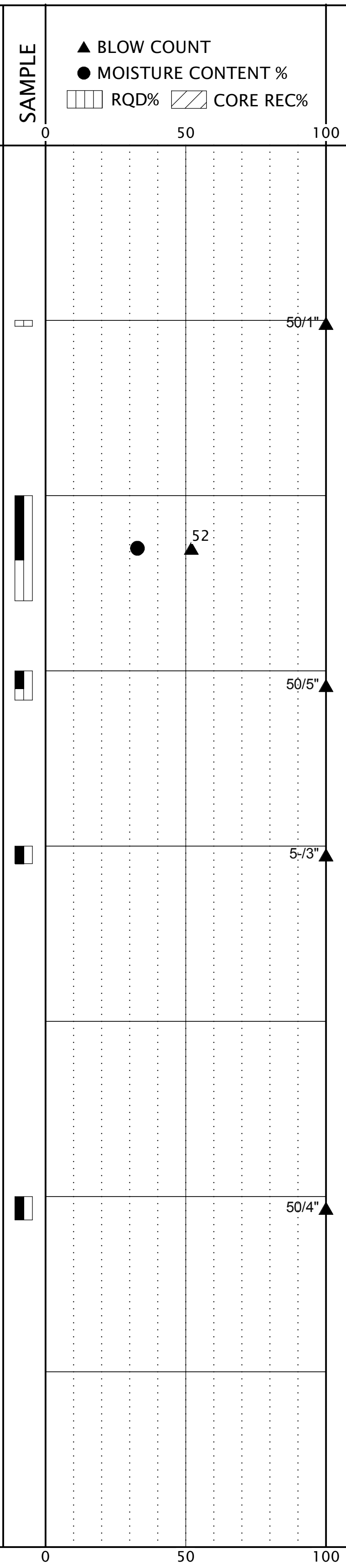
BORING B-1
(continued)

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-1

BORING LOG ODOT-71-01-B1_4-TP1_18.GPJ GEODESIGN.GDT PRINT DATE: 3/7/13:KT:AMD:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT %	INSTALLATION AND COMMENTS
0.0	Loose to medium dense, silty GRAVEL (GM), some sand; moist (colluvium).					
	large boulder at 2.0 feet					
2.5					50/1"	
5.0	Very dense, yellow-brown, silty GRAVEL with sand (GM); moist, angular, sand is fine to coarse (weathered bedrock).	5.0			52	
7.5					50/5"	
10.0	grades with less silt at 10.0 feet				5-/3"	
12.5						
15.0					50/4"	
17.5						
20.0						



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COMPLETED: 05/27/11

BORING METHOD: mud rotary (see report text)

BORING BIT DIAMETER: 3 7/8-inch

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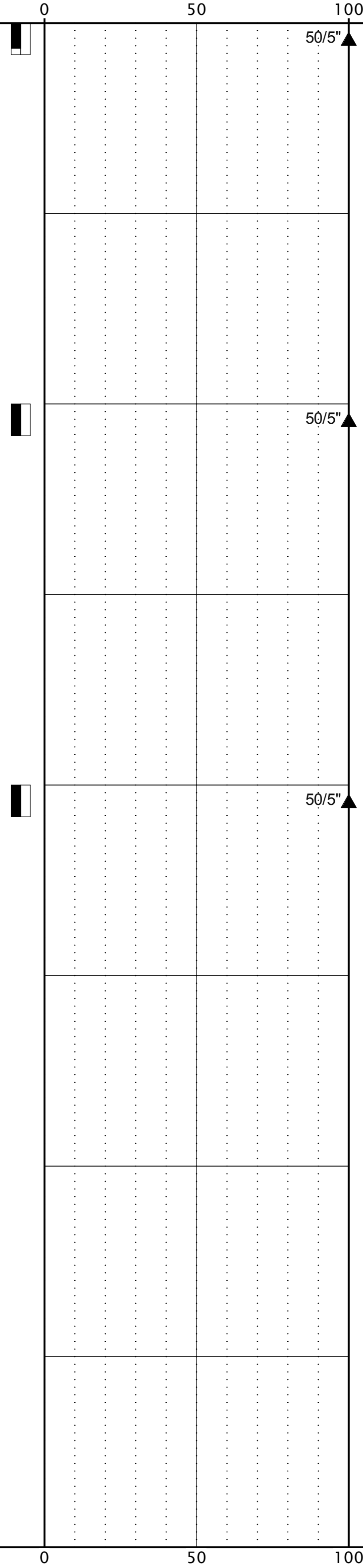
BORING B-2

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-2

DRAFT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
20.0		(continued from previous page)				50/5"	
25.0						50/5"	
30.0						50/5"	
30.5		Exploration terminated due to practical refusal on bedrock at a depth of 30.5 feet.	30.5				Surface elevation was not measured at the time of exploration.



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COMPLETED: 05/27/11

BORING METHOD: mud rotary (see report text)
BORING BIT DIAMETER: 3 7/8-inch

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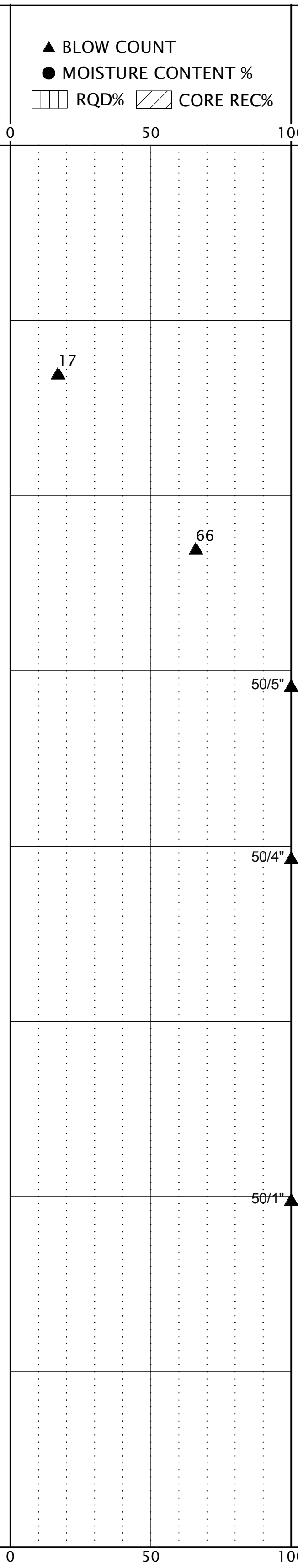
BORING B-2
(continued)

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-2

BORING LOG ODOT-71-01-B1_4-TP1_18.GPJ GEODESIGN.GDT PRINT DATE: 3/7/13:KT:AMD:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
0.0	Very stiff, brown SILT with sand (ML), minor clay, trace organics (rootlets); moist (colluvium).		
5.0	Very dense, brown-gray, silty GRAVEL (GM), some clay, minor sand; moist (decomposed bedrock).	5.0	
7.5	Very dense, gray-brown GRAVEL with sand and silt (GP-GM), minor clay; moist (weathered bedrock).	7.5	
15.1	Exploration completed at a depth of 15.1 feet.	15.1	Surface elevation was not measured at the time of exploration.



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: JGH
COMPLETED: 10/08/12

BORING METHOD: mud rotary (see report text)
BORING BIT DIAMETER:

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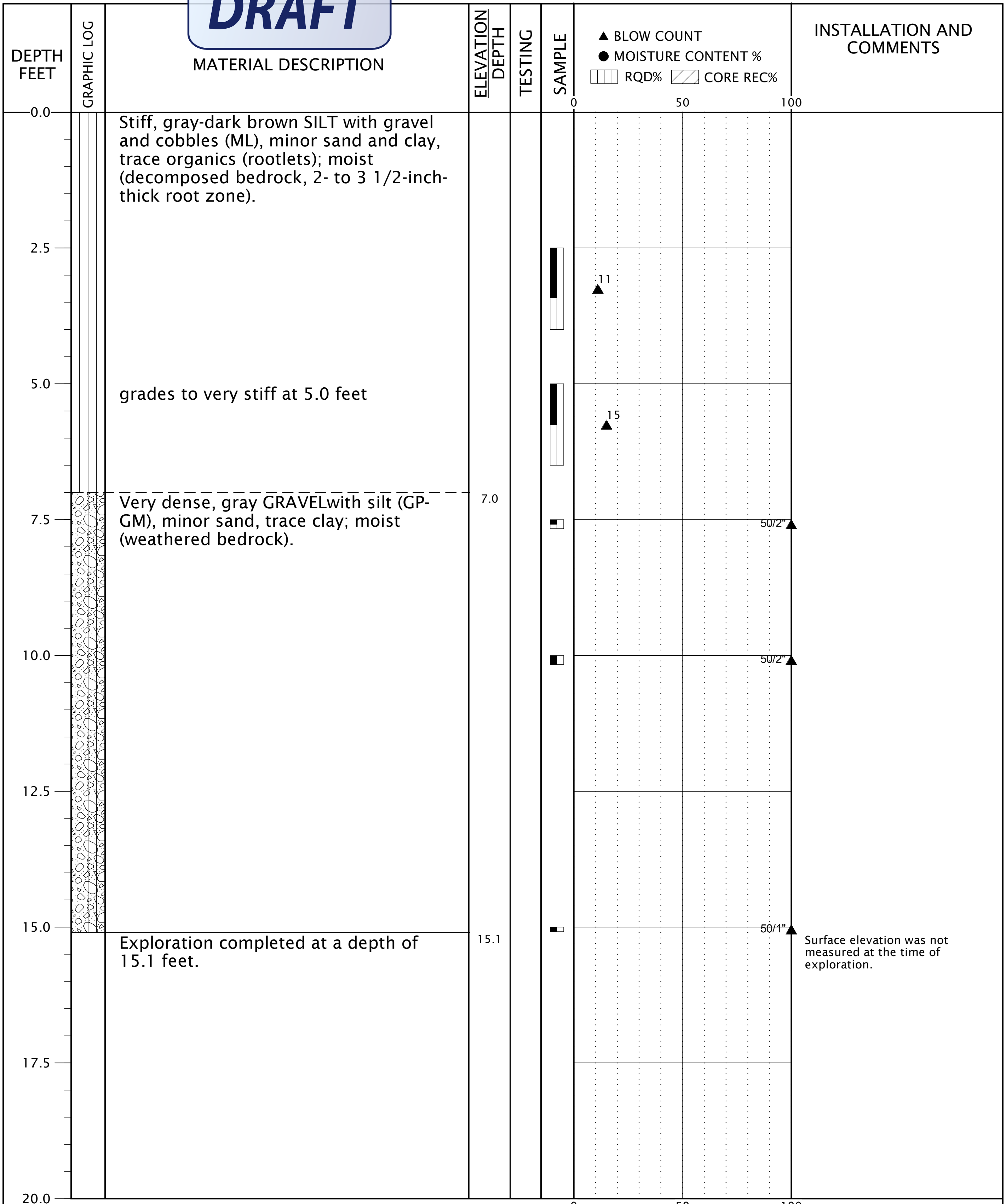
BORING B-3

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-3



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Stiff, gray-dark brown SILT with gravel and cobbles (ML), minor sand and clay, trace organics (rootlets); moist (decomposed bedrock, 2- to 3 1/2-inch-thick root zone).			
2.5			11	
5.0	grades to very stiff at 5.0 feet		15	
7.0	Very dense, gray GRAVELwith silt (GP-GM), minor sand, trace clay; moist (weathered bedrock).	7.0		
7.5			50/2"	
10.0			50/2"	
15.0			50/1"	Surface elevation was not measured at the time of exploration.
15.1	Exploration completed at a depth of 15.1 feet.	15.1		

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LOGGED BY: JGH

COMPLETED: 10/08/12

BORING METHOD: mud rotary (see report text)

BORING BIT DIAMETER:

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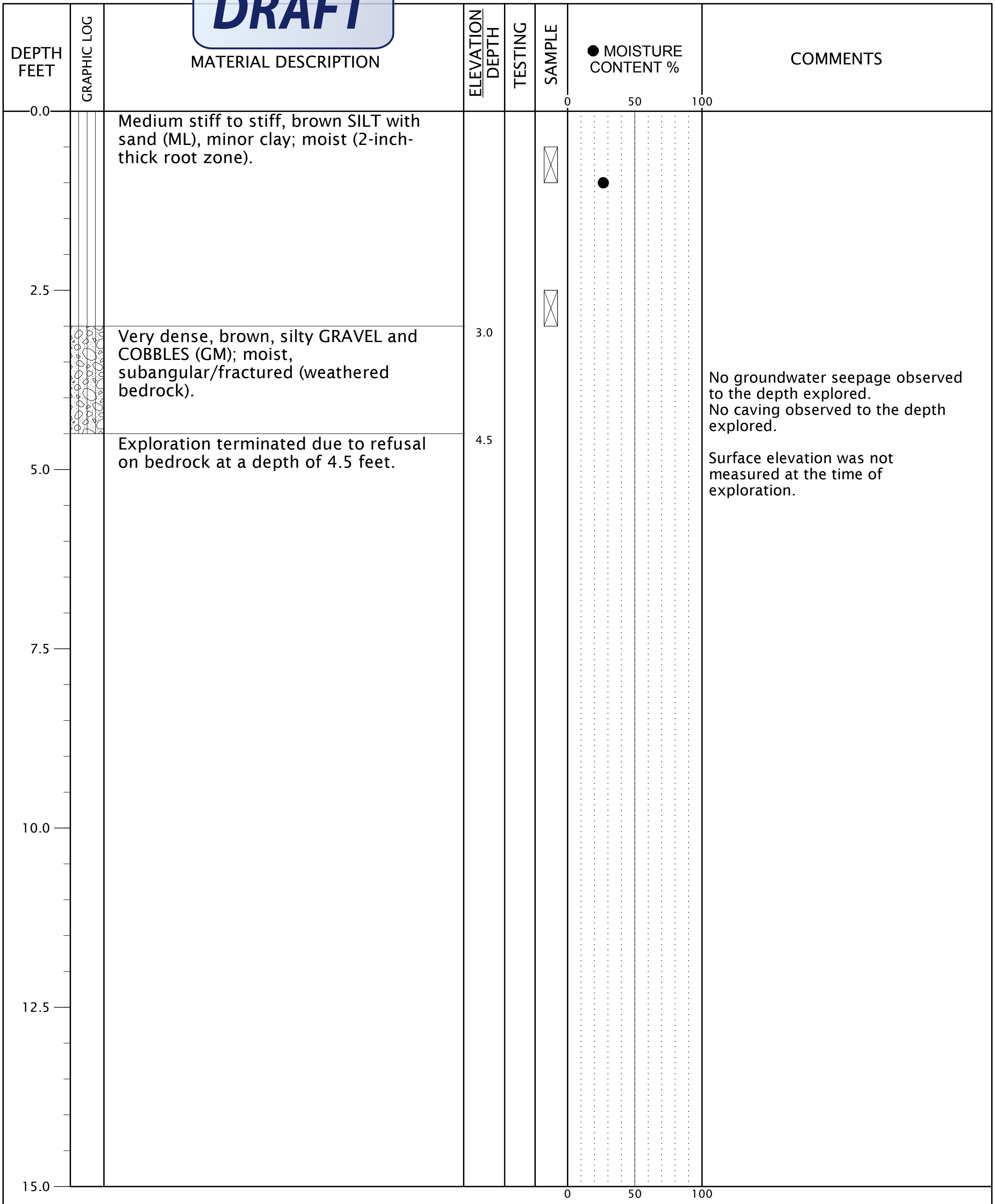
BORING B-4

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-4



Depth (ft)	Material Description
0.0	Medium stiff to stiff, brown SILT with sand (ML), minor clay; moist (2-inch-thick root zone).
3.0	Very dense, brown, silty GRAVEL and COBBLES (GM); moist, subangular/fractured (weathered bedrock).
4.5	Exploration terminated due to refusal on bedrock at a depth of 4.5 feet.

No groundwater seepage observed to the depth explored.
No caving observed to the depth explored.

Surface elevation was not measured at the time of exploration.

EXCAVATED BY: Western States Soil Conservation, Inc.

LOGGED BY: CLR

COMPLETED: 05/26/11

EXCAVATION METHOD: trackhoe (see report text)

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TEST PIT TP-8

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-12

TEST PIT LOG - 1 PER PAGE ODOT-71-01-B1_4-TP1_18.GPJ GEODESIGN.GDT PRINT DATE: 3/7/13:KT:AMD:KT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % 0 50 100	COMMENTS
0.0	▨	Very dense, brown, silty GRAVEL and COBBLES (GM); moist, subangular to fractured (1- to 2-inch-thick root zone, weathered bedrock).			☒		
2.5	▨						No groundwater seepage observed to the depth explored. No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
		Exploration terminated due to refusal on bedrock at a depth of 3.0 feet.	3.0				
5.0							
7.5							
10.0							
12.5							
15.0						0 50 100	

EXCAVATED BY: Western States Soil Conservation, Inc.
LOGGED BY: CLR
COMPLETED: 05/26/11

EXCAVATION METHOD: trackhoe (see report text)

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TEST PIT TP-9

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-13

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % 0 50 100	COMMENTS
0.0		Very dense, brown, silty GRAVEL and COBBLES (GM); moist, subangular/fractured (1- to 2--inch-thick root zone, colluvium/weathered bedrock).					
2.5					☒		
5.0		Exploration terminated due to refusal on bedrock at a depth of 5.0 feet.	5.0				No groundwater seepage observed to the depth explored. No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
7.5							
10.0							
12.5							
15.0						0 50 100	

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EXCAVATION METHOD: trackhoe (see report text)

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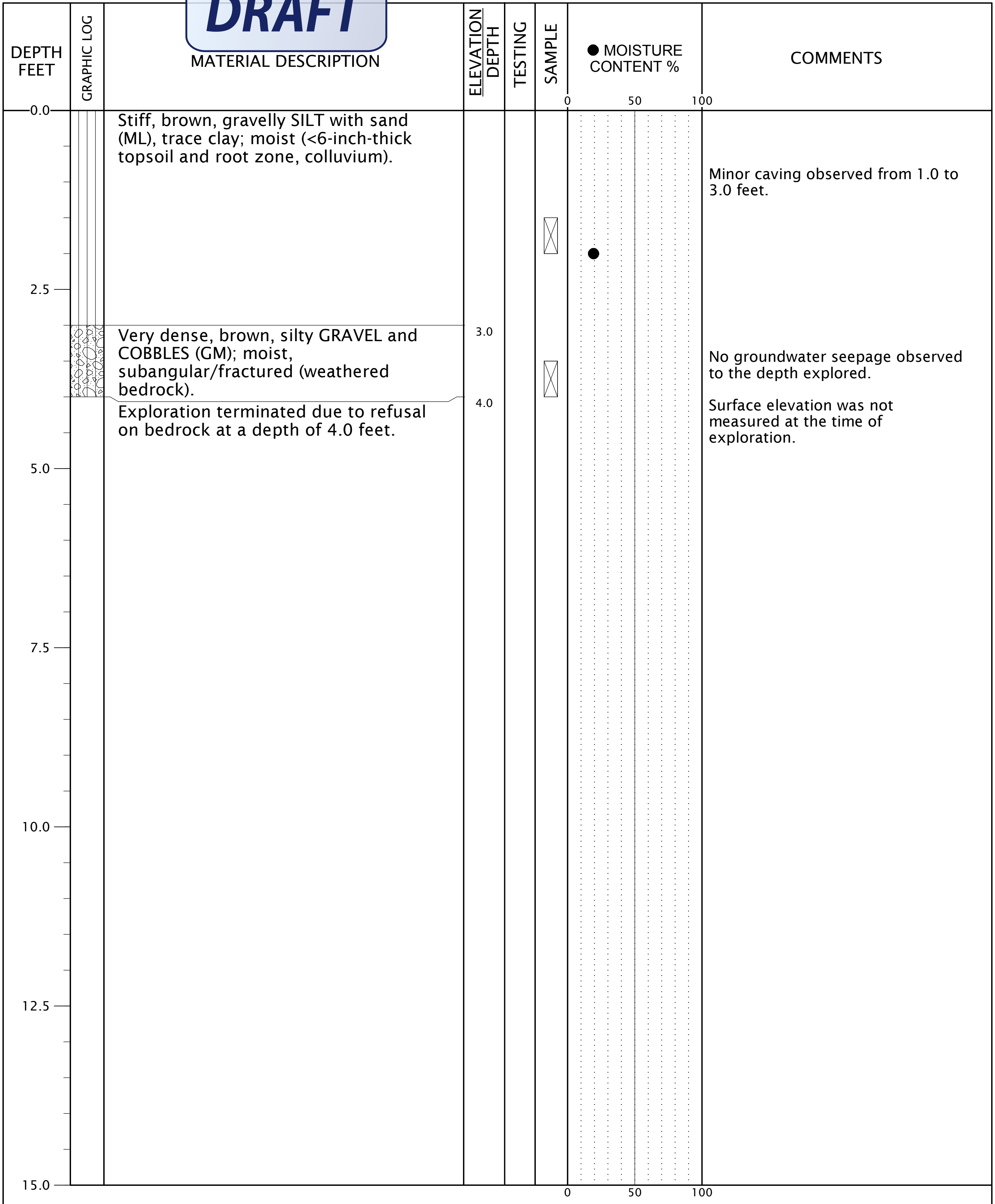
TEST PIT TP-10

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-14

TEST PIT LOG - 1 PER PAGE ODOT-71-01-B1_4-TP1_18.GPJ GEODESIGN.GDT PRINT DATE: 3/7/13:KT:AMD:KT



Stiff, brown, gravelly SILT with sand (ML), trace clay; moist (<6-inch-thick topsoil and root zone, colluvium).

Minor caving observed from 1.0 to 3.0 feet.

Very dense, brown, silty GRAVEL and COBBLES (GM); moist, subangular/fractured (weathered bedrock).

Exploration terminated due to refusal on bedrock at a depth of 4.0 feet.

No groundwater seepage observed to the depth explored.

Surface elevation was not measured at the time of exploration.

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EXCAVATION METHOD: trackhoe (see report text)

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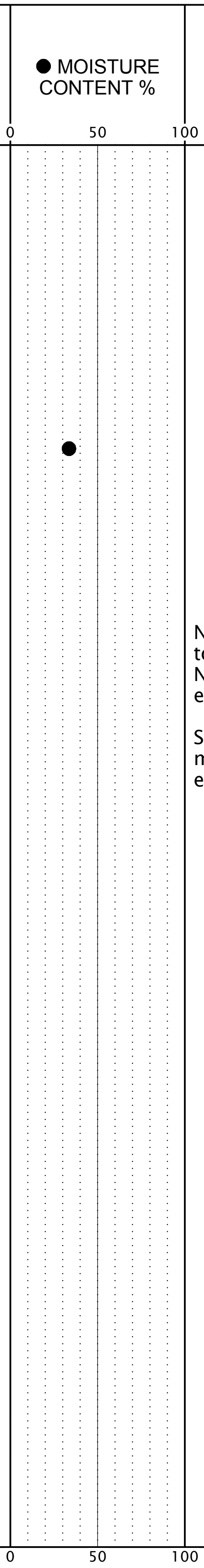
TEST PIT TP-11

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-15

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Stiff, brown, gravelly SILT with sand (ML), trace roots; moist (topsoil to 12 inches, 4-inch-thick root zone, colluvium).				
	without roots at 2.0 feet			☒	
	Dense to very dense, red-brown to brown, silty GRAVEL (GM); moist, subangular to subrounded (weathered bedrock).	3.5			No groundwater seepage observed to the depth explored. No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
	Exploration terminated due to refusal on bedrock at a depth of 6.0 feet.	6.0			



Depth scale: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0

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LOGGED BY: CMC
COMPLETED: 05/27/11

EXCAVATION METHOD: trackhoe (see report text)

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TEST PIT TP-12

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-16

DRAFT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0	Medium dense to dense, brown, fine to coarse, silty GRAVEL with sand (GM); moist, subangular (topsoil to 6 inches, 4-inch-thick root zone) - **FILL**.					
2.5				☒	●	
	Stiff to very stiff, brown SILT with gravel (ML); moist, blocky (colluvium).	3.0	PP			PP = >4.5 tsf
				☒		
5.0						
7.5						
	becomes with red-brown veining; weakly indurated at 9.0 feet					
10.0	Dense to very dense, red-brown to gray-brown, silty GRAVEL (GM), minor sand; moist, coarse and subangular (weathered bedrock).	10.0		☒		No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
	Exploration terminated due to refusal on bedrock at a depth of 11.5 feet.	11.5				Surface elevation was not measured at the time of exploration.
12.5						
15.0						

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LOGGED BY: CMC

COMPLETED: 05/27/11

EXCAVATION METHOD: trackhoe (see report text)

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TEST PIT TP-13

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-17

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % 0 50 100	COMMENTS
0.0		Medium stiff, orange-brown, silty GRAVEL with cobbles (GM), minor sand, trace organics (roots); moist, organics are up to 3/4-inch diameter(colluvium, 2- to 3 3/4-inch-thick root zone).			☒	●	
2.5		Medium dense to dense, gray-orange, silty GRAVEL with boulders and cobbles (GM), some clay, trace organics (rootlets); moist (weathered basalt),	2.5		☒		No groundwater seepage observed to the depth explored. No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
		Exploration terminated due to refusal at a depth of 4.0 feet.	4.0				
5.0							
7.5							
10.0							
12.5							
15.0						0 50 100	

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: JGH COMPLETED: 01/15/13

EXCAVATION METHOD: trackhoe (see report text)

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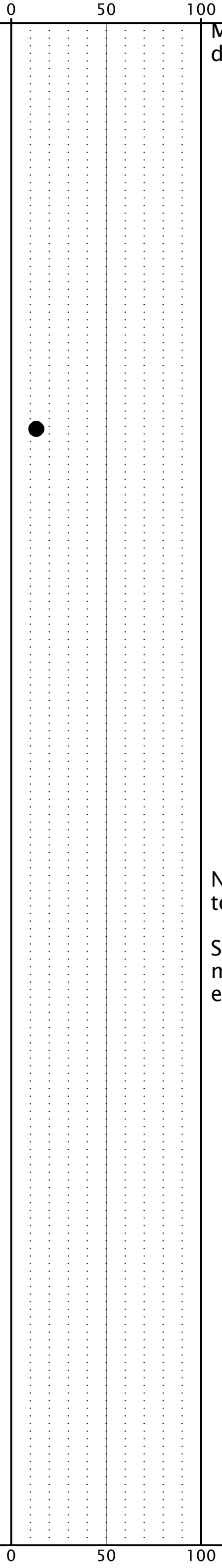
TEST PIT TP-14

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-18

DRAFT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT %	COMMENTS
0.0		Medium stiff, brown SILT (ML), some clay, minor organics (roots); moist, organics are up to 2-inch diameter (topsoil, 3-inch-thick root zone).					Minor caving observed to the depth explored.
		Medium stiff, orange-brown SILT with cobbles (ML), minor clay, trace organics (roots); moist, organics are up to 1-inch diameter (colluvium).	1.0				
2.5							
		Medium dense to dense, gray-orange GRAVEL with silt and cobbles (GP-GM), minor clay, trace organics (rootlets); moist (weathered bedrock).	3.0			●	
5.0							
7.5							No groundwater seepage observed to the depth explored. Surface elevation was not measured at the time of exploration.
		Exploration terminated due to refusal at a depth of 9.0 feet.	9.0				
10.0							
12.5							
15.0							



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EXCAVATION METHOD: trackhoe (see report text)

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TEST PIT TP-15

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-19

TEST PIT LOG - 1 PER PAGE ODOT-71-01-B1_4-TP1_18.GPJ GEODESIGN.GDT PRINT DATE: 3/7/13:KT:AMD:KT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % 0 50 100	COMMENTS
0.0		Medium stiff, gray SILT (ML), minor gravel and organics (roots), some clay; moist, organics are up to 1-inch diameter (topsoil, 2 1/2- to 3 1/2-inch-thick root zone).			☒		
		Medium stiff, orange-brown SILT with cobbles (ML), minor clay and sand; moist (colluvium).	0.8		☒		
2.5		Medium dense to dense, gray-orange GRAVEL with silt and cobbles (GP-GM), minor clay; moist (weathered bedrock).	2.5		☒		No groundwater seepage observed to the depth explored. No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
		Exploration terminated due to refusal at a depth of 4.0 feet.	4.0				
5.0							
7.5							
10.0							
12.5							
15.0						0 50 100	

EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: JGH

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EXCAVATION METHOD: trackhoe (see report text)

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TEST PIT TP-16

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-20

DRAFT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0	Medium stiff, brown SILT with gravel to cobbles (ML), some clay, trace organics (roots); moist, organics are up to 1-inch diameter (topsoil, 2-inch-thick root zone).			☒	●	Minor caving observed 0.0 to 7.0 feet.
2.5						
	Medium dense, orange-brown GRAVEL with cobbles to boulders and silt (GP-GM), minor sand, trace clay and organics (roots); moist, organics are up to 1/4-inch diameter.	3.0		☒		
5.0	Medium stiff, orange brown, clayey GRAVEL with cobbles (GC), minor sand, trace silt; moist (colluvium).	5.0		☒		
	Medium dense to dense, gray-orange GRAVEL with cobbles (GP), trace silt and clay; moist (weathered bedrock).	7.0				
7.5						No groundwater seepage observed to the depth explored. Surface elevation was not measured at the time of exploration.
	Exploration terminated due to refusal at a depth of 9.0 feet.	9.0				
10.0						
12.5						
15.0						

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: JGH COMPLETED: 01/15/13

EXCAVATION METHOD: trackhoe (see report text)

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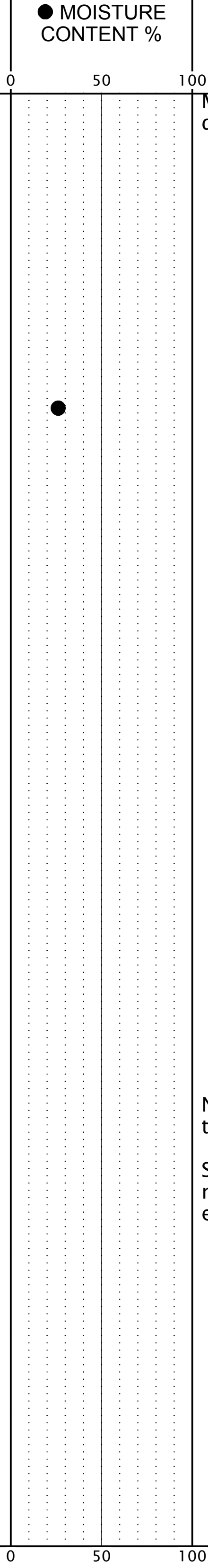
TEST PIT TP-17

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-21

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, brown SILT with gravel to cobbles (ML), some clay, trace organics (roots); moist, organics are up to 3/4-inch diameter (colluvium).			☒	Minor caving observed to the depth explored.
1.2	Medium dense to dense, orange-gray, clayey GRAVEL with cobbles (GC), minor sand, trace silt; moist (weathered bedrock).	1.2		☒ ☒ ☒	
5.0	with boulders at 5.0 feet				
10.0				☒	No groundwater seepage observed to the depth explored.
11.0	Exploration terminated due to refusal at a depth of 11.0 feet.	11.0			Surface elevation was not measured at the time of exploration.



DEPTH FEET: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0

EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: JGH

COMPLETED: 01/15/13

EXCAVATION METHOD: trackhoe (see report text)

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

ODOT-71-01

MARCH 2013

TEST PIT TP-18

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-22

DRAFT

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT (PERCENT)	PLASTIC LIMIT (PERCENT)	PLASTICITY INDEX (PERCENT)
B-1	2.5		30							
B-1	5.0		73				30			
B-2	5.0		33							
TP-1	3.0		28							
TP-2	3.0		30							
TP-3	2.0		20							
TP-4	2.0		10							
TP-7	2.5		9							
TP-8	1.0		27							
TP-11	2.0		19							
TP-12	3.0		34							
TP-13	2.0		22							
TP-14	1.0		25							
TP-15	4.0		13							
TP-17	2.0		29							
TP-18	3.0		26							

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ODOT-71-01

MARCH 2013

SUMMARY OF LABORATORY DATA

WEST LINN TRAIL BIKE/PEDESTRIAN PATH
WEST LINN, OR

FIGURE A-23

CH2M

CORNELL, HOWLAND, HAYES & MERRYFIELD

ENGINEERS AND PLANNERS

1600 WESTERN AVENUE • CORVALLIS, OREGON 97330
TELEPHONE: AREA CODE 503/752-4271
OTHER OFFICES IN: SEATTLE • BOISE • PORTLAND

10 November 1969

Project No. P5572.0

City of West Linn
West Linn, Oregon 97068

Attention: Mr. Richard W. Dippner, City Administrator

Gentlemen:

Willamette Reservoir and FAI-205 Landslide

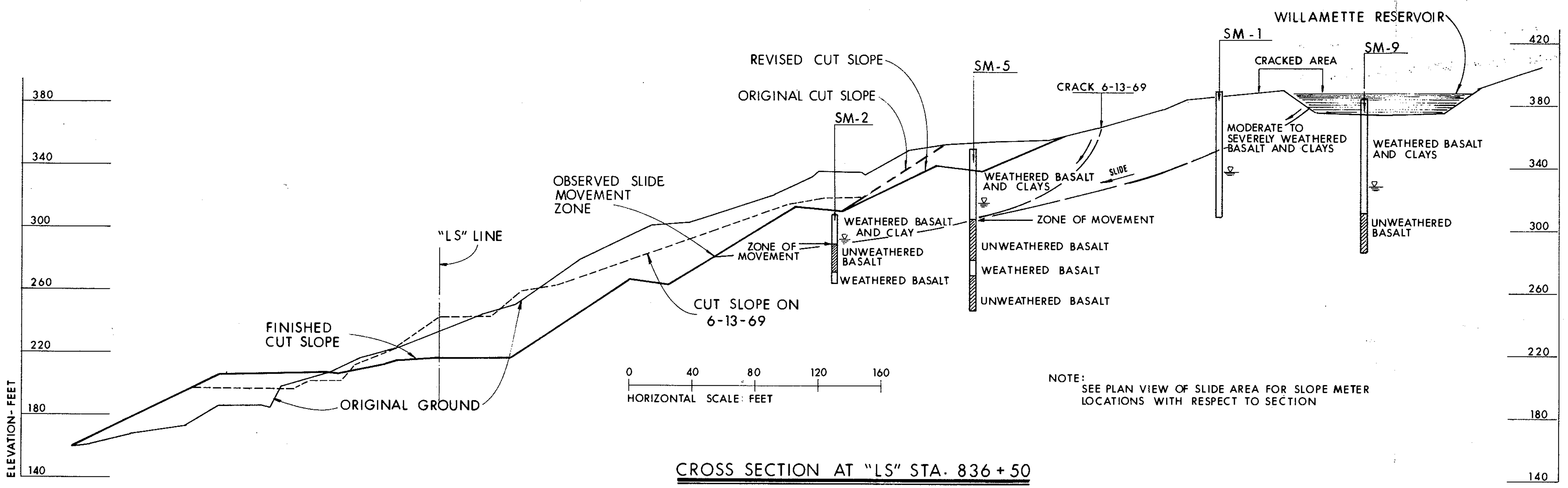
This letter presents our investigation of the condition of the Willamette Reservoir. The landslide associated with the construction of FAI-205 disrupted the reservoir and the distribution pipelines, as well as damaged several houses and other property located above the work area. This investigation was authorized in September, 1969, to examine the condition of the reservoir and to recommend its repair or abandonment. Conclusions and recommendations are presented at the end of this letter.

INTRODUCTION

Willamette Reservoir is located on the hills above the Cities of West Linn and Willamette, with a water surface elevation of 387. FAI-205 is under construction below the reservoir at about elevation 220, in a side hill excavation with designed slopes of 1 on 1.5 and three benches in the cut slope. On the evening of 10 June 1969, an Oregon State Highway Department (OSHD) inspector discovered cracks in the street and ground in the vicinity of the reservoir.

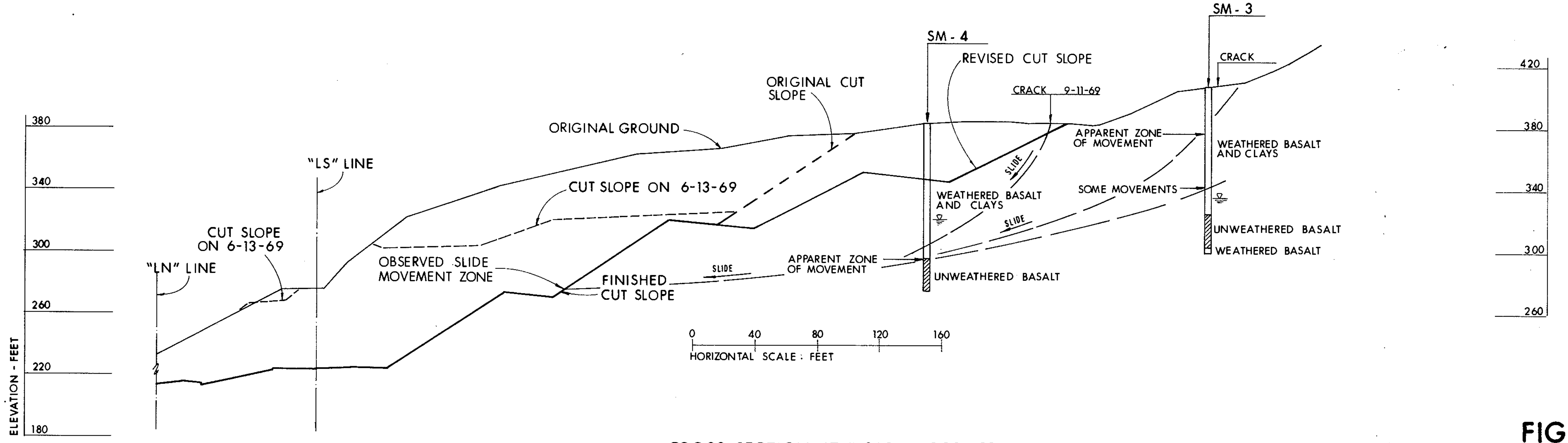
On 12 June 1969, the inflow and outflow pipes of the reservoir were closed, and the water level in the reservoir dropped 0.75-feet in 2-hours. The reservoir leakage at this time was 320-gallons per minue (gpm). The reservoir inflow and outflow pipes were then opened, but the water surface continued to drop and the reservoir could not be filled. One of the reservoir joints was observed to open 0.01-feet (1/8-inch) in a 24-hour period. On 14 June, 1969, the reservoir was drained and has been out of service since. Construction of a water main to bypass the reservoir area was authorized by the OSHD, and completed one month later on 14 July 1969. The same day the bypass main was placed in service, the 8-inch main serving the City of Willamette was pulled apart by slide movements just below the reservoir.

The condition of the reservoir prior to the June landslide activity is known to be good. In March 1969, the City water superintendent commented that the reservoir leakage had recently increased because of FAI-205 construction blasting.



NOTE:
SEE PLAN VIEW OF SLIDE AREA FOR SLOPE METER LOCATIONS WITH RESPECT TO SECTION

CROSS SECTION AT "LS" STA. 836 + 50



CROSS SECTION AT "LS" STA. 832 + 50

FIGURE 3
CITY OF WEST LINN, OREGON

FAI-205 LANDSLIDE STUDIES CROSS SECTIONS

CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE PORTLAND

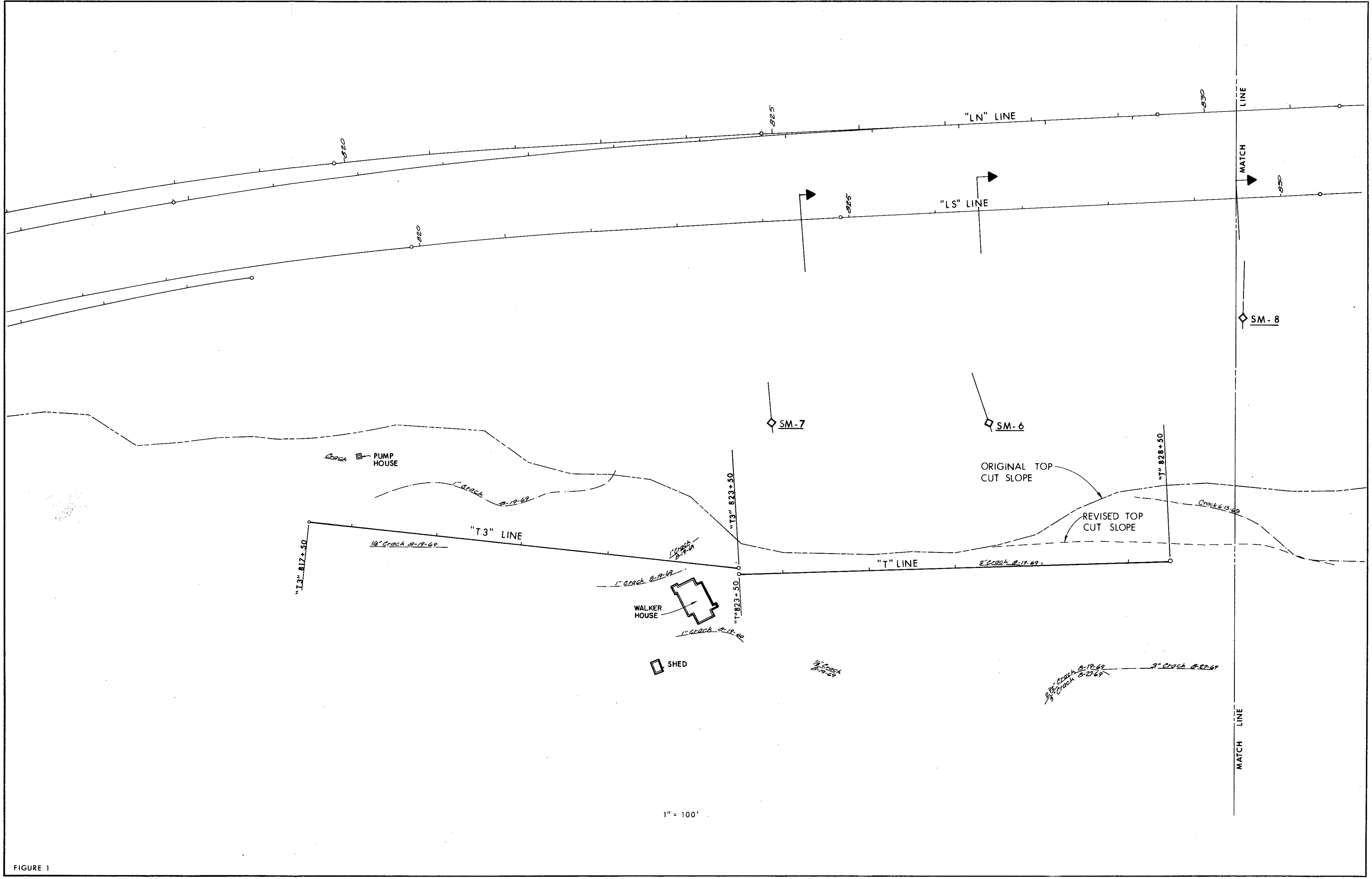

"LN" LINE
820
825
830
MATCH LINE
"LS" LINE
820
825
830
SM-8
SM-7
SM-6
Crack
PUMP HOUSE
1" Crack 8-19-69
ORIGINAL TOP CUT SLOPE
REVISED TOP CUT SLOPE
Crack 6-13-69
"Y" 828+50
"T3" 823+50
"T3" LINE
1/4" Crack 8-19-69
1" Crack 8-19-69
"T3" 817+50
"T" LINE
2" Crack 8-19-69
1" crack 8-19-69
WALKER HOUSE
"T" 823+50
1" crack 8-19-69
SHED
1/4" Crack 8-19-69
2 1/2" Crack 8-19-69
5" Crack 8-27-69
3" Crack 8-27-69
MATCH LINE
1" = 100'

FIGURE 1

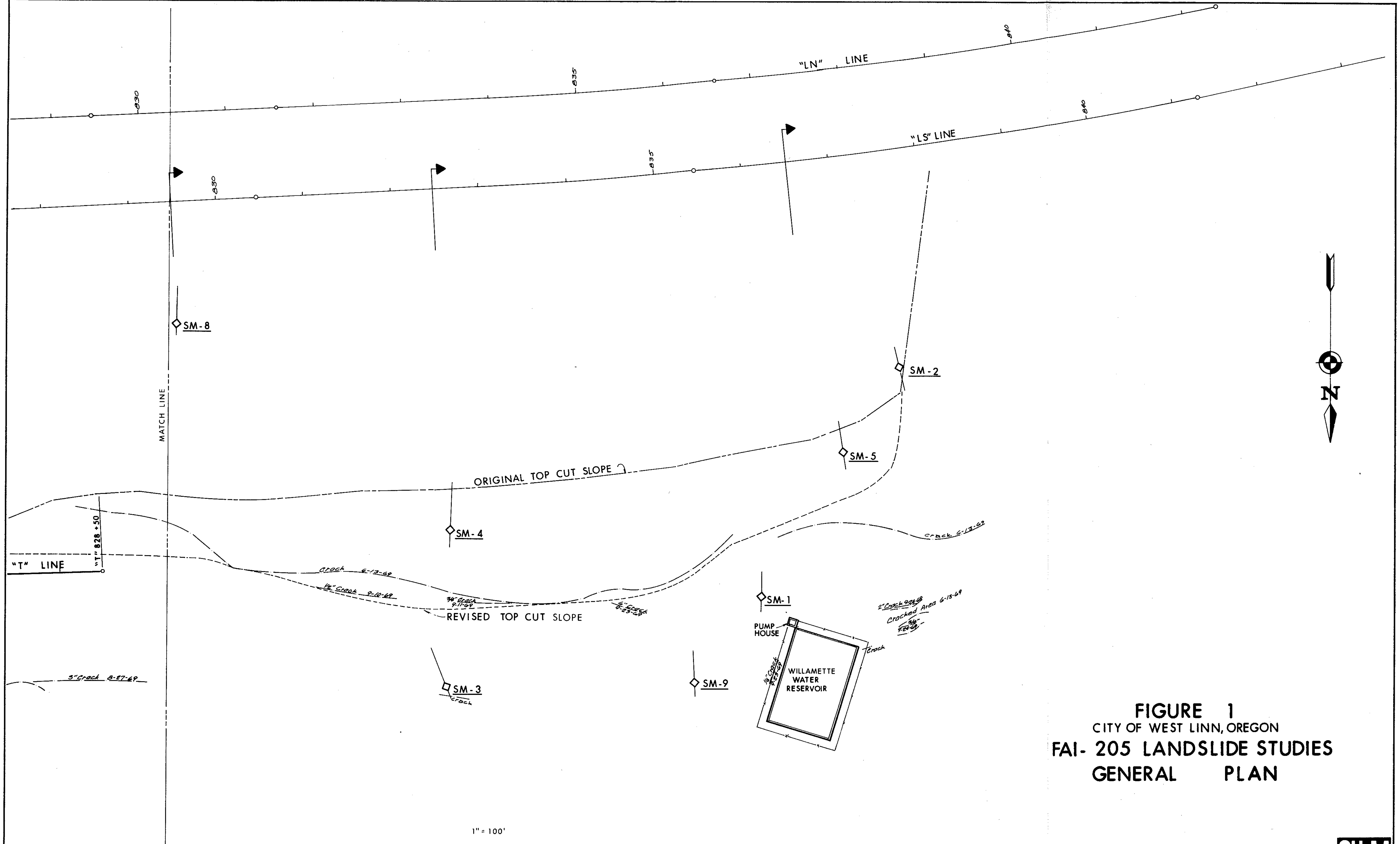


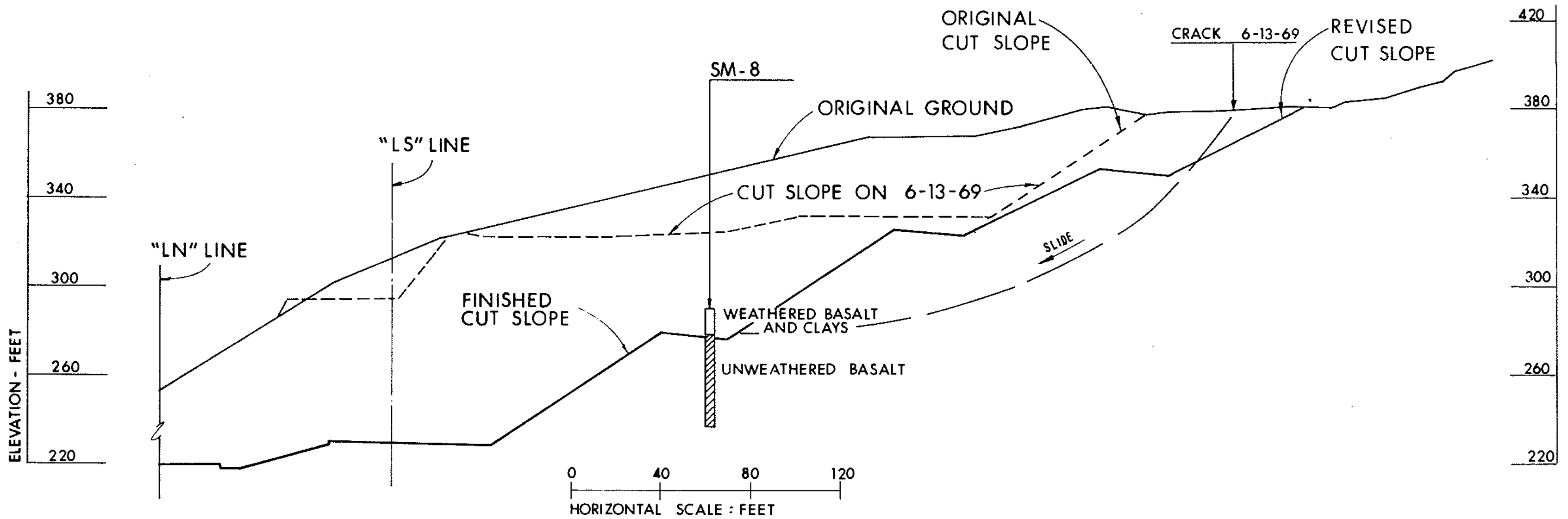
FIGURE 1

CITY OF WEST LINN, OREGON

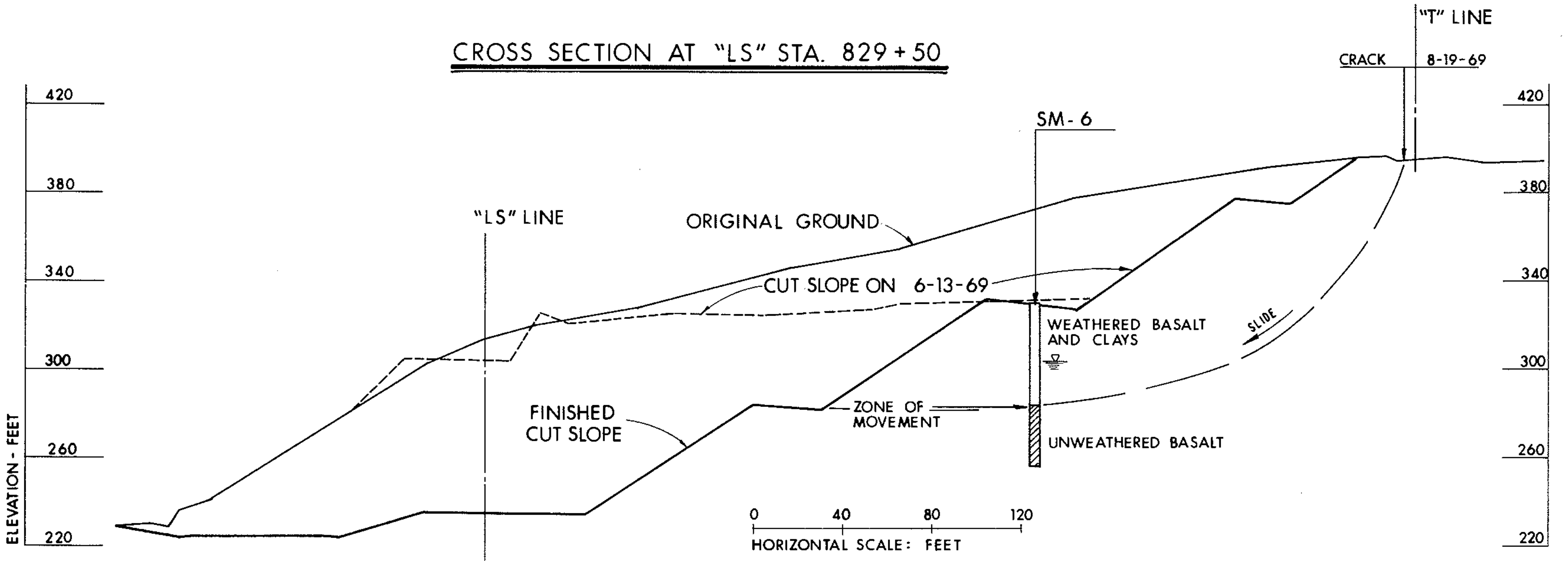
FAI- 205 LANDSLIDE STUDIES

GENERAL PLAN

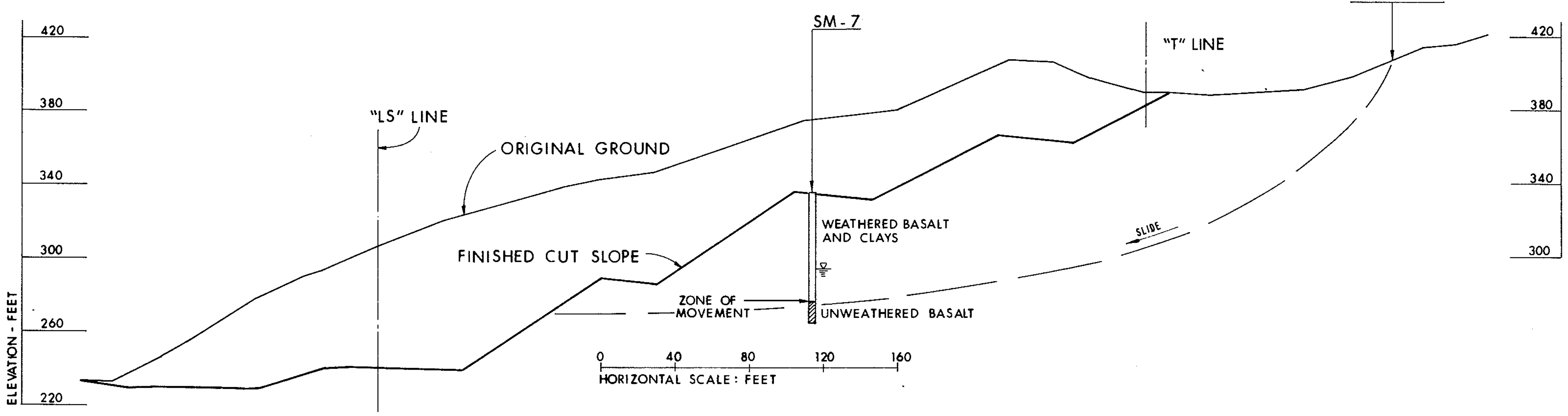
CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE PORTLAND



CROSS SECTION AT "LS" STA. 829 + 50



CROSS SECTION AT "LS" STA. 826 + 50



CROSS SECTION AT "LS" STA. 824 + 00

FIGURE 2

CITY OF WEST LINN, OREGON

FAI- 205 LANDSLIDE STUDIES
CROSS SECTIONS

CH2M

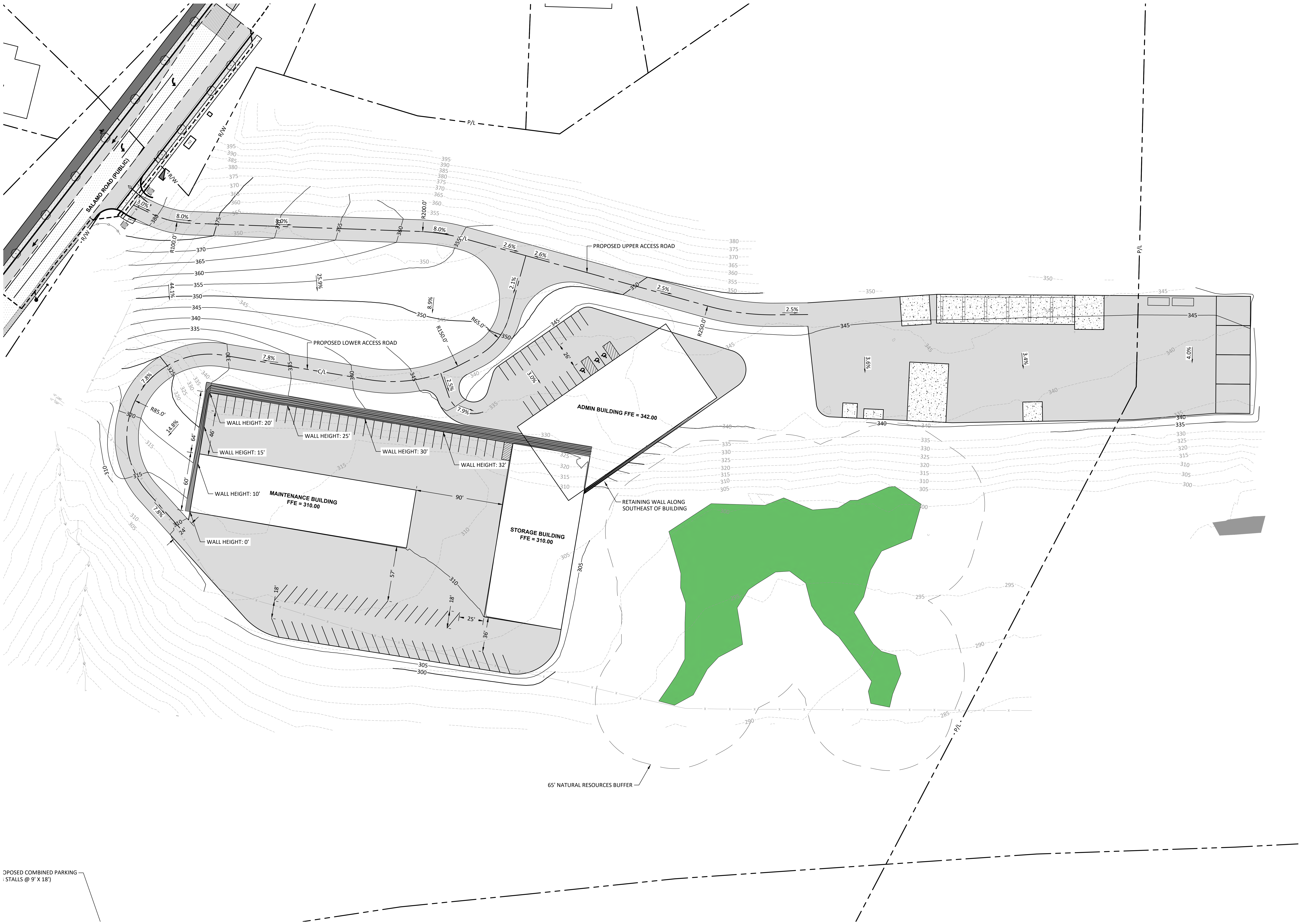
CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE PORTLAND

APPENDIX H

APPENDIX H
SCHEMATIC SITE PLANS

Relevant schematic site plans are presented in this appendix.

FILE PATH: C:\Revit_Local\23028_West Linn Public Works_tjorgensenZEPNR.rvt



PLAN VIEW

SCALE: 1" = 40'

LEGEND:

▒	ASPHALT PAVEMENT
░	CONCRETE PAVEMENT
░	GENERAL LANDSCAPING
🟩	WETLAND AREA
— · —	STORM LINE
— SAN —	SANITARY LINE
— W —	WATER LINE
— —	65' NATURAL RESOURCES BUFFER

NOT FOR CONSTRUCTION

WEST LINN PUBLIC WORKS FACILITY

Job Number: **23028**

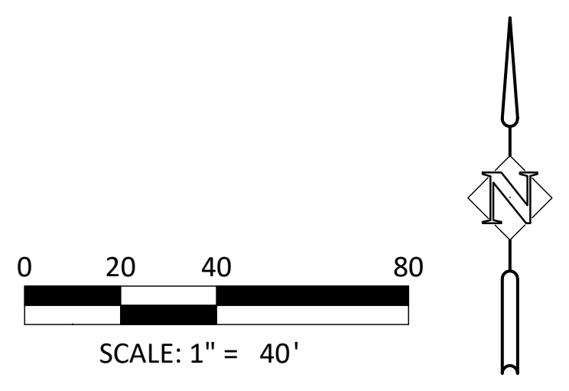
WEST LINN, OR

Harper
Houf Peterson
Righellis Inc.
ENGINEERS ◆ PLANNERS
LANDSCAPE ARCHITECTS ◆ SURVEYORS
205 SE Spokane Street, Suite 200, Portland, OR 97202
phone: 503.221.1131 www.hhpr.com fax: 503.221.1171

ISSUE	DATE
SCHEMATIC DESIGN	12.13.2024

Drawing:

SITE, UTILITY & GRADING PLAN



Sheet No:

C1.0

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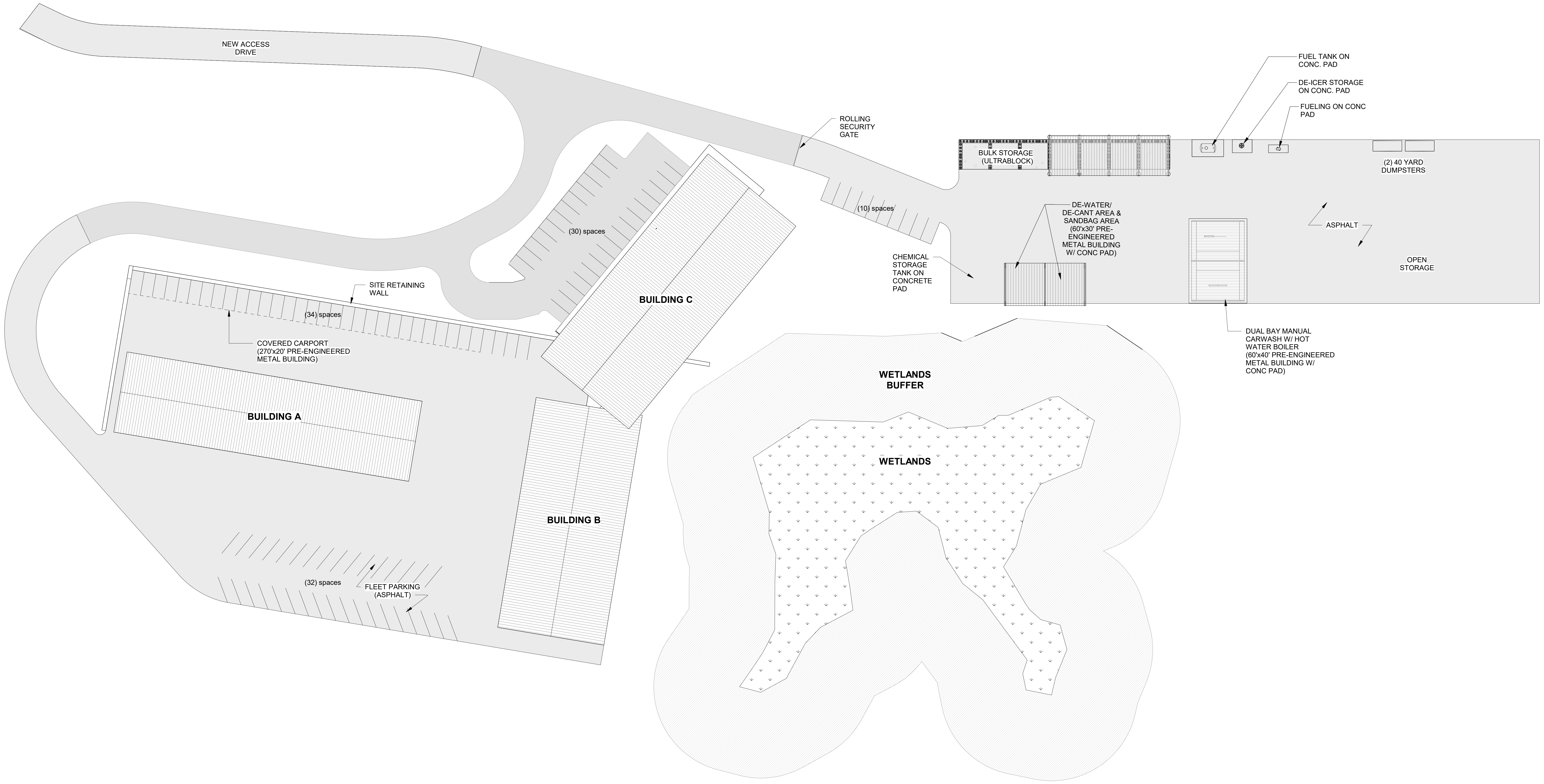

1 SITE PLAN

1" = 40'-0"

THESE DRAWINGS ARE THE ORIGINAL UNPUBLISHED WORK OF THE ARCHITECT AND MAY NOT BE DUPLICATED OR USED WITHOUT THE WRITTEN CONSENT OF THE ARCHITECT.

Scott
Edwards
Architecture

2525 E Burnside St.
Portland, OR 97214
503.226.3617
seallp.com

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WEST LINN PUBLIC WORKS FACILITY

Job Number: **23028**

WEST LINN, OR

CITY OF West Linn

100% SCHEMATIC DESIGN SET **02.14.2025**
ISSUE **DATE**

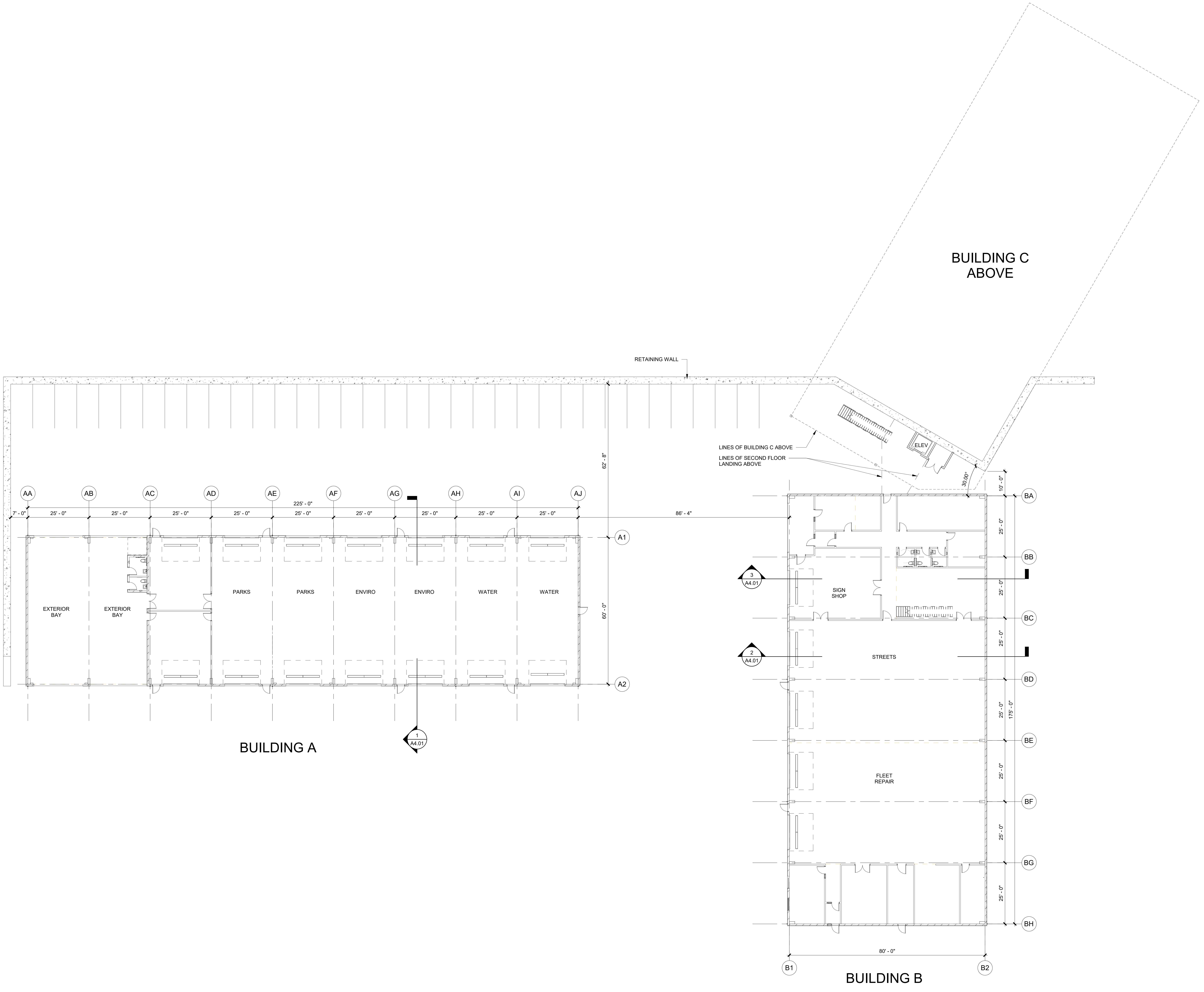
Drawing:

SITE PLAN

Sheet No:

A1.01

FILE PATH: Autodesk Docs://23028_West Linn Public Works/23028_West Linn Public Works_24.rvt



1 OVERALL FLOOR PLANS

1/16" = 1'-0"

GENERAL SHEET NOTES

A. PROVIDE FIRE EXTINGUISHERS AND FIRE EXTINGUISHER CABINETS AS SHOWN / SPECIFIED. COORDINATE FINAL LOCATIONS WITH FIRE MARSHALL.
B. ALL INTERIOR WALLS TO BE TYPE X-X, UNLESS NOTED OTHERWISE.

KEYNOTES (07-02)

Scott
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seallp.com

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CONSTRUCTION

**WEST LINN
PUBLIC WORKS
FACILITY**

Job Number: 23028

WEST LINN, OR

City of West Linn

100% SCHEMATIC DESIGN SET	02.14.2025
ISSUE	DATE

Drawing:

OVERALL FLOOR PLANS

NORTH

Sheet No:

A2.10

FILE PATH: Autodesk Docs://23028_West Linn Public Works/23028_West Linn Public Works_24.rvt

GENERAL SHEET NOTES

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KEYNOTES (07-02)

Scott
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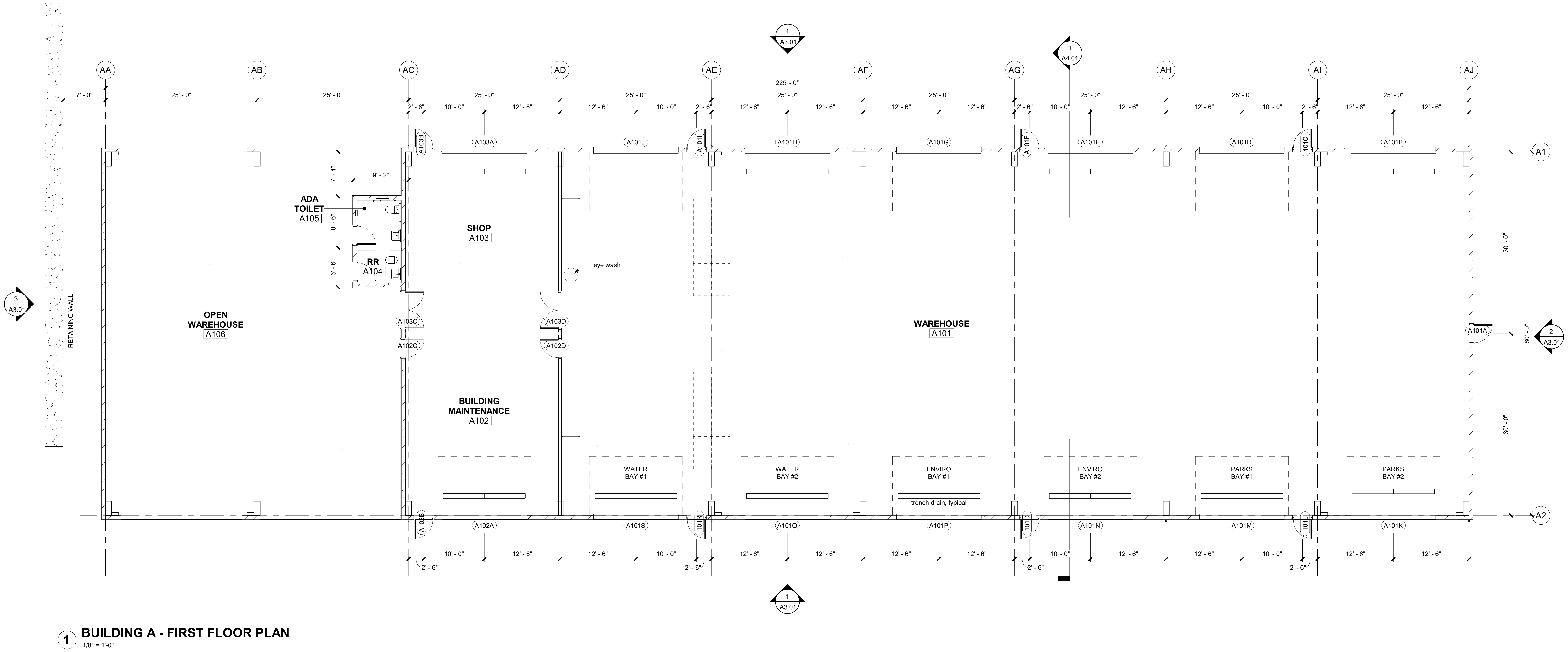
Job Number: 23028

WEST LINN, OR

City of West Linn



1 BUILDING A - FIRST FLOOR PLAN

1/8" = 1'-0"

ISSUE	DATE
100% SCHEMATIC DESIGN SET	02.14.2025

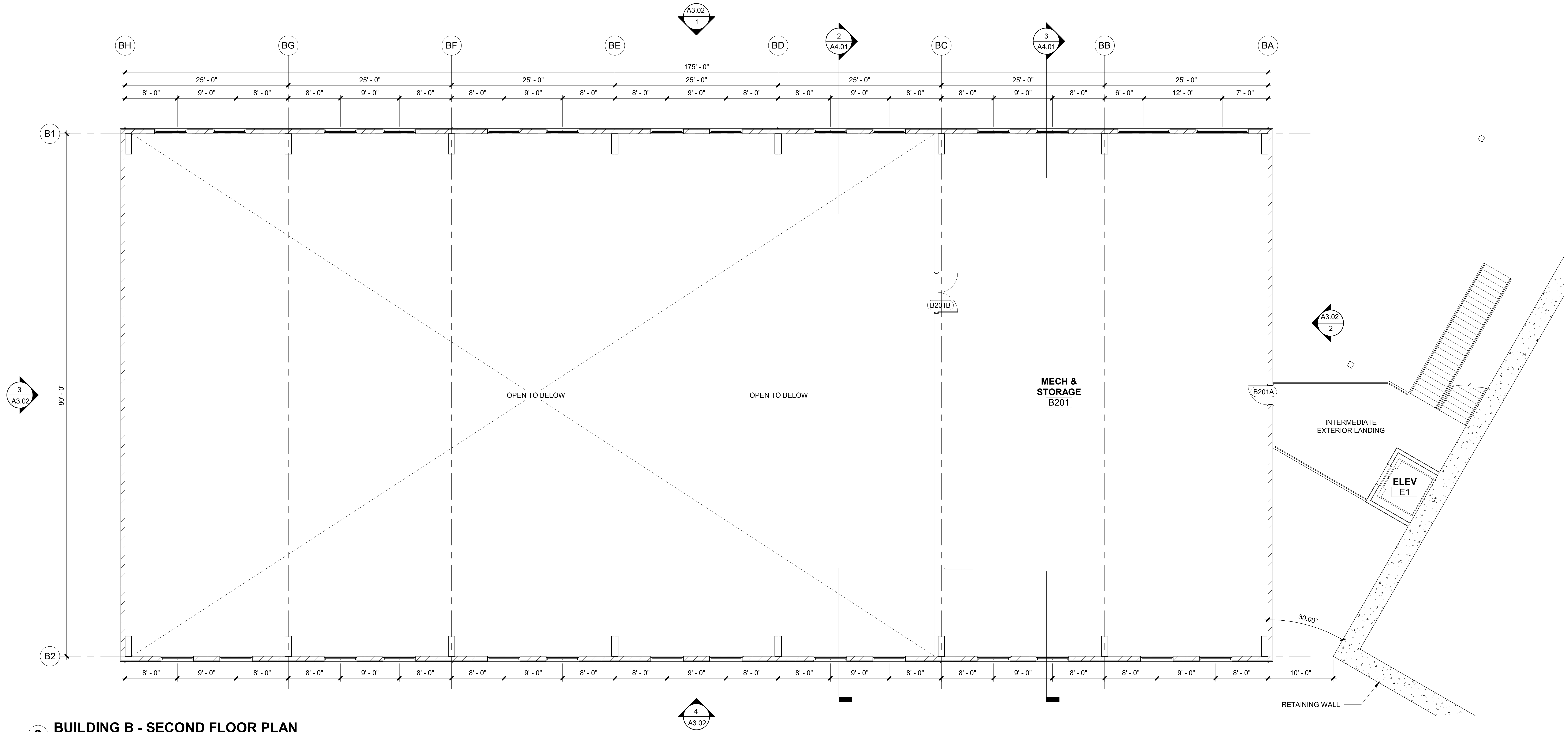
Drawing:

BUILDING A - FIRST FLOOR PLAN

Sheet No:

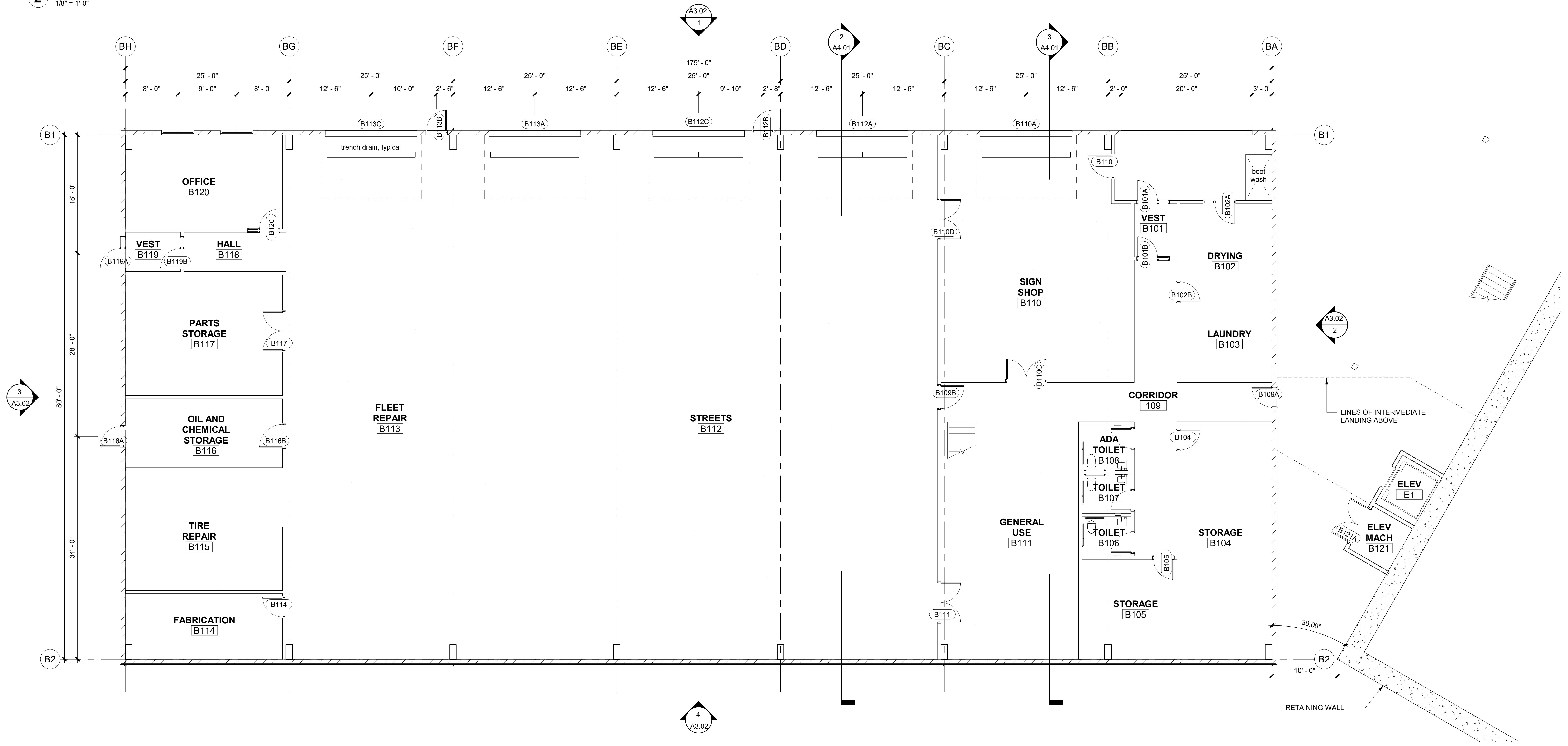
A2.11

FILE PATH: Autodesk Docs://23028_West Linn Public Works/23028_West Linn Public Works_24.rvt



2 BUILDING B - SECOND FLOOR PLAN

1/8" = 1'-0"



1 BUILDING B - FIRST FLOOR PLAN

1/8" = 1'-0"

GENERAL SHEET NOTES

A. PROVIDE FIRE EXTINGUISHERS AND FIRE EXTINGUISHER CABINETS AS SHOWN / SPECIFIED. COORDINATE FINAL LOCATIONS WITH FIRE MARSHALL.
B. ALL INTERIOR WALLS TO BE TYPE X-X, UNLESS NOTED OTHERWISE.

KEYNOTES (07-02)

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WEST LINN PUBLIC WORKS FACILITY
Job Number: 23028
WEST LINN, OR

City of West Linn

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Drawing:

BUILDING B - FIRST & SECOND FLOOR PLANS

Sheet No:

A2.12

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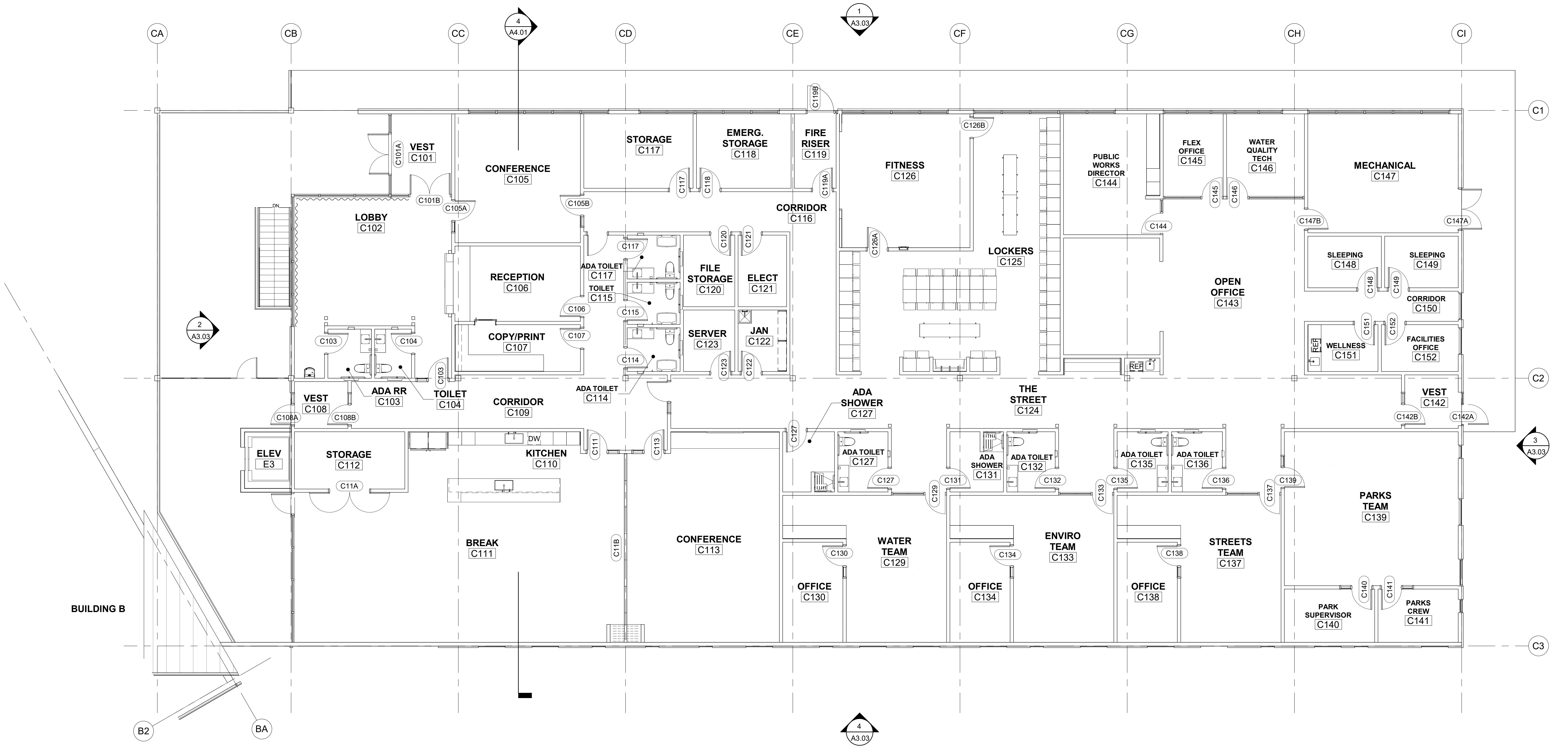

1 BUILDING C - FIRST FLOOR PLAN

1/8" = 1'-0"

GENERAL SHEET NOTES

A. PROVIDE FIRE EXTINGUISHERS AND FIRE EXTINGUISHER CABINETS AS SHOWN / SPECIFIED. COORDINATE FINAL LOCATIONS WITH FIRE MARSHALL.
B. ALL INTERIOR WALLS TO BE TYPE X-X, UNLESS NOTED OTHERWISE.

KEYNOTES 07-02

Scott
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Architecture

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Job Number: 23028

WEST LINN, OR

City of West Linn

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ISSUE	DATE

Drawing:

BUILDING C - FIRST FLOOR PLAN

NORTH

Sheet No:

A2.13

APPENDIX I

APPENDIX I
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Report Purpose, Use, and Standard of Care

This report has been prepared in accordance with standard fundamental principles and practices of geotechnical engineering and/or environmental consulting, and in a manner consistent with the level of care and skill typical of currently practicing local engineers and consultants. This report has been prepared to meet the specific needs of specific individuals for the indicated site. It may not be adequate for use by other consultants, contractors, or engineers, or if change in project ownership has occurred. It should not be used for any other reason than its stated purpose without prior consultation with Columbia West Engineering, Inc. (Columbia West). It is a unique report and not applicable for any other site or project. If site conditions are altered, or if modifications to the project description or proposed plans are made after the date of this report, it may not be valid. Columbia West cannot accept responsibility for use of this report by other individuals for unauthorized purposes, or if problems occur resulting from changes in site conditions for which Columbia West was not aware or informed.

Report Conclusions and Preliminary Nature

This geotechnical or environmental report should be considered preliminary and summary in nature. The recommendations contained herein have been established by engineering interpretations of subsurface soils based upon conditions observed during site exploration. The exploration and associated laboratory analysis of collected representative samples identifies soil conditions at specific discreet locations. It is assumed that these conditions are indicative of actual conditions throughout the subject property. However, soil conditions may differ between tested locations at different seasonal times of the year, either by natural causes or human activity. Distinction between soil types may be more abrupt or gradual than indicated on the soil logs. This report is not intended to stand alone without understanding of concomitant instructions, correspondence, communication, or potential supplemental reports that may have been provided to the client.

Because this report is based upon observations obtained at the time of exploration, its adequacy may be compromised with time. This is particularly relevant in the case of natural disasters, earthquakes, floods, or other significant events. Report conclusions or interpretations may also be subject to revision if significant development or other manmade impacts occur within or in proximity to the subject property. Groundwater conditions, if presented in this report, reflect observed conditions at the time of investigation. These conditions may change annually, seasonally or as a result of adjacent development.

Additional Investigation and Construction Observation

Columbia West should be consulted prior to construction to assess whether additional investigation above and beyond that presented in this report is necessary. Even slight variations in soil or site conditions may produce impacts to the performance of structural facilities if not adequately addressed. This underscores the importance of diligent construction observation and testing to verify soil conditions do not differ materially or significantly from the interpreted conditions utilized for preparation of this report.

Therefore, this report contains several recommendations for field observation and testing by Columbia West personnel during construction activities. Actual subsurface conditions are more readily observed and discerned during the earthwork phase of construction when soils are exposed. Columbia West cannot accept responsibility for deviations from recommendations described in this report or future performance of structural facilities if another consultant is retained during the construction phase or Columbia West is not engaged to provide construction observation to the full extent recommended.

Collected Samples

Uncontaminated samples of soil or rock collected in connection with this report will be retained for thirty days. Retention of such samples beyond thirty days will occur only at client's request and in return for payment of storage charges incurred. All contaminated or environmentally impacted materials or samples are the sole property of the client. Client maintains responsibility for proper disposal.

Report Contents

This geotechnical or environmental report should not be copied or duplicated unless in full, and even then only under prior written consent by Columbia West, as indicated in further detail in the following text section entitled Report Ownership. The recommendations, interpretations, and suggestions presented in this report are only understandable in context of reference to the whole report. Under no circumstances should the soil boring or test pit excavation logs, monitor well logs, or laboratory analytical reports be separated from the remainder of the report. The logs or reports should not be redrawn or summarized by other entities for inclusion in architectural or civil drawings, or other relevant applications.

Report Limitations for Contractors

Geotechnical or environmental reports, unless otherwise specifically noted, are not prepared for the purpose of developing cost estimates or bids by contractors. The extent of exploration or investigation conducted as part of this report is usually less than that necessary for contractor's needs. Contractors should be advised of these report limitations, particularly as they relate to development of cost estimates. Contractors may gain valuable information from this report, but should rely upon their own interpretations as to how subsurface conditions may affect cost, feasibility, accessibility and other components of the project work. If believed necessary or relevant, contractors should conduct additional exploratory investigation to obtain satisfactory data for the purposes of developing adequate cost estimates. Clients or developers cannot insulate themselves from attendant liability by disclaiming accuracy for subsurface ground conditions without advising contractors appropriately and providing the best information possible to limit potential for cost overruns, construction problems, or misunderstandings.

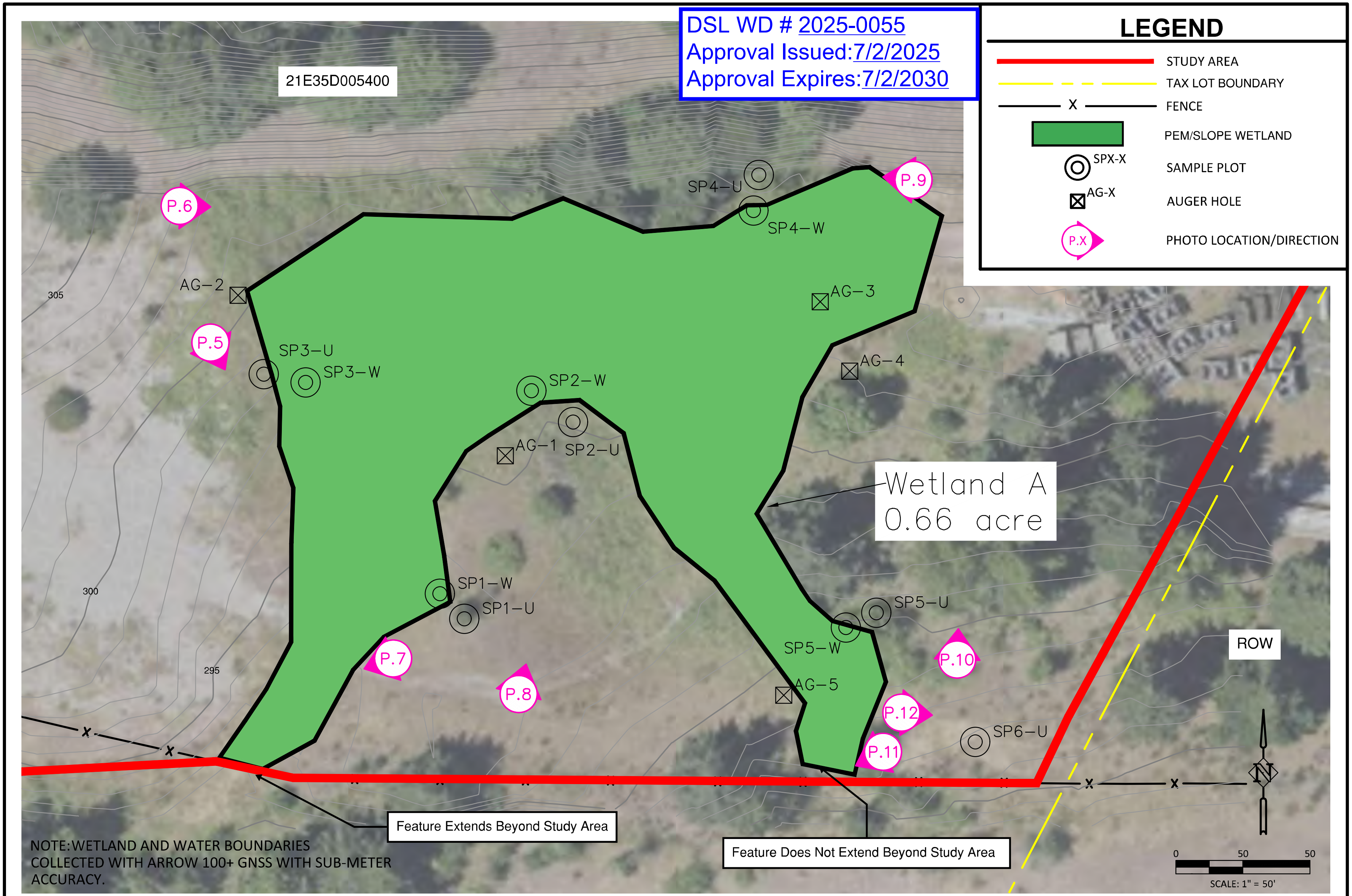
Report Ownership

Columbia West retains the ownership and copyright property rights to this entire report and its contents, which may include, but may not be limited to, figures, text, logs, electronic media, drawings, laboratory reports, and appendices. This report was prepared solely for the client, and other relevant approved users or parties, and its distribution must be contingent upon prior express written consent by Columbia West. Furthermore, client or approved users may not use, lend, sell, copy, or distribute this document without express written consent by Columbia West.

Client does not own nor have rights to electronic media files that constitute this report, and under no circumstances should said electronic files be distributed or copied. Electronic media is susceptible to unauthorized manipulation or modification, and may not be reliable.

Consultant Responsibility

Geotechnical and environmental engineering and consulting is much less exact than other scientific or engineering disciplines, and relies heavily upon experience, judgment, interpretation, and opinion often based upon media (soils) that are variable, anisotropic, and non-homogenous. This often results in unrealistic expectations, unwarranted claims, and uninformed disputes against a geotechnical or environmental consultant. To reduce potential for these problems and assist relevant parties in better understanding of risk, liability, and responsibility, geotechnical and environmental reports often provide definitive statements or clauses defining and outlining consultant responsibility. The client is encouraged to read these statements carefully and request additional information from Columbia West if necessary.



DSL WD # 2025-0055
Approval Issued:7/2/2025
Approval Expires:7/2/2030

NOTE:WETLAND AND WATER BOUNDARIES
COLLECTED WITH ARROW 100+ GNSS WITH SUB-METER
ACCURACY.

Note: GNSS located using Eos Arrow 100 Plus to sub-meter accuracy by HHPR

SHEET NO.
2
JOB NO.
CWL-11

DATE	NO.	DESCRIPTION

REVISIONS

DESIGNED: HHPR
DRAWN: HHPR
CHECKED: HHPR
DATE: 1/24/25

**Harper
Houf Peterson
Righellis Inc.**
ENGINEERS ◆ PLANNERS
LANDSCAPE ARCHITECTS ◆ SURVEYORS
205 SE Spokane Street, Suite 200, Portland, OR 97202
phone: 503.221.1131 www.hhpr.com fax: 503.221.1171

Figure 6: Wetland Delineation Exhibit

West Linn Public Works Facility

West Linn, Oregon