Geotechnical Investigation Happy Valley Community Recreation Center: Building and Park

Happy Valley, Oregon

October 6, 2025

Prepared for

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1 INTRODUCTION

As requested, GRI completed a geotechnical investigation for the proposed City of Happy Valley (City) Community Recreation Center project, which is east of the intersection of SE 172nd Avenue and SE Scouters Mountain Road in Happy Valley, Oregon. The Vicinity Map, Figure 1, shows the general location of the site. The purpose of this investigation was to evaluate subsurface conditions at the site and develop geotechnical recommendations for use in the design and construction of the proposed improvements. The investigation included a review of existing geotechnical information for the site and surrounding area, subsurface explorations, laboratory testing, and engineering analyses.

The overall project includes the construction of a new community recreation center building, a community park, new public roads, and a new vehicle crossing over Rock Creek. The overall project has been divided into two separate projects for the purposes of design and construction. This geotechnical report describes the work we accomplished during our geotechnical investigation and provides our conclusions and recommendations for use in the design and construction of the community recreation center building and community park project. GRI prepared a separate geotechnical report that provides our conclusions and recommendations for the new public roads and vehicle crossing project.

2 PROJECT DESCRIPTION

This geotechnical report provides our recommendations for the proposed community recreation center building and community park project. A separate geotechnical report provides our recommendations for the proposed new roads and Rock Creek vehicle crossing project. Because the two project areas are adjacent to each other, and because the geotechnical data we obtained for the two project areas are applicable to both projects, the geotechnical investigation data and laboratory test results we obtained for both projects are presented in Appendix A. Elevations used in this report are based on the North American Vertical Datum of 1988.

We understand that the overall project will include the construction of the improvements described in the following sections.

2.1 Community Recreation Center Building

The two-story building will initially be approximately 65,000 square feet in size but may be increased to approximately 83,000 square feet by future expansions. The building is proposed to include an aquatics center and pool, a multi-purpose gymnasium, an indoor walking and jogging track, a large community room and kitchen, reservable gathering spaces, weight and cardio spaces, a group exercise room, and childcare services. The building will primarily feature steel-frame construction, along with concrete tilt up panels in some areas. Some of the building walls will act as retaining walls that retain up to approximately 13 feet of soil. The building will generally be approximately at grade, although there may be some deeper areas used for elevator pits, mechanical pits, and the swimming pool. The overall building is anticipated to be constructed in separate phases. The project structural engineer has informed us that the building will likely be designed for Risk Category III and in accordance with the upcoming 2025 *Oregon Structural Specialty Code* (OSSC), which references the 2022 American Society of Civil Engineers 7-22 document titled *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. Maximum building column loads will be approximately 225 kips. The likely location of the new building is shown on the Site Plan, Figure 2.

2.2 Community Park

The property around the community recreation center building will be developed as a park that includes athletic fields, walking paths, additional recreational amenities, and asphalt concrete (AC) parking lots. Some amenities may include small structures such as concession stands or covered picnic areas. Some amenities may include hardscape surfaces such as walking paths, tennis courts, basketball courts, a splash pad, or a plaza. Light poles will likely be constructed throughout the site to illuminate the athletic fields and parking lots. The current layout of the park and the amenities shown on Figure 2 are preliminary and may be later modified as plans for the park continue to be developed.

2.3 Paved Asphalt Concrete Roads

New public AC roads will be constructed to provide access to the site from SE 172nd Avenue. These roads will include extensions of SE Scouters Mountain Road, a new SE 177th Avenue segment, and a new roundabout. Light poles will be constructed to illuminate the roads. The new roads that will be part of this project are shown on Figure 2. The grading plan along SE Scouters Mountain Road shows cuts will be up to approximately 2 feet and fills will be up to approximately 25 feet. The grading plan along SE 177th Avenue shows cuts and fills will generally be less than approximately 5 feet. Additional road improvements may be performed in the future, such as connecting the project site to SE Foster Road or constructing new onsite local roads.

2.4 A Vehicle Crossing over Rock Creek

A crossing is proposed over Rock Creek to provide access to the community center from SE 172nd Avenue. We understand that the preferred option for the crossing is to construct a new culvert. The culvert is anticipated to consist of an arch culvert that is approximately 43 feet wide, 20 feet high, and 90 feet long. The culvert will be supported on shallow foundations or deep foundations.

3 SITE DESCRIPTION

3.1 General

The community recreation center building and community park project site consists of an approximately 37-acre former agricultural field that measures approximately 1,280 feet by

1,250 feet. The project site is bordered by an agricultural field and private residence to the north, a private residence and shop buildings to the east, private residences to the south, and private residences and the City right-of-way to the west. The surrounding residential properties generally consist of multiple-acre parcels that include outbuildings, open fields, and forested areas. The project site was covered with grass vegetation at the time of our explorations. There are no structures or pavement at the site. Based on historical aerial photography, the site appears relatively unchanged since at least 1994. Records indicate that the field was used primarily for agricultural purposes since at least 1937.

According to the U.S. Geological Survey (USGS, 2020) topographic map of the Damascus Quadrangle, Oregon, the community recreation center and community park project site elevations range from approximately 340 feet on the western edge of the property to approximately 380 feet on the east side of the property. The project site slopes gently down to the west toward Rock Creek, the nearest surface water body, which is approximately 600 feet west of the project site. The elevation of Rock Creek is approximately 312 feet.

3.2 Geology

Published geologic mapping and our results from field explorations indicate the project site is mantled with Pleistocene fine-grained facies of catastrophic flood deposits (Madin, 1994; Wells et al., 2020). These deposits include stratified clay, silt, sand, and smaller amounts of gravel that are together classified as Willamette Silt. Mapped nearby is the Pliocene to Pleistocene Basalt of Boring Lava and Springwater Formation. The Boring Lava originates from a series of local vents and is separated into several different chemically distinct basalt flows, typically gray basalt and basaltic andesite flows and associated scoria (Madin, 1994). West of the site is mapped as Basalt of Mount Scott and east is Basalt of Winston Road and Basalt of Borges Road. Cross-sections show the Boring basalts interfingering with the slightly older Springwater Formation. The Springwater Formation is mapped as a fluvial conglomerate, volcaniclastic sandstone, siltstone, and debris flows derived from the Cascade Range (Madin, 1994).

3.3 Faults and Seismicity

A discussion of the faults and seismicity in the vicinity of the project site is provided in the Site-Specific Seismic Hazard Evaluation in Appendix E.

4 SUBSURFACE CONDITIONS

4.1 General

Subsurface materials and conditions at the overall project site were investigated between April 7 and 21, 2025, by drilling 23 borings, advancing two cone penetration test (CPT) probes, advancing two flat dilatometer test (DMT) probes, performing 10 Kessler Dynamic Cone Penetration (DCP) tests, performing two geophysical test profiles, and performing a pebble count in Rock Creek. The borings were designated B-1 through B-23 and were advanced to depths between 6.5 feet and 71.5 feet below existing site grades and were completed using hollow-stem auger drilling, mud-rotary drilling, and HQ rock coring methods. The CPTs were designated CPT-1 and CPT-2 and were advanced to depths of 19.7 feet and 25.9 feet below the existing site grades using a track-mounted Geoprobe CPT rig with 20 tons of push force. The DMTs were designated DMT-1 and DMT-2 and were advanced to depths of 14.4 feet and 18.4 feet below the existing site grades using a Geoprobe CPT rig with 20 tons of push force to advance the DMT equipment. The DCP tests, designated DCP-1 through DCP-10, were completed with a Kessler Dynamic Cone Penetrometer. The geophysical profiles consisted of two refraction microtremor (ReMi) arrays that were designated ReMi Array 1 and ReMi Array 2 and had lengths of 345 feet and geophone spacing of 15 feet. The pebble count was performed in Rock Creek, approximately 60 feet north of the existing Rock Creek culvert. The approximate locations of the explorations are shown on Figure 2.

Logs of the GRI borings are provided on Figures 1A through 23A in Appendix A. Photographs of the rock core samples collected during drilling are provided on Figures 24A through 26A. Logs of the DMT soundings are provided on Figures 27A and 28A. DCP test results are provided on Figures 29A through 38A. The GRI laboratory program conducted to evaluate the physical engineering properties of the materials encountered in the explorations is described in Appendix A and the results are provided on Figures 39A through 47A. Subgrade resilient modulus values approximated from the DCP tests are provided in Table 1A. The terms and symbols used to describe the materials encountered in the explorations are defined in Tables 2A through 4A and the attached legend.

Results from the CPT testing performed by Oregon Geotechnical Explorations are provided in Appendix B. The geophysical report prepared by Earth Dynamics, LLC is provided in Appendix C. Figures, logs, and laboratory test results from a previous geotechnical exploration program performed at the site by others are provided in Appendix D.

4.2 Soil Sampling

Disturbed and undisturbed soil samples were generally obtained from the borings at 2.5-foot intervals of depth in the upper 15 feet and at 5-foot intervals below 15 feet. Disturbed soil samples were generally obtained using a 2-inch outside-diameter standard split-spoon sampler, although a 3-inch outside-diameter standard split-spoon sampler was occasionally used to obtain additional sample material. Standard Penetration Tests (SPTs) were conducted by driving the sampler into the soil a distance of 18 inches using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is known as the Standard Penetration Resistance (SPT N-value). SPT N-values provide a measure of the relative density of granular soil and the relative consistency of cohesive soil.

Relatively undisturbed soil samples were collected by pushing a 3-inch outside-diameter Shelby tube into the undisturbed soil a maximum of 24 inches using the hydraulic ram of the drill rig. The soils in the Shelby tubes were extruded in our laboratory, and field vane or Torvane shear-strength measurements were recorded on selected samples. We also collected several disturbed bulk samples of the auger cuttings.

4.3 Cone Penetration Tests

CPT testing was performed at the Rock Creek crossing location and at the building location. Soil shear wave velocity readings were collected during our investigation. Additional details of our CPT program and logs of the data collected are presented in Appendix B.

4.4 Dilatometer Tests

DMTs were performed at the Rock Creek crossing location and at the building location. Additional details of our DMTs are presented in Appendix A. The logs of the test data and the test results are provided on Figures 27A and 28A in Appendix A.

4.5 Dynamic Cone Penetration Testing

Kessler DCP testing was performed to approximate subgrade resilient modulus at 10 locations along new roads and parking lots. Additional details of our DCP testing are presented in Appendix A. Logs of the test data and the test results are provided on Figures 29A through 38A in Appendix A.

4.6 Geophysical Testing

Geophysical testing was performed at the Rock Creek crossing location and at the building location. The geophysical testing consisted of collecting data from two ReMi arrays that had lengths of 345 feet and geophone spacing of 15 feet. The purpose of the geophysical testing was to provide an estimate of shear wave velocities at the site, which will be used to evaluate seismic conditions at the site. The geophysical report that was prepared for this project is provided in Appendix C.

4.7 Pebble Count

We performed a pebble count in Rock Creek, approximately 60 feet north of the existing Rock Creek culvert. However, the stream bed material consisted of silty sand with only trace gravel. Because measurable rocks were relatively absent from the stream bed, the pebble count could not be completed according to the standard method. To obtain gradation data, we collected a grab sample of the steam bed material and returned it to our laboratory for further evaluation and gradation testing. Additional details of our pebble count are presented in Appendix A. The laboratory gradation test results are provided on Figure 41A in Appendix A.

4.8 Soils

For the purpose of discussion, the subsurface soils disclosed by our investigation have been grouped into the following categories based on their physical characteristics and engineering properties. They are listed as they were encountered below the ground surface:

a. SILT and CLAY (Willamette Silt)
b. SILT and Silty SAND (Springwater Formation)
c. BASALT (Boring Lava)

The following paragraphs provide a description of the soil layers encountered in the explorations completed by GRI for this project. The soil conditions we observed at the site are consistent with what geologic mapping of the area shows.

a. SILT and CLAY (Willamette Silt)

In all the borings drilled at the building and park project site, we encountered a layer of silt and clay at the ground surface that extended to depths between 6.5 feet and 20 feet. We interpret this silt and clay layer to be part of the Willamette Silt geologic unit. This unit ranges from silt to clay, with lesser amounts of sand and, in a few places, trace gravel. The soil in this unit is typically brown, gray, and orange in color, moist to wet, has medium to high plasticity, and contains trace to some fine- to coarse-grained sand. The relative consistency of the unit is generally medium stiff to very stiff based on SPT N-values. The upper 6 inches to 18 inches generally consists of topsoil that contains roots.

Natural moisture contents, Atterberg-limits indices, fines contents, consolidation results, and other laboratory testing data for the Willamette Silt layer are provided in Appendix A. Soil such as this generally exhibits low to moderate strength and low to moderate compressibility.

b. SILT and Silty SAND (Springwater Formation)

In the deeper borings drilled at the building and park project site (B-16, B-17, B-18, and B-19), we encountered the Springwater Formation at depths of 11 feet, 20 feet, 14.5 feet, and 18.5 feet, respectively. These borings were drilled to depths of 50.8 feet to 71.5 feet, and the Springwater Formation extended deeper than the bottoms of these holes. In borings B-11, B-12, and B-21, we encountered possible Springwater Formation soils at depths of 7.5 feet, 12.5 feet, and 13.5 feet, respectively.

The Springwater Formation varies in soil type and at this site includes clayey silt, silt with trace to some sand, sandy silt, and silty sand with varying amounts of mostly subangular to subrounded gravel. The Springwater Formation is brown, gray, and red/yellow-brown and sometimes has a more blocky and older-looking structure than the overlying Willamette Silt. Another distinguishing characteristic of the Springwater Formation is that it has more sand and gravel and is generally less plastic than the Willamette Silt. The plasticity of the unit ranges from low to high plasticity, but is mostly in the low to medium range. The Springwater Formation samples we observed were also generally moist to wet and had trace to some fine- to coarse-grained sand and gravel. The relative consistency of the unit is generally very stiff to very hard for silt and dense to very dense for sand, based on SPT N-values.

Natural moisture contents, fines contents, and other laboratory testing data for the Springwater Formation are provided in Appendix A. Soil such as this generally exhibits moderate strength and low compressibility.

c. BASALT (Boring Lava)

In boring B-3 drilled near Rock Creek, we encountered basalt of the Boring Lava unit that was present beneath the Willamette Silt unit. The Boring Lava basalt layer was not encountered in borings drilled at the building and park site, but it could still be present at this location due to the irregular nature of the contact with this unit. In boring B-3, decomposed basalt was encountered at a depth of 15 feet that had generally decomposed to a sandy SILT with trace gravel and had a relative consistency that was very stiff. Below 20 feet, the unit became less decomposed and consists of silty gravel that is dark gray and teal, moist, subangular to angular, contains fine- to coarse-grained sand, contains nonplastic fines, and has a relative density that is very dense based on SPT N-values. Rock coring started at 50 feet and yielded intact basalt that is dark gray, slightly weathered, medium hard (R3), and contains some vesicles, joints, and fractures. It is likely that the transition from gravel-sized basalt fragments to intact basalt occurred at a depth somewhere between 30 and 50 feet. Boring B-3 was terminated in the basalt layer at a depth of 67 feet.

Natural moisture content laboratory testing data for the Boring Lava Basalt layer are provided in Appendix A. Soil and rock such as this generally exhibits high strength and very low compressibility.

4.9 Groundwater

We measured groundwater levels in the borings listed below after leaving the holes open overnight. The depth to groundwater below the ground surface that we measured in the borings are summarized below in Table 4-1.

Table 4-1: SUMMARY OF GROUNDWATER DEPTH OBSERVED IN BORINGS

Location	Depth of Groundwater, feet
B-1	2.4
B-2	2.3
B-5	1.3
B-6	8.1
B-7	11.6
B-8	>12.5 (dry hole)
B-9	0.5
B-11	4.1
B-12	1.4
B-13	1.5
B-14	4.2
B-15	0.8
B-16	7.7
B-17	4.8
B-18	1.6
B-19	11.8
B-21	1.8
B-22	9.0
B-23	3.3

In addition to the groundwater depths listed above, while drilling boring B-17 we noted that the drilling mud became watered-down below a depth of 40 feet, which typically indicates flowing groundwater. In October of 2024, Shannon & Wilson drilled a boring and installed a piezometer in the northwest corner of the field, where they measured groundwater at a depth of 10 feet. We reviewed published depth to groundwater mapping, which indicates groundwater is generally at a depth of 30 feet to 40 feet below the ground surface throughout the project site, but decreases to less than 10 feet below the ground surface at Rock Creek (Snyder, 2008). We also reviewed well logs filed with the Oregon Water Resources Department but did not find records of any previous wells drilled at the project site.

Based on the information described above, it is our opinion that shallow perched groundwater is present throughout the project site within the upper 10 feet of soil, with the regional groundwater level likely at a depth of approximately 30 feet to 40 feet below the ground surface. The depth to groundwater will fluctuate in response to seasonal changes, prolonged rainfall, changes in surface topography, irrigation, and other factors not observed in this study. Perched water could develop near the ground surface, especially following periods of wet weather and/or heavy rain.

4.10 Infiltration Testing

Infiltration testing was performed in general accordance with the infiltration testing requirements for encased falling head tests that are provided in the Clackamas County Water Environment Services Stormwater Standards manual. The infiltration testing was performed in borings B-2, B-4, B-10, and B-20 at a depth of 5 feet, which is the approximate location and depth where stormwater infiltration is being considered by the design team. The infiltration testing was performed inside 6-inch-diameter augers that were drilled into the ground. We placed approximately one foot of water inside the augers and allowed the material to soak for at least four hours. We then continued to collect water readings for multiple hours and even allowed the tests to run overnight. No water infiltration occurred in any of the tests. In boring B-2, the water level inside the augers actually increased slightly overnight, which indicates the tests were performed below the perched groundwater level.

Based on the results of our infiltration testing, it appears that on-site stormwater infiltration is not feasible at the project site due to the relatively impermeable soil and the shallow perched groundwater levels that are frequently near the ground surface. We understand that stormwater will likely be conveyed to a City stormwater system and disposed of off site.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Subsurface explorations completed for this investigation indicate the building and park site is generally mantled with an upper layer of silt and clay that we classified as Willamette Silt, which overlies the Springwater Formation. Shallow perched groundwater is also present throughout the site. Hazard mapping shows that an earthquake fault is present in the northeast portion of the site.

The primary geotechnical considerations associated with design and construction of the proposed improvements include building and retaining wall foundation support; retaining wall design; construction and permanent dewatering; waterproofing buried portions of the building; placement and compaction of on-site soil; not constructing the building in the northeast portion of the site where there is a mapped earthquake fault, and sequencing the work so that later building expansions do not undermine existing foundations or cause existing foundations to excessively settle. The following sections of this report provide our conclusions and recommendations for use in the design and construction of the project.

5.2 Seismic Considerations and Geologic Hazards

5.2.1 Design Acceleration Parameters

We understand that seismic design for the project is being completed in accordance with the 2025 OSSC and American Society of Civil Engineers (ASCE) 7-22. A site-specific seismic hazard study was completed for the project to fulfill the requirements of amended Section 1803 of the 2025 OSSC for special occupancy structures. Details of the site-specific seismic hazard study and development of the recommended response spectrum are provided in Appendix E.

A ground-motion hazard analysis was completed in accordance with Section 21.2 of ASCE 7-22 to develop the site-specific ground motion values. Based on the shear wave data obtained from a seismic CPT probe and a shear wave ReMi test completed at the site, it is our opinion the site can generally be classified as Site Class C in accordance with Chapter 20 of ASCE 7-22. The average shear wave velocity in the upper 100 feet was estimated to be approximately 1,760 feet per second (ft/s), which represents a Site Class C condition. The recommended response spectra for structural design were developed by comparing the site-specific spectra based on ground motion hazard analysis with the code-based spectra based on Site Class C conditions. Our recommended MCE_R and design response spectral values for design of the project are summarized in Table 5-1. The table presents multi-period and two-period spectral values. The two-period spectral values are derived in accordance with the guidelines provided in Section 21.4 of ASCE 7-22. In accordance with Section 21.4, the 0.2-second MCE_R spectral value can be taken as 90% of the maximum spectral acceleration obtained from the site-specific response spectrum at any period within the range of 0.2 seconds to 5.0 seconds. The 1.0-second MCE_R spectral value can be derived based on 90% of the maximum value of the product of spectral accelerations and corresponding periods for periods ranging from 1.0 seconds to 5.0 seconds for sites with a V_{S30} value less than 1,450 ft/s but not less than 100% of the spectral value at 1 second.

Table 5-1: RECOMMENDED MCE_R AND DESIGN RESPONSE SPECTRAL VALUES, 5% DAMPING

	Recommended Multi-Period Spectral Values	
Period, seconds	MCE_R-Level Response Spectral Values, g	Design-Level Response Spectral Values, g
PGA	0.44	0.30
0.05	0.56	0.38
0.1	0.87	0.58
0.2	1.06	0.71
0.3	0.97	0.64
0.4	0.83	0.55

Period, seconds	Recommended Multi-Period Spectral Values	
	MCE$_R$-Level Response Spectral Values, g	Design-Level Response Spectral Values, g
0.5	0.72	0.48
0.75	0.55	0.37
1	0.44	0.29
1.5	0.31	0.21
2	0.24	0.16
3	0.15	0.10
4	0.11	0.07
5	0.09	0.06
Parameter	**Recommended Two-Period Spectral Values**	
0.2 seconds	0.96	0.64
1 second	0.44	0.29

Abbreviations: MCE$_R$ = Risk-Targeted Maximum Considered Earthquake; PGA = peak ground acceleration

5.2.2 *Liquefaction and Cyclic Softening Hazard*

Liquefaction is a process by which loose, saturated, granular materials such as clean sand and, to a somewhat lesser degree, nonplastic and low-plasticity silts temporarily lose stiffness and strength during and immediately after a seismic event. This degradation in soil properties may be substantial and abrupt, particularly in loose sands. Liquefaction occurs as seismic shear stresses propagate through a saturated soil and distort the soil structure, causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure causes the pore-water pressure to increase between the soil grains. If the pore-water pressure becomes sufficiently large, the intergranular stresses become small and the granular layer temporarily behaves as a viscous liquid rather than a solid. After liquefaction is triggered, there is an increased risk of settlement, loss of bearing capacity, lateral spreading, and/or slope instability, particularly along waterfront areas. Liquefaction-induced settlement occurs as the elevated pore-water pressures dissipate and the soil consolidates after the earthquake.

The cyclic behavior of fine-grained material is generally different from that of granular material; therefore, the term "cyclic softening" is used to differentiate the behavior of fine-grained materials from liquefaction. Cyclic softening describes a relatively gradual and progressive increase in shear strain with seismic load cycles. Excess pore-water pressures may increase due to cyclic loading but will generally not approach total overburden stress. Shear strains accumulate with additional loading cycles; however, an abrupt or sudden decrease in shear stiffness is not typically observed. Settlement due to post-seismic consolidation can occur, particularly in lower-plasticity silts; however, settlement does not generally occur to the same degree as in sandy soils. Large shear strains can develop, and strength loss related to soil sensitivity may occur in some fine-grained soils.

The potential for liquefaction and cyclic softening in the project area was evaluated using two different methods. The first method was to evaluate the CPT-2 data using the software program CLiq, developed by GeoLogismiki of Neo Souli, Greece. The second method was to evaluate the CPT-2 data and the SPT data from borings B-16, B-17, B-18, and B-19 using the methods recommended by Idriss and Boulanger (2008), with subsequent revisions (Boulanger and Idriss, 2014). The USGS Unified Hazard Tool was used to determine the contributing earthquake magnitudes that represent the seismic exposure of the site (USGS, 2025). A crustal event on the Portland Hills fault and an event on the Cascadia Subduction Zone (CSZ) were determined to represent the maximum sources of seismic shaking. For our analysis, we considered a moment magnitude (M_W) 7.0 crustal earthquake, a M_W 9.0 CSZ earthquake, and a peak ground acceleration value of 0.43 g that we obtained from ASCE hazard tool (ASCE, 2025). We modeled a groundwater depth of about 3 feet below the ground surface, which corresponds to the average perched groundwater level at the site. The results of our analyses indicate that the risk of liquefaction at the project site is very low or absent due to the relatively dense/stiff soil conditions and the relatively high plasticity of the near-surface fine-grained soil.

5.2.3 *Lateral Spreading Hazard*

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. Because it is our opinion that the soil at the site is not susceptible to liquefaction, it is also our opinion that lateral spreading is not a hazard at this site.

5.2.4 *Fault Rupture*

The project site is mapped as being within the Damascus-Tickle Creek Fault zone (USGS Earthquake Hazards Program, 2025). One fault strand is mapped as crossing the northeast portion of the site where a parking lot and athletic fields will likely be located. Seismic mapping indicates that the location of this fault is inferred and is not well defined, so the exact location of the fault may vary slightly from what is shown on maps. The seismic mapping also shows that this fault is not contributing to the overall seismic hazard at the site, which indicates a lower risk of this fault rupturing. It is our opinion that the risk of fault rupture occurring at the site is low.

5.2.5 *Landslide Hazard*

The building and park project site was observed by members of GRI's engineering and geology staff during the field exploration program. The site was observed to be gently sloping down from east to west, but did not contain steep slopes that are susceptible to large landslides. Published lidar mapping of the site does not show any steep slopes at the site. The Oregon Department of Geology and Mineral Industries (DOGAMI) Statewide Landslide Information Database for Oregon mapping does not show any landslides at the project site; however, it does show that some previous landslides have occurred in the hills east of the project site (designation of closest landslide: Damascus_109), although no landslide debris has come within approximately 800 feet of the project site. Based on the lack of steep slopes at the project site and the relatively large distance between the project site and the nearest mapped landslide debris deposits, it is our opinion that the risk of landslides at the project site is low.

5.2.6 Other Geologic Hazards

According to the DOGAMI (2018) online statewide geohazards viewer, there are no mapped flood hazards or volcanic hazards at the site. The risk of damage by tsunami and/or seiche at the site is absent.

5.3 Earthwork

5.3.1 General

The fine-grained soil that mantles the site is moisture sensitive. As a result, it is our opinion that earthwork can be completed most economically during the dry summer months, which typically extend from June to mid-October. It has been our experience that the moisture content of the upper few feet of fine-grained soils will decrease during extended warm, dry weather. However, below this depth, the moisture content of the soil tends to remain relatively unchanged and well above the optimum moisture content for compaction. As a result, the contractor must use construction equipment and procedures that reduce disturbance and softening of the subgrade soils. To reduce disturbance of the moisture-sensitive, fine-grained soils, site grading can be completed using track-mounted hydraulic excavators. The excavations should be finished using a smooth-edged bucket to produce a firm, undisturbed surface. It may also be necessary to construct granular haul roads and work pads concurrently with earthwork to reduce subgrade disturbance. If the subgrade is disturbed during construction, soft, disturbed soils should be overexcavated to firm soil and backfilled with structural fill.

The trafficability of fine-grained soil at the ground surface may be difficult when the moisture content of the surface soil is more than a few percentage points above optimum, which will likely be the case during most of the year. If not carefully executed, earthwork activities can create extensive soft areas, resulting in significant repair costs. If very soft subgrade conditions are encountered during construction, especially during wet weather, granular work pads will be required to protect the underlying fine-grained subgrade and provide a firm working surface for construction activities. In our opinion, a 12- to 18-inch-thick granular work pad should be sufficient to reduce subgrade disturbance by lighter construction equipment and limited traffic by dump trucks. To reduce the risk of subgrade deterioration, haul roads and other high-density traffic areas (such as those trafficked by fork lifts) will require a minimum of 18 inches to 24 inches of crushed rock up to 6-inch nominal size. We recommend placing a geotextile fabric over the subgrade to reduce maintenance during construction. Although we have presented typical recommendations for granular work pads, the actual thickness and material should be determined by the contractor based on their sequencing of the project and the type and frequency of construction equipment. We also note that the base rock thickness for structural areas is intended to support post-construction design loads and will not support construction traffic when the subgrade soil is wet. If construction is planned for periods when the subgrade soil is wet, an increased thickness of base rock will be required.

5.3.2 Cement Amendment

As an alternative to the use of a thickened section of crushed rock to support construction activities and protect the subgrade, the fine-grained subgrade soil can be treated with cement. It has been our experience in this area that treating the subgrade soil to a depth of 12 inches to 16 inches with an approximately 6% to 8% admixture of cement overlain by 6 inches to 12 inches of crushed rock will typically support construction equipment and provide a good all-weather working surface. The actual cement content required will depend on multiple factors and will need to be determined by the contractor during construction based on their means and methods. We do not recommend attempting to cement amend subgrade soil that contains significant amounts of gravel, cobbles, or boulders. We recommend a minimum curing time of four days 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 at least 4 inches to 6 inches of crushed rock before construction traffic is allowed. It is common for localized areas of cement-treated soil to require retreatment or replacement with crushed rock.

Portland cement-amended soil is hard and has low permeability. This soil does not drain well and 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. Cement amendment should not be used if runoff during construction cannot be directed away from wetlands (if present). It is not possible to amend soil during heavy or continuous rainfall. Cement amendment should not be performed if the ground temperature is less than 40 degrees.

Cement can also be added to the on-site fine-grained soil to allow it to be placed as structural fill when its moisture content is wet of optimum. 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).

5.3.3 Site Preparation

The ground surface beneath all new foundations, retaining walls, hardscapes, athletic fields, and areas to receive structural fill should be stripped of existing vegetation, surface organics, and loose surface soils or fill. We observed the thickness of the topsoil zone to be highly variable across the site, generally ranging from 6 inches to 18 inches, with an overall average topsoil depth of approximately 12 inches. The actual stripping depth should be based on field observations at the time of construction. The stripping should extend at least 5 feet beyond the limits of the proposed improvement areas. The organic strippings should be transported off site for disposal or used as fill in landscaped areas. Excavations required to remove unsuitable soil, vegetation, and trees should be backfilled with structural fill.

Following stripping of excavations to design elevations, the exposed subgrade should be evaluated by a qualified member of GRI's geotechnical engineering or geology staff to evaluate the presence of areas of unsuitable or unstable soil. The subgrade should be evaluated using moisture-density testing, a hand probe, or proof rolling with a fully loaded dump truck (or similar heavy, rubber tire construction equipment). Any soft areas or areas of unsuitable material disclosed by the evaluation should be overexcavated to firm material and backfilled with structural fill. When excavations occur in areas containing undocumented fill, GRI staff should be present to observe the earthwork and evaluate the excavated soil. If the excavated soil contains significant organics (tree trunks, leaf piles, logs, etc.), oversized material, or other unsuitable material, additional site preparation work may be required beneath new structures to remove unsuitable material and reduce the risk of future damage to overlying structures. Depending on the conditions observed, GRI staff may also recommend additional test pits during construction to further evaluate subgrade beneath new structures. Construction documents should include costs for overexcavation and structural fill.

5.3.4 Prior Site Development and Demolition

The site has been used primarily as an agricultural field since at least 1937. We are not aware of any prior development activity at the site that should be considered by the proposed project. There are scattered fences, gates, and utilities around the perimeter of the project site that may need to be removed. Project plans also indicate there is a buried concrete retaining wall in the southwest corner of the field, which may need to be removed. If any additional existing footings, walls, slabs, utilities, pavement, or other similar improvements are unexpectedly found during construction, they should be completely removed from beneath new structures. Any monitoring wells or underground storage tanks that may be found on the property should be abandoned in accordance with state and local regulations prior to site development. Excavations resulting from the demolition of existing improvements should be backfilled with compacted structural fill as recommended in this report. The base of the excavations should expose firm subgrade. The sides of the temporary excavations should be cut into firm material and sloped no steeper than 1.5H:1V (Horizontal to Vertical).

5.3.5 Site Grading

We anticipate that the maximum fill height required for this project to raise grades will be approximately 5 feet. The on-site soil is susceptible to erosion. Consequently, we recommend that permanent slopes be covered with an appropriate erosion control product if construction occurs during periods of wet weather before new vegetation becomes established. Erosion control measures such as straw bales, sediment fences, and temporary detention and settling basins should be used in accordance with local and state ordinances. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face.

Final grading across the project site should provide for positive drainage of surface water away from exposed slopes to reduce the potential for erosion. Permanent cut and fill slopes, if planned, should not be steeper than 2H:1V and should be protected with vegetation as soon as practical to reduce the risk of surface erosion due to rainfall.

5.4 Excavation

5.4.1 General

According to the project grading plans, the maximum depth of cuts to establish final site grades will be up to approximately 10 feet. We anticipate that additional cuts up to approximately 10 feet may be required to install utilities, buried features like elevator pits, and the swimming pool. While we have described certain approaches to performing excavations in this report, it is the contractor’s responsibility to select the excavation and dewatering methods, monitor the excavations for safety, and provide any shoring required to protect personnel and adjacent improvements. All excavation work should be performed in accordance with applicable local, state, and federal safety regulations, including the current Occupational Safety and Health Administration excavation and trench safety standards. The means, methods, and sequencing of construction operations and site safety are the responsibility of the contractor. The information provided below is for the use of our client and should not be interpreted to imply that we are assuming responsibility for the contractor’s actions or site safety.

5.4.2 *Temporary Excavations*

Temporary excavations will be required to construct the proposed project. Conventional earthmoving equipment in proper working condition should be capable of making the necessary excavations in soil (silt, clay, sand, and gravel). The soil becomes much denser/stiffer below a depth of approximately 15 feet to 20 feet below the existing ground surface. Temporary excavation sidewalls will likely stand nearly vertical in silt and clay to depths of up to 4 feet, provided groundwater is maintained below the base of the excavations, but could experience raveling within sand layers that may result in excavations being wider than anticipated. Excavations deeper than 4 feet will require shoring or should be sloped. The contractor should be responsible for selecting the appropriate shoring system. We recommend a minimum horizontal distance of 5 feet from the edge of existing improvements to the top of any temporary slope.

Sloped excavations in soil may be used to depths of 20 feet and should have side slopes no steeper than 1.5H:1V, provided groundwater seepage does not occur. If seepage, sloughing, or instability is observed, the slope should be flattened or shored. All cut slopes should be protected from erosion by being covered during wet weather. Shoring will be required where slopes are not possible. If temporary excavation slopes encounter perched groundwater, a blanket of relatively clean, well-graded crushed rock placed on the slopes may be required to reduce the risk of raveling-soil conditions. We recommend the use of relatively clean, free-draining material, such as 2- to 4-inch-minus crushed rock, for this purpose. The thickness of the granular blanket should be evaluated based on actual conditions but would likely be on the order of 12 inches. The contractor should also consider adding geotextile fabric beneath the granular blanket to keep fines from mixing with the rock, depending on the upcoming weather conditions, amount of groundwater flow, duration of construction, and other factors.

In our opinion, the short-term stability of temporary slopes will be adequate if surcharge loads due to construction traffic, vehicle parking, material laydown, etc., are kept away from the top of the slope at a horizontal distance that is equal to the depth of the cut. Other measures that should be implemented to reduce the risk of localized failures of temporary slopes include the following: 1) using plastic or geotextile fabric to protect the exposed cut slopes from surface erosion; 2) providing positive drainage away from the tops and bottoms of the cut slopes; 3) constructing and backfilling walls as soon as practical after completing the excavation; 4) backfilling overexcavated areas as soon as practical after completing the excavation; 5) periodically monitoring the slopes for evidence of seepage, sloughing, and instability; and 6) periodically monitoring the area around the top of the excavation for evidence of ground cracking. Following these recommendations will not guarantee that sloughing or movement of the temporary cut slopes will not occur; however, the measures should serve to reduce the risk of a major slope failure. Blocks of ground and/or localized slumps may tend to move into the excavation during construction. The contractor should review the site conditions at the time of construction with the project team and evaluate factors impacting temporary slope stability.

Excavations should not be allowed to undermine adjacent improvements. If existing hardscape, retaining walls, buildings, or other structures are located near a proposed excavation, unsupported excavations can be maintained outside of a 1H:1V downward projection that starts 5 feet from the base of the existing elements. Excavations that must be inside of this zone should be supported by properly designed temporary or permanent shoring.

5.4.3 *Utility Excavations*

In our opinion, there are three major considerations associated with design and construction of new utilities:

1. Provide stable excavation sideslopes or support for trench sidewalls to reduce the loss of ground and undermining of adjacent structures.

2. Provide a safe working environment during construction.

3. Minimize post-construction settlement of the utility and ground surface.

In our opinion, trenches shallower than 4 feet deep that do not encounter groundwater may be cut vertically and left unsupported during the normal construction sequence, assuming trenches are excavated and backfilled in the shortest possible sequence and the trenches are not located near settlement-sensitive structures. Utility excavations more than 4 feet deep should be laterally supported or, alternatively, provided with side slopes of 1.5H:1V or flatter. In our opinion, adequate lateral support may be provided by common methods such as the use of a trench shield or hydraulic shoring systems. The shoring systems used in trench excavations should be designed to resist active soil pressures and designed to accommodate surcharge loading from adjacent settlement-sensitive structures. We recommend that shored trench excavations near settlement-sensitive structures or near steep slopes be limited to having no more than approximately 20 feet of trench excavation open at a time.

5.4.4 *Groundwater Management*

Excavations for this project will likely be below perched groundwater levels in some locations, especially after periods of wet weather or heavy rain. Groundwater seepage, running-soil conditions, and unstable excavation sidewalls or excavation subgrades, if encountered during construction, will require dewatering of the excavation and sidewall support. The impact of these conditions can be reduced by completing excavations during the summer months, when groundwater levels are lowest, and by limiting the depths of the excavations.

We anticipate that groundwater inflow, if encountered, can generally be controlled by pumping water from sumps. To facilitate dewatering, it will be necessary to overexcavate the base of the excavation to permit installation of a granular working blanket. We estimate the required thickness of the granular working blanket will be on the order of 1 foot, or as required to maintain a stable excavation base. The actual required depth of overexcavation will depend on the conditions exposed in the excavations and the effectiveness of the contractor's dewatering efforts. The thickness of the granular blanket must be determined based on field observations during construction. For this purpose, we recommend the use of relatively clean, free-draining material such as 2- to 4-inch-minus open crushed rock. The material should have a maximum particle size of 4 inches, should have less than 5% 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 organics and other deleterious material. The use of a geotextile fabric over the excavation base will assist in subgrade stability and dewatering. Water generated during dewatering operations should be treated, if required by Clackamas County or the City, and pumped to a suitable disposal point. Water should not be pumped onto existing slopes or stored in a temporary pond at the top of a slope.

We are informed that the contractor for this project is considering installing an interceptor drain along the east side of the building to assist with dewatering during construction and to permanently reduce the long-term risk of building water intrusion. The interceptor drain would consist of a drainpipe installed at the base of a trench approximately 10 feet deep and 2 feet wide, backfilled with crushed rock and filter fabric. The interceptor drain would capture shallow groundwater flowing downhill toward the building and direct it away from the building to a suitable discharge point.

In our opinion, this interceptor drain could be beneficial during construction as part of the contractor's dewatering efforts; however, it should not be relied upon as a permanent dewatering measure, since the drainpipe may eventually become clogged, and shallow groundwater could still migrate toward the building from other directions, bypassing the interceptor drain.

We note that these recommendations are for guidance only. Dewatering of excavations is the sole responsibility of the contractor, as the contractor is in the best position to select the appropriate system based on their means and methods.

5.5 Structural Fill

5.5.1 *Imported Granular Material*

We anticipate that significant amounts of structural fill will be required to establish final site grades. We recommend that imported structural fill for this project consist of granular material such as crushed rock, sandy gravel, or sand with a maximum size of 2 inches. Granular material that has less than 5% passing the No. 200 sieve (washed analysis) can usually be placed during periods of wet weather. Granular backfill should be placed in lifts and compacted with vibratory equipment to at least 95% of the material's maximum dry density, as determined by ASTM International (ASTM) D1557. Appropriate lift thicknesses will depend on the type of compaction equipment used. For example, if hand-operated, vibratory-plate equipment is used, lift thicknesses should be limited to 6 inches to 8 inches. If smooth-drum vibratory rollers are used, lift thicknesses up to 12 inches are appropriate, and if backhoe- or excavator-mounted vibratory plates are used, lift thicknesses up to 2 feet may be acceptable.

We recommend not importing fine-grained soil due to the likely challenges associated with moisture-conditioning the soil before it can be properly compacted.

5.5.2 *Trench Backfill Material*

All public utilities that are installed should be backfilled in accordance with the applicable agency's trench backfill requirements. All private utility trench excavations within structural or hardscape areas should be backfilled with relatively clean, granular material such as crushed rock, sandy gravel, or sand of up to 1-inch maximum size and having less than 5% passing the No. 200 sieve (washed analysis). The bottom of the excavation should be thoroughly cleaned to remove loose materials. The utilities should be underlain by a minimum 6-inch thickness of bedding material, and the utilities should also be surrounded with this material in the pipe zone. The bedding and pipe zone material should be compacted to at least 90% of the maximum dry density, as determined by ASTM D1557, or as recommended by the pipe manufacturer. The granular backfill material above the pipe zone should be compacted to at least 95% of the material's maximum dry density, as determined by ASTM D1557, in the upper 4 feet of the trench and to at least 92% of this density below a depth of 4 feet. The use of hoe-mounted, vibratory-plate compactors is usually most efficient for this purpose. Flooding or jetting as a means of compacting the trench backfill should not be permitted. Outside of structural areas, trench backfill material should be compacted to at least 90% of the maximum dry density, as determined by ASTM D1557.

5.5.3 *On-Site Soil*

The on-site fine-grained soil will be suitable for use as structural fill only if it can be moisture conditioned. Based on our experience, fine-grained soil is sensitive to small changes in moisture content and may be difficult, if not impossible, to compact adequately during wet weather or when the moisture content is more than a few percentage points above optimum. Available fine-grained soil may require extensive drying if it is used as structural fill. This material will only be suitable for use as fill during the dry season. The material should be placed in lifts with a maximum uncompacted thickness of 8 inches and compacted to not less than 95% of the maximum dry density, as determined by ASTM D698. We recommend using imported granular material for structural fill if the moisture content of the on-site fine-grained soil cannot be reduced. It may also be possible to cement amend on-site fine-grained soil for use as structural fill if the moisture content cannot be reduced, as described in the "Cement Amendment" section of this report.

5.5.4 *Recycled Cement Concrete*

Recycled cement concrete can be used for structural fill, provided the concrete is broken to a maximum particle size of 2 inches. This material must be durable so that there is minimal visible degradation of the material during and after compaction as structural fill. Recycled cement concrete can be used as trench backfill if it meets the size requirements for that application and the requirements for imported granular material. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95% of the maximum dry density, as determined by ASTM D1557.

5.5.5 *Geofoam*

Geofoam is a lightweight, engineered material composed primarily of expanded polystyrene (EPS). Geofoam's low density, high strength, and ability to be easily cut and shaped make it suitable for various lightweight fill applications. Geofoam is typically manufactured in blocks that measure approximately 4 feet wide by 4 feet high by 8 feet long and weigh approximately 1% as much as traditional soil. Geofoam installation can occur rapidly in nearly any type of weather. However, geofoam is vulnerable to damage by contact with petroleum products, is combustible, and is buoyant.

If desired, EPS geofoam may be used to backfill the building basement walls instead of onsite soil or imported crushed rock. The use of geofoam would result in reduced lateral loads on the basement walls, which could allow the walls to be designed with reduced thickness. If the use of geofoam is considered for this project, we recommend that the project structural engineer evaluate the impact of the reduced wall loads, and that the design and construction team evaluate any potential cost savings resulting from the use of geofoam. The current project plans show that the tallest basement walls are located on the east and north sides of the building and will be up to approximately 13 feet tall at the end of Phase 1 construction. The basement walls will retain landscape areas and sidewalk/pavement areas at the end of Phase 1 construction. We understand that a future development phase may replace some of the landscape area with a gymnasium building addition. The building addition finish floor elevation may match the existing building floor elevation, which would require the removal of the basement wall backfill material. Alternatively, the building addition finish floor elevation may be approximately 15 feet above the existing building floor elevation, which would require the addition of approximately 2 feet of additional fill during Phase 2 construction.

We recommend that geofoam be covered with a minimum of 18 inches to 24 inches of soil or pavement section to protect the geofoam from damage and allow for root growth in landscaped areas. We recommend that the geofoam be covered with a protective, hydrocarbon-resistant geomembrane compatible with EPS if exposure to petroleum products is anticipated. Appropriate fire precautions should also be implemented at the project site where open-flame activities such as welding may occur.

Geofoam is typically available in various densities ranging from about 0.7 pounds per cubic foot (pcf) to 2.85 pcf, with higher densities corresponding to greater compressive resistance. Loads acting on the geofoam will consist of soil cover, pavement, construction traffic, potential future building additions, and other surcharges. We recommend selecting a geofoam density that limits vertical loading compressive strain to 1% of the material’s compressive resistance. Vertical loads acting on the geofoam will result in uniform horizontal pressures equal to one-tenth of the vertical loads. Geofoam blocks should be installed according to the manufacturer’s recommendations and should include the use of gripper plates to prevent block movement. We note that installed geofoam may be difficult to dig through in the future due to the tendency of blocks to move during digging. During the installation of geofoam blocks, we recommend that the ground surface be benched so that the geofoam blocks are placed on horizontal surfaces. If heavy traffic loads will be placed over the geofoam, it may be necessary to construct a reinforced concrete slab above the geofoam to provide a firm pavement base layer.

Geofoam should be installed behind basement walls in a stair-step configuration to reduce static and dynamic wall pressures. The minimum width of geofoam at the base of the wall should be equal to one-half the height of the wall. If the geofoam and retained soil slope angle is 2H:1V or flatter, the static retained soil load on the non-yielding basement walls can be reduced from an equivalent fluid unit weight of 55 pcf to 2 pcf. The seismic retained soil load can be reduced from an equivalent fluid unit weight of 16 pcf to a resultant force of 1 H^2 psf per unit length of wall, where H is the height of the wall in feet and the resultant force acts at a height of 0.6H above the base of the wall. Although we have included recommended loads for a 2H:1V interface slope between the geofoam and retained soil that results in wall loads being reduced to nearly zero, we note that alternate slope angles are possible and we can provide load recommendations for other configurations upon request. In general, steeper slope angles will use less geofoam, but will result in higher wall loads than what is noted above.

In addition to static and seismic wall loads from the retained soil, surcharge loads from soil cover, pavement, construction or vehicle traffic, and future building additions should be included in the design. The load recommendations provided above assume drained conditions. We recommend that drains be installed beneath and behind the geofoam blocks to prevent buoyant uplift forces from raising the geofoam blocks. There should be a continuous zone of drain rock at least 12 inches thick beneath and behind the geofoam blocks. The drain rock should be wrapped in a nonwoven geotextile fabric.

As noted above, a future phase of development may construct a new building addition adjacent to the basement wall location. If the new building addition finish floor elevation matches the existing building floor elevation, the geofoam backfill installed would need to be removed. If the new building addition finish floor elevation is about 15 feet above the existing finish floor elevation, the geofoam will need to be designed to support the new building addition footings. To avoid overstressing the geofoam and to minimize the amount of differential settlement between footings supported on geofoam and footings supported on native soil, we recommend new building footings supported on geofoam be sized based on an allowable bearing pressure of 1,500 psf under static loads, plus a one-third increase for short-term loading such as wind and seismic.

Prior to installation, the contractor shall provide a submittal on the proposed geofoam material to GRI for review to confirm that it will perform as intended.

5.6 Foundation Support

5.6.1 General

As previously discussed, there is a mapped earthquake fault in the northeast portion of the site. We recommend that the proposed building not be constructed over the fault; therefore, we recommend that the proposed building not be constructed in the northeast portion of the site unless additional explorations to determine the exact location of the fault are performed. Current plans show the building will be in the southwest portion of the site, which has a low risk of fault rupture.

Based on the results of our explorations and analyses, we recommend that the proposed structures for this project be supported on spread footings that are underlain by firm native soil, or structural fill placed over firm native soil. If soft subgrade soil is encountered during construction, the unsuitable material should be overexcavated and replaced with crushed rock structural fill. If used, the crushed rock should extend laterally at least 6 inches beyond the footing perimeter for every foot that the crushed rock extends below subgrade. Our footing overexcavation recommendations are shown on the Footing Overexcavation Detail, Figure 3.

Footings supported on firm native soil or structural fill placed over firm native soil should be sized using an allowable bearing pressure of 3,000 pounds per square foot (psf). This value may be increased by one-third for short-term loads such as wind and seismic forces. We recommend that individual column and continuous wall footings have minimum widths of 24 inches and 18 inches, respectively. The bottoms of exterior footings should be founded at least 18 inches below the lowest adjacent grade. Interior footings should be founded at least 12 inches below the base of the floor slab. We recommend a minimum horizontal spacing between adjacent footings that is equal to twice the width of the footings.

Our experience indicates that subgrade consisting of fine-grained soil is easily disturbed by excavation and construction activities, especially during the wet season. Therefore, we recommend installing a minimum 3-inch-thick layer of compacted crushed rock over prepared footing subgrade consisting of fine-grained soil if the subgrade is exposed to rain. Relatively clean, ¾-inch-minus, crushed rock is suitable for this purpose and should be compacted with a lightweight vibratory compactor. A thicker section of crushed rock may be recommended by wall designers as a leveling course beneath retaining wall footings.

All footing subgrade should be evaluated by the project geotechnical engineer or their representative to evaluate bearing conditions. Observations should determine whether all loose or soft material, organic material, unsuitable fill, prior topsoil zones, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate unsuitable material.

If any foundations, floor slabs, retaining walls, hardscapes, utilities, or other structures will be located near areas where new fill is placed, we recommend that the work be sequenced so the new fill is placed before the new structures are built. We also recommend that settlement monitoring be performed to confirm that primary consolidation settlement is complete before nearby structures are built.

5.6.2 Lateral Resistance

Horizontal shear forces on footings can be resisted by friction on the base of the footings and by soil passive resistance. We recommend that an allowable friction coefficient of 0.30 be used to compute the frictional resistance for footings bearing on silt or clay. We recommend that a friction coefficient of 0.50 be used to compute the frictional resistance for footings bearing on crushed rock. Passive earth pressures against embedded footings can be computed based on an equivalent fluid having a unit weight of 250 pcf. Lateral deformations of approximately 0.01H (where H is the height of the embedded portion of the structure) will result from full mobilization of this passive pressure. This design passive earth pressure is applicable only if the footing is cast neat against undisturbed soil or if backfill for the footings is placed as structural fill. The top 1 foot of soil should be neglected when calculating lateral earth pressures unless the soil is covered with pavement or a concrete slab. These values assume that the ground adjacent to footings is level and that groundwater remains below the base of the footings.

5.6.3 Settlement

We anticipate that the total post-construction settlement of building footings bearing on firm native soil or structural fill will be less than 1 inch. Differential settlement between similarly loaded adjacent building footings is expected to be up to ½ inch. We anticipate that seismic settlement at this site will be negligible.

5.7 Slab-On-Grade Support and Underslab Drain System

We anticipate that the existing subgrade soil will generally provide adequate support for concrete slabs-on-grade. We recommend that the slab subgrade be evaluated during construction by a qualified member of GRI's geotechnical engineering or geology staff. If any loose undocumented fill or unsuitable soil is present beneath the floor slabs, the subgrade soil should be scarified and recompacted or overexcavated. A modulus of subgrade reaction of 120 pounds per cubic inch for a 1-foot-by-1-foot loaded area can be used for design of the floor slabs, provided the subgrade is prepared in accordance with the recommendations presented in this report. Settlement of slabs supporting the anticipated design loads and constructed as recommended is anticipated to be less than 1 inch of total settlement and ½ inch of differential settlement.

Because of the relatively shallow groundwater levels, finish floor elevations typically being below adjacent grades, and the use of moisture-sensitive flooring in certain building areas, we recommend that an underslab drain system be installed beneath the entire building footprint. The underslab drain system should be installed directly beneath the floor slab and consist of 4-inch-diameter perforated pipes spaced 15 feet to 20 feet apart, embedded in a minimum 12-inch-thick layer of drain rock.

The drainpipes should be located between footings, not directly beneath them. The drainpipes should be sloped to drain to the perimeter footing drains or the stormwater system, with appropriate backflow prevention devices installed. The drainpipes should be protected so that they are not damaged by construction traffic. At least 2 inches of drain rock should be present beneath the drainpipes and at least 4 inches of drain rock should be placed above the drainpipes. The drain rock should consist of crushed rock with a maximum particle size of 1½ inches, have at least two mechanically fractured faces, and have less than 2% by dry weight passing the U.S. Standard No. 200 sieve. The drain rock should be compacted using vibratory equipment until it is firm and unyielding. If desired for constructability purposes, it is acceptable for the upper 2 inches of the drain rock layer to be replaced with a cap of ¾-inch-minus crushed rock. Depending on the gradation of the two rock types, it may be necessary to install a separation geotextile between the two rock layers to minimize the migration of smaller particles. If building codes require an alternate rock be used beneath the floor slabs for radon mitigation purposes, we request that GRI be allowed to review the radon rock submittal to confirm that it will also provide adequate floor slab support. Our underslab drain recommendations are shown on the Typical Underslab Drain Detail, Figure 4.

We recommend that a vapor barrier be installed beneath floor slabs to reduce the risk of moisture intrusion through the floor slab. The vapor barrier should be installed directly beneath the concrete floor slab and above the granular material. The granular material should not be saturated before the vapor barrier is placed. Care should be taken during construction to avoid puncturing or tearing the vapor barrier. If the vapor barrier is damaged during construction, it should be immediately repaired per the manufacturer's recommendations.

5.8 Work Sequence and Fill Considerations

Due to the planned phased construction of the new building, care should be taken by the design and construction team to avoid having future building expansions damage the existing building foundations. New foundations should not be allowed to undermine existing foundations and new loads that would cause excessive settlement should not be allowed on existing foundations. This can be accomplished by installing the initial building foundations deeper, so they are less likely to be undermined by future excavations; by installing oversized foundations for the initial building, so that there is enough capacity to accommodate new loads from the building expansion; and by other methods. The design team should also consider how underslab and wall drainage systems for future expansions can be constructed without interfering with existing drainage systems.

Although significant fills are not anticipated for this project, some fill will need to be placed to backfill basement walls and other excavations, as well as to raise grades in some locations. We anticipate that the maximum fill height placed to raise grades will not exceed 5 feet. Before new fill is placed on slopes, we recommend that all vegetation and topsoil be removed. GRI staff should then observe the site to confirm organic material has been removed. Once the organic material has been removed, we recommend that the exposed slope be benched so that the new fill can be keyed into the existing slope. We recommend that benches be approximately 4 feet to 8 feet wide and 2 feet to 4 feet high. The width of the bench should correspond to the width of the compaction equipment being used. A detail showing our benching recommendations is provided on the Benching Detail for Fill Placed on Slopes, Figure 5.

The fill placed on the slope should be compacted as structural fill. Because compaction equipment cannot compact fill directly to the edge of a slope, the slope should be slightly overbuilt, and the loose soil at the face of the slope should be removed with an excavator bucket until firm soil is encountered.

If any foundations, floor slabs, retaining walls, hardscapes, utilities, or other structures will be located near areas where new fill is placed, we recommend that the work be sequenced so the new fill is placed before the new structures are built. We also recommend that settlement monitoring be performed to confirm that primary consolidation settlement is complete before nearby structures are built. Settlement monitoring should also be performed if existing settlement-sensitive features are present near where fill is placed, with near defined as being within a horizontal distance that is equal to the width of the fill area.

We recommend that settlement be monitored using survey hubs. After the new fill has been placed, we recommend that multiple survey hubs be installed at each fill location. The elevations of the survey hubs should be measured twice per week until the data show that primary settlement is complete. The survey hubs should be monitored using survey equipment with accuracy to 0.01 feet and referenced to a minimum of two stationary points such as construction control points or permanently installed benchmarks located outside of the area of construction. The construction team should protect the survey hubs from being disturbed during construction. The survey data should be sent to GRI for evaluation after each measurement. The contractor should also prepare a figure showing the installed locations of all the survey hubs. As the project geotechnical engineer of record, GRI will determine when primary consolidation settlement due to the fill is complete and when the construction of overlying or nearby structures can begin.

If existing buried utilities are present near the fill locations, they could also settle as fill is placed. The settlement may be irregular and low points (bellies) in the pipelines could develop. The utility owners should verify that such utilities are capable of withstanding the estimated settlement without being damaged.

5.9 Drainage

The finished ground surface around the building should be sloped away from foundations at a minimum 2% gradient for a distance of at least 5 feet. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. Runoff water should not be directed to the top of slopes or discharged onto the slope face.

We recommend that perimeter foundation drains be installed at all exterior footings. We recommend that roof downspouts or scuppers discharge to a solid pipe that carries the collected water to an appropriate stormwater system that is designed to prevent backflow. As described in the Slab-on-Grade Support and Underslab Drain System section of this report, we recommend that an underslab drain system be installed beneath the entire building footprint.

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

5.10 Waterproofing and Buoyancy Resistance

Below-grade enclosed spaces such as mechanical pits, elevator pits, and other similar spaces that will extend deeper than the underslab drain system should be fully waterproofed to reduce water intrusion. We recommend that a waterproofing consultant be retained to select the appropriate waterproofing products and to develop installation details. We anticipate that the waterproofing will consist of bentonite panels or another approved waterproofing product that fully covers the underside of the floor slab and the walls of the below-grade enclosed spaces.

Below-grade structures that will extend below the underslab drain system, such as swimming pools, mechanical pits, elevator pits, and other similar spaces, should be designed to resist buoyant uplift forces. This can typically be accomplished by increasing the thickness of concrete floor slabs in these areas to add weight that will offset buoyant forces. We recommend that buoyancy calculations be based on the assumption that groundwater is present to the elevation of the underslab drain system.

5.11 Retaining Walls

We anticipate that permanent retaining walls with a maximum height of approximately 13 feet may be required to maintain final site grades. Design lateral earth pressures for retaining walls depend on the type of construction and the ability of the walls to yield. Possible conditions are 1) a wall that is laterally supported at its base and top and therefore unable to yield to the active state and 2) a retaining wall, such as a typical cantilevered or gravity wall, which yields to the active state by tilting about its base. A conventional basement wall and cantilevered retaining wall are examples of non-yielding and yielding walls, respectively.

For completely drained conditions and horizontal backfill, yielding and non-yielding retaining walls may be designed based on equivalent fluid unit weights of 35 pcf and 55 pcf, respectively. To account for seismic loading, the earth pressure should be increased by 7 pcf and 16 pcf for yielding and non-yielding walls, respectively. These earth pressures assume the walls are fully drained and that hydrostatic pressure cannot build up on the back of the wall. This results in a triangular distribution with the resultant acting at ⅓H up from the base of the wall, where H is the height of the wall in feet. The seismic earth pressures were estimated based on the methods described in Agusti and Sitar (2013).

For yielding walls that retain slopes with a steepness of up to 2H:1V, we recommend that an increased static earth pressure equivalent fluid unit weight of 55 pcf be used for design, along with an increased seismic pressure value of 24 pcf.

We recommend that continuous retaining wall footings have minimum widths of 18 inches. The bases of the wall footings should extend a minimum of 18 inches below the lowest adjacent grade. Deeper embedment may be required to satisfy global stability concerns. Global stability analyses should be performed by the wall designer or GRI as part of the retaining wall design. At locations where there is a slope in front of the retaining wall, we recommend that a minimum 5-foot-wide, horizontal bench be placed between the wall and the top of the slope. Excavations for retaining wall foundations should be made with a smooth-edged bucket to reduce subgrade disturbance.

The pressures provided above will allow moderate relaxation of the wall, which will cause some ground surface settlement behind the wall. Consequently, we recommend that construction of flatwork adjacent to retaining walls be postponed until survey data indicate that primary consolidation settlement is complete. We anticipate that settlement will become negligible beyond a horizontal distance from the wall that is equal to the height of the wall.

If surcharges such as retained slopes, building foundations, vehicles, terraced walls, etc. are within a horizontal distance from the back of the wall that is equal to the height of the wall, additional pressures will need to be accounted for in the wall design. The Surcharge-Induced Lateral Pressure, Figure 6, can be used to determine surcharge pressures resulting from some common loading scenarios. Our office should be contacted for additional pressures resulting from alternate loading scenarios. We recommend a vertical live load of 250 psf be applied at the surface of the retained soil where the wall retains roadways with vehicle traffic. At locations where walls will not retain roadways or vehicle traffic, vertical live loads that correspond to the intended use should be applied.

The lateral earth pressure criteria presented above are only appropriate if the retaining walls are fully drained. Perched groundwater may occur within the shallow fine-grained soils during periods of prolonged or intense precipitation. Based on these considerations, we recommend installation of a permanent drainage system behind all retaining walls. For wall backfill consisting of granular material, the drainage system can either consist of a drainage blanket of crushed rock or continuous drainage panels between the retained soil/backfill and the face of the wall. For wall backfill consisting of fine-grained soil, the drainage system should consist of a chimney drain that extends along the original slope face and a vertical drainage blanket or continuous drainage panels against the wall. The drainage blanket and chimney drain should have a minimum width of 12 inches and should consist of crushed drain rock that contains less than 2% fines content (washed analysis). A typical drainage system for retaining walls is shown on the Typical Wall Subdrainage Detail, Figure 7. The drainage blanket or drainage panels should extend to the base of the wall, where water should be collected in a 4-inch-diameter perforated pipe and discharged to a suitable outlet such as a sump or approved storm drain system that includes measures to prevent backflow into the drainage system of the wall. A geotextile fabric should be placed between the crushed rock and fine-grained soil to minimize the migration of fines into the crushed rock. If foam insulation is placed on the embedded portion of the wall, the drainage layer should be installed on the soil side of the foam. In addition, the wall design should include positive drainage measures to avoid ponding of surface water behind the top of the wall.

For the below-grade swimming pool walls, the walls can be designed for drained conditions if a drain is installed at the base of the walls, as described above. Alternatively, the swimming pool walls do not need drains if the walls are designed to resist full hydrostatic pressure. Undrained non-yielding swimming pool walls should be designed using a static earth pressure equivalent fluid unit weight of 90 pcf. To account for seismic loading, the earth pressure should be increased by 16 pcf for non-yielding walls. We note that the additional wall thickness required to resist hydrostatic forces may be beneficial by providing additional uplift resistance due to the extra wall weight. The swimming pool walls should be designed for the worst-case condition of saturated wall backfill and no water inside the swimming pool.

We anticipate that the wall backfill material could range from on-site fine-grained soil to imported granular material. Overcompaction of backfill behind walls should be avoided. Heavy compactors and large pieces of construction equipment should not operate within 5 feet of any retaining wall to avoid the buildup of excessive lateral earth pressures. Compaction close to the walls should be accomplished with hand-operated, vibratory plate compactors and lifts up to 6 inches thick. Backfill located within a horizontal distance of 3 feet from the retaining walls should be compacted to approximately 90% of the maximum dry density, as determined by ASTM D698 for fine-grained soil or ASTM D1557 for granular material. Overcompaction of backfill could significantly increase lateral earth pressures behind walls and cause damage to cast-in-place concrete retaining walls. If hardscape such as slabs, sidewalks, or pavement will be placed adjacent to the wall, we recommend that the upper 2 feet of fill consist of granular material and be compacted to 95% of the maximum dry density, as determined by ASTM D1557.

5.12 Pavement Design

5.12.1 General

Based on our discussions with the design team, we understand that two new AC-surfaced parking lots will be constructed on the northeast and southwest ends of the proposed community park.

We conducted pavement design analyses using flexible pavement to accommodate the estimated traffic loading over a 25-year design period for conventional flexible pavement (i.e., AC over aggregate base rock) and AC over cement-stabilized soil. The details of our analysis approach, details of the design parameters used in our analyses, and the findings from our analyses are provided below.

5.12.2 Traffic-Loading Analysis

We approximated the annual 18-kip Equivalent Single Axle Load (ESAL) repetitions of Federal Highway Administration Class 4 through 13 vehicles based on traffic, vehicle class, and frequency data provided to us by the design team. The calculation methodology and other inputs to approximate the cumulative ESAL repetitions for the design period are presented in Appendix F. The 25-year traffic loading approximation used in our pavement design analysis for the parking lots is 17,000 ESALs. As requested by the City, we did not design the parking lot pavement to accommodate construction traffic. Construction traffic should be limited to haul roads. If construction traffic is allowed to operate on the new pavement, the design-life of the pavement could be reduced and it may be necessary to repair some of the pavement that becomes damaged.

5.12.3 Subgrade Resilient Modulus

We used the DCP test data summarized in Appendix A to approximate the design subgrade resilient modulus values within the project limits. The DCP test results are used to approximate the California bearing ratio, which is correlated to the subgrade resilient modulus based on the relationship developed by Chen et al. (1999). Based on the average of the approximated resilient modulus values at each test location within the project limits, we approximated a design subgrade resilient modulus value of 4,000 psi for the parking lots.

5.12.4 Pavement Design Recommendations

We used the methods described by Giroud and Han (2004a and 2004b) to develop the design thickness of aggregate base rock needed above the subgrade, in combination with a geotextile, to support road construction traffic and the procedures presented in the 1993

American Association of State Highway and Transportation Officials *Guide for Design of Pavement Structures,* the 2019 Oregon Department of Transportation Pavement Design Guide (ODOT PDG), and the City's 2025 *Engineering Design Standards Details Manual* (EDM). Accordingly, we developed new pavement construction options for the proposed parking lot based on the input parameters and design details provided in Tables 2F and 3F in Appendix F, which provide designs for conventional flexible pavement (AC over aggregate base rock) and AC over cement-stabilized soil, respectively. Our recommendations for the new parking lot pavement are provided below.

Option 1: New Construction with Geotextile

- 3-inch-thick, Level 2, ½-inch, Dense Asphalt Concrete Pavement (ACP), Performance Grade (PG) 64-22 Asphalt Binder
- 2.0-inch-thick Crushed Aggregate Leveling Course (¾-inch-0)
- 8.0-inch-thick Crushed Aggregate Base Rock Course (1½-inch-0)
- Non-woven Subgrade Geotextile

Option 2: New Construction with Cement-Stabilized Soil

- 3-inch-thick, Level 2, ½-inch, Dense ACP, PG 64-22 Asphalt Binder
- 12.0-inch-thick Cement-Stabilized Soil Layer

5.12.5 Construction Considerations

Construction materials and procedures should comply with the applicable sections of the 2024 ODOT *Oregon Standard Specifications for Construction* and the modifications given in Table 5-2, below.

Table 5-2: OREGON STANDARD SPECIFICATIONS FOR CONSTRUCTION

Materials/Activity	Specification
Subgrade Compaction	Special Provision 00330
Subgrade Stabilization	Special Provision 00331
Subgrade Geosynthetics	Special Provision 00350. Non-woven Separation Geotextile.
Aggregate Base and Subbase	Special Provision 00641 (1½-inch-0 or ¾-inch-0).
Asphalt Concrete	Special Provision 00744. Use Level 2, ½-inch dense asphalt concrete pavement. The minimum lift thickness is 2 inches, and the maximum lift thickness is 3 inches.
Asphalt Binder	Use Performance Grade 64-22 Asphalt Cement.

Materials/Activity	Specification
Treated Subgrade	Special Provision 00344. We recommend portland cement as the stabilizing agent.

5.12.6 Pavement Subgrade Stabilization/Wet-Weather Construction

Subgrade stabilization should be completed where the subgrade soils exhibit unsuitable conditions based on proof rolling or foundation probing and/or where the finished aggregate base rock exhibits deflections or pumping based on proof rolling. Our recommendations for subgrade stabilization are presented below:

- 12-inch-thick subgrade stabilization consisting of additional aggregate base rock
- Non-woven subgrade geotextile
- On undisturbed subgrade

For extremely soft conditions or in periods of wet weather, an additional thickness of subgrade stabilization may be required.

For the pavement section option with cement-stabilized soil, we did not conduct laboratory testing on subgrade soil samples to determine the design cement content for the in-place stabilization. The cement and moisture content should be adjusted according to the field conditions at the time of construction. The in-place stabilization work consists of constructing a reclaimed cement-treated base by pulverizing and mixing the existing subgrade materials with portland cement and water, then grading and compacting the cement-treated base to the lines, grades, thicknesses, and cross-sections required.

The cement-stabilized soil layer should be constructed in accordance with the recommendations in the Cement Amendment section of this report and the following requirements:

- The work of cement or cement slurry application, mixing, spreading, compacting, shaping, and finishing should be continuous and completed within three hours from the start of mixing.

- After completion of final grading and compaction of the in-place cement-stabilized base, the surface should be sealed with a CSS-1 asphalt cement cure seal, or the first lift of AC should be placed. The cure seal or AC lift should be placed as soon as feasible and not later than four hours after the initial mixing of the reclaimed material with cement. If the contractor elects to place the AC lift, a proof roll of the cement-treated material should be performed as described below before the AC is placed.

- After the cure seal or AC lift has been placed, all heavy vehicle traffic should be prohibited from using the roadway for a period of 96 hours.

- Before AC is placed and after complete curing, the cement-treated material base should be proof rolled with a loaded dump truck. If unsuitable conditions are observed during proof rolling, the area should be overexcavated using hoe-type equipment equipped with a smooth-edged bucket and stabilized using aggregate base rock or controlled density fill. For extremely soft conditions, stabilization with an additional 16 inches of aggregate base rock backfill or 10 inches of controlled density fill may be required.

- After the 96-hour cure period and the proof roll, the AC (or remaining lifts of AC) should be constructed.

6 DESIGN REVIEW AND CONSTRUCTION SERVICES

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. To observe compliance with the intent of our recommendations, the design concepts, and the plans and specifications, we are of the opinion that all construction operations dealing with earthwork, foundations, and retaining walls should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in our report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions different from those described in this report.

7 LIMITATIONS

This report has been prepared to aid the project team in the design of this project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the proposed improvements. In the event that any changes in the design and location of the project elements as outlined in this report are planned, we should be given the opportunity to review the changes and modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations submitted in this report are based on the data obtained from the explorations made at the locations indicated on Figure 2 and other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times.

However, variations in soil conditions may exist between exploration locations. This report does not reflect any variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If subsurface conditions during construction differ from those encountered in the explorations, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in this report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

We have included as Appendix G the Geoprofessional Business Association guidance document "Important Information about This Geotechnical-Engineering Report" to assist you and others in understanding the use and limitations of this report. We recommend you read this document.

Submitted for GRI,

REGISTERED PROFESSIONAL ENGINEER
85582
OREGON
SEPT. 11, 2018
JASON DONALD BOCK

RENEWS: 12-2025

Ryan Lawrence

Jason D. Bock, PE
Principal

Ryan T. Lawrence, PE
Associate

This document has been submitted electronically.

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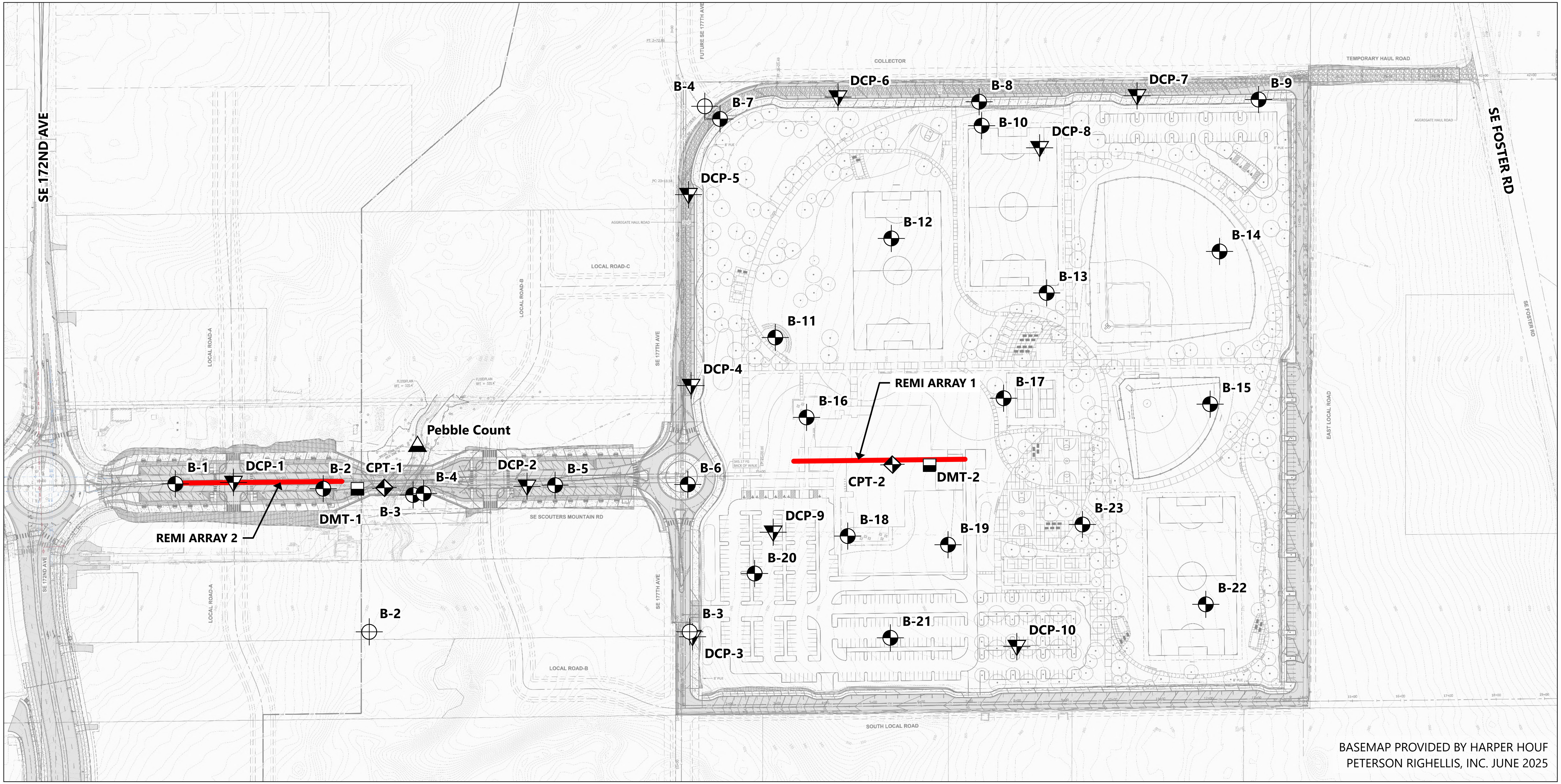
CITY OF HAPPY VALLEY
HAPPY VALLEY COMMUNITY RECREATION
CENTER: BUILDING AND PARK

VICINITY MAP

OCT. 2025 JOB NO. 7072-A FIG. 1



BASEMAP PROVIDED BY HARPER HOUF PETERSON RIGHELLIS, INC. JUNE 2025

LEGEND:

- ⊕ APPROXIMATE LOCATION OF BORING COMPLETED BY GRI
- ◆ APPROXIMATE LOCATION OF CONE PENETRATION TEST COMPLETED BY GRI
- ▬ APPROXIMATE LOCATION OF DILATOMETER TEST COMPLETED BY GRI
- ▲ APPROXIMATE LOCATION OF PEBBLE COUNT COMPLETED BY GRI
- ▼ APPROXIMATE LOCATION OF DYNAMIC CONE PENETRATRION TEST COMPLETED BY GRI
- ⊕ APPROXIMATE LOCATION OF BORING COMPLETED BY SHANNON & WILSON (2024)
- 🟥 APPROXIMATE LOCATION OF OF GEOPHYSICAL TESTING COMPLETED BY GRI

N

0 200 400

1 INCH = 200 FEET

GRI

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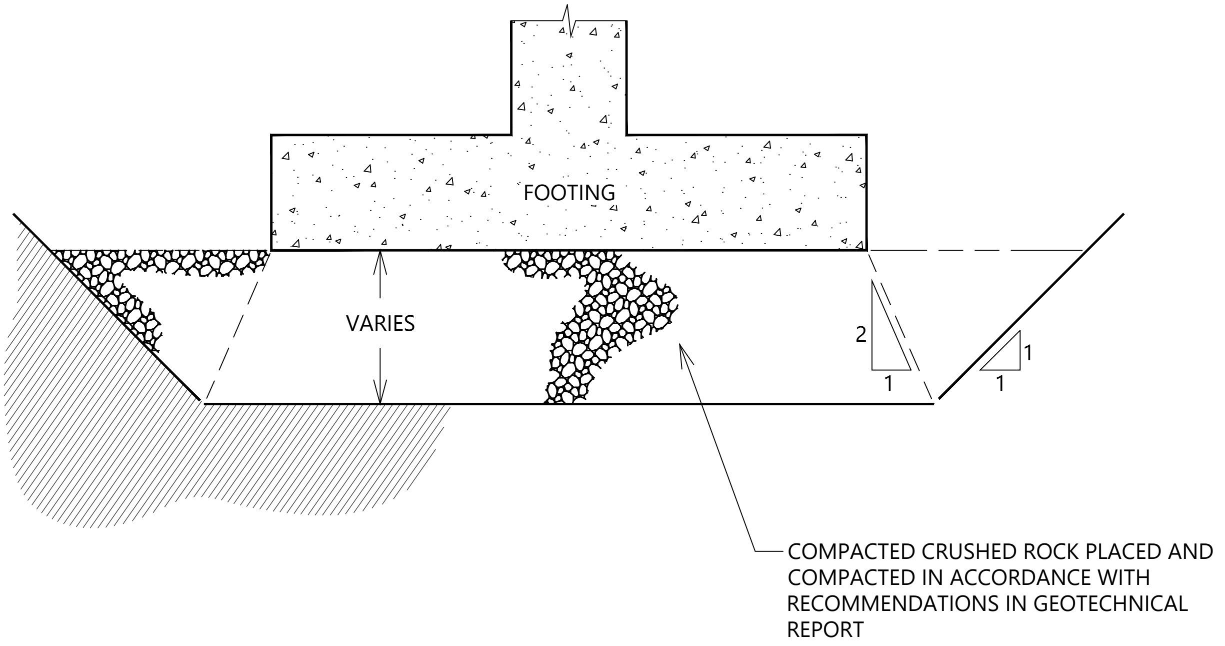
SITE PLAN

OCT. 2025 | JOB NO. 7072-A | FIG. 2



NOT TO SCALE

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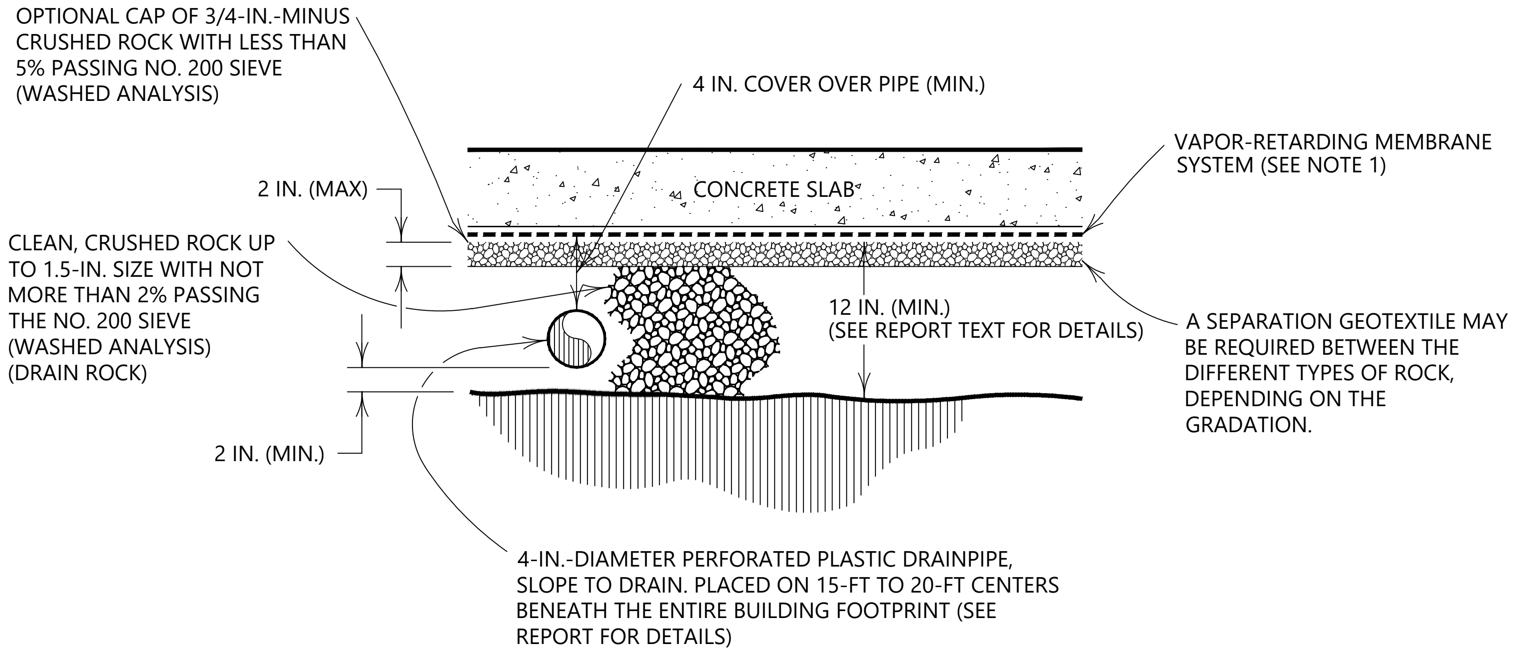
FOOTING OVEREXCAVATION DETAIL

OCT. 2025 JOB NO. 7072-A FIG. 3



NOTES:

1. A VAPOR-RETARDING MEMBRANE SYSTEM IS RECOMMENDED.
2. DETAILS REGARDING INSTALLATION OF THE SYSTEM SHOULD BE REVIEWED BY THE DESIGN TEAM.

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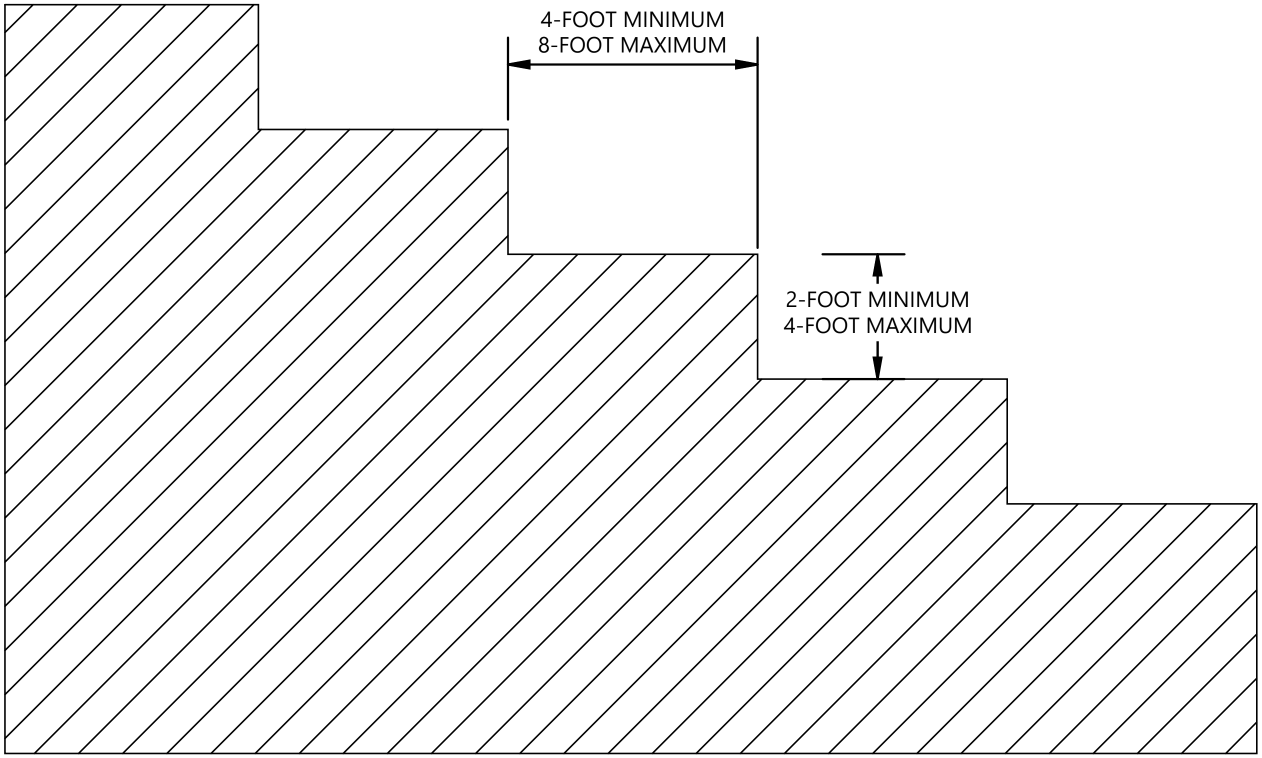
TYPICAL UNDERSLAB DRAIN DETAIL

OCT. 2025 JOB NO. 7072-A FIG. 4



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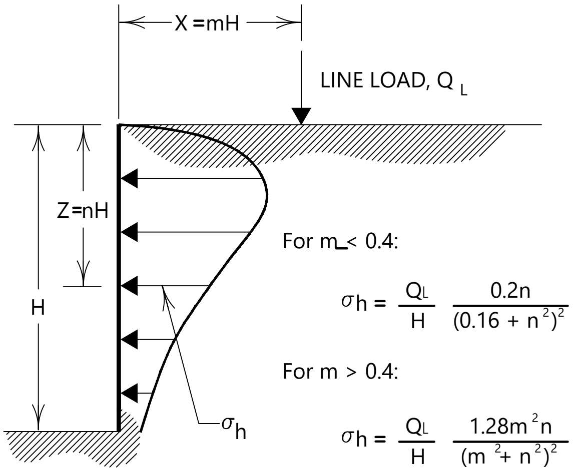
BENCHING DETAIL FOR FILL PLACED ON SLOPES

OCT. 2025 JOB NO. 7072-A FIG. 5



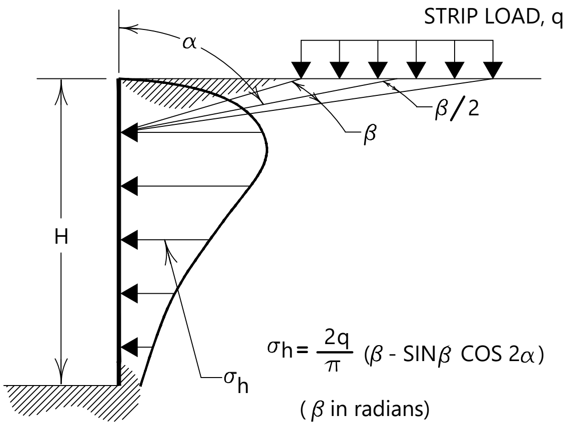
For $m \le 0.4$:

$$\sigma_h = \frac{Q_L}{H} \frac{0.2n}{(0.16 + n^2)^2}$$

For $m > 0.4$:

$$\sigma_h = \frac{Q_L}{H} \frac{1.28m^2 n}{(m^2 + n^2)^2}$$

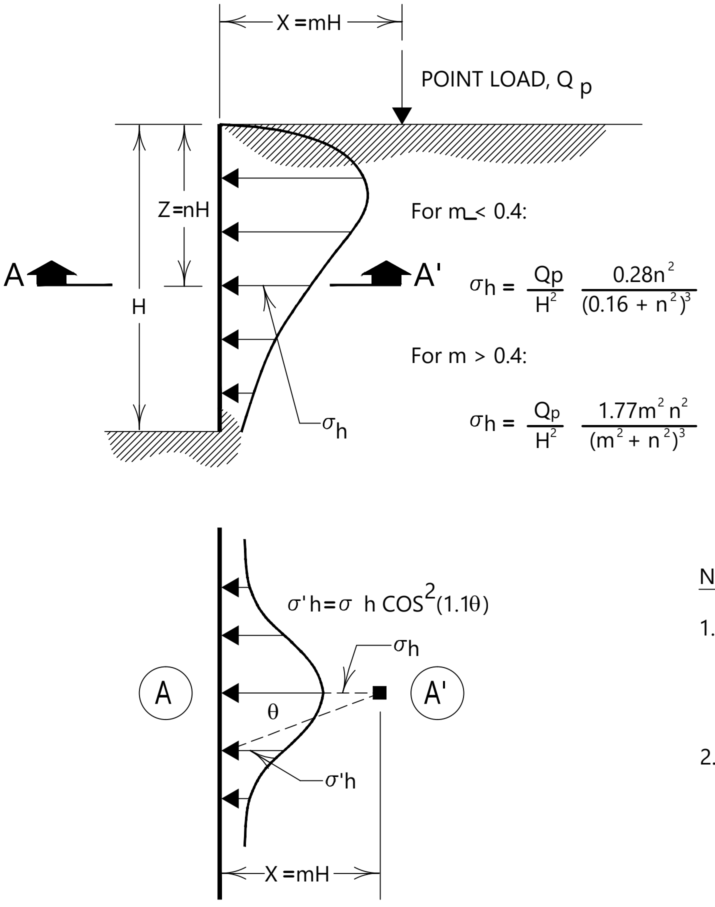
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{\pi} (\beta - \text{SIN}\beta \ \text{COS} \ 2\alpha)$$

(β in radians)

STRIP LOAD PARALLEL TO WALL



For $m \le 0.4$:

$$\sigma_h = \frac{Q_p}{H^2} \frac{0.28n^2}{(0.16 + n^2)^3}$$

For $m > 0.4$:

$$\sigma_h = \frac{Q_p}{H^2} \frac{1.77m^2 n^2}{(m^2 + n^2)^3}$$

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS. IF WALLS ARE DESIGNED TO YIELD, A LATERAL LOAD REDUCTION MAY BE APPROPRIATE. REFER TO REPORT TEXT FOR ADDITIONAL RECOMMENDATIONS FOR YIELDING WALLS.

2. LATERAL PRESSURES FROM ANY COMBINATION OF THE ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

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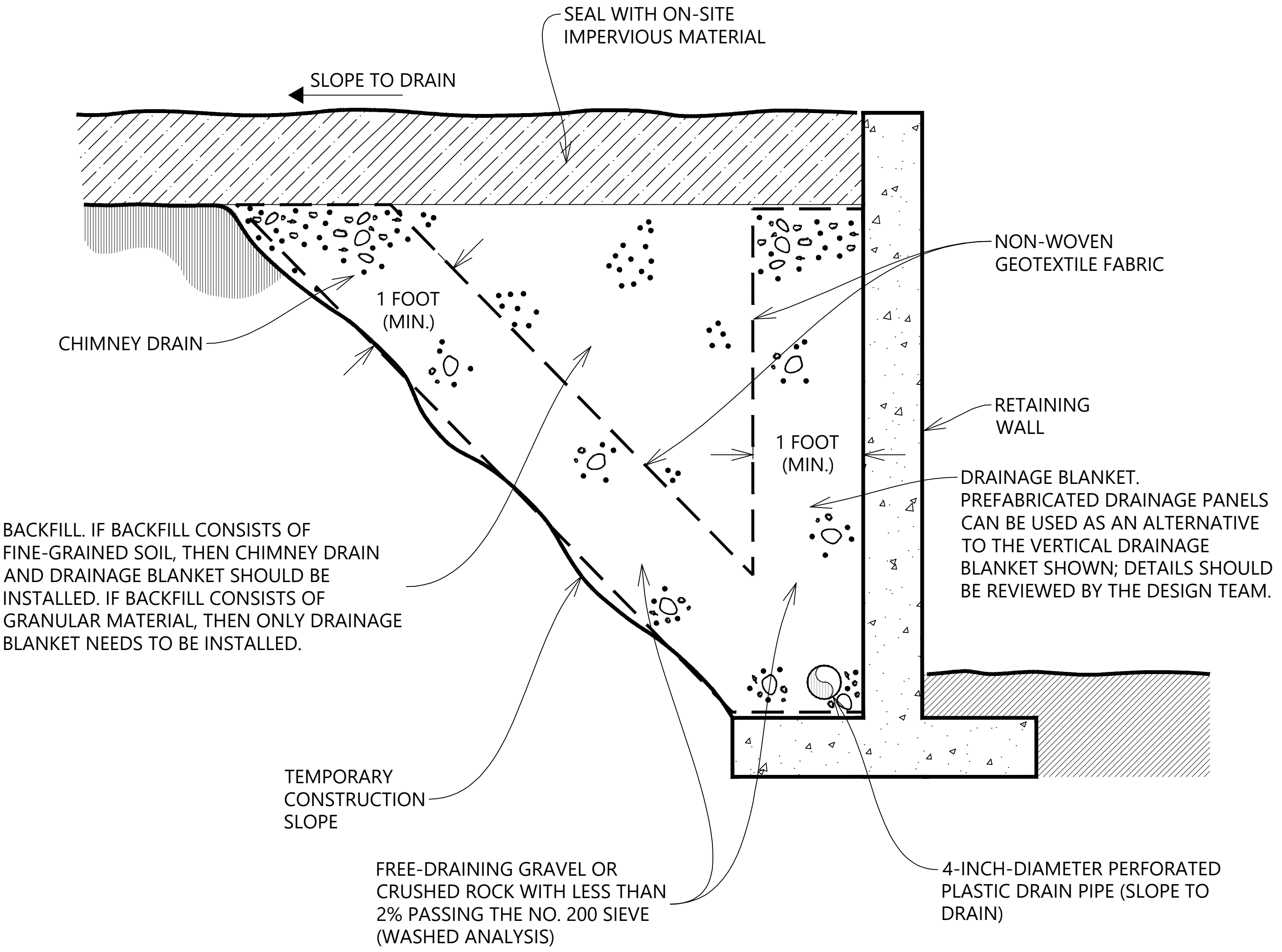
SURCHARGE-INDUCED LATERAL PRESSURE

OCT. 2025 JOB NO. 7072-A FIG. 6



NOT TO SCALE

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TYPICAL WALL SUBDRAINAGE DETAIL

OCT. 2025 JOB NO. 7072-A FIG. 7

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

A.1 FIELD EXPLORATIONS

A.1.1 General

Subsurface materials and conditions at the overall site were investigated between April 7 and 21, 2025, by drilling 23 borings, advancing two flat dilatometer test (DMT) probes, performing 10 Dynamic Cone Penetration (DCP) tests, and performing a pebble count in Rock Creek. The approximate locations of the borings, DMT probes, DCP tests, and pebble count completed for this investigation are shown on the Site Plan, Figure 2. The above fieldwork is discussed in more detail below. The field exploration work was coordinated and documented by experienced members of GRI's geology and engineering staff, who maintained logs of the materials and conditions disclosed during the course of work.

A.1.2 Borings

The 23 borings were designated B-1 through B-23 and were advanced to depths between 6.5 feet and 71.5 feet below existing site grades. The borings were drilled with hollow-stem auger, mud-rotary, and HQ rock coring techniques using a track-mounted drill rig provided and operated by Western States Soil Conservation, Inc., of Hubbard, Oregon. Disturbed and undisturbed soil samples were generally obtained from the borings at 2.5-foot intervals of depth in the upper 15 feet and at 5-foot intervals below this depth. Disturbed soil samples were obtained using a standard split-spoon sampler. The Standard Penetration Test (SPT) was completed while obtaining disturbed soil samples. This test is performed by driving a 2-inch outside-diameter, split-spoon sampler into the soil a distance of 18 inches using the force of a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is known as the Standard Penetration Resistance (SPT N-value). The SPT N-values provide a measure of the relative density of granular soils and the relative consistency of cohesive soils. Samples obtained from the borings were placed in sealed plastic bags and returned to our laboratory for further classification and testing.

In addition, relatively undisturbed samples were collected by pushing a 3-inch outside-diameter Shelby tube into the undisturbed soil a maximum distance of 24 inches using the hydraulic ram of the drill rig. The soil exposed at the end of the Shelby tube was examined and classified in the field. After classification, the tubes were sealed with rubber caps and returned to our laboratory for further examination and testing. Bulk samples of auger cuttings were also collected from selected borings, placed in buckets, and returned to our laboratory for further examination and compaction testing.

The intact rock we encountered was drilled using wireline drilling techniques and cored using an HQ diamond coring bit attached to a split-core barrel. All rock core samples were examined and classified in the field, photographed, placed in core boxes, and returned to our laboratory for further examination and testing.

Logs of the borings are provided on Figures 1A through 23A. The logs present a summary of the various types of materials encountered in the borings and note the depth at which the materials and/or characteristics of the materials change. To the right of the summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, SPT N-values, moisture contents, Atterberg limits, field vane and Torvane shear-strength values, and percent material passing the No. 200 sieve are shown graphically. The terms and symbols used to describe the materials encountered in the borings are defined in Tables 2A through 4A and the attached legend. Rock core photographs are provided on Figures 24A through 26A.

A.1.3 Dilatometer Test Soundings

Two DMT soundings, designated DMT-1 and DMT-2, were advanced to depths of 14.4 feet and 18.4 feet, respectively, using a track-mounted rig provided and operated by Oregon Geotechnical Explorations, Inc., of Kaiser, Oregon. DMT soundings provide additional geotechnical information to characterize the subsurface materials. The DMT is performed by pushing a blade-shaped instrument into the soil. The blade is equipped with an expandable membrane on one side that is pressurized until the membrane moves horizontally into the surrounding soil. Readings of pressure required to move the membrane to a point that is flush with the blade (P_0) and to a point 1.1 millimeters into the surrounding soil (P_1) are recorded. The test sequence was performed at 8-inch intervals to obtain a comprehensive soil profile. A material index (I_D), a horizontal stress index (K_D), and a dilatometer modulus (E_D) are obtained directly from the dilatometer data. The constrained modulus (M) is then obtained from the DMT data. The terms used to describe the materials encountered in the DMT are defined in Table 4A.

DMT results are summarized on Figures 27A and 28A. The results show the dilatometer pressure readings (P_0, P_1) and three dilatometer-derived parameters: horizontal stress index (K_D), material index (I_D), and constrained modulus (M).

A.1.4 Dynamic Cone Penetration Testing

A.1.4.1 Overview

GRI completed DCP testing at 10 locations along proposed new roadways and parking lots on April 7, 2025. We advanced the DCP test probe below the existing ground surface and into the subgrade soil.

We used our Kessler Dynamic Cone Penetrometer manufactured by KSE Testing Equipment to complete the tests in general accordance with ASTM International (ASTM) D6951 by driving a ⅝-inch-diameter steel rod with a cone tip into the soil using a 17.6-pound sliding hammer dropped to a fixed height of 22.6 inches. We recorded the number of hammer drops (blows) required to drive the probe in increments of approximately 2 inches, or the penetration depth for each blow, and terminated testing at refusal of penetration or end of rod length.

A.1.4.2 Subgrade Resilient Modulus Approximation

Using the recorded test data, we plotted cumulative blows against cumulative penetration depths and visually assessed the resulting curve to delineate regions with approximately linear slopes. We then used least squares regression to calculate the slope of the approximately linear regions along the curve and used the procedures described by Chen et al. (1999) to approximate the resilient modulus for each region. Where a curve exhibited more than one slope, we used Odemark's Method of Equivalent Thickness described by Ullidtz (1998) to calculate an equivalent modulus value for the entire data set of similar soil types.

A.1.4.3 Dynamic Cone Penetration Test Results

Cumulative blows versus cumulative penetration depths are listed and presented graphically on Figures 29A to 38A in this appendix. Also shown on the figures are the approximated resilient modulus values and the equivalent modulus value if the data exhibit more than one approximately linear region for a given soil type. We approximated an average subgrade modulus value of 4,000 pounds per square inch. A summary of the approximated resilient modulus values at each test location and the average value for the project are provided in Table 1A, below.

Table 1A: APPROXIMATED SUBGRADE RESILIENT MODULUS BASED ON DCP TESTING

DCP Test Number	Approximate Subgrade Modulus, psi	Recommended Design Subgrade Modulus, psi
DCP-1	3,420	4,000
DCP-2	2,970	
DCP-3	3,360	
DCP-4	4,890	
DCP-5	4,500	
DCP-6	3,750	
DCP-7	6,460	
DCP-8	4,450	
DCP-9	3,940	
DCP-10	3,160	

Abbreviations: DCP = Dynamic Cone Penetration; psi = pounds per square inch

A.1.5 Pebble Count

A pebble count is a method used to determine the particle-size distribution of streambed or bank materials. It involves collecting representative samples of the bed materials and measuring the size of each particle, then grouping the measurements into size classes. We performed a pebble count in Rock Creek, approximately 60 feet north of the existing Rock Creek culvert. However, the stream bed material consisted of silty sand with only trace gravel. Since measurable rocks were relatively absent from the stream bed, the pebble count could not be completed according to the standard method. To obtain gradation data, we collected a grab sample of the stream bed material and returned it to our laboratory for further evaluation and gradation testing. The laboratory gradation test results are provided on Figure 41A.

A.2 LABORATORY TESTING

A.2.1 General

The samples obtained from the borings were returned to our laboratory, where the physical characteristics of the samples were noted and the field classifications modified where necessary. At the time of classification, the natural moisture content of each sample was determined. Additional testing included field vane and Torvane shear strength testing, Atterberg limits testing, grain-size analyses, dry unit weight determinations, one-dimensional consolidation testing, monotonic direct simple shear (MDSS) testing, and compaction testing. The following sections describe the testing program in more detail. The results of the testing are summarized in Table 5A.

A.2.2 Natural Moisture Contents

Natural moisture content determinations were made in general accordance with ASTM D2216. The results are summarized on Figures 1A through 23A and in Table 5A.

A.2.3 Vane Shear Strength

The approximate undrained shear strength of relatively undisturbed fine-grained soil was determined using a Torvane and/or a field vane shear device. The Torvane and field vane devices are a handheld apparatus with vanes that are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of the Torvane and/or field vane shear-strength testing are summarized on Figures 1A through 23A.

A.2.4 Atterberg Limits

Atterberg-limits testing was performed on selected samples of fine-grained soil in general accordance with ASTM D4318. The test results are summarized on the Plasticity Chart (Figures 39A and 40A), Figures 1A through 23A, and in Table 5A.

A.2.5 Grain-Size Analysis

A.2.5.1 Washed-Sieve Method

To assist in classification of the soils, samples of known dry weight were washed over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed. The percentage of material passing the No. 200 sieve is then calculated. The results are summarized on Figures 1A through 23A and in Table 5A.

A.2.5.2 Dry Sieve Method

Sieve analyses were performed on a sample of soil in general accordance with ASTM D6913. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed, and the percentage of material passing the No. 200 sieve is calculated. The soil retained on the No. 200 sieve is then screened through a series of sieves of various sizes using a sieve shaker. The weight of the soil retained on each sieve is recorded and expressed as a percentage of the total sample weight. The test data are summarized on Figure 41A.

A.2.6 Dry Unit Weight

The dry unit weight, or density, of selected undisturbed samples was determined in the laboratory in general accordance with ASTM D2937 by cutting a cylindrical specimen of soil from a Shelby tube sample. The dimensions of the specimen were carefully measured, the volume calculated, and the specimen weighed. After oven-drying, the specimen was reweighed and the water content calculated. The dry unit weight was then computed. The dry unit weight is shown on Figures 1A through 23A and is summarized in Table 5A.

A.2.7 One-Dimensional Consolidation

One-dimensional consolidation tests were performed in general accordance with ASTM D2435 on selected relatively undisturbed soil samples extruded from a Shelby tube. This test provides data on the compressibility of fine-grained soils. The test results are summarized on Figures 42A through 44A in the form of a curve showing percent strain versus applied effective stress. The initial moisture content and unit weight of the sample are also provided on the figure.

A.2.8 Monotonic Direct Simple Shear Testing

A single-stage, consolidated, undrained MDSS test with pore pressure measurements was performed in general accordance with ASTM D6528 on a relatively undisturbed soil sample extruded from a Shelby tube. The MDSS test provides data on the peak shear strength and associated shear strain of the fine-grained soil selected for testing. Results of the testing are shown on Figure 45A.

A.2.9 Compaction Testing

Laboratory compaction testing using standard effort was performed in general accordance with ASTM D698 on bulk soil samples collected in the field. The test results indicate the optimum moisture content that will result in the maximum dry density, which will be used during construction to confirm that adequate soil compaction is achieved. The test results are summarized on Figures 46A and 47A.

A.3 REFERENCES

Chen, J., Hossain, M., Latorella, T. M., 1999, Use of Falling Weight Deflectometer and Dynamic Cone Penetrometer in pavement evaluation, *Journal of the Transportation Research Board*, vol. 1655, iss. 1, pp. 145–151.

Ullidtz, P., 1998, Modelling flexible pavement response and performance: Tech Univ. of Denmark Polytekn.

Table 2A

GUIDELINES FOR DESCRIPTION OF SOIL[1]

Description of Relative Density for Cohesionless (Coarse-Grained) Soils

Relative Density	Standard Penetration Resistance (N-values) blows/foot (ft)	3-inch Sampler, 140-lb hammer approx. N-Value (blows/ft)[2]	3-inch Sampler, 300-lb hammer approx. N-Value (blows/ft)[1]
Very Loose	0 - 4	0 – 10	0 – 5
Loose	4 - 10	10 – 24	5 – 11
Medium Dense	10 - 30	24 – 73	11 – 34
Dense	30 - 50	73 – 122	34 – 57
Very Dense	over 50	over 122	over 57

Description of Relative Consistency for Cohesive (Fine-Grained) Soils

Relative Consistency	Standard Penetration Resistance (N-values) blows/ft	3-inch Sampler, 140 lb hammer approx. N-Value (blows/ft)[1]	3-inch Sampler, 300 lb hammer approx. N-Value (blows/ft)[2]	Torvane or Undrained Shear Strength, tsf
Very Soft	0 - 2	0 – 3	0 – 1	less than 0.125
Soft	2 - 4	3 – 6	1 – 3	0.125 - 0.25
Medium Stiff	4 - 8	6 – 12	3 – 6	0.25 - 0.50
Stiff	8 - 15	12 – 23	6 – 11	0.50 - 1.0
Very Stiff	15 - 30	23 – 46	11 – 22	1.0 - 2.0
Hard	30 – 60	46 – 92	22 – 43	over 2.0
Very Hard	over 60	over 92	over 43	

Grain-Size Classification

Boulders:
  >12 inches
Cobbles:
  3 inches - 12 inches
Gravel:
  ¼ inch - ¾ inch (fine)
  ¾ inch - 3 inches (coarse)
Sand:
  No. 200 - No. 40 sieve (fine)
  No. 40 - No. 10 sieve (medium)
  No. 10 - No. 4 sieve (coarse)
Silt/Clay:
  Pass No. 200 sieve

Modifier for Subclassification

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

1. Soil descriptions are developed using visual-manual procedures (ASTM D2488) and generally follow ODOT Geotechnical Design Manual (Chapter 5) guidelines.
2. Oversized sampler (OD = 3 inches, ID = 2.4 inches) blow counts converted to SPT N-Value using equations provided by Burmister, D.M., 1948, The importance and practical use of relative density in soil mechanics: Proceedings of ASTM, v. 48:1249.

Table 3A

GUIDELINES FOR CLASSIFICATION OF ROCK

Relative Rock Weathering Scale

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

Relative Rock Strength Scale

Term	Hardness Designation	Field Identification	Approximate Unconfined Compressive Strength
Extremely Weak	R0	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	35 - 150 psi
Very Weak	R1	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocketknife and scratched with fingernail.	150 - 725 psi
Weak	R2	Can be peeled by a pocketknife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.	725 – 3,500 psi
Medium Strong	R3	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.	3,500 – 7,250 psi
Strong	R4	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.	7,250 – 14,500 psi
Very Strong	R5	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	14,500 – 36,250 psi
Extremely Strong	R6	Can only be chipped with a rock hammer	>36,250 psi

RQD and Rock Quality

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

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

Table 4A

SOIL CHARACTERIZATION
BASED ON MARCHETTI FLAT PLATE DILATOMETER TEST (DMT)

Description of Relative Consistency for Cohesive (Fine-Grained) Soils

	Soil Type[(a)]	
	CH, CL	ML, MH
	DMT Constrained Modulus (M_{DMT}), tsf	
Relative Consistency	I_D[(b)] < 0.6	$0.6 < I_D$[(b)] < 1.8
Very Soft	0 - 30	0 - 50
Soft	30 - 60	50 - 100
Medium Stiff	60 - 100	100 - 200
Stiff	100 - 175	200 - 375
Very Stiff	175 +	375 +

Description of Relative Density for Cohesionless (Coarse-Grained) Soils

	Soil Type[(a)]	
	SM, SC	SP, SW
	DMT Constrained Modulus (M_{DMT}), tsf	
Relative Density	$1.8 < I_D$[(b)] < 3.3	$3.3 < I_D$[(b)]
Very Loose	0 - 75	0 - 100
Loose	75 - 150	100 - 200
Medium Dense	150 - 300	200 - 425
Dense	300 - 550	425 - 850
Very Dense	550 +	850 +

Notes:

a) Unified Soil Classification System

b) I_D = Material Index

Table 5A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-1	S-1	2.5	344.5	32	--	--	--	--	Sandy SILT
	S-2	5.0	342.0	34	--	--	--	31	Silty SAND
	S-3	7.5	339.5	35	--	--	--	--	Sandy SILT
	S-4	10.0	337.0	26	--	--	--	--	Silty SAND
	S-5	12.5	334.5	36	--	--	--	--	SILT
	S-6	15.0	332.0	27	--	--	--	--	Clayey SILT
B-2	S-1	2.5	324.5	33	--	--	--	--	SILT
	S-2	5.0	322.0	29	--	--	--	68	Sandy SILT
B-3	S-1	2.5	315.5	33	--	--	--	70	SILT
	S-2	5.5	312.5	30	--	31	11	--	Clayey SAND
	S-2	6.0	312.0	30	94	--	--	--	Clayey SAND
	S-2	6.3	311.8	30	91	--	--	37	Clayey SAND
	S-3	7.0	311.0	33	--	--	--	51	Clayey SAND
	S-4	10.0	308.0	28	--	47	26	--	Silty CLAY
	S-5	12.5	305.5	39	--	--	--	88	SILT
	S-6	15.0	303.0	58	--	--	--	--	Sandy SILT
	S-7	20.0	298.0	34	--	--	--	--	Silty GRAVEL
	S-8	25.0	293.0	33	--	--	--	--	Silty GRAVEL
B-4	S-1	2.5	315.5	31	--	--	--	--	Sandy SILT
	S-2	5.0	313.0	34	--	--	--	39	Silty SAND
	S-2b	6.0	312.0	36	--	--	--	55	Sandy SILT
B-5	S-1	2.5	330.5	24	--	--	--	--	Clayey SILT
	S-2	5.0	328.0	25	--	--	--	--	Clayey SILT
	S-3	7.5	325.5	34	--	--	--	--	SILT
	S-4	10.0	323.0	36	--	--	--	--	SILT
	S-5	12.5	320.5	33	--	--	--	--	SILT
	S-6	15.0	318.0	37	--	--	--	--	Silty CLAY
B-6	S-1	2.5	339.5	34	--	--	--	--	Clayey SILT
	S-2	5.0	337.0	28	--	--	--	--	Clayey SILT
	S-3	7.5	334.5	34	--	--	--	--	Sandy SILT
	S-4	10.0	332.0	28	--	--	--	--	Sandy SILT
	S-5	12.5	329.5	40	--	--	--	--	SILT
	S-6	15.0	327.0	36	--	--	--	--	CLAY
B-7	S-1	2.5	337.5	35	--	--	--	--	Clayey SILT
	S-2	5.0	335.0	22	--	--	--	--	SILT
	S-3	7.5	332.5	39	--	--	--	61	Sandy SILT
	S-4	10.0	330.0	34	--	--	--	--	SILT
	S-5	12.5	327.5	39	--	--	--	--	Clayey SILT
	S-6	15.0	325.0	25	--	--	--	--	SILT
B-8	S-1	2.5	355.5	21	--	--	--	--	Silty CLAY

Table 5A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	**Sample**	**Depth, ft**	**Elevation, ft**	**Moisture Content, %**	**Dry Unit Weight, pcf**	**Liquid Limit, %**	**Plasticity Index, %**	**Fines Content, %**	**Soil Type**
B-8	S-2	5.0	353.0	25	--	--	--	--	Clayey SILT
	S-3	7.5	350.5	26	--	--	--	--	Clayey SILT
	S-4	10.0	348.0	27	--	--	--	--	Silty CLAY
	S-5	12.5	345.5	23	--	--	--	--	Silty CLAY
	S-6	15.0	343.0	42	--	--	--	--	Clayey SILT
B-9	S-1	2.5	381.5	31	--	--	--	--	SILT
	S-2	5.0	379.0	27	--	--	--	--	Clayey SILT
	S-3	7.5	376.5	28	--	--	--	--	Clayey SILT
	S-4	10.0	374.0	31	--	--	--	--	Clayey SILT
	S-5	12.5	371.5	23	--	--	--	--	SILT
	S-6	15.0	369.0	30	--	--	--	--	CLAY
B-10	S-1	2.5	357.5	28	--	--	--	--	Clayey SILT
	S-2	5.0	355.0	27	--	--	--	85	Clayey SILT
B-11	S-1	2.5	341.5	25	--	--	--	--	SILT
	S-2	5.0	339.0	23	--	--	--	--	Silty CLAY
	S-3	7.5	336.5	28	--	--	--	--	Clayey SILT
	S-4	10.0	334.0	32	--	--	--	51	Sandy SILT
	S-5	12.5	331.5	29	--	--	--	--	SILT
	S-6	15.0	329.0	34	--	--	--	--	SILT
B-12	S-1	2.5	349.5	23	--	--	--	--	SILT
	S-2	5.0	347.0	27	--	--	--	--	Clayey SILT
	S-3	7.5	344.5	26	--	--	--	--	Silty CLAY
	S-4	10.0	342.0	26	--	--	--	--	Silty CLAY
	S-5	12.5	339.5	24	--	--	--	--	Silty SAND
	S-6	15.0	337.0	26	--	--	--	--	SILT
B-13	S-1	2.5	366.5	30	--	--	--	--	SILT
	S-2	5.0	364.0	24	--	--	--	--	SILT
	S-3	7.5	361.5	26	--	--	--	--	Clayey SILT
	S-4	10.0	359.0	26	--	--	--	--	Clayey SILT
	S-5	12.5	356.5	25	--	--	--	--	Clayey SILT
	S-6	15.0	354.0	27	--	--	--	--	Silty CLAY
B-14	S-1	2.5	378.5	30	--	--	--	--	SILT
	S-2	5.0	376.0	25	--	--	--	--	Silty CLAY
	B-1	5.1	375.9	25	--	--	--	93	Silty CLAY
	S-3	7.5	373.5	27	--	--	--	--	Silty CLAY
	S-4	10.0	371.0	26	--	--	--	--	Silty CLAY
	S-5	12.5	368.5	23	--	--	--	--	Silty CLAY
	S-6	15.0	366.0	24	--	--	--	--	Silty CLAY
B-15	S-1	2.5	376.5	30	--	--	--	--	SILT
	S-2	5.0	374.0	24	--	--	--	--	Clayey SILT

Table 5A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-15	S-3	7.5	371.5	26	--	--	--	--	Clayey SILT
	S-4	10.0	369.0	29	--	--	--	--	Silty CLAY
	S-5	12.5	366.5	23	--	--	--	--	Clayey SILT
	S-6	15.0	364.0	23	--	--	--	--	Clayey SILT
B-16	S-2	4.5	344.5	24	--	--	--	--	Clayey SILT
	S-3	7.5	341.5	30	--	60	38	--	CLAY
	S-5	11.3	337.8	12	--	--	--	59	Sandy SILT
	S-6	15.0	334.0	21	--	--	--	--	Sandy SILT
	S-7	20.0	329.0	19	--	--	--	--	Sandy SILT
	S-8	25.0	324.0	19	--	--	--	--	Sandy SILT
	S-9	30.0	319.0	37	--	--	--	89	SILT
	S-10	35.0	314.0	32	--	--	--	--	SILT
	S-11	40.0	309.0	42	--	--	--	--	SILT
	S-12	45.0	304.0	32	--	--	--	--	Clayey SILT
	S-13	50.0	299.0	47	--	--	--	--	SILT
B-17	S-1	2.5	363.5	33	--	44	19	--	Silty CLAY
	S-2	5.0	361.0	29	--	--	--	--	Silty CLAY
	S-3	8.0	358.0	28	--	--	--	--	Silty CLAY
	S-4	10.0	356.0	27	--	--	--	--	Silty CLAY
	S-5	12.5	353.5	30	--	63	43	99	CLAY
	S-6	15.5	350.5	23	103	--	--	--	CLAY
	S-7	17.0	349.0	22	--	--	--	--	CLAY
	S-8	20.0	346.0	28	--	--	--	--	Clayey SILT
	S-9	25.0	341.0	28	--	--	--	31	Silty SAND
	S-10	30.0	336.0	24	--	--	--	--	Silty SAND
	S-12	40.0	326.0	20	--	--	--	--	Silty SAND
	S-13	45.0	321.0	18	--	--	--	--	Silty SAND
	S-14	50.0	316.0	17	--	--	--	--	Silty SAND
	S-15	55.0	311.0	19	--	--	--	--	Silty SAND
	S-16	60.0	306.0	18	--	--	--	--	Silty SAND
	S-17	65.0	301.0	35	--	--	--	--	SILT
	S-18	70.0	296.0	36	--	--	--	--	SILT
B-18	S-1	2.5	353.5	32	--	--	--	86	SILT
	S-2	5.0	351.0	28	--	--	--	--	Silty CLAY
	S-3	8.0	348.0	34	90	65	43	99	CLAY
	S-3	8.4	347.6	34	87	--	--	--	CLAY
	S-4	9.5	346.5	30	--	--	--	--	CLAY
	S-5	13.0	343.0	34	88	62	37	99	CLAY
	S-5	13.4	342.6	40	80	--	--	--	CLAY
	S-6	14.5	341.5	30	--	--	--	72	SILT

Table 5A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-18	S-7	20.0	336.0	20	--	--	--	22	Silty SAND
	S-8	25.0	331.0	20	--	--	--	--	Silty SAND
	S-9	30.0	326.0	23	--	--	--	--	Silty SAND
	S-10	35.0	321.0	31	--	--	--	--	SILT
	S-11	40.0	316.0	24	--	--	--	--	Sandy SILT
	S-12	45.0	311.0	29	--	--	--	--	SILT
	S-13	50.0	306.0	43	--	--	--	--	SILT
B-19	S-1	2.5	359.5	29	--	--	--	--	SILT
	S-2	5.5	356.5	22	107	46	25	--	Silty CLAY
	S-3	7.0	355.0	27	--	--	--	--	Silty CLAY
	S-4	10.0	352.0	28	--	--	--	94	Silty CLAY
	S-6	14.5	347.5	29	--	--	--	--	CLAY
	S-7	20.0	342.0	22	--	--	--	26	Silty SAND
	S-8	25.0	337.0	19	--	--	--	--	Sandy SILT
	S-9	30.0	332.0	20	--	--	--	--	Sandy SILT
	S-10	35.0	327.0	20	--	--	--	--	Sandy SILT
	S-11	40.0	322.0	20	--	--	--	--	Sandy SILT
	S-12	45.0	317.0	18	--	--	--	--	Sandy SILT
	S-13	50.0	312.0	18	--	--	--	--	Sandy SILT
B-20	S-1	5.0	342.0	35	--	--	--	99	Silty CLAY
B-21	S-1	2.5	355.5	29	--	--	--	--	SILT
	S-2	5.0	353.0	28	--	--	--	--	SILT
	S-3	7.5	350.5	26	--	--	--	--	SILT
	S-4	10.0	348.0	28	--	--	--	--	Silty CLAY
	S-5	12.5	345.5	32	--	--	--	--	Silty CLAY
	S-6	15.0	343.0	21	--	--	--	23	Silty SAND
B-22	S-1	2.5	370.5	33	--	--	--	--	Silty CLAY
	B-1	3.0	370.0	28	--	41	18	83	Silty CLAY
	S-2	5.0	368.0	29	--	--	--	--	Silty CLAY
	S-3	7.5	365.5	28	--	--	--	--	Silty CLAY
	S-4	10.0	363.0	28	--	--	--	--	Silty CLAY
	S-5	12.5	360.5	25	--	--	--	--	Silty CLAY
	S-6	15.0	358.0	24	--	--	--	--	SILT
B-23	S-1	2.5	365.5	31	--	--	--	--	SILT
	S-2	5.0	363.0	26	--	--	--	--	Clayey SILT
	S-3	7.5	360.5	26	--	--	--	91	Clayey SILT
	S-4	10.0	358.0	26	--	--	--	--	Silty CLAY
	S-5	12.5	355.5	28	--	--	--	--	Silty CLAY
	S-6	15.0	353.0	22	--	--	--	--	Clayey SILT
PC-1	S-1	0.0	312.0	35	--	--	--	25	Silty SAND

BORING AND TEST PIT LOG LEGEND

SOIL SYMBOLS

Symbol	Typical Description
⁘	LANDSCAPE MATERIALS
▩	FILL
○	GRAVEL; clean to some silt, clay, and sand
○	Sandy GRAVEL; clean to some silt and clay
○	Silty GRAVEL; up to some clay and sand
○	Clayey GRAVEL; up to some silt and sand
░	SAND; clean to some silt, clay, and gravel
░	Gravelly SAND; clean to some silt and clay
░	Silty SAND; up to some clay and gravel
░	Clayey SAND; up to some silt and gravel
▥	SILT; up to some clay, sand, and gravel
▥	Gravelly SILT; up to some clay and sand
▥	Sandy SILT; up to some clay and gravel
▥	Clayey SILT; up to some sand and gravel
▨	CLAY; up to some silt, sand, and gravel
▨	Gravelly CLAY; up to some silt and sand
▨	Sandy CLAY; up to some silt and gravel
▨	Silty CLAY; up to some sand and gravel
≋	PEAT

BEDROCK SYMBOLS

Symbol	Typical Description
✚	BASALT
▤	MUDSTONE
▤	SILTSTONE
▤	SANDSTONE

SURFACE MATERIAL SYMBOLS

Symbol	Typical Description
■	Asphalt concrete PAVEMENT
▩	Portland cement concrete PAVEMENT
○	Crushed rock BASE COURSE

SAMPLER SYMBOLS

Symbol	Sampler Description
▮	2.0 in. O.D. split-spoon sampler and Standard Penetration Test with recovery (ASTM D1586)
▮	Shelby tube sampler with recovery (ASTM D1587)
▮	3.0 in. O.D. split-spoon sampler with recovery (ASTM D3550)
⊠	Grab Sample
▯	Rock core sample interval
▮	Sonic core sample interval
▯	Push probe sample interval

INSTALLATION SYMBOLS

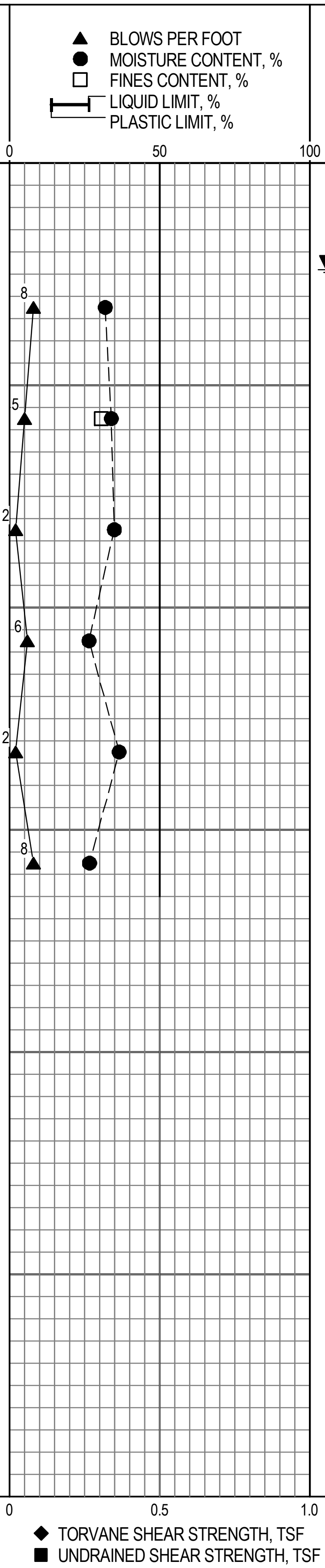
Symbol	Symbol Description
◫	Flush-mount monument set in concrete
◫	Concrete, well casing shown where applicable
▨	Bentonite seal, well casing shown if applicable
░	Filter pack, machine-slotted well casing shown where applicable
◫	Grout, vibrating-wire transducer cable shown where applicable
Ⓟ	Vibrating-wire pressure transducer
◫	1-in.-diameter solid PVC
⋮	1-in.-diameter hand-slotted PVC
◫	Grout, inclinometer casing shown where applicable

FIELD MEASUREMENTS

Symbol	Typical Description
▽	Groundwater level during drilling and date measured
▼	Groundwater level after drilling and date measured
▨	Rock/sonic core or push probe recovery (%)
▨	Rock quality designation (RQD, %)

Surface Elevation: 347.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Sandy SILT, contains organics; ML; brown to red-brown and gray; medium plasticity; moist; medium stiff to stiff; fine sand; organics consist of fine roots; micaceous (Willamette Silt)				Surface: grass
					▼ 2.4 ft (4/16/2025)
			S-1	1 4 4	
5	Silty SAND; SM; brown to red-brown; low-plasticity fines; moist; loose; fine to coarse sand; micaceous (Willamette Silt)	342.0 5.0	S-2	1 2 3	
	Sandy SILT; ML; bluish gray to dark gray; medium plasticity; wet; soft; fine sand (Willamette Silt)	339.5 7.5	S-3	0 1 1	
10	Silty SAND; SM; gray; low-plasticity fines; moist to wet; loose; fine sand (Willamette Silt)	337.0 10.0	S-4	1 3 3	
	SILT, trace sand; ML; gray; medium plasticity; moist; soft; fine sand (Willamette Silt)	334.5 12.5	S-5	0 1 1	
15	Clayey SILT, trace sand; MH; blue-gray; medium to high plasticity; moist; medium stiff to stiff; fine sand (Willamette Silt)	332.0 15.0	S-6	2 4 4	
	(4/16/2025)	330.5 16.5			
20					
25					
30					



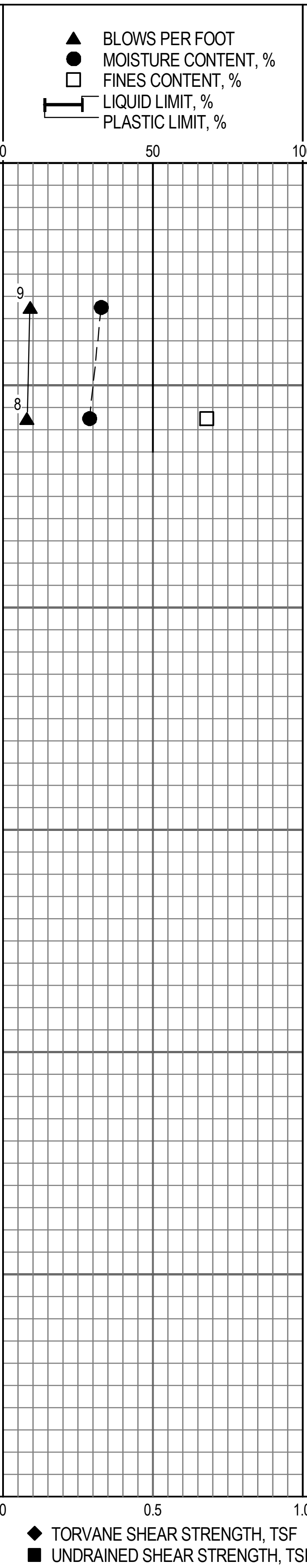
Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/15/25	**GPS Coordinates:** 45.440736° N 122.484647° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 4 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

GRI

BORING B-1

OCT. 2025 JOB NO. 7072-A FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** 327.0 ft [±] (NAVD 88)	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
0		SILT, some sand; ML; brown, red-brown, gray, and orange; low to medium plasticity; moist; stiff; fine to coarse sand (Willamette Silt)						Surface: dirt road ▼2.3 ft (4/16/2025)
					S-1	I	3 5 4	
5		Sandy SILT, trace gravel; ML; brown and gray; low plasticity; moist; medium stiff to stiff; fine to coarse sand; subrounded gravel; micaceous (Willamette Silt)	322.0 5.0		S-2	I	3 4 4	Performed infiltration test at 5 feet
		(4/16/2025)	320.5 6.5					
10								
15								
20								
25								
30								



GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 10/6/25

Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/15/25	**GPS Coordinates:** 45.440707° N 122.483444° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 6 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

GRI

BORING B-2

OCT. 2025 JOB NO. 7072-A FIG. 2A

Surface Elevation: 318.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
0	SILT, some sand to sandy, contains organics; ML; brown, gray, orange; medium plasticity; wet; soft; fine to coarse sand; organics consist of fine roots and wood fibers (Possible Willamette Silt)			S-1		0 1 2	Surface: dirt road
5	Clayey SAND, trace gravel, contains organics; SC; brown mottled red, black, and yellow; low-plasticity fines; moist; soft; fine to coarse sand; subangular to rounded gravel; organics consist of charred wood and fine roots; contains layers of sandy, silty CLAY with trace gravel (CL) (Possible Willamette Silt)	313.0 5.0		S-2			Shelby tube pushed at 0 psi Dry Density = 91 pcf Field vane = 1.7 tsf
	Sandy SILT, trace gravel; ML; brown, red-brown, and dark gray; medium plasticity; moist; very stiff; fine to coarse sand; angular to subangular gravel (Possible Willamette Silt)	310.5 7.5		S-3		3 7 11	
10	Silty CLAY, trace sand; CL; orange-brown; medium plasticity; moist; very stiff; fine to coarse sand (Willamette Silt)	308.0 10.0		S-4		5 8 11	
	SILT, trace sand and gravel; ML; red-brown with scattered teal and dark gray gravel; medium plasticity; moist; very stiff; fine to coarse sand; subangular to subrounded gravel (Willamette Silt)	305.5 12.5		S-5		6 9 9	
15	Sandy SILT, trace gravel; ML; red-brown; low plasticity; moist; very stiff; fine to coarse sand; subangular gravel (Decomposed Boring Lava Basalt)	303.0 15.0		S-6		5 9 11	
20	Silty GRAVEL, trace to some sand; GM; dark gray and teal; nonplastic fines; moist; very dense; fine to coarse sand; angular to subangular gravel (Decomposed Boring Lava Basalt)	298.0 20.0		S-7		50/5.5"	Slower drilling below 20 feet. Driller notes consistent drilling from 20 feet to 35 feet
25				S-8		50/4"	
30		288.0					

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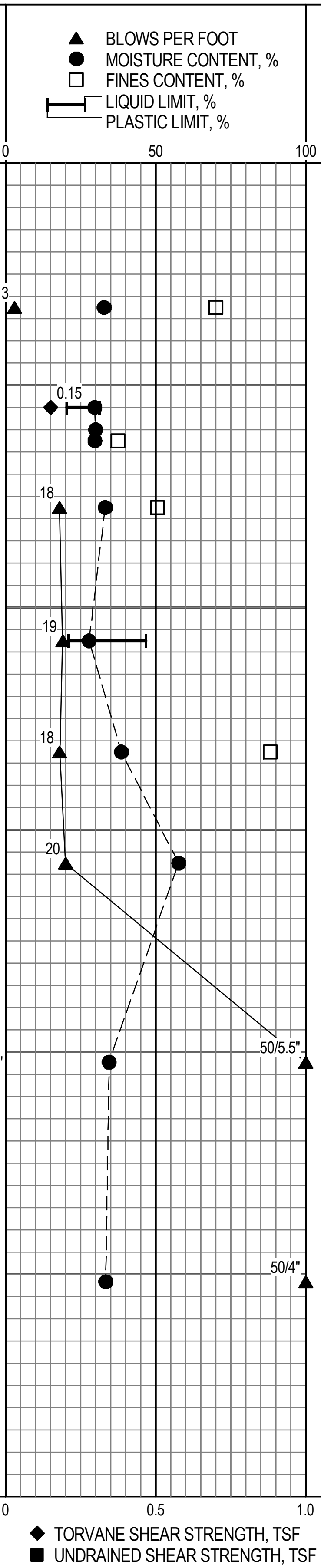


Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/14/25	**GPS Coordinates:** 45.44067° N 122.482713° W (WGS 84)
Drilling Method: Mud Rotary **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

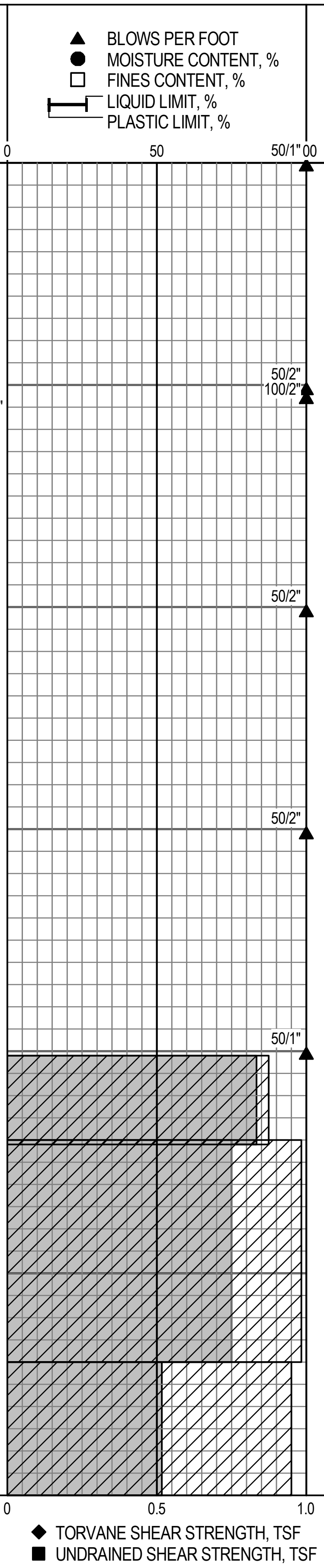
GRI

BORING B-3

OCT. 2025 JOB NO. 7072-A FIG. 3A

Surface Elevation: 318.0 ft [±] (NAVD 88)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	+++	Gravel-sized ROCK FRAGMENTS of Boring Lava Basalt	30.0		S-9		50/1"	
35	+++				S-10 S-11		50/2" 100/2"	
40	+++				S-12		50/2"	
45	+++				S-13		50/2"	
50	+++	BASALT; some very small vesicles; dark gray; slightly weathered; medium hard (R3); close to moderately close joint spacing (Boring Lava Basalt)	268.0 50.0		S-14		50/1"	Stopped drilling for the day at 50 feet. Switched to rock coring on 4/15/2025
	+++				Run-1			
55	+++				Run-2			
60	+++		258.0		Run-3			



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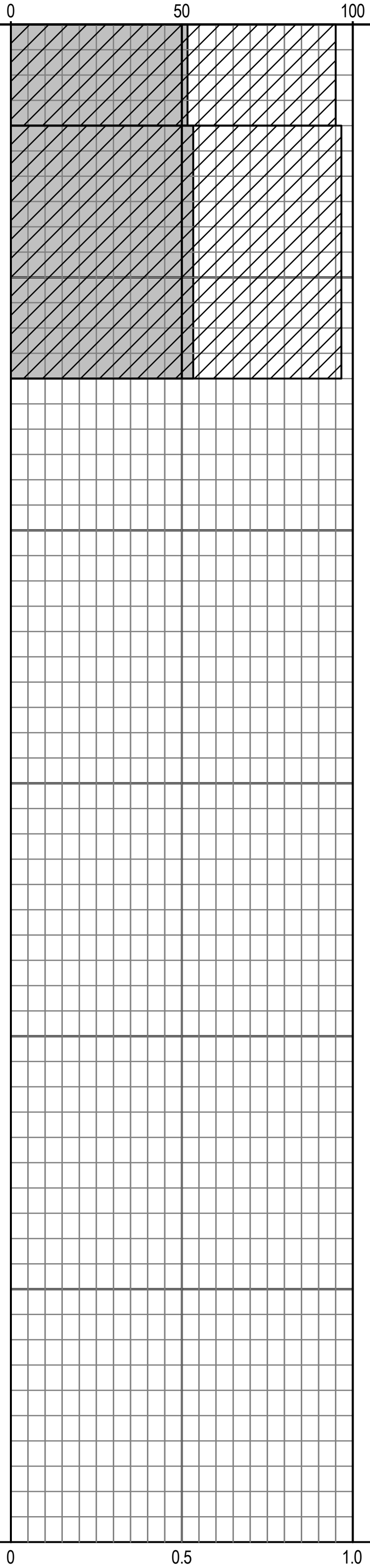
DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** 318.0 ft [±] (NAVD 88)	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
65	+++	BASALT; highly vesicular; dark gray; moderately weathered (yellow, brown, and teal coloring); medium hard (R3) very close to close joint spacing (Boring Lava Basalt) --- some very small vesicles; slightly weathered; close to moderately close joint spacing below 64 feet	60.0		Run-4			
70 75 80 85 90		(4/15/2025)	251.0 67.0					

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %
☐ FINES CONTENT, %
LIQUID LIMIT, %
PLASTIC LIMIT, %



◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 10/6/25

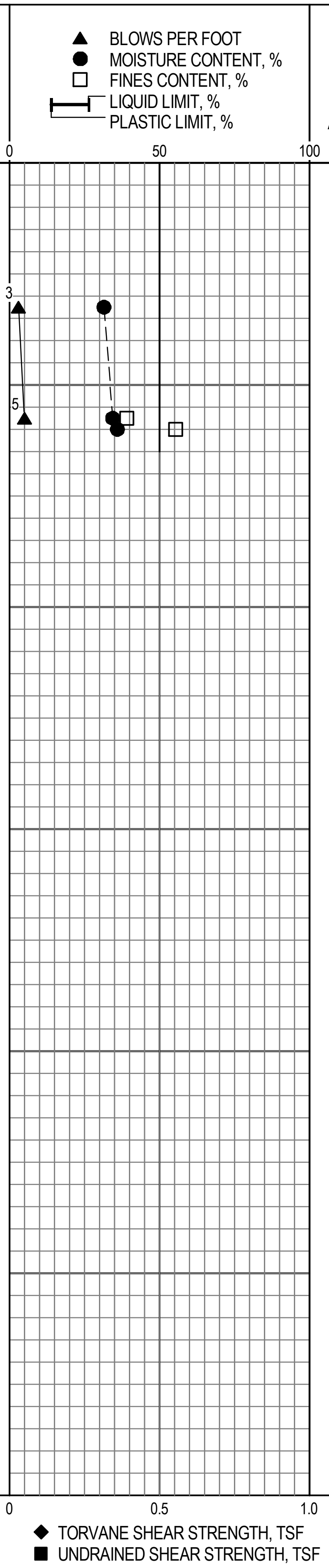
GRI

BORING B-3

OCT. 2025 JOB NO. 7072-A FIG. 3A

Surface Elevation: 318.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
0	Sandy SILT, contains organics; ML; brown, gray, and orange; low plasticity; wet; soft; fine sand; organics consist of roots and grasses mostly concentrated at top of sample (Willamette Silt)						Surface: dirt road
				S-1		0 2 1	
5	Silty SAND; SM; brown, gray, and orange; low-plasticity fines; wet; loose; fine sand (Willamette Silt)	313.0 5.0		S-2		0 1 4	Performed infiltration test at 5 feet
	Sandy SILT, trace gravel; ML; red-brown, black, and gray; low-plasticity; moist; medium stiff; fine to coarse sand; subangular to subrounded gravel (Possible Willamette Silt) (4/15/2025)	311.5 6.5					



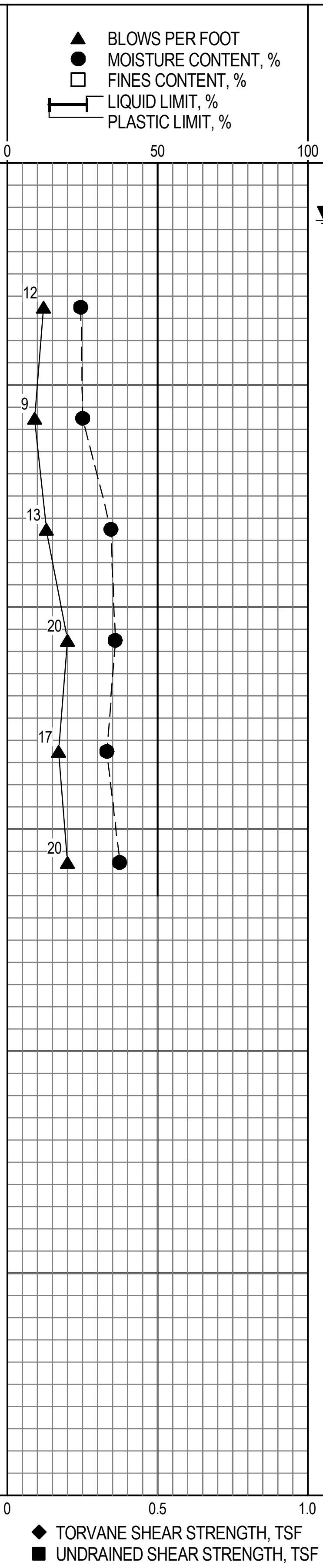
Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/14/25	**GPS Coordinates:** 45.440679° N 122.482627° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 6 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

GRI

BORING B-4

OCT. 2025 JOB NO. 7072-A FIG. 4A

DEPTH, FT	CLASSIFICATION OF MATERIAL **Surface Elevation:** 333.0 ft [±] (NAVD 88)	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Clayey SILT, trace sand, contains organics; MH; brown and gray; medium plasticity; moist; stiff; fine sand; organics consist of fine roots; 15- to 24-inch-thick rooted zone at ground surface (Willamette Silt)					Surface: grass ▼1.3 ft (4/14/2025)
				S-1	3 6 6	
5	--- red-brown; gray, dark gray/black; organics absent below 5 feet			S-2	3 5 4	
	SILT, trace sand; ML; gray; low plasticity; moist; stiff; fine sand (Willamette Silt)	325.5 7.5		S-3	6 6 7	
10	--- trace gravel; red-brown, gray, and brown; moist to wet; very stiff; fine to coarse sand; angular gravel; micaceous below 10 feet			S-4	6 8 12	
	--- light brown; fine sand below 12.5 feet			S-5	4 7 10	
15	Silty CLAY; CL; gray and orange; medium to high plasticity; moist; very stiff (Willamette Silt)	318.0 15.0		S-6	5 9 11	
	(4/11/2025)	316.5 16.5				
20						
25						
30						

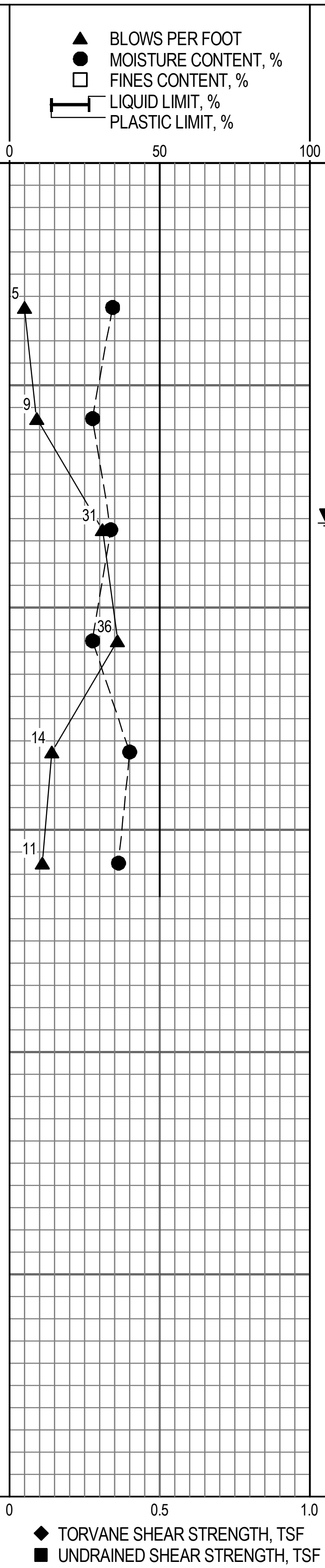


Logged By: A. Horst		**Drilled by:** Western States Soil Conservation, Inc.	
Date Started: 4/11/25	**GPS Coordinates:** 45.440725° N 122.481558° W (WGS 84)		
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 4 in.		**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828	
Note: See Legend for Explanation of Symbols			

BORING B-5

OCT. 2025 JOB NO. 7072-A FIG. 5A

DEPTH, FT	CLASSIFICATION OF MATERIAL Surface Elevation: 342.0 ft [±] (NAVD 88)	ELEVATION, FT DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Clayey SILT; MH; brown and gray; high plasticity; moist; medium stiff; 18- to 24-inch-thick rooted zone at ground surface (Willamette Silt)						Surface: grass
				S-1		1 2 3	
5	--- moist to wet; stiff below 5 feet			S-2		2 4 5	
	Sandy SILT; ML; gray, brown, black; low plasticity; damp to moist; hard; fine to coarse sand (Willamette Silt)	334.5 7.5		S-3		6 14 17	▼8.1 ft (4/14/2025)
10	--- red/orange-brown below 10 feet			S-4		8 19 17	
	SILT, some sand; ML; gray and brown; medium plasticity; moist; stiff; fine sand (Willamette Silt)	329.5 12.5		S-5		3 6 8	
15	CLAY; CH; gray and dark brown; high plasticity; moist; stiff (Willamette Silt)	327.0 15.0		S-6		4 5 6	
	(4/11/2025)	325.5 16.5					
20							
25							
30							



Logged By: A. Horst		**Drilled by:** Western States Soil Conservation, Inc.	
Date Started: 4/11/25	**GPS Coordinates:** 45.440727° N 122.480477° W (WGS 84)		
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 4 in.		**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828	
Note: See Legend for Explanation of Symbols			

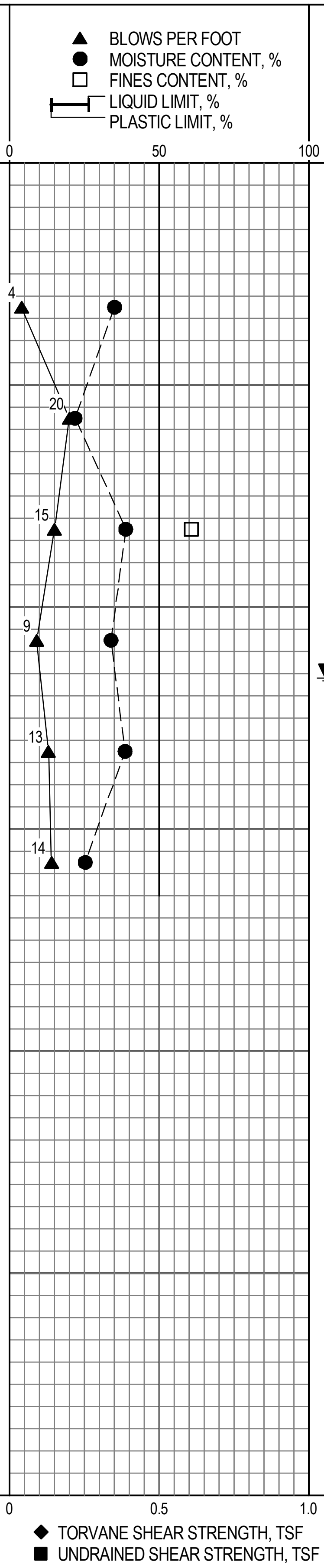
GRI

BORING B-6

OCT. 2025 JOB NO. 7072-A FIG. 6A

Surface Elevation: 340.0 ft [±] (NAVD 88)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
0		Clayey SILT, trace sand and gravel; MH; brown-gray; high plasticity; moist; soft to medium stiff; fine to coarse sand; subangular to subrounded gravel; 18-inch-thick rooted zone at ground surface (Willamette Silt)			S-1		1 2 2	Surface: grass
5		SILT, some sand; ML; brown; low to medium plasticity; damp to moist; very stiff; fine to coarse sand (Willamette Silt)	335.0 5.0		S-2		5 9 11	
		Sandy SILT; ML; brown, orange, red-brown, and black; low to medium plasticity; moist; stiff to very stiff; fine to coarse sand (Willamette Silt)	332.5 7.5		S-3		4 7 8	
10		SILT, trace sand; ML; light gray; medium plasticity; moist; stiff; fine sand (Willamette Silt)	330.0 10.0		S-4		3 4 5	▼11.6 ft (4/11/2025)
		Clayey SILT, trace sand; MH; gray, orange, and brown; medium plasticity; moist; stiff; fine to coarse sand (Willamette Silt)	327.5 12.5		S-5		3 6 7	
15		SILT; ML; light gray; medium plasticity; moist; stiff (Willamette Silt)	325.0 15.0		S-6		5 7 7	
		(4/10/2025)	323.5 16.5					
20								
25								
30								



Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/10/25	**GPS Coordinates:** 45.442822° N 122.480211° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 4 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

GRI

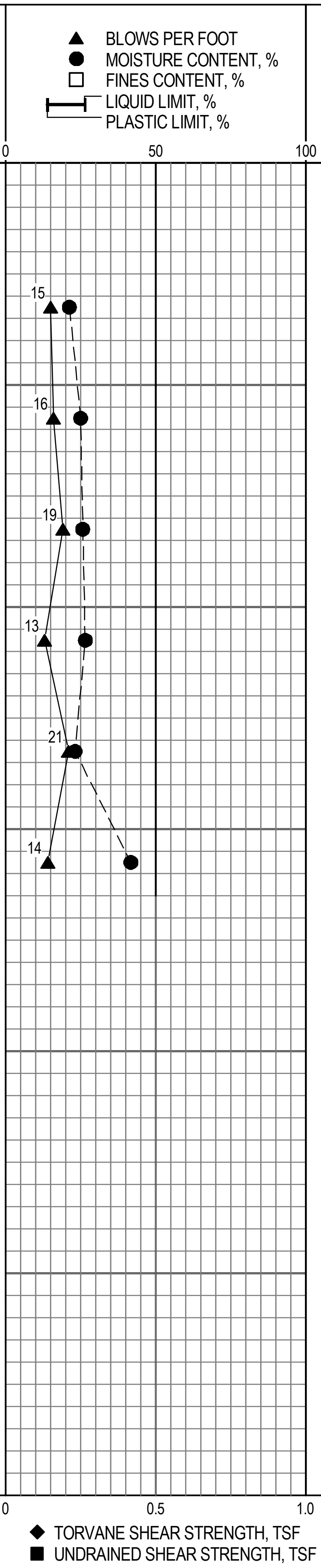
BORING B-7

OCT. 2025 JOB NO. 7072-A FIG. 7A

GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 10/6/25

Surface Elevation: 358.0 ft [±] (NAVD 88)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Silty CLAY, trace sand; CL; brown, orange, and gray; medium to high plasticity; moist; stiff to very stiff; fine sand; 6-inch-thick rooted zone at ground surface (Willamette Silt)						Surface: grass
					S-1		3 7 8	
5		Clayey SILT, trace to some sand; MH; brown and gray; medium plasticity; moist; very stiff; fine to coarse sand (Willamette Silt)	353.0 5.0					
					S-2		4 7 9	
		--- contains black silty/sandy veins below 7.5 feet						
					S-3		3 8 11	
10		Silty CLAY; CL; dark brown; medium to high plasticity; moist; stiff (Willamette Silt)	348.0 10.0					
					S-4		2 6 7	
		--- gray to brown; very stiff below 12.5 feet						
					S-5		4 8 13	
15		Clayey SILT; MH; gray with some black veins and spots; medium plasticity; moist; stiff (Willamette Silt)	343.0 15.0					
					S-6		4 7 7	
		(4/10/2025)	341.5 16.5					
20								
25								
30								



Logged By: A. Horst		**Drilled by:** Western States Soil Conservation, Inc.	
Date Started: 4/10/25	**GPS Coordinates:** 45.442918° N 122.478102° W (WGS 84)		
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 4 in.		**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828	
Note: See Legend for Explanation of Symbols			

GRI

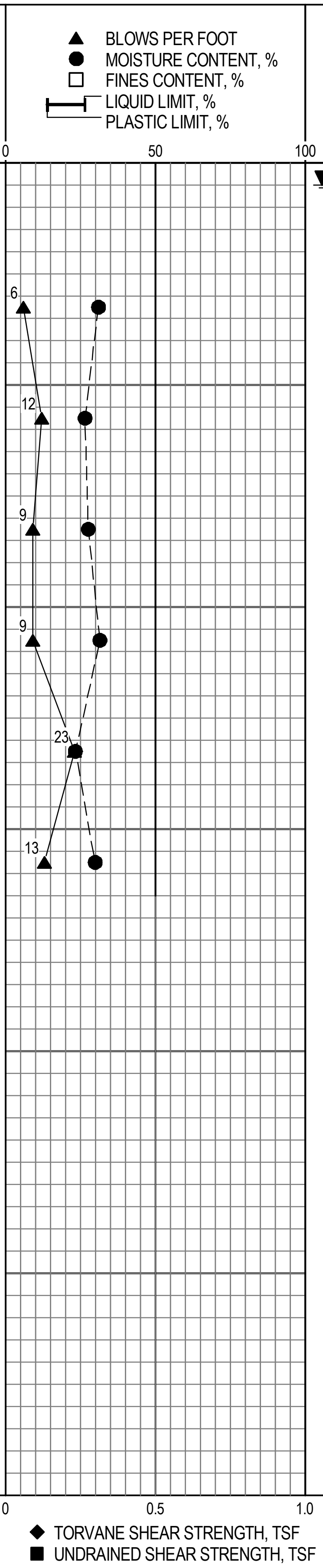
BORING B-8

OCT. 2025 JOB NO. 7072-A FIG. 8A

GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 10/6/25

Surface Elevation: 384.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	SILT, trace sand; contains organics; ML; brown and orange; medium plasticity; moist; medium stiff; fine sand; organics consist of roots and pieces of wood; 24-inch-thick rooted zone at ground surface (Willamette Silt)					▼0.5 ft (4/11/2025) Surface: grass
				S-1	1 3 3	
5	Clayey SILT, trace sand; MH; brown and orange; medium to high plasticity; moist; stiff; fine sand (Willamette Silt)	379.0 5.0				
				S-2	3 6 6	
	--- brown; medium plasticity below 7.5 feet					
				S-3	3 4 5	
10	--- orange and gray below 10 feet					
				S-4	3 4 5	
	SILT, trace sand; ML; brown and gray; low to medium plasticity; moist; very stiff; fine to coarse sand (Willamette Silt)	371.5 12.5				
				S-5	5 10 13	
15	CLAY; CH; gray and orange-brown; high plasticity; moist; stiff (Willamette Silt)	369.0 15.0		S-6	3 5 8	
	(4/10/2025)	367.5 16.5				
20						
25						
30						

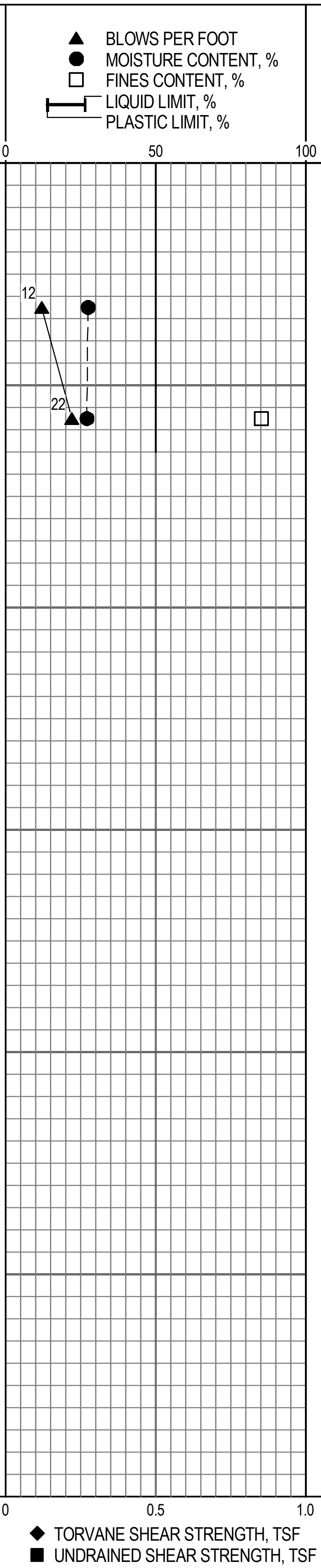


Logged By: A. Horst **Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/10/25 **GPS Coordinates:** 45.442927° N 122.475829° W (WGS 84)
Drilling Method: Hollow-Stem Auger
Equipment: CME 850 Track-Mounted Drill Rig
Hole Diameter: 4 in.
Note: See Legend for Explanation of Symbols
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: 0.828

GRI

BORING B-9

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL Surface Elevation: 360.0 ft [±] (NAVD 88)	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Clayey SILT; MH; brown; medium plasticity; moist; stiff; 8-inch-thick rooted zone at ground surface (Willamette Silt)						Surface: grass
					S-1	I	1 3 9	
5		--- trace sand and gravel; brown and gray; very stiff; fine to coarse sand; subangular to subrounded gravel below 5 feet			S-2	I	5 10 12	Performed infiltration test at 5 feet
		(4/10/2025)	353.5 6.5					
10								
15								
20								
25								
30								



Logged By: A. Horst | **Drilled by:** Western States Soil Conservation, Inc.

Date Started: 4/9/25 | **GPS Coordinates:** 45.44278° N 122.478083° W (WGS 84)

Drilling Method: Hollow-Stem Auger
Equipment: CME 850 Track-Mounted Drill Rig
Hole Diameter: 6 in.

Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: 0.828

Note: See Legend for Explanation of Symbols

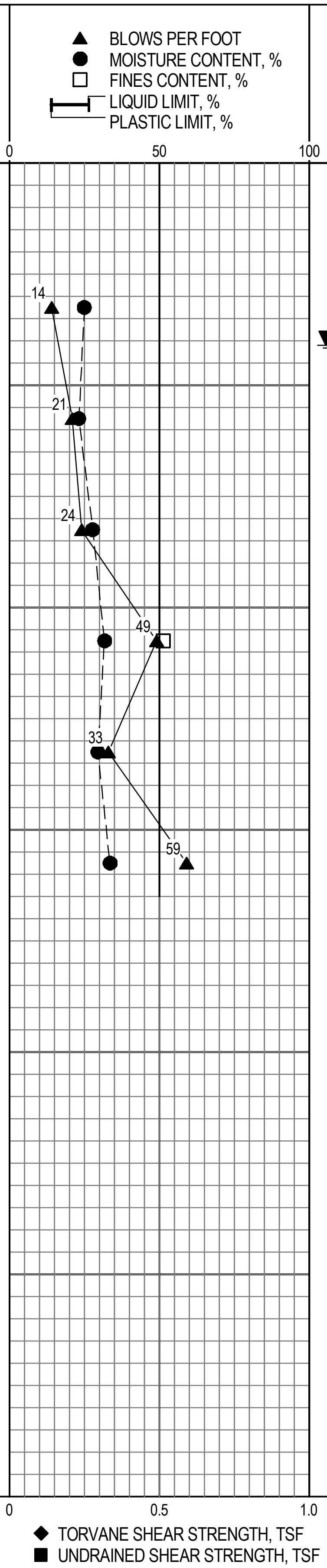
GRI

BORING B-10

OCT. 2025 JOB NO. 7072-A FIG. 10A

Surface Elevation: 344.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
0	SILT; ML; brown and gray; medium plasticity; moist; stiff; 10- to 12-inch-thick rooted zone at ground surface (Willamette Silt)					Surface: grass
				S-1	4 6 8	▼ 4.1 ft (4/14/2025)
5	Silty CLAY, trace sand; CL; dark gray, some brown; medium to high plasticity; moist; very stiff; fine to coarse sand (Willamette Silt)	339.0 5.0		S-2	5 9 12	
	Clayey SILT; MH; brown and gray; medium plasticity; damp to moist; very stiff (Possible Springwater Formation)	336.5 7.5		S-3	4 9 15	
10	Sandy SILT, trace gravel; ML; brown and dark gray; nonplastic to low plasticity; damp to moist; hard; fine to coarse sand; angular to subangular gravel (Possible Springwater Formation)	334.0 10.0		S-4	7 19 30	
	SILT, some sand; ML; brown; low to medium plasticity; damp to moist; hard; fine to coarse sand (Possible Springwater Formation)	331.5 12.5		S-5	8 15 18	
15	--- light brown to tan; low plasticity; moist below 15 feet			S-6	8 19 40	
	(4/10/2025)	327.5 16.5				
20						
25						
30						



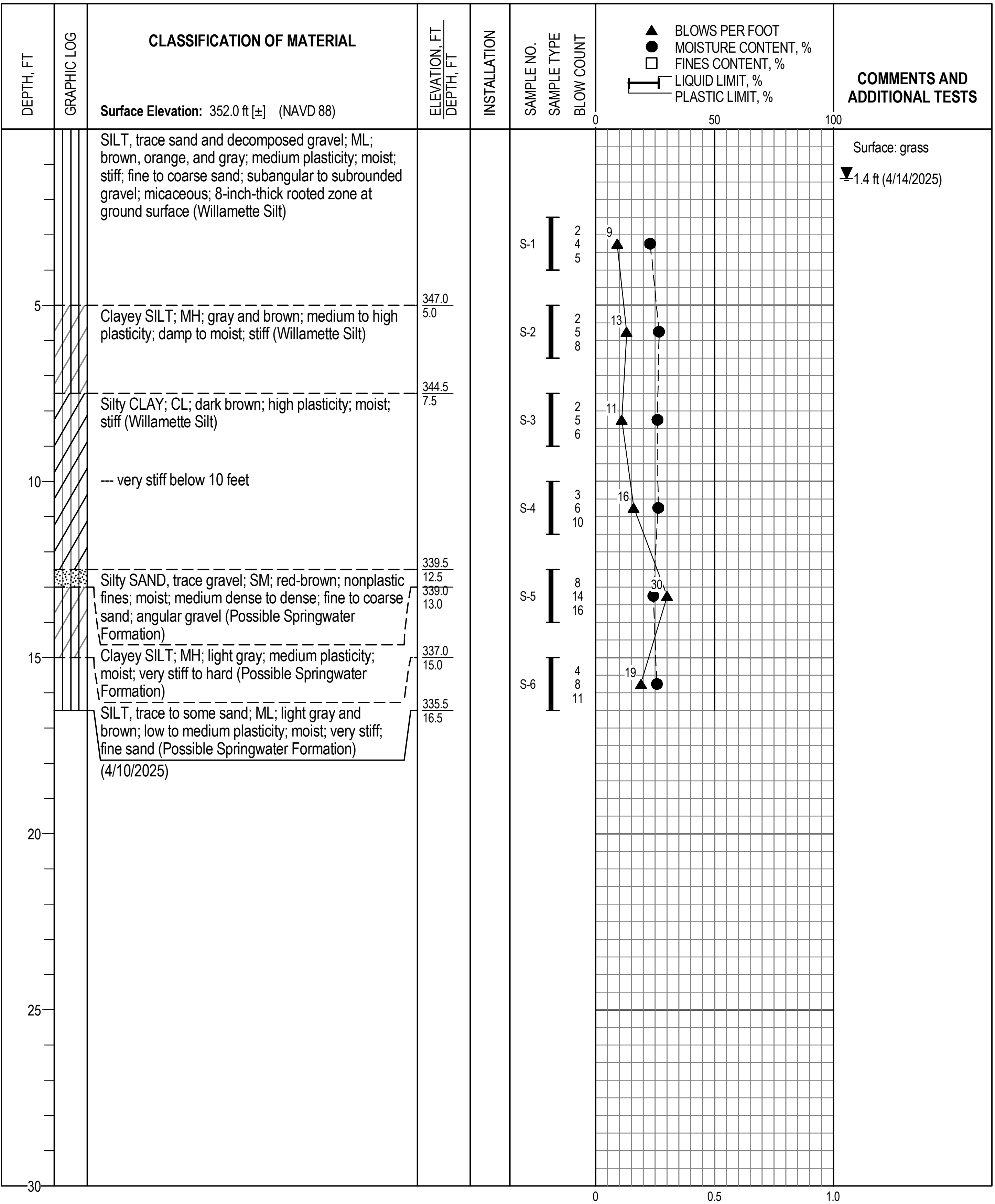
Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/10/25	**GPS Coordinates:** 45.441569° N 122.479763° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 4 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 10/6/25

GRI

BORING B-11

OCT. 2025 JOB NO. 7072-A FIG. 11A



CLASSIFICATION OF MATERIAL

Surface Elevation: 352.0 ft [±] (NAVD 88)

Elevation, ft / Depth, ft	Classification of Material
	SILT, trace sand and decomposed gravel; ML; brown, orange, and gray; medium plasticity; moist; stiff; fine to coarse sand; subangular to subrounded gravel; micaceous; 8-inch-thick rooted zone at ground surface (Willamette Silt)
347.0 / 5.0	Clayey SILT; MH; gray and brown; medium to high plasticity; damp to moist; stiff (Willamette Silt)
344.5 / 7.5	Silty CLAY; CL; dark brown; high plasticity; moist; stiff (Willamette Silt) --- very stiff below 10 feet
339.5 / 12.5	Silty SAND, trace gravel; SM; red-brown; nonplastic fines; moist; medium dense to dense; fine to coarse sand; angular gravel (Possible Springwater Formation)
339.0 / 13.0	
337.0 / 15.0	Clayey SILT; MH; light gray; medium plasticity; moist; very stiff to hard (Possible Springwater Formation)
335.5 / 16.5	SILT, trace to some sand; ML; light gray and brown; low to medium plasticity; moist; very stiff; fine sand (Possible Springwater Formation) (4/10/2025)

COMMENTS AND ADDITIONAL TESTS

Surface: grass

▼1.4 ft (4/14/2025)

GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 10/6/25

Logged By: A. Horst **Drilled by:** Western States Soil Conservation, Inc.

Date Started: 4/10/25 **GPS Coordinates:** 45.442136° N 122.478819° W (WGS 84)

Drilling Method: Hollow-Stem Auger
Equipment: CME 850 Track-Mounted Drill Rig
Hole Diameter: 4 in.

Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: 0.828

Note: See Legend for Explanation of Symbols

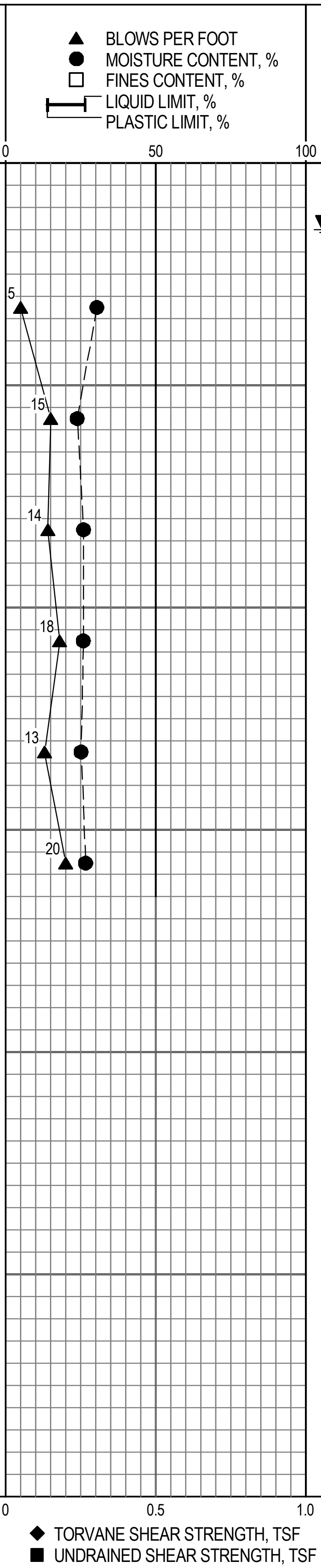
GRI

BORING B-12

OCT. 2025 JOB NO. 7072-A FIG. 12A

Surface Elevation: 369.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	SILT; ML; brown, gray, and orange; medium plasticity; moist; medium stiff; 15-inch-thick rooted zone at ground surface (Willamette Silt)					Surface: grass ▼ 1.5 ft (4/14/2025)
				S-1	2 1 4	
5	--- stiff to very stiff below 5 feet			S-2	3 7 8	
	Clayey SILT; MH; gray; medium to high plasticity; moist; stiff (Willamette Silt)	361.5 7.5		S-3	2 5 9	
10	--- trace sand; gray and brown; very stiff; fine to coarse sand below 10 feet			S-4	3 7 11	
	--- gray and orange; stiff below 12.5 feet			S-5	3 5 8	
15	Silty CLAY; CL; brown; medium to high plasticity; moist; very stiff (Willamette Silt)	354.0 15.0		S-6	3 8 12	
	(4/10/2025)	352.5 16.5				
20						
25						
30						



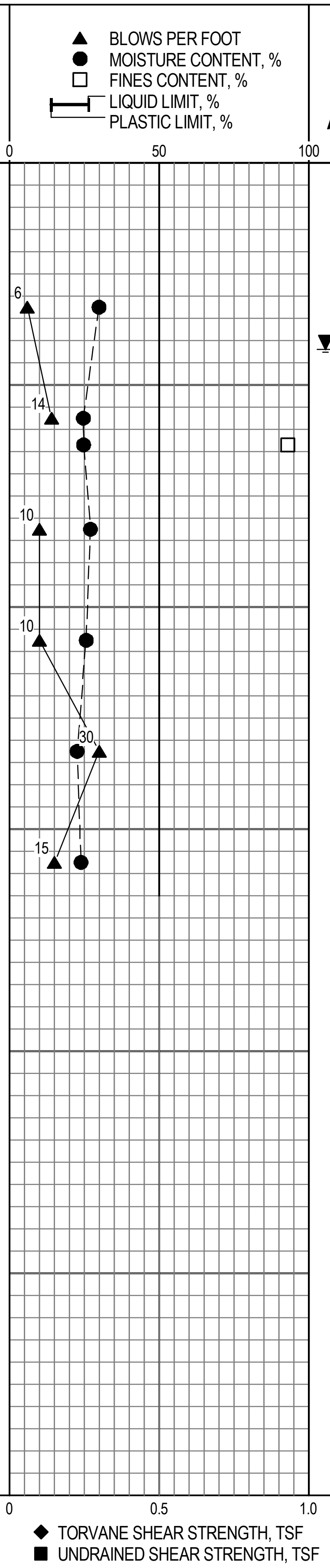
Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/10/25	**GPS Coordinates:** 45.441822° N 122.477555° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 4 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

GRI

BORING B-13

OCT. 2025 JOB NO. 7072-A FIG. 13A

DEPTH, FT	CLASSIFICATION OF MATERIAL **Surface Elevation:** 381.0 ft [±] (NAVD 88)	ELEVATION, FT DEPTH, FT	INSTALLATION	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	SILT, trace sand; ML; gray, brown, and orange; medium plasticity; moist; medium stiff; fine to coarse sand; 14-inch-thick rooted zone at ground surface (Willamette Silt)					Surface: grass
				S-1	1 3 3	
						▼4.2 ft (4/14/2025)
5	Silty CLAY, trace sand; CL; brown and gray; medium to high plasticity; moist; stiff; fine to coarse sand (Willamette Silt)	376.0 5.0		S-2 B-1	4 6 8	Bulk sample taken from 5 to 7.5 feet
				S-3	3 4 6	
10	--- moist to wet; trace black decomposed gravel; subrounded gravel below 10 feet			S-4	3 4 6	
	--- moist; very stiff to hard below 12.5 feet			S-5	6 13 17	
15	--- stiff to very stiff below 15 feet			S-6	4 7 8	
	(4/10/2025)	364.5 16.5				
20						
25						
30						



GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 10/6/25

Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/10/25	**GPS Coordinates:** 45.442058° N 122.47615° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 4 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

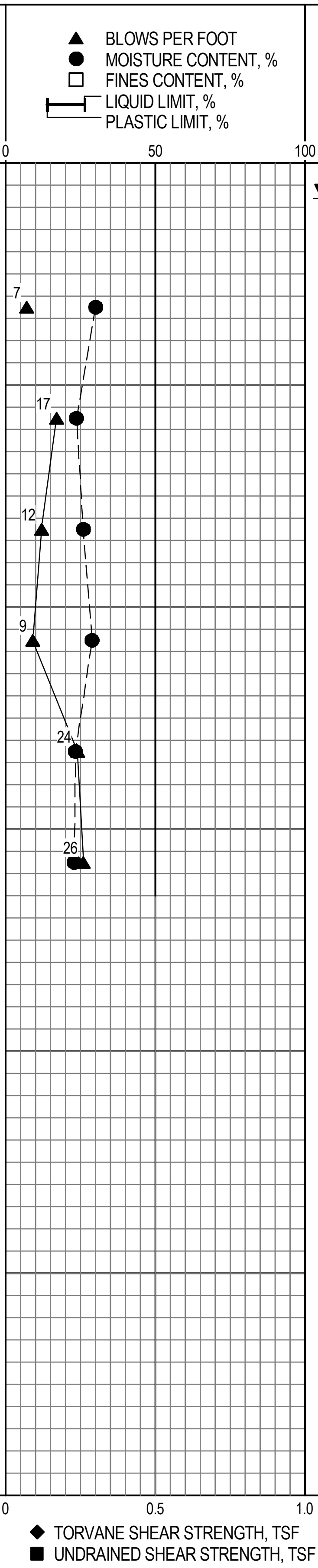
GRI

BORING B-14

OCT. 2025 JOB NO. 7072-A FIG. 14A

Surface Elevation: 379.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT DEPTH, FT	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
0	SILT; ML; gray, orange, and brown; medium plasticity; moist; medium stiff; 10- to 12-inch-thick rooted zone at ground surface (Willamette Silt)				▼0.8 ft (4/14/2025) Surface: grass
			S-1 B-1	1 2 5	Bulk sample taken from 2 to 4 feet
5	Clayey SILT; MH; gray and orange; medium to high plasticity; moist; very stiff (Willamette Silt)	374.0 5.0	S-2	3 8 9	
	--- stiff below 7.5 feet		S-3	3 5 7	
10	Silty CLAY; CL; gray and brown; medium to high plasticity; moist; stiff (Willamette Silt)	369.0 10.0	S-4	2 3 6	
	Clayey SILT, trace sand and decomposed gravel; MH; brown and gray; medium plasticity; moist; very stiff; fine to coarse sand; subrounded gravel (Willamette Silt)	366.5 12.5	S-5	4 9 15	
15			S-6	5 11 15	
	(4/11/2025)	362.5 16.5			
20					
25					
30					



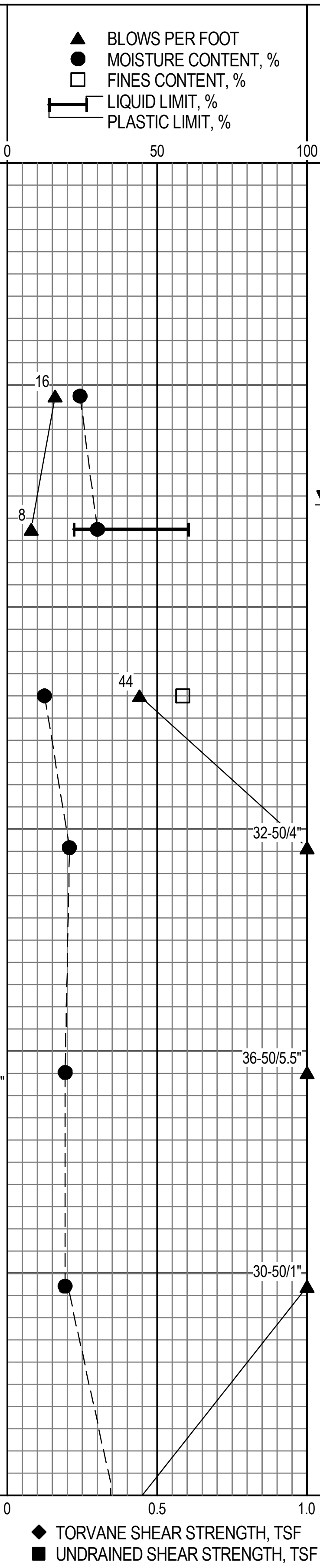
Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/11/25	**GPS Coordinates:** 45.441183° N 122.476227° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 4 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

GRI

BORING B-15

OCT. 2025 JOB NO. 7072-A FIG. 15A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL Surface Elevation: 349.0 ft [±] (NAVD 88)	ELEVATION, FT DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
0		Clayey SILT, trace to some sand; MH; brown, orange, and gray; medium plasticity; moist; very stiff; fine to coarse sand; approximately 12-inch-thick rooted zone at ground surface (Willamette Silt)						Surface: grass
					S-1			Shelby tube pushed at 400, 250, 500, 750 psi for each 6-inch increment
5					S-2		4 7 9	
		CLAY; CH; gray; high plasticity; moist; medium stiff to stiff (Willamette Silt)	341.5 7.5					7.7 ft (4/10/2025)
					S-3		2 3 5	
10					S-4			Shelby tube pushed at 100, 600 to 1,000 psi for each 6-inch increment
		Sandy SILT; trace gravel; ML; brown and gray; moist; low plasticity; hard; fine to coarse sand; subangular to subrounded gravel (Springwater Formation)	338.0 11.0		S-5		14 19 25	
15		--- gray; very hard below 15 feet			S-6		32 50/4"	Chattery drilling between 15 and 20 feet
20								Increased drilling chatter at 19.5 feet
					S-7		36 50/5.5"	
25					S-8		30 50/1"	Slower drilling from 25 to 26 feet, then smooth drilling below 26 feet
30			319.0					



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Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/9/25	**GPS Coordinates:** 45.441111° N 122.479511° W (WGS 84)
Drilling Method: Mud Rotary **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

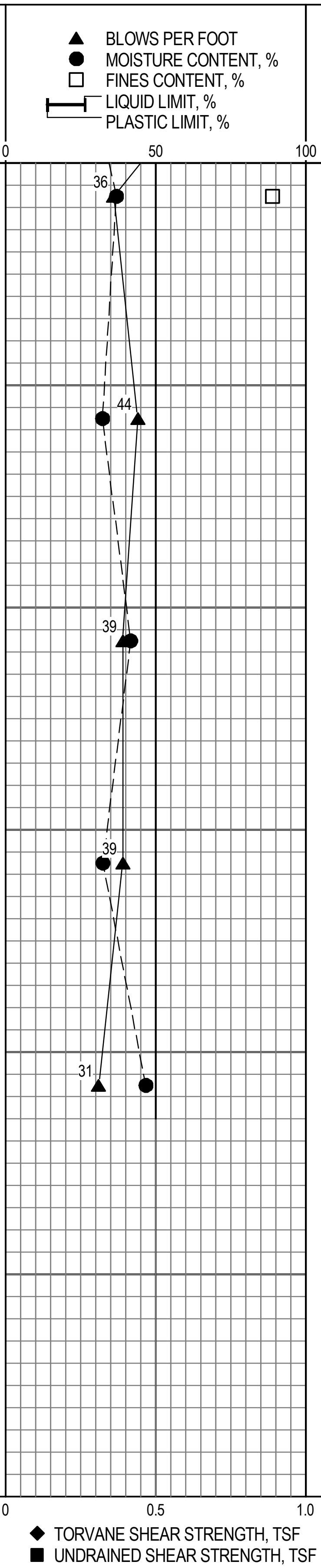
GRI

BORING B-16

OCT. 2025 JOB NO. 7072-A FIG. 16A

Surface Elevation: 349.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	SILT, trace sand; ML; brown; low plasticity; moist; hard; fine sand (Springwater Formation)	30.0		S-9		10 14 22	
35	--- trace to some sand; low to medium plasticity; micaceous below 35 feet			S-10		11 19 25	
40	--- moist to wet below 40 feet			S-11		11 18 21	
45	Clayey SILT, trace sand and gravel; MH; brown and gray; medium to high plasticity; moist; hard; fine to coarse sand; subrounded gravel (Springwater Formation)			S-12		14 18 21	
50	SILT, some sand; ML; gray; low plasticity; wet; hard; fine sand; micaceous (Springwater Formation)	299.0 50.0		S-13		9 14 17	
	(4/10/2025)	297.5 51.5					
55							
60							



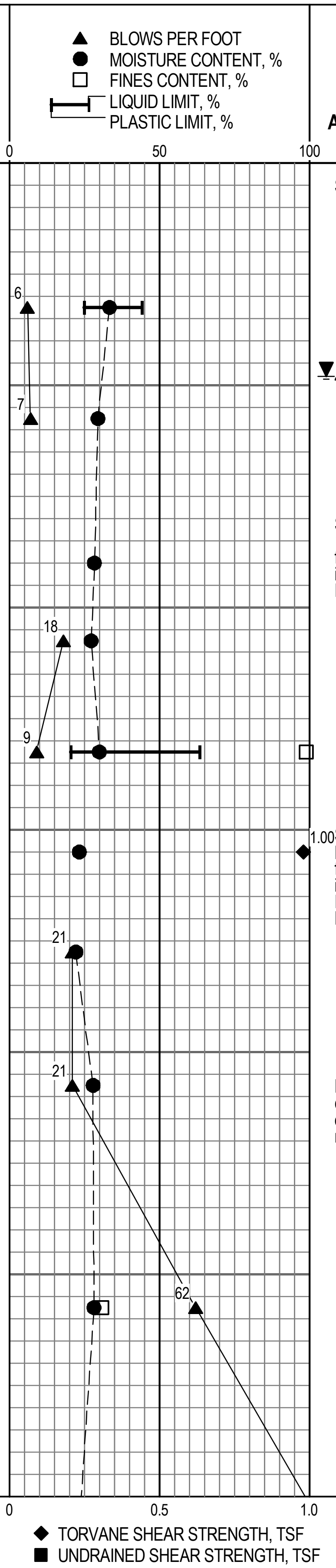
GRI

BORING B-16

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT
	Surface Elevation: 366.0 ft [±] (NAVD 88)	
0	Silty CLAY, trace to some sand, contains organics; CL; brown and gray; medium plasticity; moist; medium stiff; fine to coarse sand; organics consist of roots; micaceous; approximately 12-inch-thick rooted zone at ground surface (Willamette Silt)	
5	--- fine sand; organics absent below 5 feet	
	--- trace sand and gravel; brown mottled black and red-brown; very stiff; fine to coarse sand; subrounded gravel; blocky texture below 8 feet	
10	--- brown and gray; medium to high plasticity; fine sand below 10 feet	
	CLAY, trace sand; CH; gray and brown; high plasticity; moist; stiff; fine sand (Willamette Silt)	353.5 12.5
15	--- brown mottled black and white; medium to high plasticity; stiff to hard; blocky texture below 15 feet	
	--- high plasticity; very stiff; micaceous below 17 feet	
20	Clayey SILT, some sand; MH; dark gray; medium plasticity; moist; very stiff; fine to coarse sand; blocky structure (Springwater Formation)	346.0 20.0
25	Silty SAND, trace gravel; SM; brown, orange, and gray; low-plasticity fines; moist; very dense; fine to coarse sand; angular to subrounded gravel (Springwater Formation)	341.0 25.0
30		336.0

(CONTINUED NEXT PAGE)

SAMPLE NO.	BLOW COUNT	BLOWS PER FOOT
S-1	2 3 3	6
S-2	1 3 4	7
S-3		
S-4	4 7 11	18
S-5	2 4 5	9
S-6		
S-7	6 10 11	21
S-8	3 7 14	21
S-9	13 26 36	62



COMMENTS AND ADDITIONAL TESTS

Surface: grass

4.8 ft (4/8/2025)

Shelby tube pushed at 100, 400, 500, 600 psi for each 6-inch increment
Field vane = 1.5 tsf

Shelby tube pushed at 100, 500, 750, 900 psi
Dry Density = 1[illegible] pcf
for each 6-inch increment
Field vane = 2.76 tsf
Dry Density = 103 pcf

Driller indicated drilling conditions in and out of softer and harder material

Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/7/25	**GPS Coordinates:** 45.441219° N 122.477908° W (WGS 84)

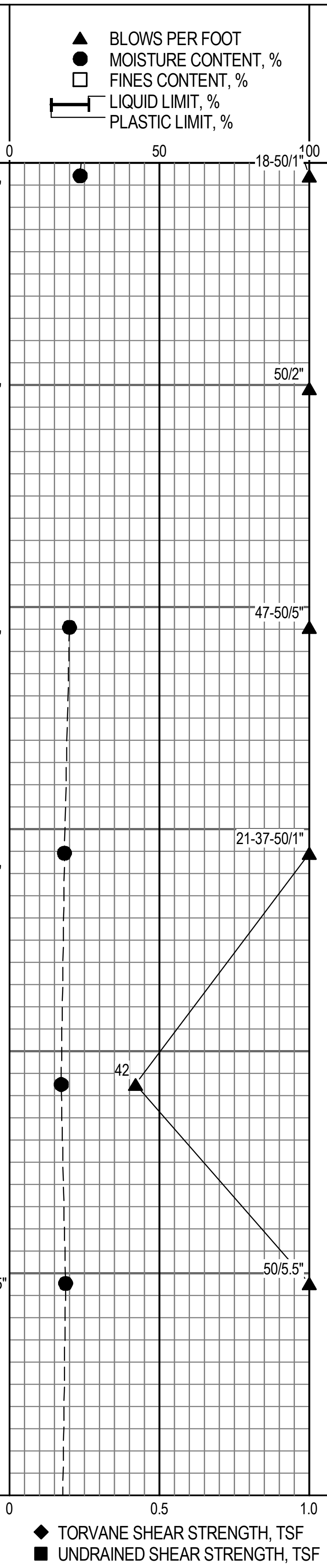
Drilling Method: Mud Rotary
Equipment: CME 850 Track-Mounted Drill Rig
Hole Diameter: 5 in.
Note: See Legend for Explanation of Symbols

Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: 0.828

GRI

BORING B-17

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL Surface Elevation: 366.0 ft [±] (NAVD 88)	ELEVATION, FT DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
30		Silty SAND, trace gravel; SM; dark gray; nonplastic fines; moist; very dense; fine to coarse sand; subangular to subrounded gravel (Springwater Formation)	30.0		S-10		18 50/1"	Slower drilling and lots of chatter between 30 and 35 feet. Zones of softer sediment also observed in this zone
35					S-11		50/2"	Recovery in S-11 at 35 feet was only a few gravels
40		--- trace to some gravel; brown and gray below 40 feet			S-12		47 50/5"	Soft drilling 40-42 feet
								Mud loss around 42 feet and after adding more mud, it became very watery
45		--- some gravel; fine sand below 45 feet			S-13		21 37 50/1"	
50		--- dense below 50 feet			S-14		33 12 30	In and out of soft and hard material below 40 feet
55		--- very dense below 55 feet			S-15		50/5.5"	Mud loss and thinning of mud continual below 40 feet
60			306.0					



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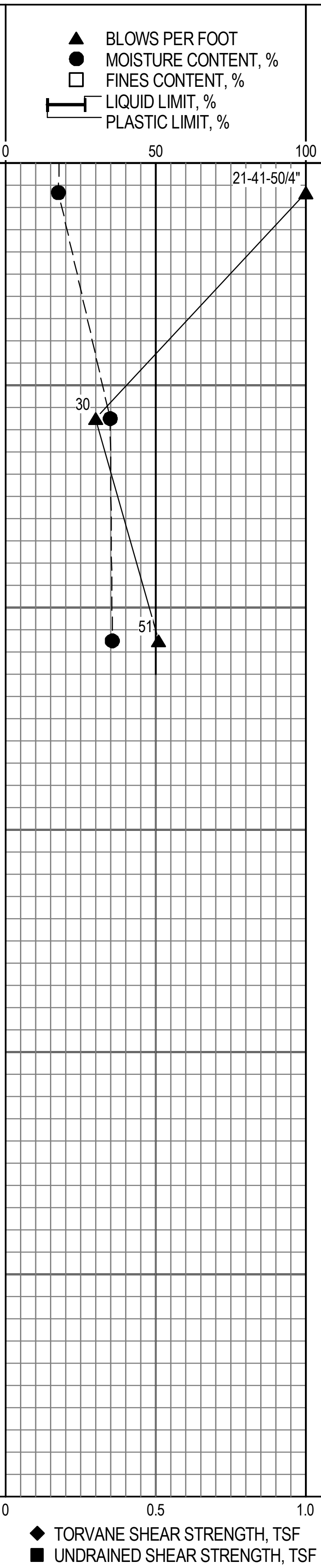
GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 10/6/25

Surface Elevation: 366.0 ft [±] (NAVD 88)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Silty SAND, some gravel; SM; brown and gray; nonplastic fines; moist; very dense; fine sand; subangular to subrounded gravel (Springwater Formation)	60.0		S-16		21 41 50/4"	Occasional drill rig chatter from 60 to 65 feet
65		SILT, trace sand; ML; light brown; low plasticity; moist; very stiff to hard; fine sand (Springwater Formation)	301.0 / 65.0		S-17		10 13 17	Smooth and fast drilling from 65 to 70 feet
70		--- light gray to brown; hard; micaceous below 70 feet			S-18		12 24 27	
		(4/8/2025)	294.5 / 71.5					
75								
80								
85								
90								



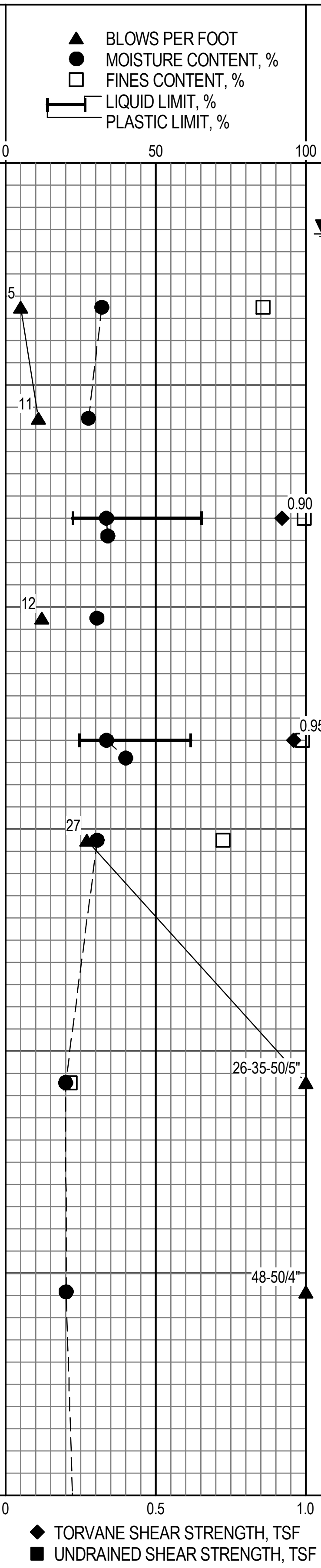
GRI

BORING B-17

OCT. 2025 JOB NO. 7072-A FIG. 17A

Surface Elevation: 356.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	SILT, trace sand; ML; brown; medium plasticity; moist; medium stiff; fine to coarse sand; micaceous; approximately 12-inch-thick rooted zone at ground surface (Willamette Silt)					Surface: grass 1.6 ft (4/9/2025)
				S-1	2 2 3	
5	Silty CLAY; CL; gray and brown; medium plasticity; moist; stiff; micaceous (Willamette Silt)	351.0 5.0		S-2	2 5 6	
	CLAY, trace sand; CH; light gray mottled red-brown; high plasticity; moist; very stiff; fine sand (Willamette Silt)	348.5 7.5		S-3		Shelby tube pushed at 0, 0, 250, 400 psi for Dry Density = 93 pcf each 6-inch increment Field Vane = 1.6 tsf Dry Density = 90 pcf
	--- gray and brown; stiff below 9.5 feet					
10				S-4	4 5 7	
	--- gray brown mottled white and black; stiff to hard below 12.5 feet			S-5		Shelby tube pushed at 0, 0, 100, 150 psi for Dry Density = 88 pcf each 6-inch increment Field Vane = 2.56 tsf Dry Density = 88 pcf
15	SILT, some sand, trace gravel; ML; brown and gray; medium plasticity; moist; very stiff; fine to coarse sand; subrounded gravel (Possible Springwater Formation)	341.5 14.5		S-6	13 10 17	Slower drilling between 15 and 20 feet; chatter at 17 feet
20	Silty SAND, trace gravel; SM; brown to gray; low-plasticity fines; moist; very dense; fine to coarse sand; subangular gravel (Springwater Formation)	336.0 20.0		S-7	26 35 50/5"	Chatter right below 20 feet
25	--- trace to some gravel below 25 feet			S-8	48 50/4"	
30		326.0				



◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

(CONTINUED NEXT PAGE)

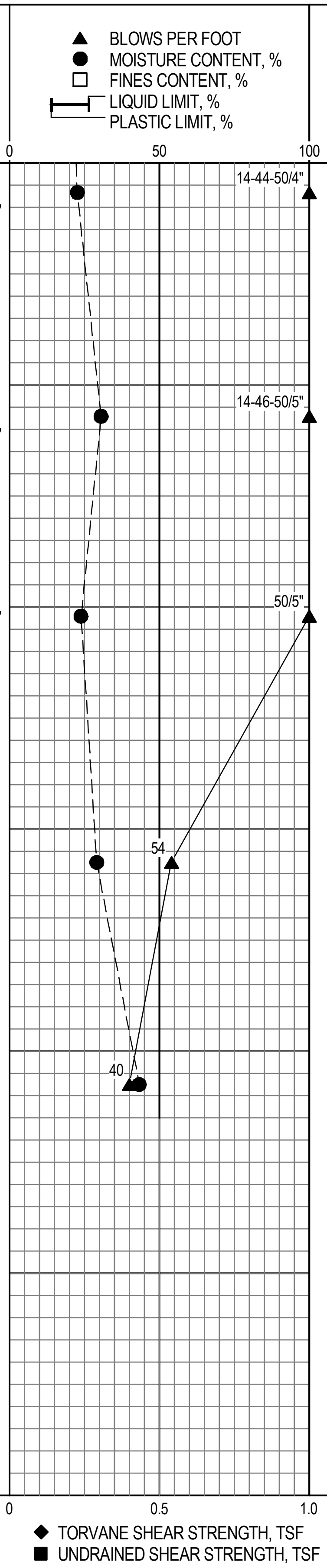
Logged By: A. Horst
Drilled by: Western States Soil Conservation, Inc.
Date Started: 4/8/25
GPS Coordinates: 45.44043° N 122.479177° W (WGS 84)
Drilling Method: Mud Rotary
Equipment: CME 850 Track-Mounted Drill Rig
Hole Diameter: 5 in.
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: 0.828
Note: See Legend for Explanation of Symbols

BORING B-18

OCT. 2025 JOB NO. 7072-A FIG. 18A

Surface Elevation: 356.0 ft [±] (NAVD 88)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Silty SAND, trace to some gravel; SM; yellow-brown, dark gray, and red-brown; low-plasticity fines; moist; very dense; fine to coarse sand; subangular gravel (Springwater Formation)	30.0		S-9		14 44 50/4"	Chatter between 30 and 35 feet
35		SILT, trace to some sand; ML; yellow-brown and red-brown; low plasticity; moist; very hard; fine to coarse sand (Springwater Formation)	321.0 35.0		S-10		14 46 50/5"	
40		Sandy SILT, trace gravel; ML; low plasticity; moist; very hard; fine to coarse sand; subangular gravel (Springwater Formation)	316.0 40.0		S-11		50/5"	
45		SILT, trace to some sand, trace gravel; ML; red/orange-brown; low plasticity; moist; hard; fine to coarse sand; subangular gravel (Springwater Formation)	311.0 45.0		S-12		21 30 24	
50		--- trace sand; dark gray below 50 feet			S-13		9 16 24	
		(4/9/2025)	304.5 51.5					
55								
60								



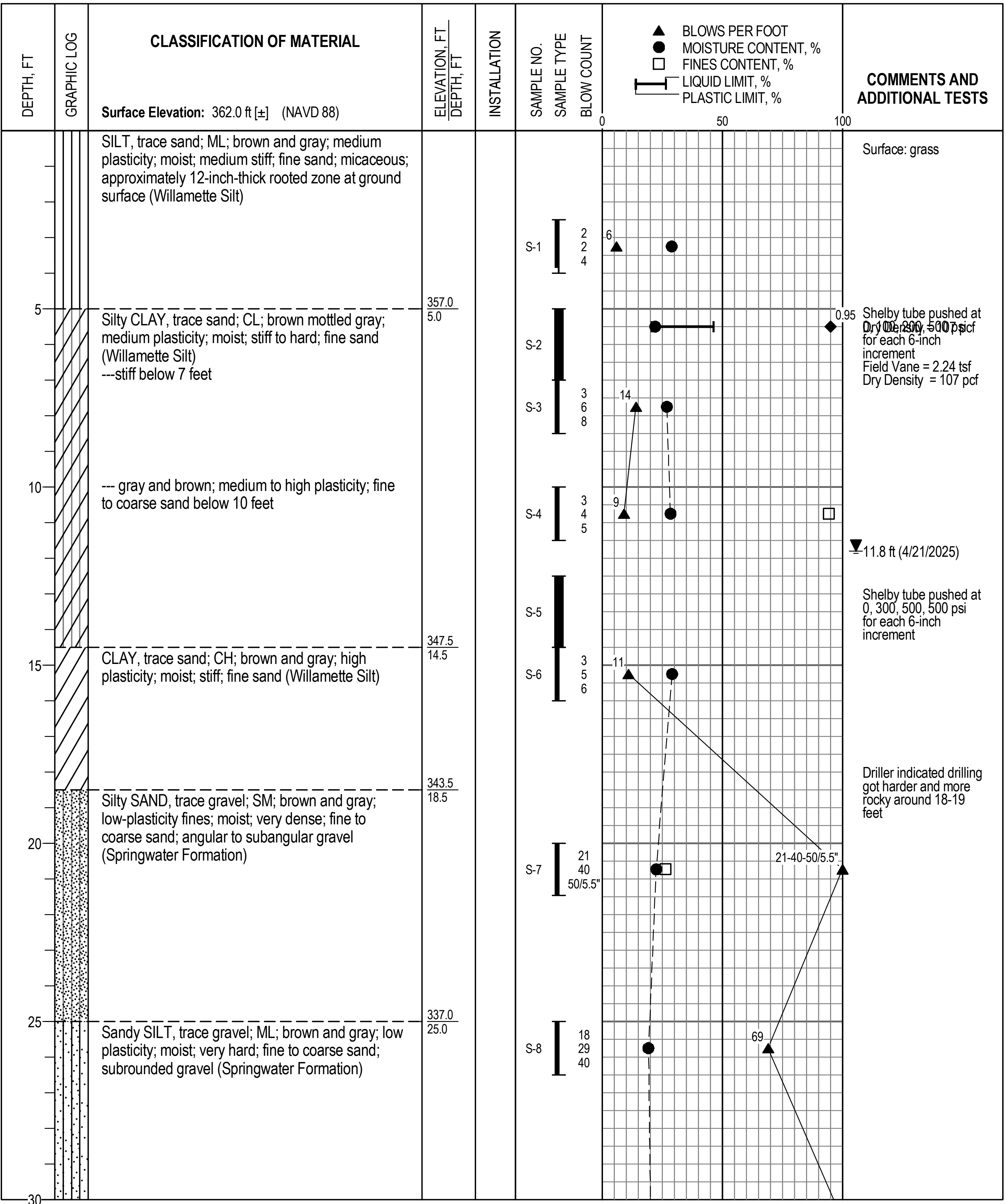
GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 10/6/25

GRI

BORING B-18

OCT. 2025 JOB NO. 7072-A FIG. 18A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: 362.0 ft [±] (NAVD 88)						
0		SILT, trace sand; ML; brown and gray; medium plasticity; moist; medium stiff; fine sand; micaceous; approximately 12-inch-thick rooted zone at ground surface (Willamette Silt)						Surface: grass
					S-1		2 2 4	
5		Silty CLAY, trace sand; CL; brown mottled gray; medium plasticity; moist; stiff to hard; fine sand (Willamette Silt) ---stiff below 7 feet	357.0 5.0		S-2			Shelby tube pushed at 0, 100, 200, 500 psi for each 6-inch increment Dry Density = 107 pcf Field Vane = 2.24 tsf Dry Density = 107 pcf
					S-3		3 6 8	
10		--- gray and brown; medium to high plasticity; fine to coarse sand below 10 feet			S-4		3 4 5	
								▼11.8 ft (4/21/2025)
					S-5			Shelby tube pushed at 0, 300, 500, 500 psi for each 6-inch increment
15		CLAY, trace sand; CH; brown and gray; high plasticity; moist; stiff; fine sand (Willamette Silt)	347.5 14.5		S-6		3 5 6	
								Driller indicated drilling got harder and more rocky around 18-19 feet
20		Silty SAND, trace gravel; SM; brown and gray; low-plasticity fines; moist; very dense; fine to coarse sand; angular to subangular gravel (Springwater Formation)	343.5 18.5		S-7		21 40 50/5.5"	
25		Sandy SILT, trace gravel; ML; brown and gray; low plasticity; moist; very hard; fine to coarse sand; subrounded gravel (Springwater Formation)	337.0 25.0		S-8		18 29 40	
30								



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◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

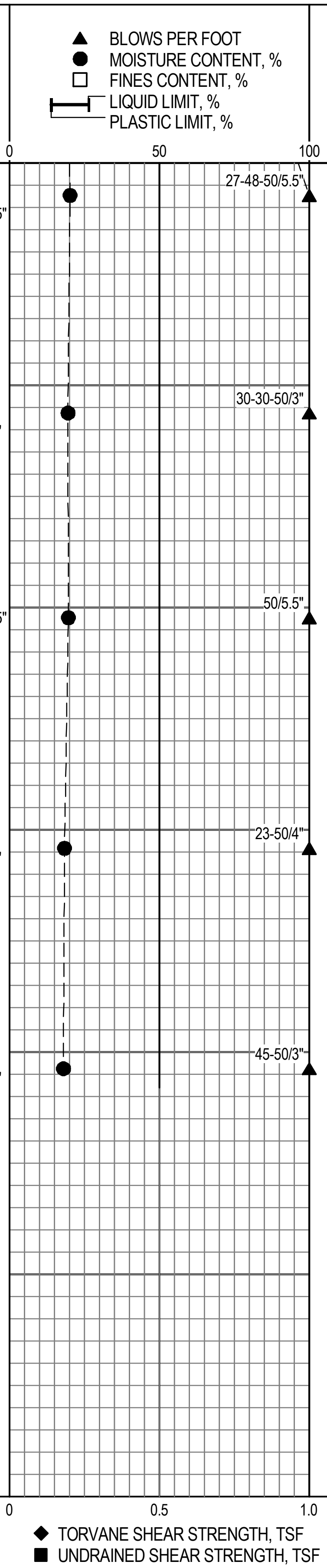
Logged By: A. Horst
Drilled by: Western States Soil Conservation, Inc.
Date Started: 4/8/25
GPS Coordinates: 45.440379° N 122.478362° W (WGS 84)
Drilling Method: Mud Rotary
Equipment: CME 850 Track-Mounted Drill Rig
Hole Diameter: 5 in.
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: 0.828
Note: See Legend for Explanation of Symbols

GRI

BORING B-19

OCT. 2025 JOB NO. 7072-A FIG. 19A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL Surface Elevation: 362.0 ft [±] (NAVD 88)	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Sandy SILT, trace gravel; ML; brown and gray; low plasticity; moist; very hard; fine to coarse sand; subrounded gravel (Springwater Formation)			S-9	I	27 48 50/5.5"	
35					S-10	I	30 30 50/3"	
40		--- trace to some gravel below 40 feet			S-11	I	50/5.5"	
45		--- trace gravel below 45 feet			S-12	I	23 50/4"	
50					S-13	I	45 50/3"	
		(4/8/2025)	311.2 50.8					
55								
60								



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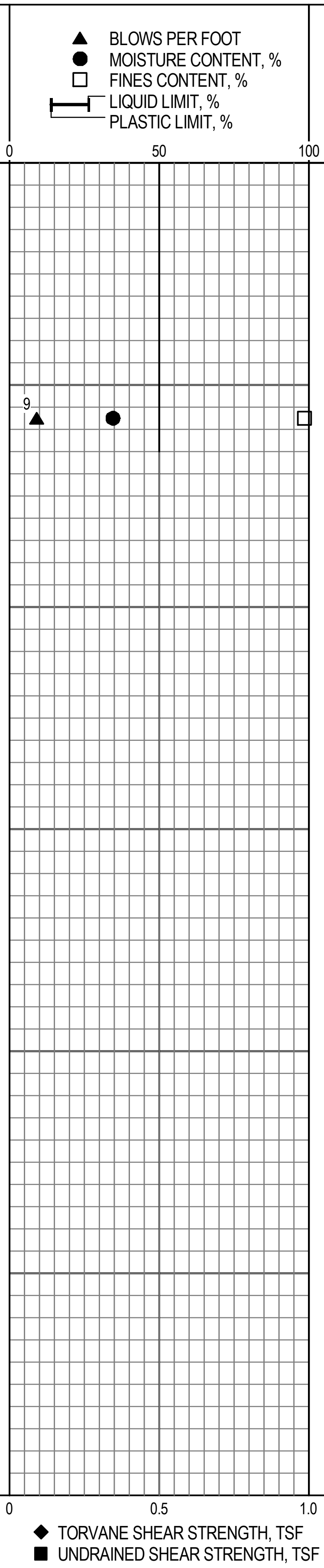
BORING B-19

OCT. 2025 JOB NO. 7072-A FIG. 19A

Surface Elevation: 347.0 ft [±] (NAVD 88)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	▨	Silty CLAY, trace sand, contains organics; CL; gray and brown; medium to high plasticity; moist; stiff; fine to coarse sand; organics consist of wood fibers; 12-inch-thick rooted zone at ground surface (Willamette Silt)						Surface: grass
5					S-1	I	3 4 5	Performed infiltration test at 5 feet
		(4/9/2025)	340.5 6.5					
10								
15								
20								
25								
30								



Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/8/25	**GPS Coordinates:** 45.440216° N 122.479936° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 6 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

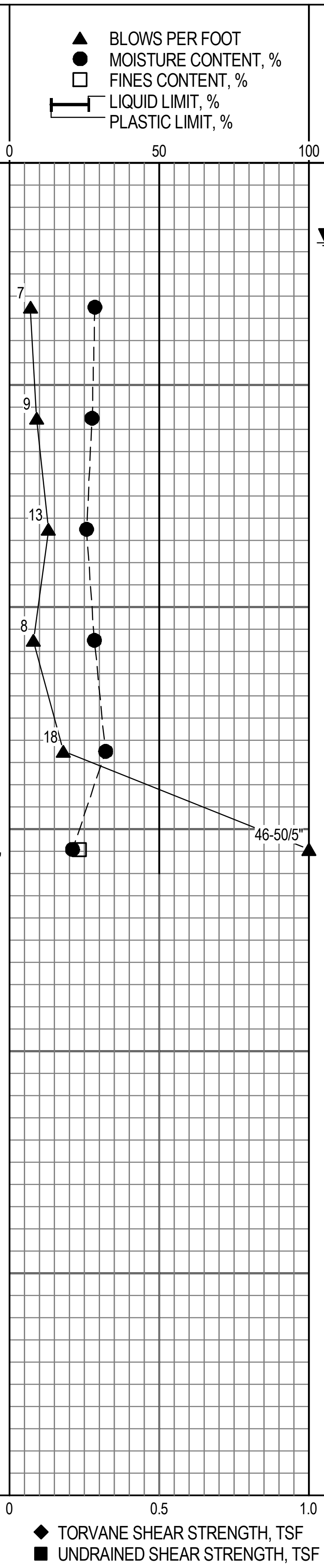
GRI

BORING B-20

OCT. 2025 JOB NO. 7072-A FIG. 20A

Surface Elevation: 358.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	SILT; ML; gray and orange-brown; medium plasticity; moist; medium stiff; micaceous; 9-inch-thick rooted zone at ground surface (Willamette Silt)						Surface: grass 1.8 ft (4/14/2025)
				S-1		2 3 4	
5	--- stiff below 5 feet			S-2		3 4 5	
	--- some sand; black decomposed gravel; fine to coarse sand below 7.5 feet			S-3		5 6 7	
10	Silty CLAY; CL; gray to light gray; medium to high plasticity; moist; medium stiff to stiff (Willamette Silt)	348.0 10.0		S-4		2 4 4	
	--- very stiff below 12.5 feet			S-5		2 6 12	
15	Silty SAND, trace gravel; SM; brown and gray; medium-plasticity fines; moist; medium dense; fine to coarse sand; subangular to subrounded gravel (Possible Springwater Formation) --- very dense below 15 feet	344.5 13.5		S-6		46 50/5"	
	(4/11/2025)	342.0 16.0					
20							
25							
30							



Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/11/25	**GPS Coordinates:** 45.439847° N 122.478833° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 4 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

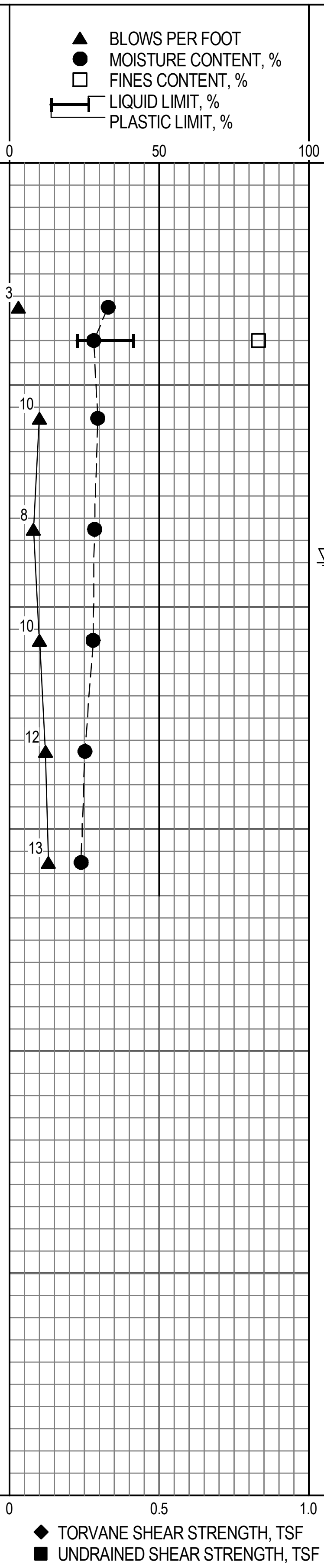
GRI

BORING B-21

OCT. 2025 JOB NO. 7072-A FIG. 21A

Surface Elevation: 373.0 ft [±] (NAVD 88)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Silty CLAY, some sand, contains organics; CL; gray and brown; medium plasticity; moist; soft; fine to coarse sand; organics consist of fine roots (Willamette Silt)						Surface: grass
					S-1		1 1 2	
					B-1			Bulk sample taken from 3 to 5 feet
5		--- organics absent; trace sand; medium to high plasticity; stiff below 5 feet			S-2		3 4 6	
		--- medium stiff to stiff; micaceous below 7.5 feet			S-3		3 4 4	
								∇9.0 ft (4/11/2025)
10		--- orange and brown; stiff below 10 feet			S-4		2 4 6	
					S-5		3 6 6	
15		SILT, some sand; ML; gray and orange-brown; medium plasticity; moist; stiff; fine to coarse sand (Willamette Silt)	358.0 15.0		S-6		3 6 7	
		(4/11/2025)	356.5 16.5					
20								
25								
30								



Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/11/25	**GPS Coordinates:** 45.440036° N 122.476259° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 4 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

GRI

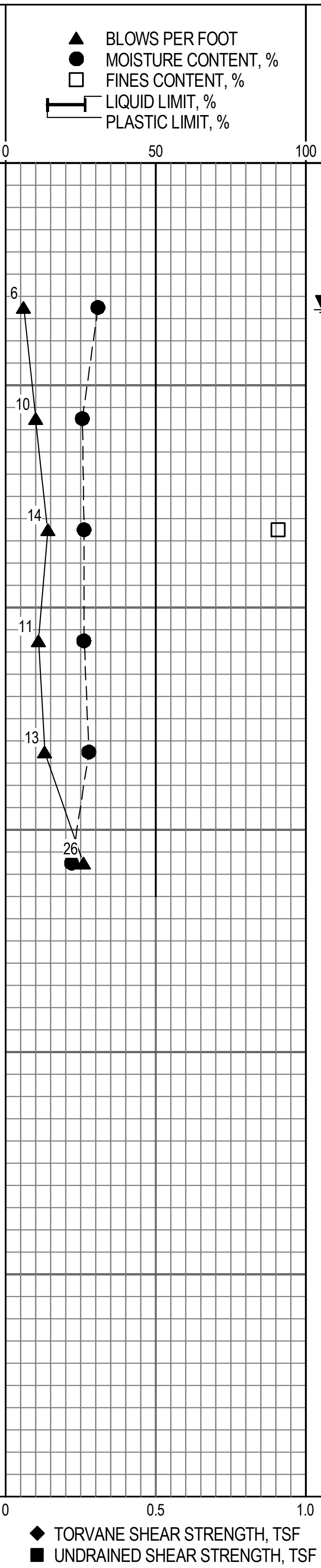
BORING B-22

OCT. 2025 JOB NO. 7072-A FIG. 22A

CLASSIFICATION OF MATERIAL

Surface Elevation: 368.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
0	SILT; ML; brown/orange and gray; medium plasticity; moist; medium stiff; micaceous; 18-inch-thick rooted zone at ground surface (Willamette Silt)				Surface: grass
			S-1	1 3 3	▼3.3 ft (4/14/2025)
5	Clayey SILT; MH; brown and gray; medium plasticity; moist; stiff (Willamette Silt)	363.0 5.0	S-2	2 3 7	
	--- trace sand; fine to coarse sand below 7.5 feet		S-3	3 6 8	
10	Silty CLAY; CL; gray and brown; medium to high plasticity; moist; stiff (Willamette Silt)		S-4	2 5 6	
	--- high plasticity below 12.5 feet		S-5	2 5 8	
15	Clayey SILT; MH; brown; high plasticity; damp to moist; very stiff (Willamette Silt)	353.0 15.0	S-6	6 12 14	
	(4/11/2025)	351.5 16.5			
20					
25					
30					



Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 4/11/25	**GPS Coordinates:** 45.440495° N 122.477268° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 4 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.828
Note: See Legend for Explanation of Symbols	

GRI

BORING B-23

OCT. 2025 JOB NO. 7072-A FIG. 23A

a. ROCK CORE RECOVERED FROM BORING B-3, RUNS 1 AND 2.

GRI

ROCK CORE PHOTOGRAPHS

OCT. 2025 JOB NO. 7072-A FIG. 24A

b. ROCK CORE RECOVERED FROM BORING B-3, RUN 3.

GRI

ROCK CORE PHOTOGRAPHS

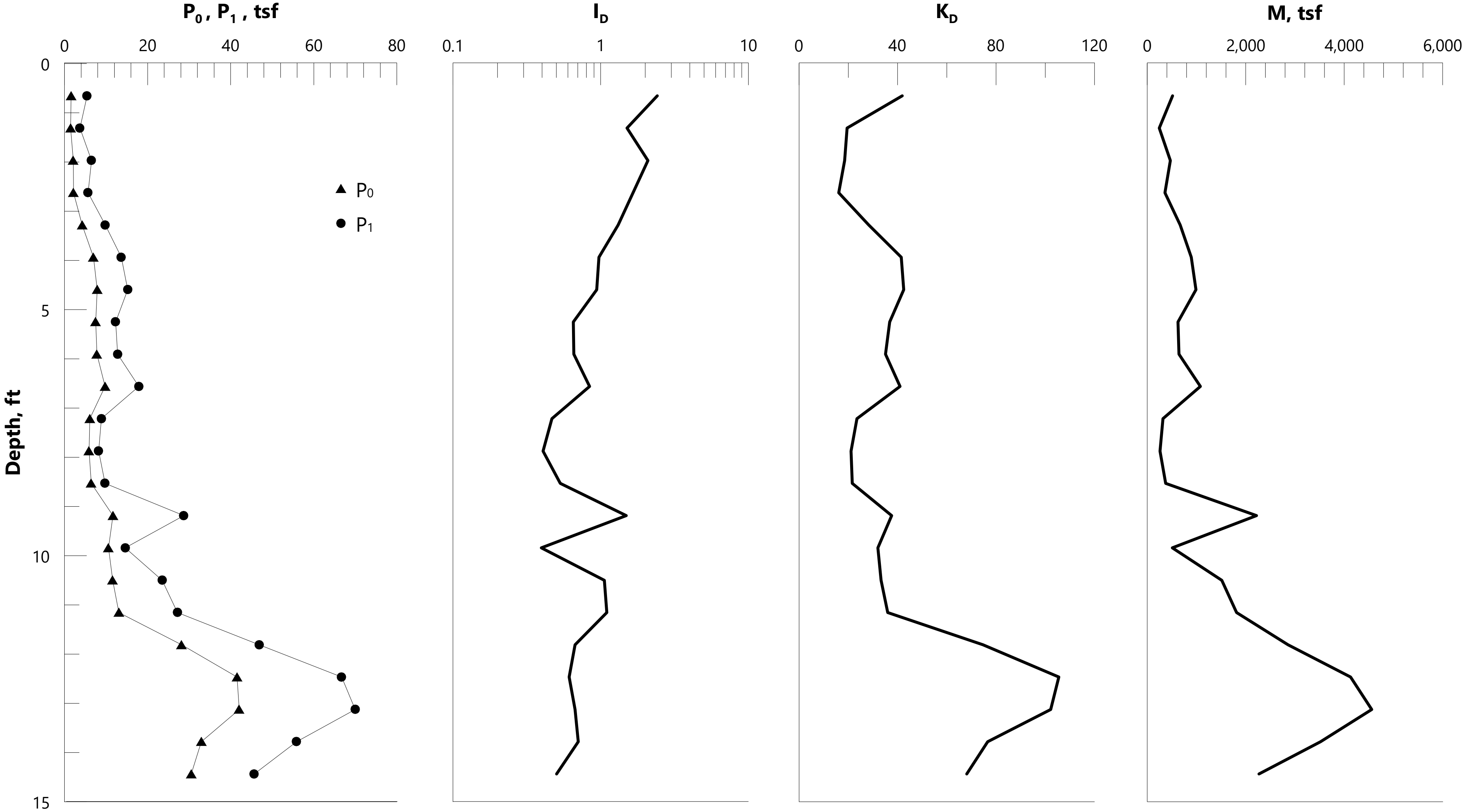
OCT. 2025 JOB NO. 7072-A FIG. 25A

c. ROCK CORE RECOVERED FROM BORING B-3, RUN 4.

GRI

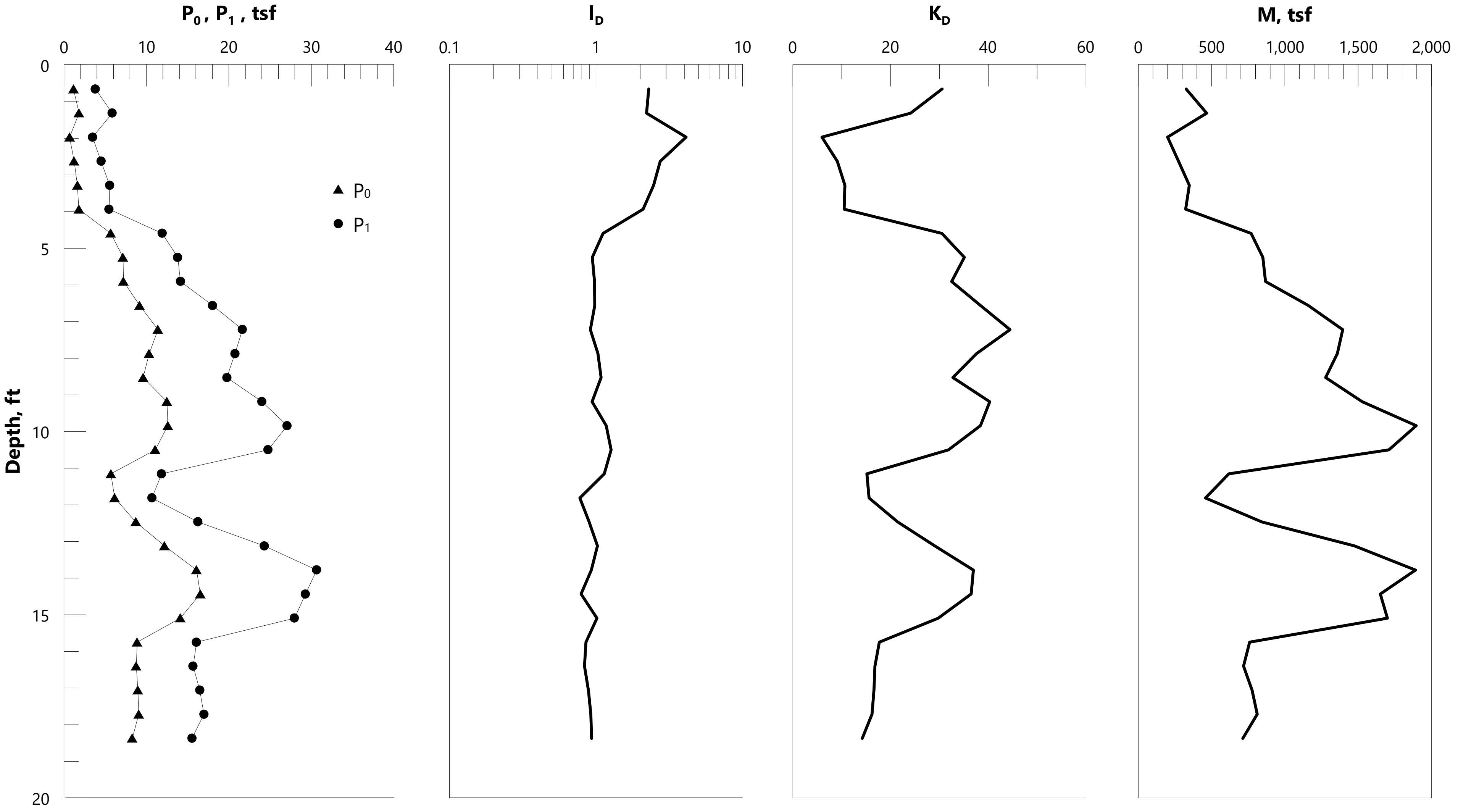
ROCK CORE PHOTOGRAPHS

OCT. 2025 JOB NO. 7072-A FIG. 26A



DILATOMETER SOUNDING DMT-1

OCT. 2025 JOB NO. 7072-A FIG. 27A



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DILATOMETER SOUNDING DMT-2

OCT. 2025 JOB NO. 7072-A FIG. 28A

KESSLER DYNAMIC CONE PENETROMETER LOG

JOB NO. 7072-A DRAWN BY JGH TESTING DATE 4/7/2025

Test Number	DCP-1	Surface Type	Grass
Location	Happy Valley, OR	Hammer	17.6 pounds

Depth, inches	Depth, millimeters	Cumulative Blows
0.0	0	0
4.5	115	1
8.3	210	2
12.0	306	3
15.2	385	4
17.8	453	5
20.0	507	6
22.0	558	7
23.6	599	8
24.6	624	9
25.7	654	10
26.8	680	11
28.4	721	12
29.8	758	13
31.3	795	14
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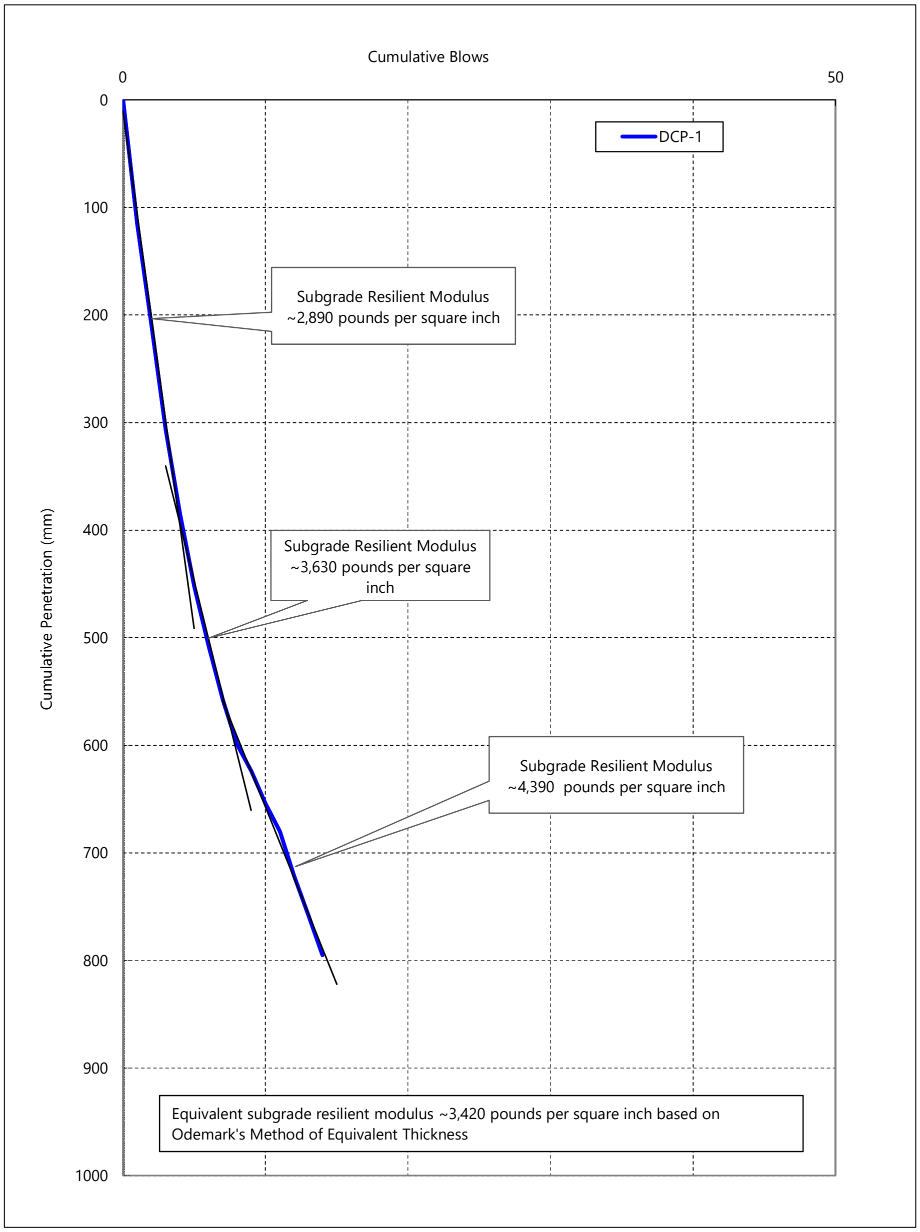

Subgrade Resilient Modulus ~2,890 pounds per square inch

Subgrade Resilient Modulus ~3,630 pounds per square inch

Subgrade Resilient Modulus ~4,390 pounds per square inch

Equivalent subgrade resilient modulus ~3,420 pounds per square inch based on Odemark's Method of Equivalent Thickness

GRI

DYNAMIC CONE PENETROMETER

OCT. 2025 JOB NO. 7072-A FIG. 29A

KESSLER DYNAMIC CONE PENETROMETER LOG

JOB NO. 7072-A   DRAWN BY JGH   TESTING DATE 4/7/2025

Test Number DCP-2   Surface Type Grass
Location Happy Valley, OR   Hammer 17.6 pounds

Depth, inches	Depth, millimeters	Cumulative Blows
0.0	0	0
13.7	348	1
16.6	421	2
19.1	486	3
22.2	565	4
24.4	621	5
26.4	671	6
27.8	705	7
28.7	730	8
29.6	752	9
30.5	774	10
31.2	793	11
31.9	810	12
32.6	827	13
33.2	844	14
33.8	858	15
34.3	870	16
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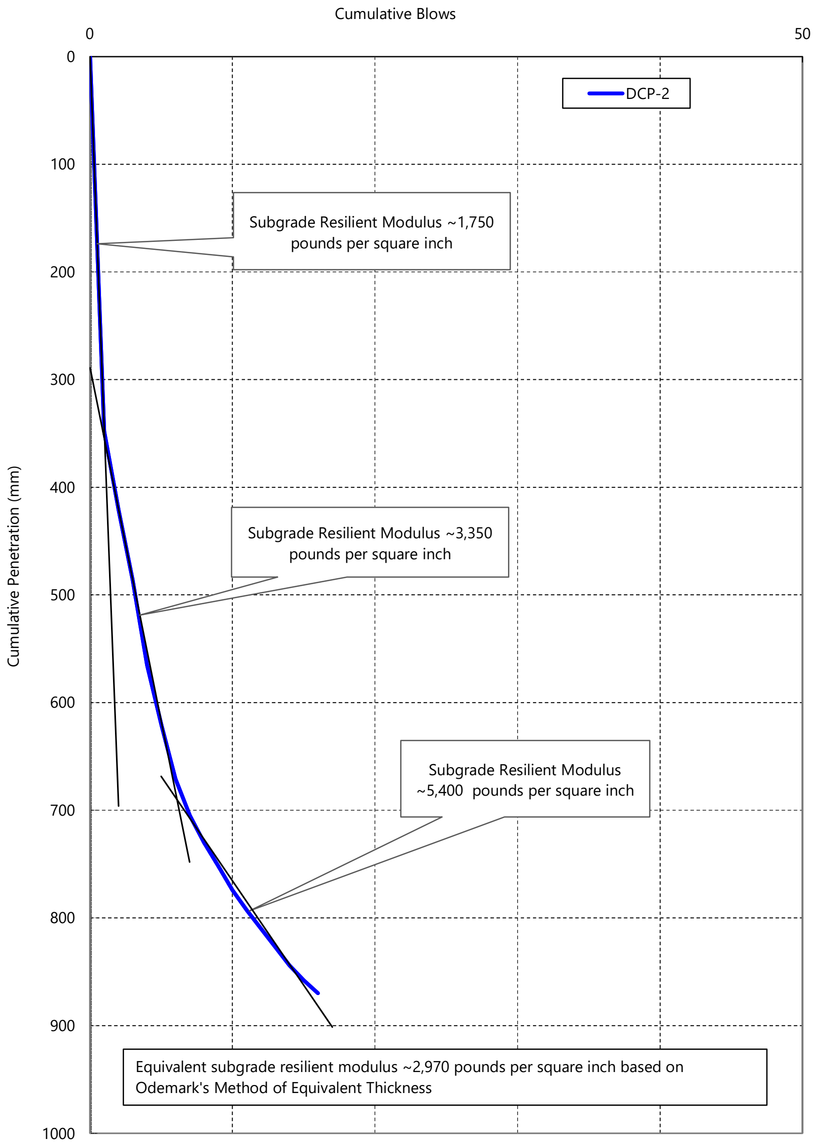

Subgrade Resilient Modulus ~1,750 pounds per square inch

Subgrade Resilient Modulus ~3,350 pounds per square inch

Subgrade Resilient Modulus ~5,400 pounds per square inch

Equivalent subgrade resilient modulus ~2,970 pounds per square inch based on Odemark's Method of Equivalent Thickness

DYNAMIC CONE
PENETROMETER

OCT. 2025   JOB NO. 7072-A   FIG. 30A

KESSLER DYNAMIC CONE PENETROMETER LOG

JOB NO. 7072-A | DRAWN BY JGH | TESTING DATE 4/7/2025

Test Number DCP-3
Location Happy Valley, OR
Surface Type Grass
Hammer 17.6 pounds

Depth, inches	Depth, millimeters	Cumulative Blows
0.0	0	0
1.7	44	1
3.2	82	2
4.8	123	3
7.0	179	4
11.4	289	5
16.5	418	6
20.2	514	7
23.0	583	8
25.4	644	9
27.2	691	10
28.7	728	11
29.9	759	12
30.9	786	13
31.9	811	14
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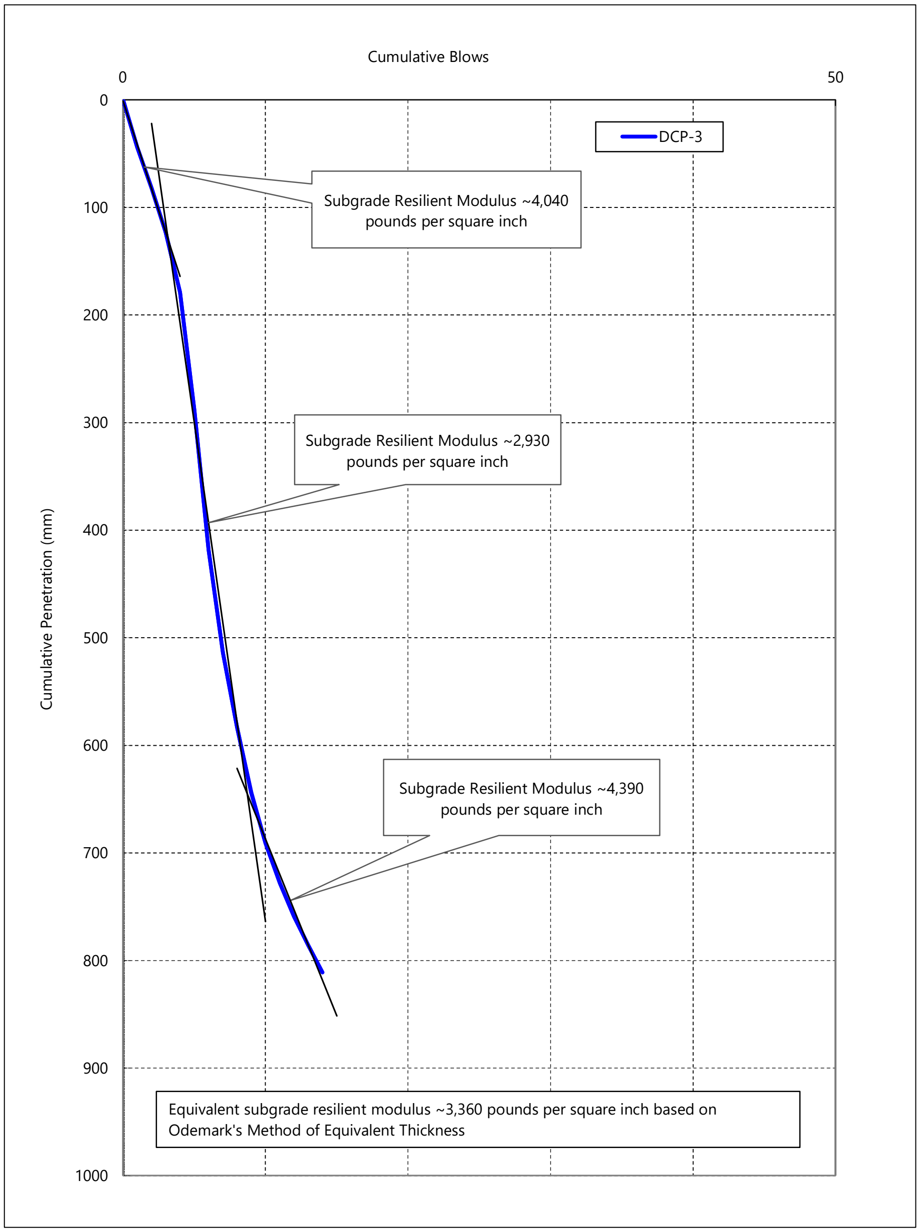

Subgrade Resilient Modulus ~4,040 pounds per square inch

Subgrade Resilient Modulus ~2,930 pounds per square inch

Subgrade Resilient Modulus ~4,390 pounds per square inch

Equivalent subgrade resilient modulus ~3,360 pounds per square inch based on Odemark's Method of Equivalent Thickness

GRI

DYNAMIC CONE PENETROMETER

KESSLER DYNAMIC CONE PENETROMETER LOG

JOB NO. 7072-A
DRAWN BY JGH
TESTING DATE 4/7/2025

Test Number	DCP-4	Surface Type	Grass
Location	Happy Valley, OR	Hammer	17.6 pounds

Depth, inches	Depth, millimeters	Cumulative Blows
0.0	0	0
0.5	13	1
2.0	52	2
3.4	86	3
4.8	122	4
6.2	157	5
7.2	184	6
8.1	207	7
9.1	232	8
10.3	262	9
11.3	288	10
12.4	314	11
13.4	340	12
14.4	366	13
15.4	390	14
16.4	417	15
17.5	444	16
18.3	466	17
19.3	490	18
20.2	514	19
21.1	537	20
22.0	560	21
22.9	581	22
23.7	602	23
24.5	623	24
25.4	644	25
26.1	664	26
27.0	685	27
27.8	706	28
28.6	727	29
29.5	749	30
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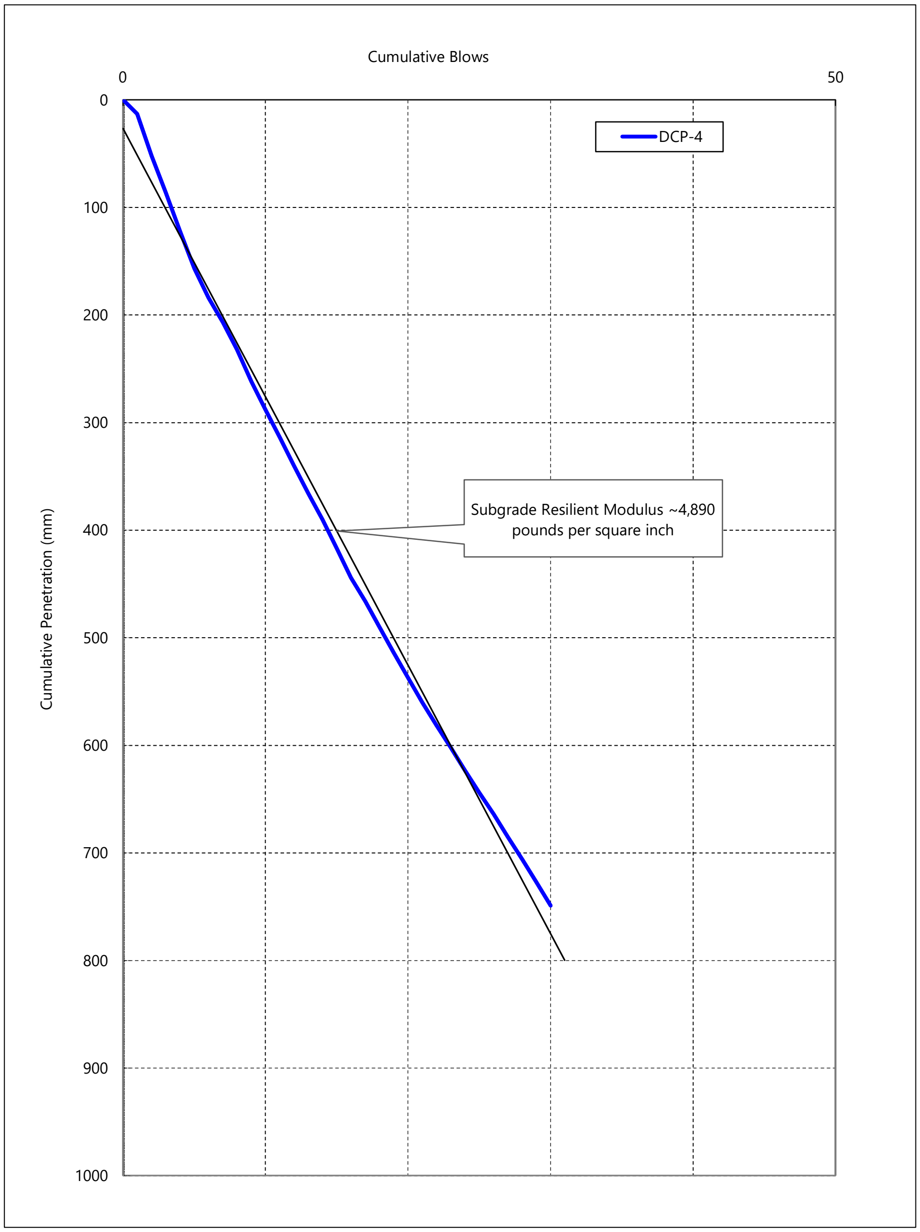

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DYNAMIC CONE PENETROMETER

KESSLER DYNAMIC CONE PENETROMETER LOG

JOB NO. 7072-A
DRAWN BY JGH
TESTING DATE 4/7/2025

Test Number DCP-5
Location Happy Valley, OR
Surface Type Grass
Hammer 17.6 pounds

Depth, inches	Depth, millimeters	Cumulative Blows
0.0	0	0
3.8	96	1
5.0	127	2
6.0	152	3
6.9	175	4
8.1	207	5
9.7	247	6
11.5	292	7
13.1	334	8
14.8	375	9
16.1	409	10
17.4	442	11
18.7	476	12
20.0	507	13
21.2	538	14
22.3	567	15
23.3	592	16
24.3	617	17
25.3	642	18
26.2	666	19
27.1	688	20
28.0	711	21
28.8	732	22
29.6	752	23
30.3	769	24
30.9	786	25
31.6	802	26
32.2	817	27
32.8	832	28
33.3	847	29
33.9	860	30
34.4	875	31
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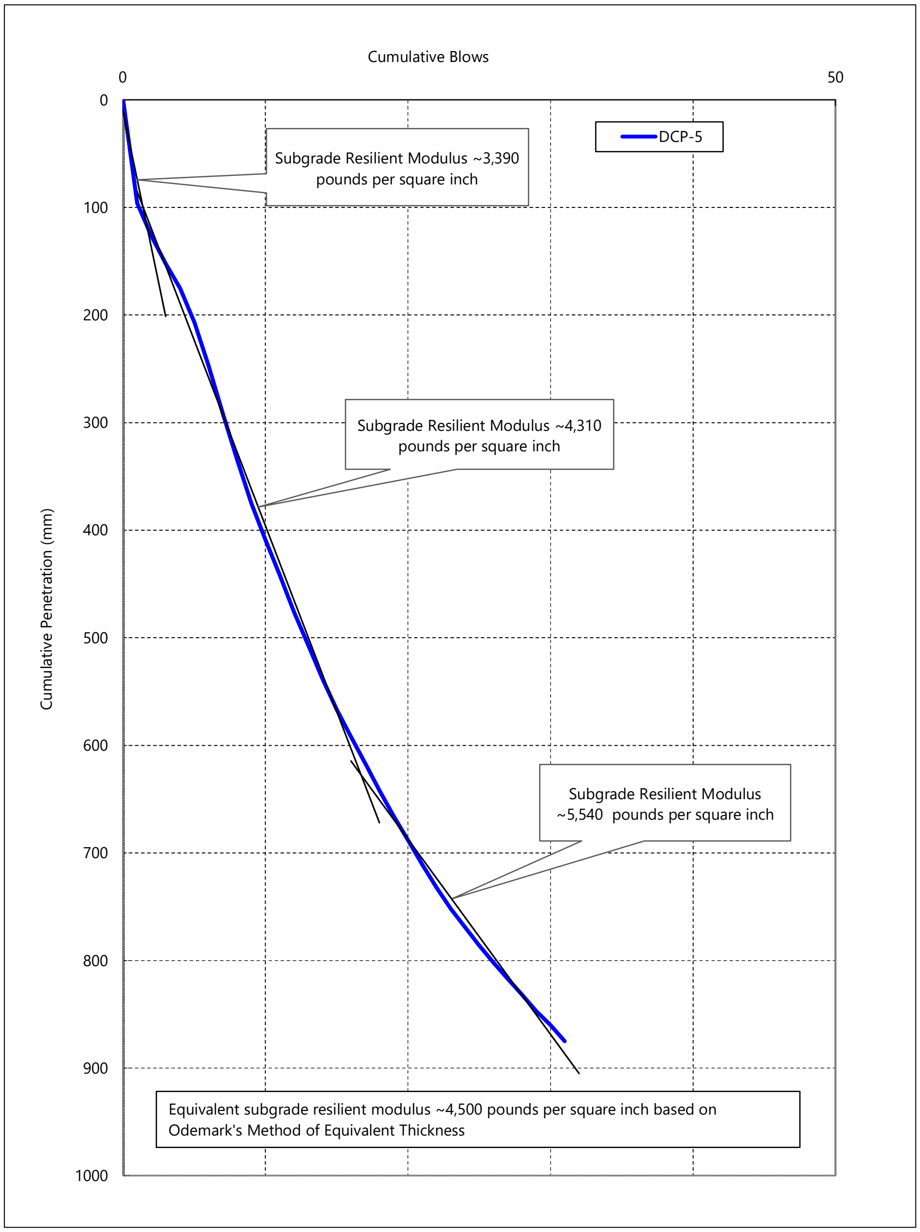

Subgrade Resilient Modulus ~3,390 pounds per square inch

Subgrade Resilient Modulus ~4,310 pounds per square inch

Subgrade Resilient Modulus ~5,540 pounds per square inch

Equivalent subgrade resilient modulus ~4,500 pounds per square inch based on Odemark's Method of Equivalent Thickness

GRI

DYNAMIC CONE PENETROMETER

OCT. 2025 JOB NO. 7072-A FIG. 33A

KESSLER DYNAMIC CONE PENETROMETER LOG

JOB NO. 7072-A DRAWN BY JGH TESTING DATE 4/7/2025

Test Number DCP-6 Surface Type Grass
Location Happy Valley, OR Hammer 17.6 pounds

Depth, inches	Depth, millimeters	Cumulative Blows
0.0	0	0
1.8	45	1
3.0	75	2
4.0	102	3
5.2	132	4
7.0	177	5
9.1	232	6
12.5	317	7
15.5	394	8
18.4	468	9
21.1	536	10
23.4	595	11
25.5	647	12
27.0	687	13
28.5	724	14
29.7	755	15
31.0	787	16
32.2	817	17
33.1	841	18
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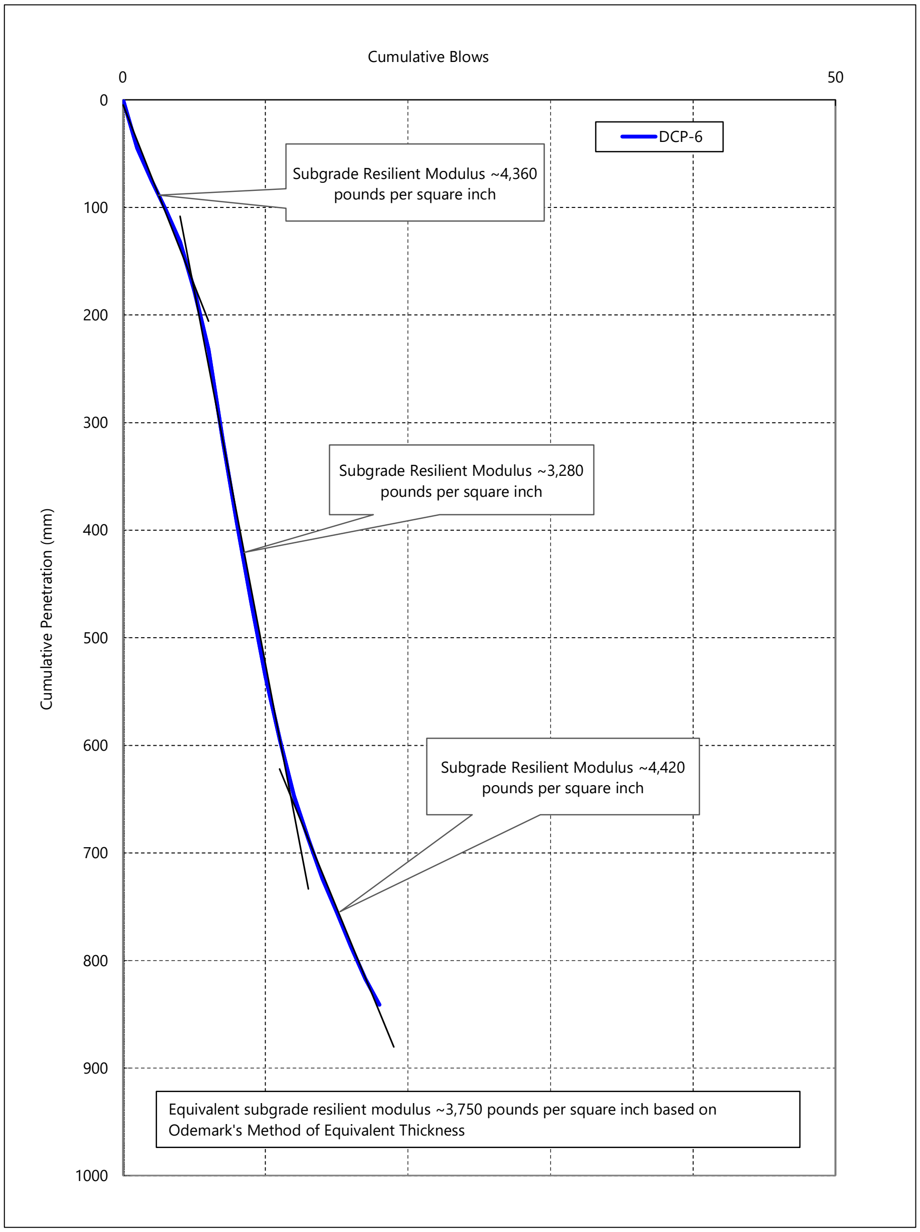

Equivalent subgrade resilient modulus ~3,750 pounds per square inch based on Odemark's Method of Equivalent Thickness

GRI

DYNAMIC CONE PENETROMETER

OCT. 2025 JOB NO. 7072-A FIG. 34A

KESSLER DYNAMIC CONE PENETROMETER LOG

JOB NO. 7072-A DRAWN BY JGH TESTING DATE 4/7/2025

Test Number DCP-7 Surface Type Grass
Location Happy Valley, OR Hammer 17.6 pounds

Depth, inches	Depth, millimeters	Cumulative Blows
0.0	0	0
3.7	94	1
5.8	147	2
10.2	258	3
14.1	358	4
17.0	433	5
19.4	492	6
21.0	533	7
22.6	574	8
24.0	610	9
25.2	639	10
26.2	665	11
27.1	689	12
28.0	712	13
28.9	733	14
29.6	753	15
30.4	772	16
31.0	787	17
31.6	803	18
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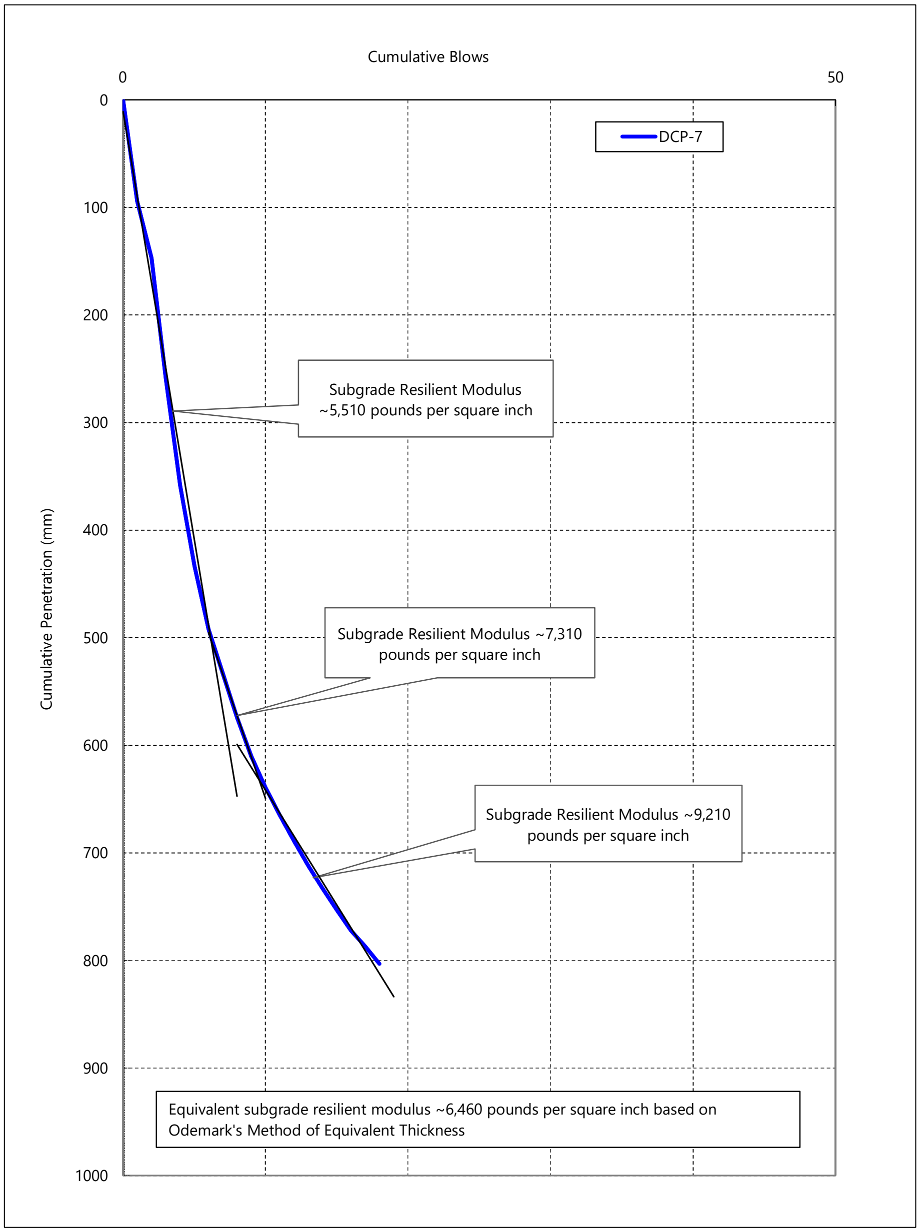

Subgrade Resilient Modulus ~5,510 pounds per square inch

Subgrade Resilient Modulus ~7,310 pounds per square inch

Subgrade Resilient Modulus ~9,210 pounds per square inch

Equivalent subgrade resilient modulus ~6,460 pounds per square inch based on Odemark's Method of Equivalent Thickness

GRI

DYNAMIC CONE PENETROMETER

OCT. 2025 JOB NO. 7072-A FIG. 35A

KESSLER DYNAMIC CONE PENETROMETER LOG

JOB NO. 7072-A DRAWN BY JGH TESTING DATE 4/7/2025

Test Number	DCP-8	Surface Type	Grass
Location	Happy Valley, OR	Hammer	17.6 pounds

Depth, inches	Depth, millimeters	Cumulative Blows
0.0	0	0
3.1	80	1
4.3	110	2
6.8	172	3
9.4	238	4
12.0	305	5
13.8	351	6
15.4	392	7
16.9	428	8
18.0	458	9
19.1	485	10
20.2	513	11
21.4	543	12
22.5	572	13
23.5	598	14
24.3	618	15
25.0	636	16
25.7	654	17
26.4	670	18
27.0	685	19
27.6	700	20
28.1	714	21
28.6	727	22
29.1	738	23
29.5	750	24
29.9	760	25
30.4	771	26
30.7	780	27
31.1	790	28
31.5	800	29
31.9	809	30
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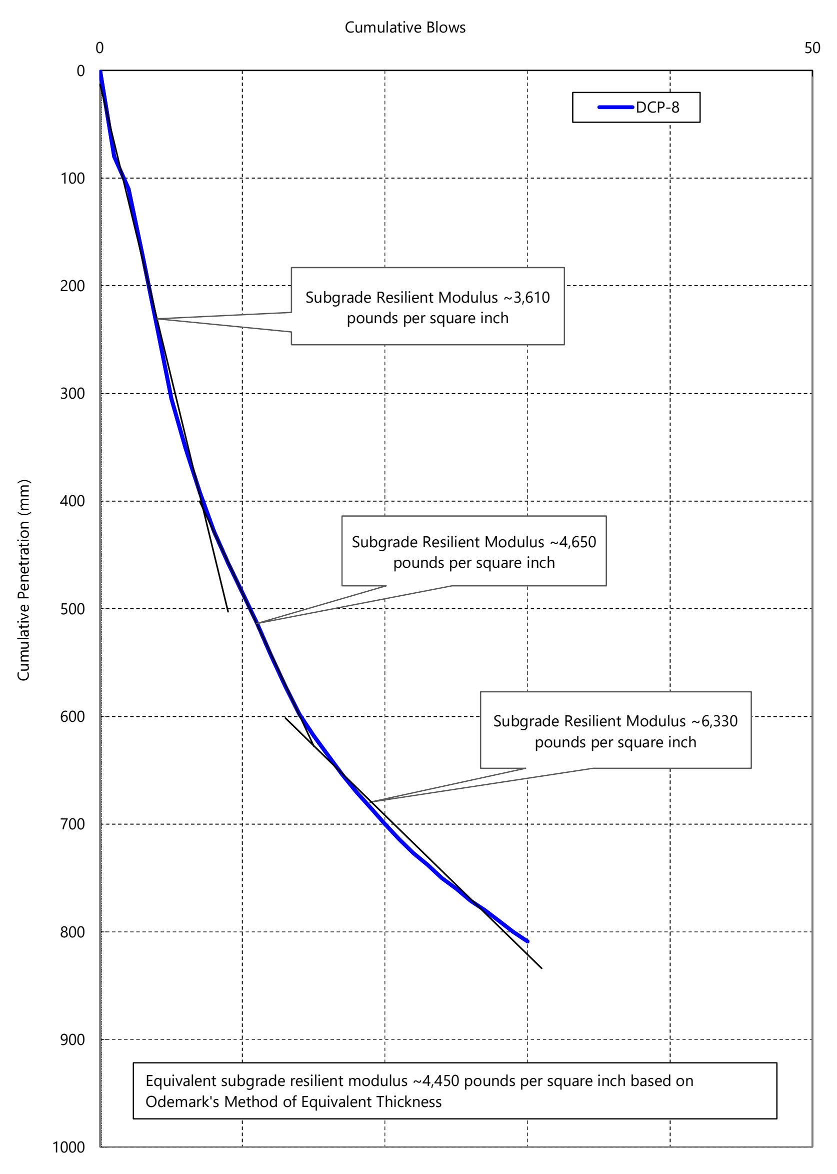

Subgrade Resilient Modulus ~3,610 pounds per square inch

Subgrade Resilient Modulus ~4,650 pounds per square inch

Subgrade Resilient Modulus ~6,330 pounds per square inch

Equivalent subgrade resilient modulus ~4,450 pounds per square inch based on Odemark's Method of Equivalent Thickness

GRI

DYNAMIC CONE
PENETROMETER

OCT. 2025 JOB NO. 7072-A FIG. 36A

KESSLER DYNAMIC CONE PENETROMETER LOG

JOB NO. 7072-A DRAWN BY JGH TESTING DATE 4/7/2025

Test Number DCP-9 Surface Type Grass
Location Happy Valley, OR Hammer 17.6 pounds

Depth, inches	Depth, millimeters	Cumulative Blows
0.0	0	0
3.6	91	1
6.9	175	2
9.2	233	3
11.4	290	4
13.1	334	5
14.5	369	6
16.1	408	7
17.4	442	8
18.8	477	9
20.2	512	10
21.8	553	11
23.2	589	12
24.5	622	13
25.7	652	14
26.9	682	15
28.0	710	16
29.1	739	17
30.2	768	18
31.3	795	19
32.4	822	20
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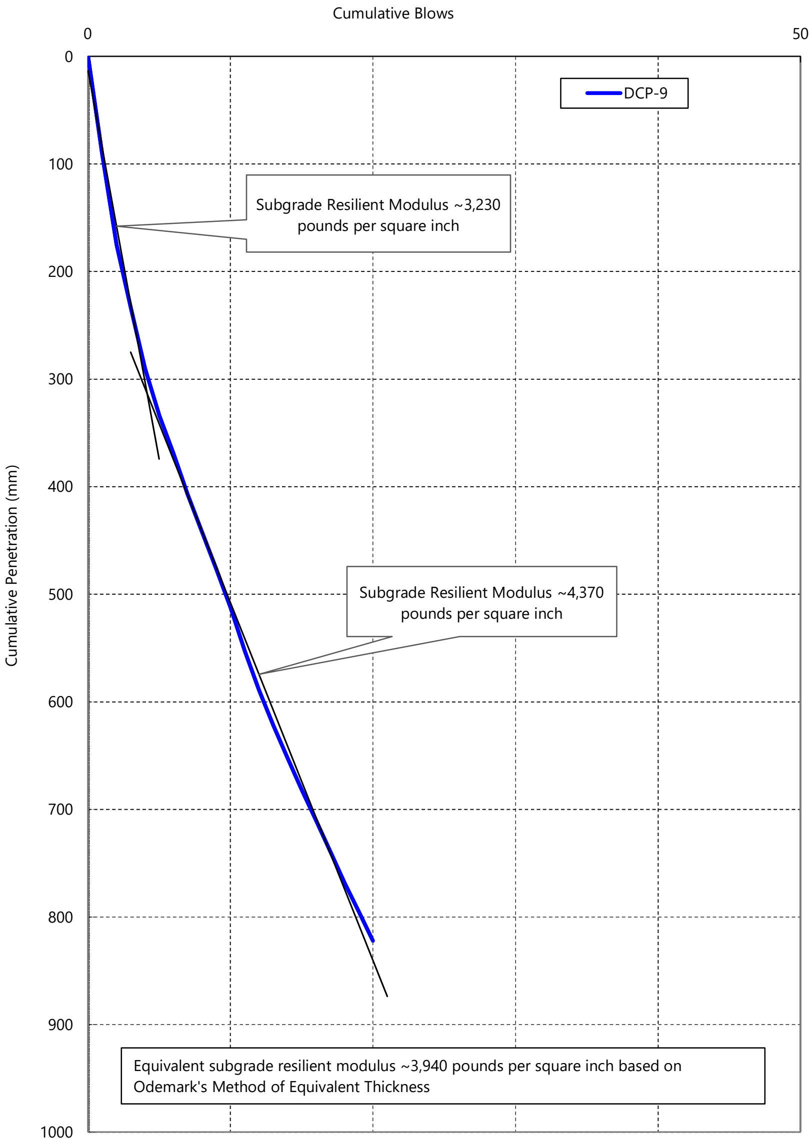

Subgrade Resilient Modulus ~3,230 pounds per square inch

Subgrade Resilient Modulus ~4,370 pounds per square inch

Equivalent subgrade resilient modulus ~3,940 pounds per square inch based on Odemark's Method of Equivalent Thickness

GRI

DYNAMIC CONE PENETROMETER

OCT. 2025 JOB NO. 7072-A FIG. 37A

KESSLER DYNAMIC CONE PENETROMETER LOG

JOB NO. 7072-A DRAWN BY JGH TESTING DATE 4/7/2025

Test Number	DCP-10	Surface Type	Grass
Location	Happy Valley, OR	Hammer	17.6 pounds

Depth, inches	Depth, millimeters	Cumulative Blows
0.0	0	0
7.0	178	1
9.7	246	2
11.8	300	3
14.5	369	4
17.6	448	5
19.9	505	6
21.9	556	7
23.8	605	8
25.6	650	9
27.2	691	10
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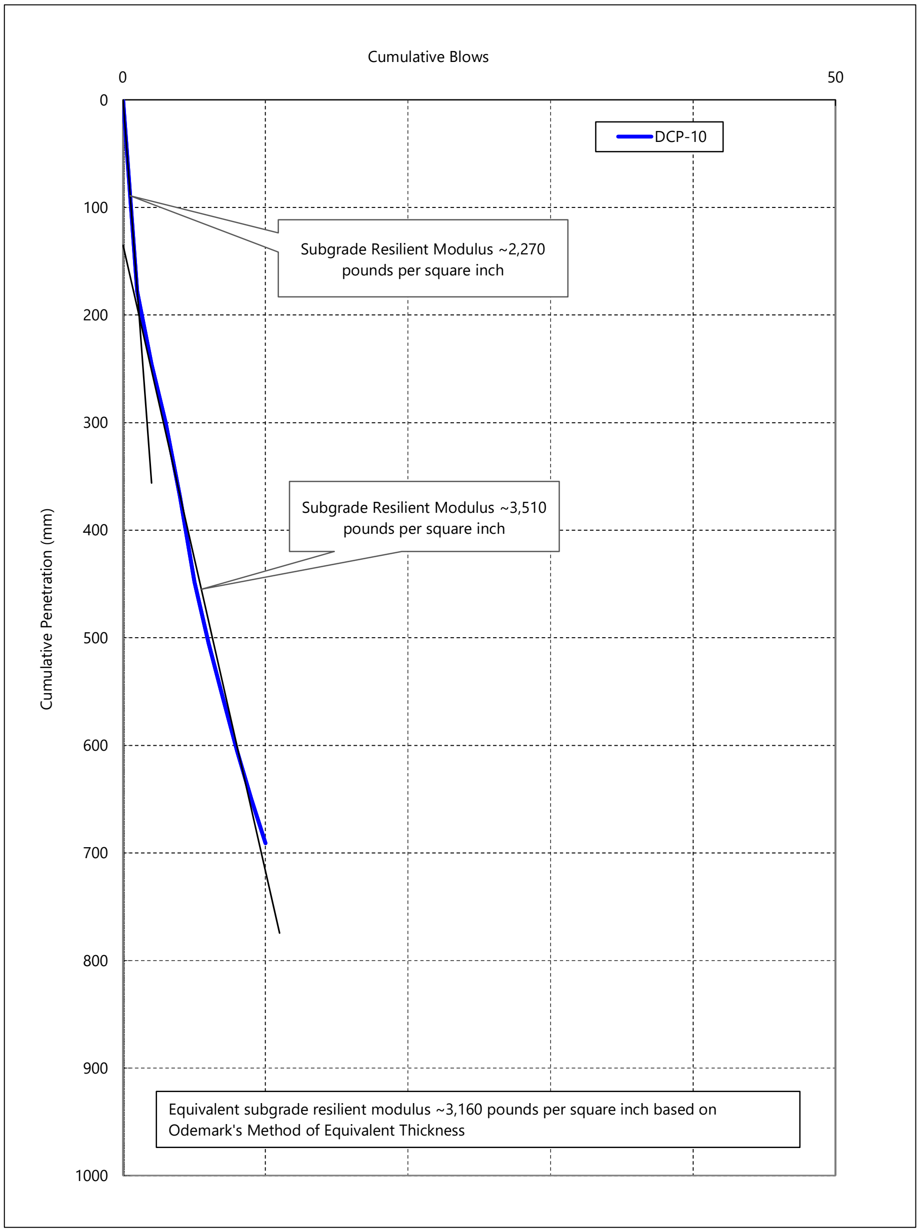


Subgrade Resilient Modulus ~2,270 pounds per square inch

Subgrade Resilient Modulus ~3,510 pounds per square inch

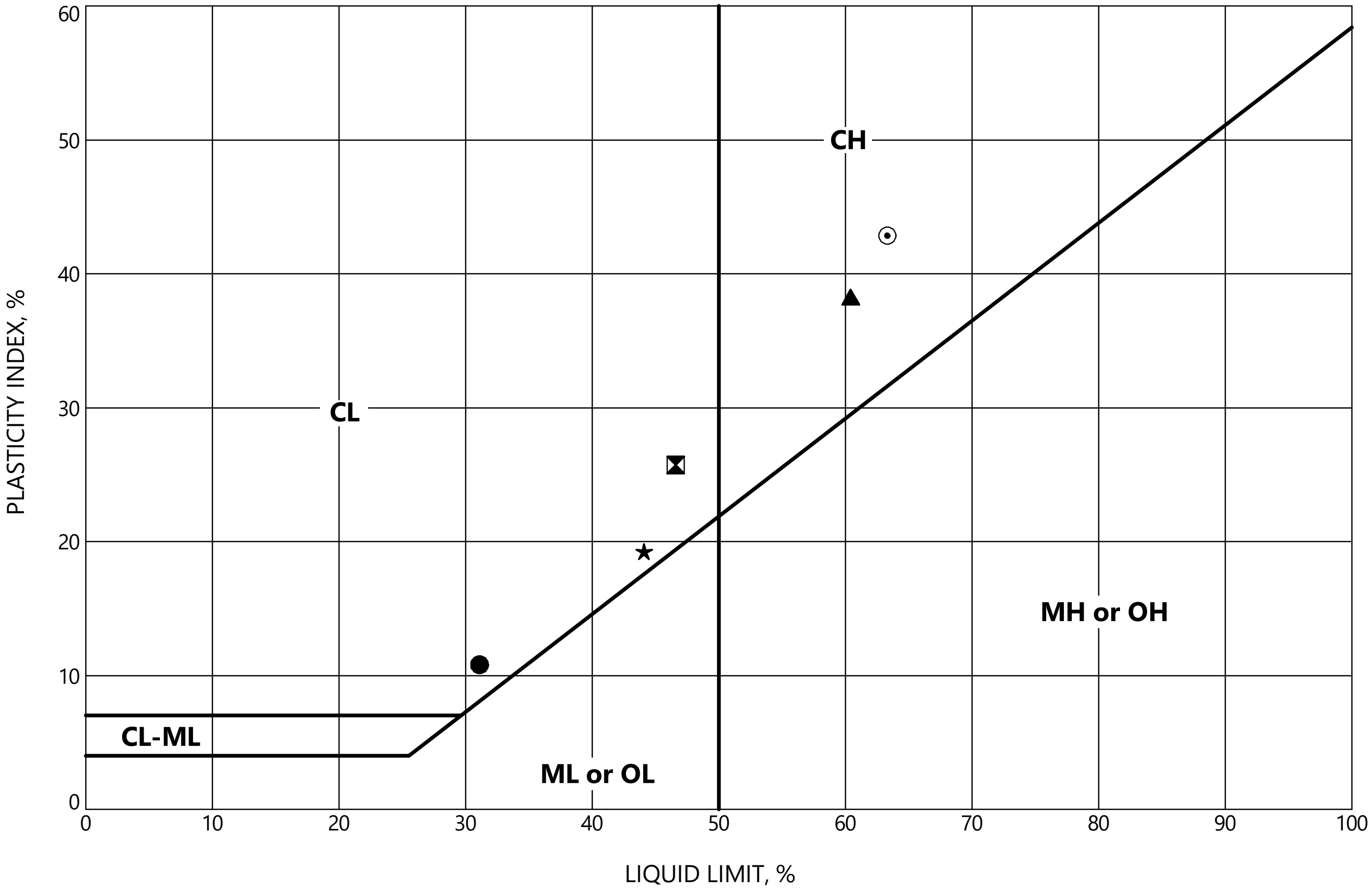
Equivalent subgrade resilient modulus ~3,160 pounds per square inch based on Odemark's Method of Equivalent Thickness

GRI

DYNAMIC CONE PENETROMETER

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-3	S-2	5.5	Clayey SAND, trace gravel, contains organics; SC; brown mottled red, black, and yellow	31	20	11	30
⊠	B-3	S-4	10.0	Silty CLAY, trace sand; CL; orange-brown	47	21	26	28
▲	B-16	S-3	7.5	CLAY; CH; gray	60	22	38	30
★	B-17	S-1	2.5	Silty CLAY, trace to some sand, contains organics; CL; brown and gray	44	25	19	33
⊙	B-17	S-5	12.5	CLAY, trace sand; CH; gray and brown	63	20	43	30

GRI

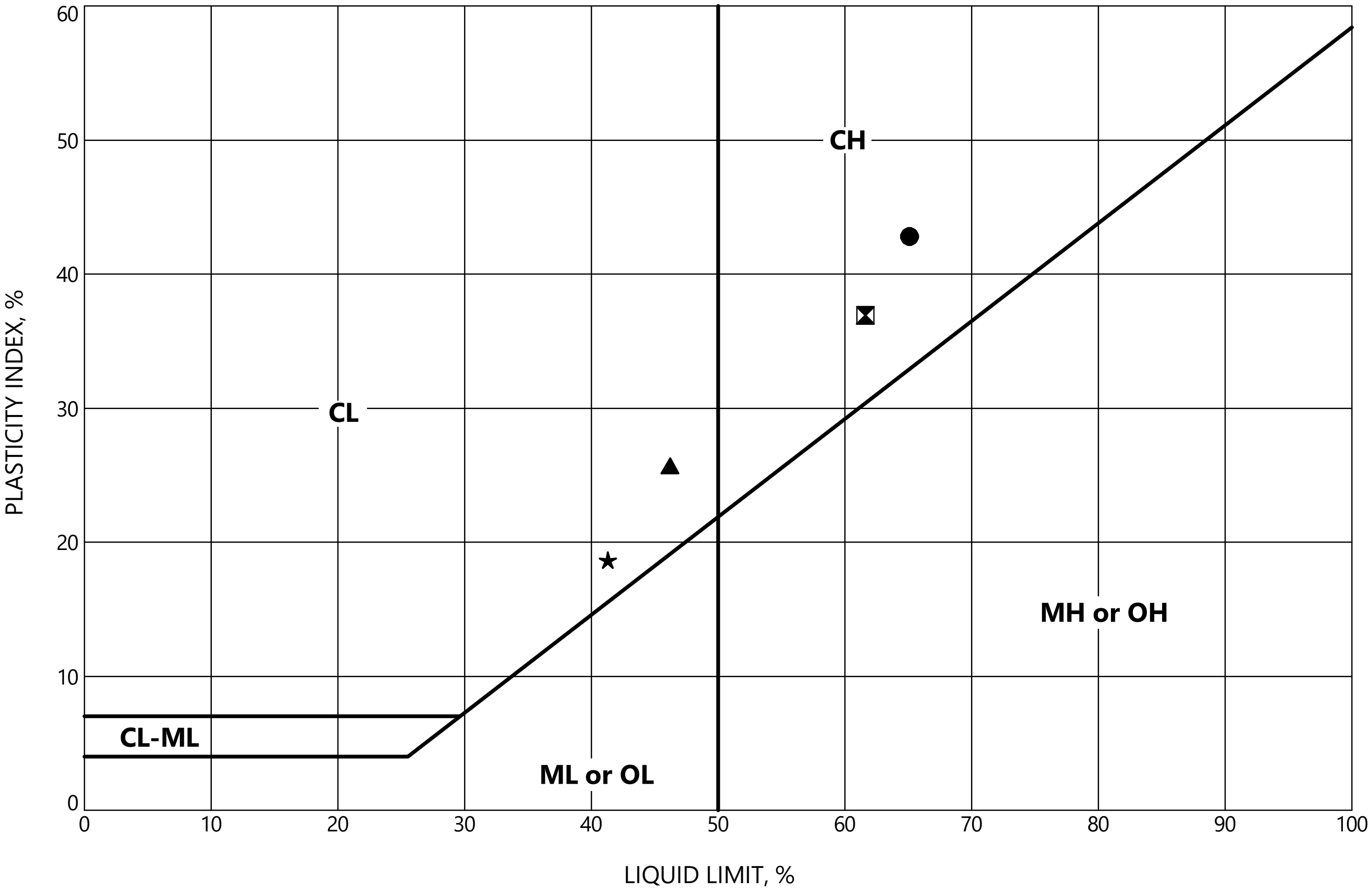
PLASTICITY CHART

OCT. 2025 JOB NO. 7072-A FIG. 39A

ATTERBERG-PLASTICITY 5 PER PAGE GRI DATA TEMPLATE.GDT 10/6/25

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



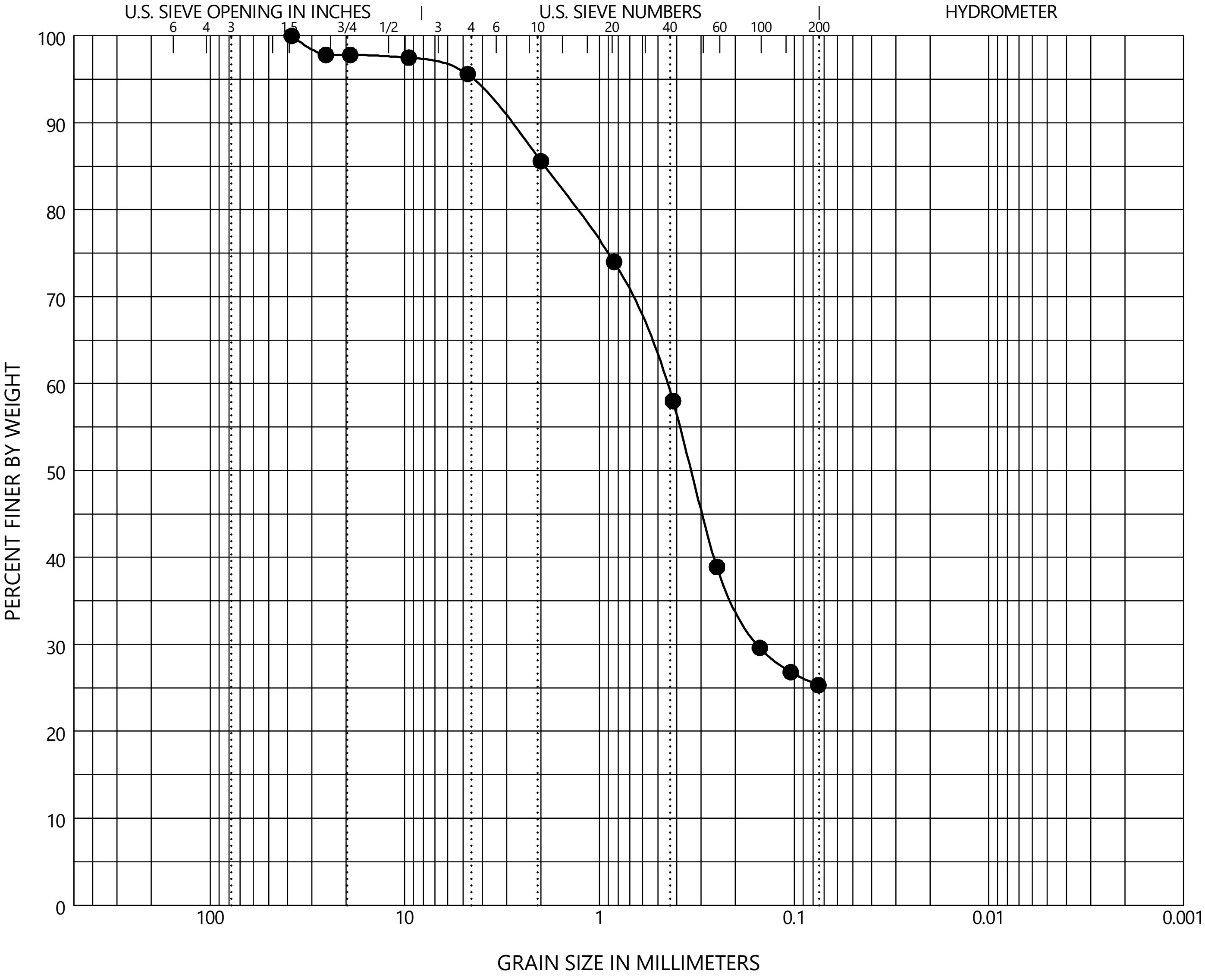
	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-18	S-3	8.0	CLAY, trace sand; CH; light gray mottled red-brown	65	22	43	34
⧗	B-18	S-5	13.0	CLAY, trace sand; CH; gray brown mottled white and black	62	25	37	34
▲	B-19	S-2	5.5	Silty CLAY, trace sand; CL; brown mottled gray	46	21	25	22
★	B-22	B-1	3.0	Silty CLAY, some sand, contains organics; CL; gray and brown	41	23	18	28

GRI

PLASTICITY CHART

OCT. 2025 JOB NO. 7072-A FIG. 40A

ATTERBERG-PLASTICITY 4 PER PAGE GRI DATA TEMPLATE.GDT 10/6/25



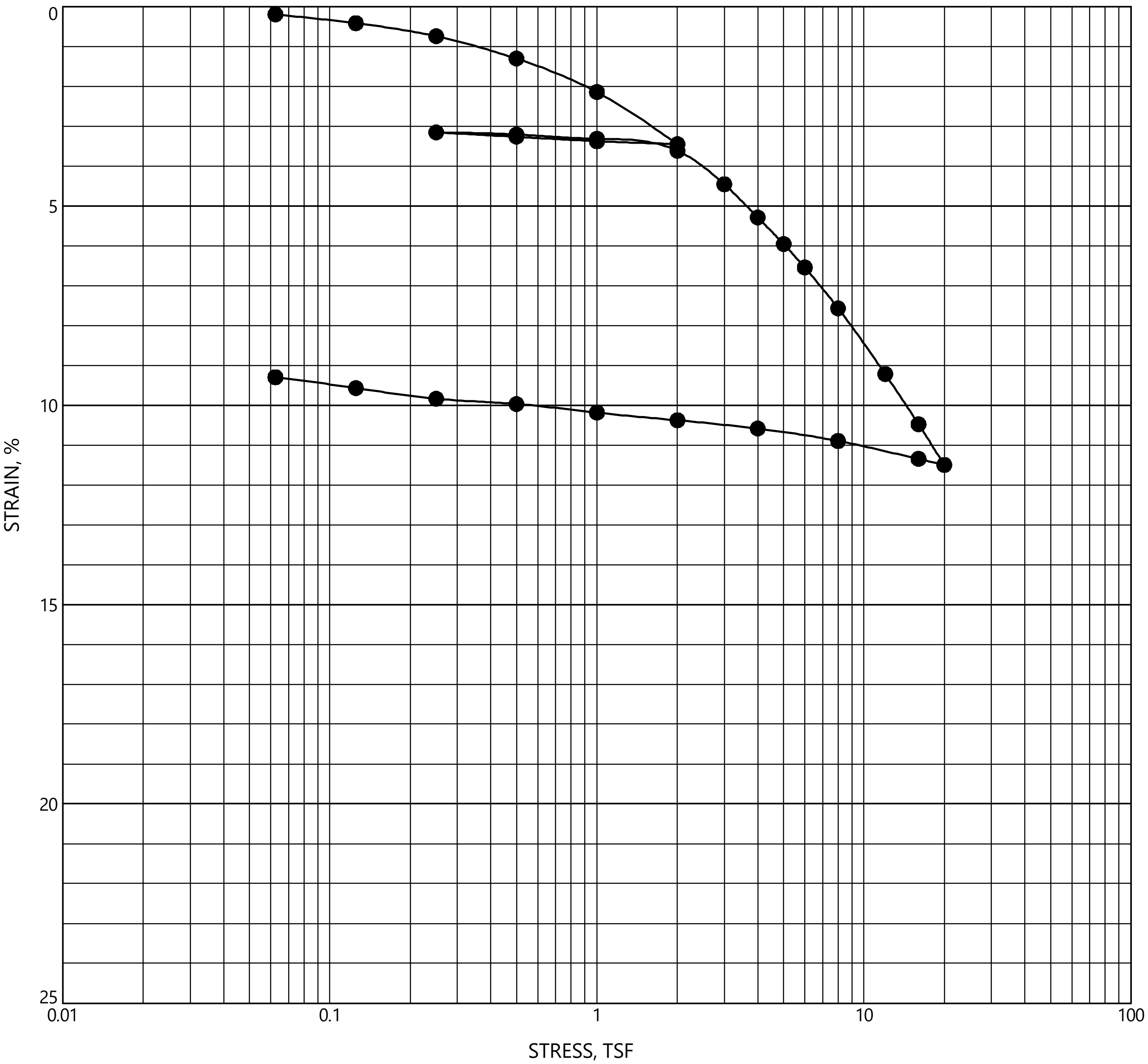
<table>
<tr><td rowspan="2">COBBLES</td><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT OR CLAY</td></tr>
<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

	Location	Sample	Depth, ft	Classification	Gravel, %	Sand, %	Fines, %
●	PC-1	S-1	0.0	Silty SAND; trace gravel; SM	4.4	70.3	25.3

GRI

GRAIN SIZE DISTRIBUTION

OCT. 2025 JOB NO. 7072-A FIG. 41A

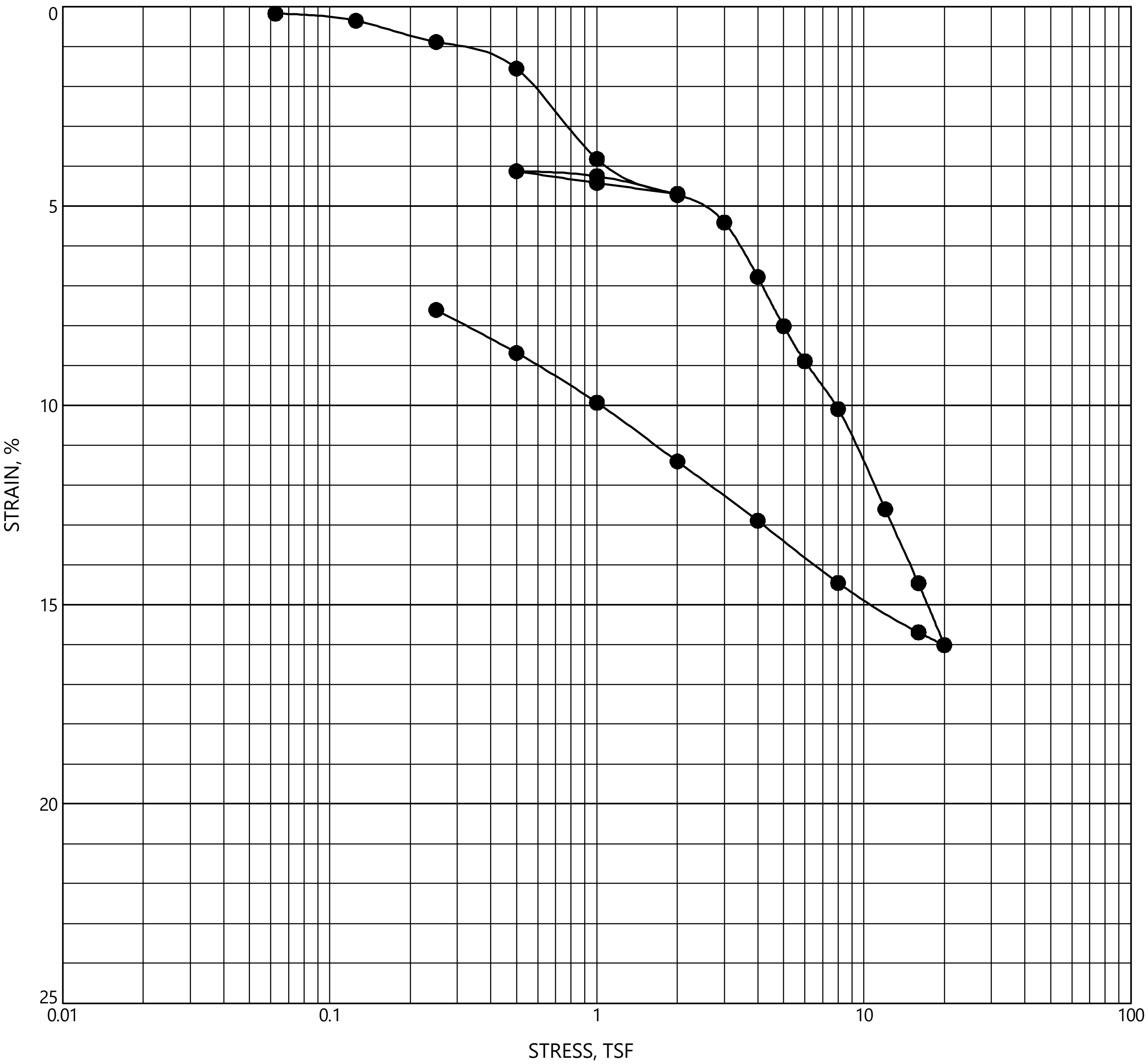


	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	MC, %
●	B-3	S-2	6.0	Clayey SAND, trace gravel, contains organics; SC; brown mottled red, black, and yellow	94	30

GRI

CONSOLIDATION TEST

OCT. 2025 JOB NO. 7072-A FIG. 42A

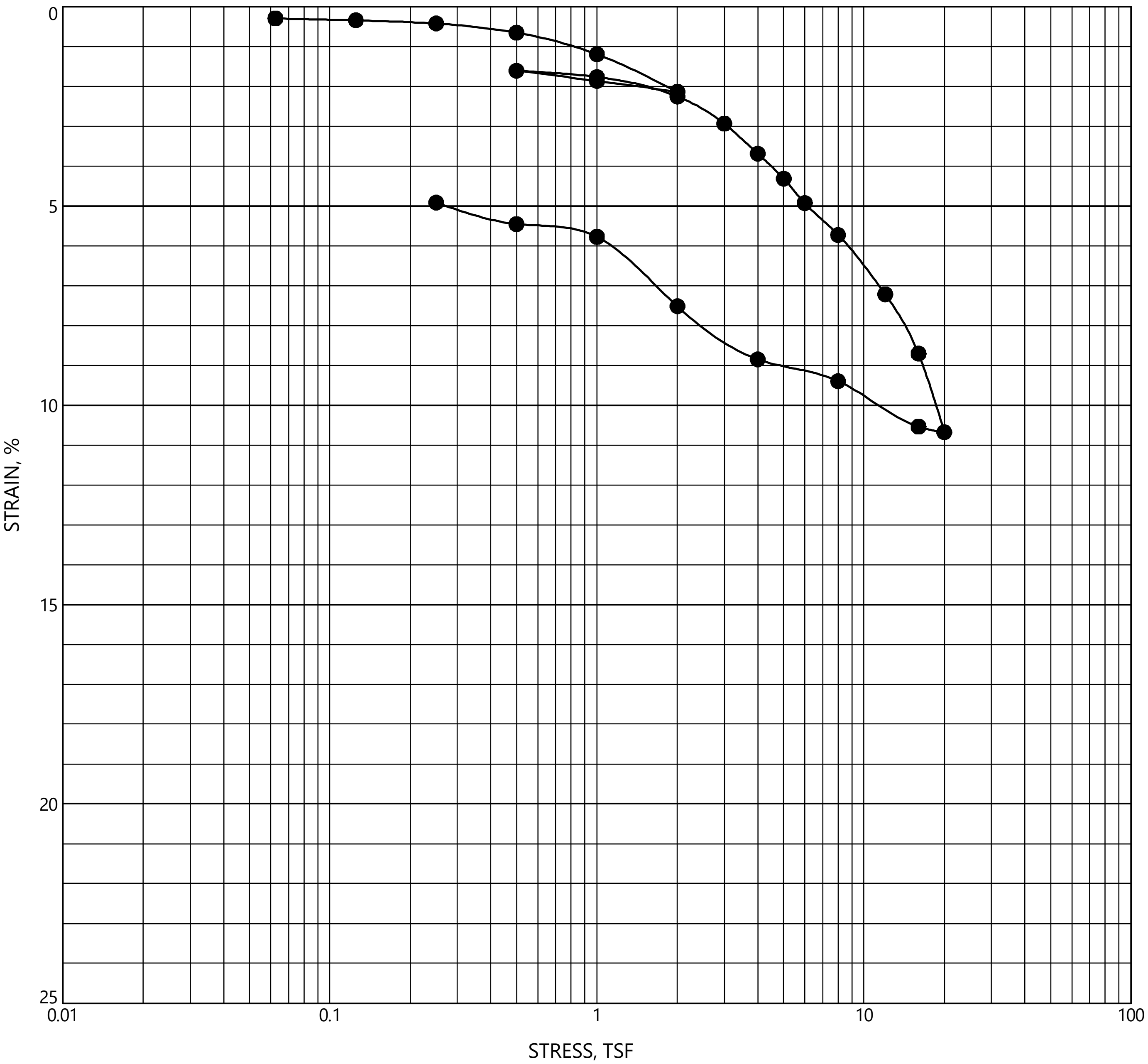


	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	MC, %
●	B-18	S-3	8.4	CLAY, trace sand; CH; light gray mottled red-brown	87	34

GRI

CONSOLIDATION TEST

OCT. 2025 JOB NO. 7072-A FIG. 43A

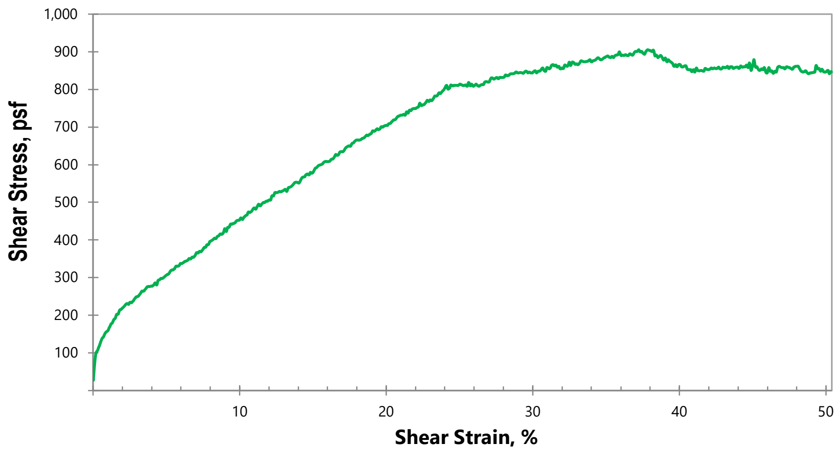


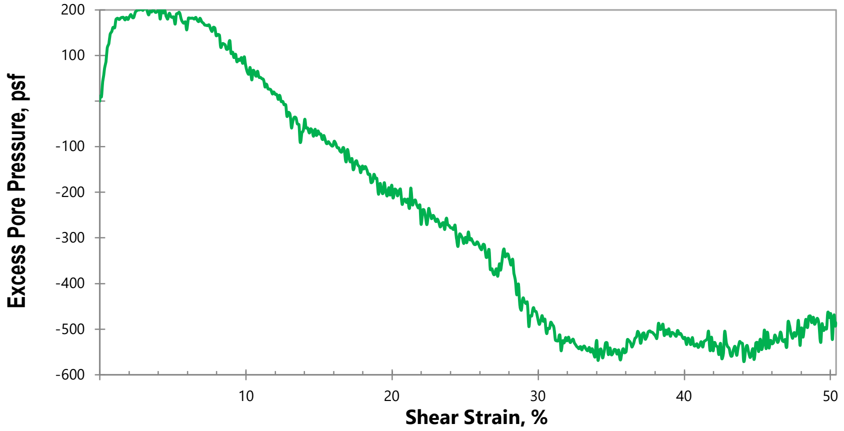
	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-18	S-5	13.4	CLAY, trace sand; CH; gray brown mottled white and black	80	40

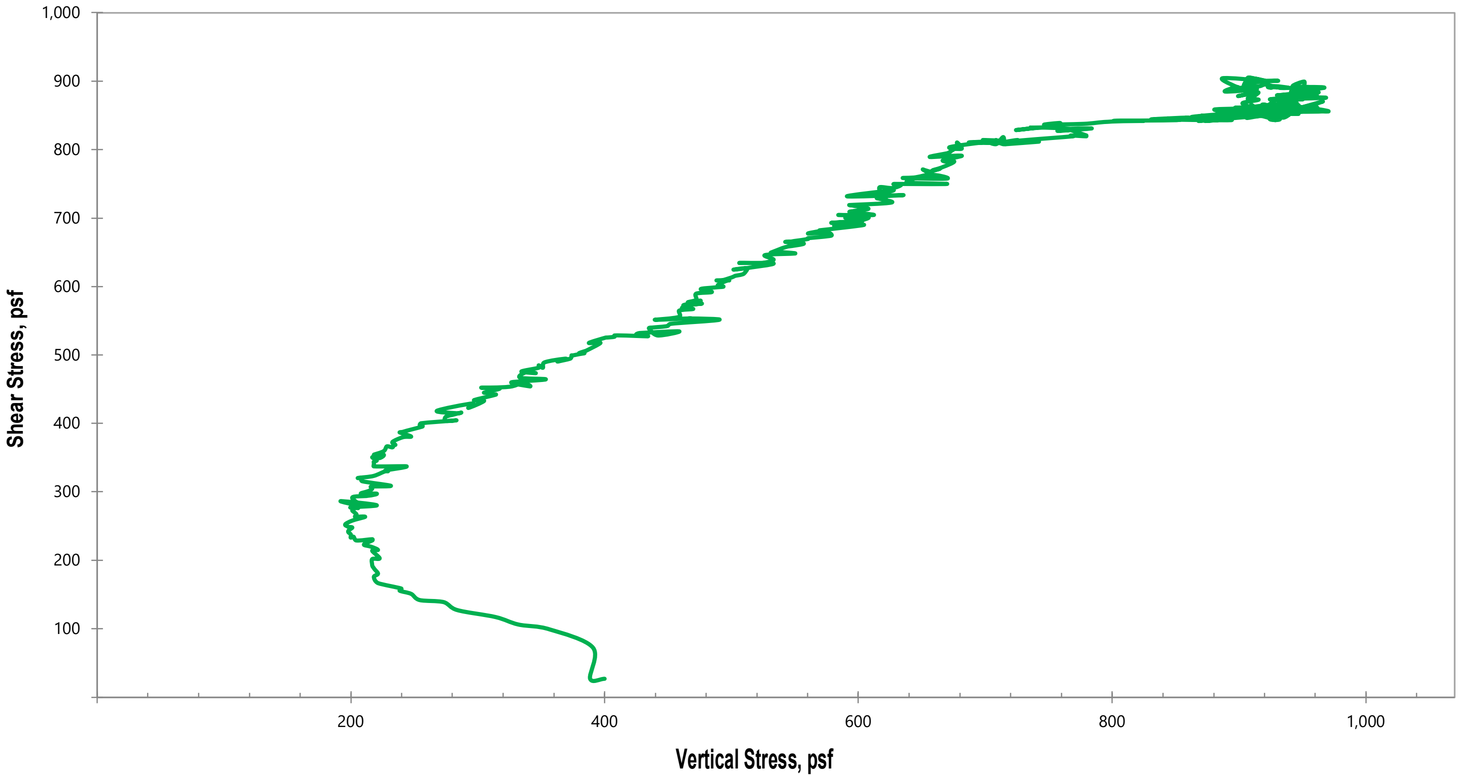
CONSOLIDATION TEST

OCT. 2025 JOB NO. 7072-A FIG. 44A

CONSOL STRAIN GRI-0 TO 25-1 PER PAGE GRI DATA TEMPLATE.GDT 10/6/25







	TEST 1
TEST SYMBOL	🟩
BORING NO.	B-3
SAMPLE NO.	S-2
DEPTH (FT)	6.3
VERTICAL EFFECTIVE CONSOLIDATION STRESS (PSF)	400
EST. OVERCONSOLIDATION RATIO	1.0
LIQUID LIMIT (%)	31
PLASTICITY INDEX (%)	11
FINES CONTENT (%)	67
DRY UNIT WEIGHT (PCF)	95
INITIAL WATER CONTENT (%)	28
FINAL WATER CONTENT (%)	29
STRAIN RATE (%/HR)	5

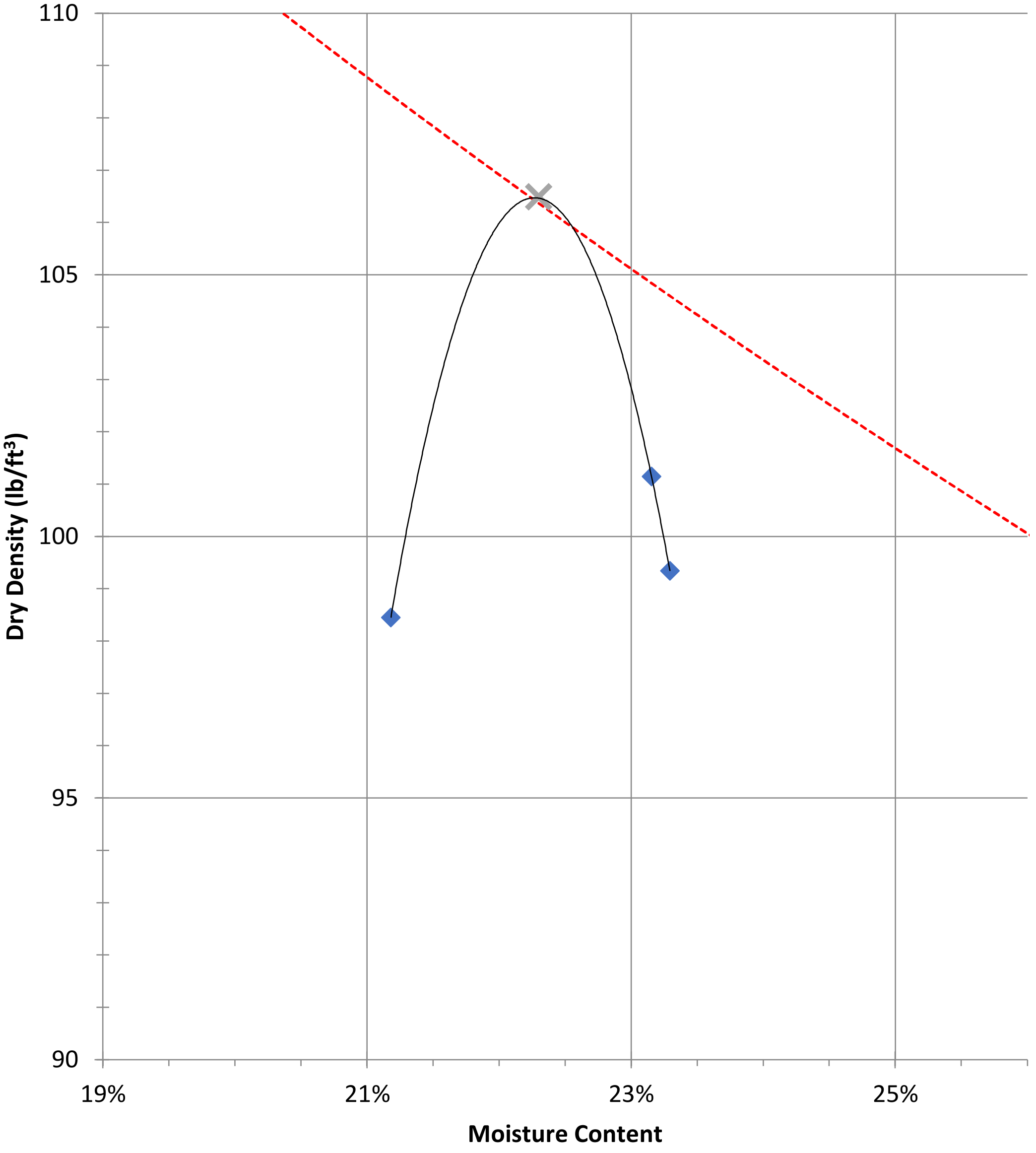
TYPE OF TEST:	■ CU	☐ CD
FAILURE CRITERIA:	■ MAX. SHEAR STRESS	☐ % SHEAR STRAIN
TYPE OF SAMPLE:	■ UNDISTURBED	☐ REMOLDED

SOIL CLASSIFICATION: Sandy Sity CLAY, trace gravel, contains organics; CL

GRI

DIRECT SIMPLE SHEAR STRENGTH
(BORING B-3, S-2)

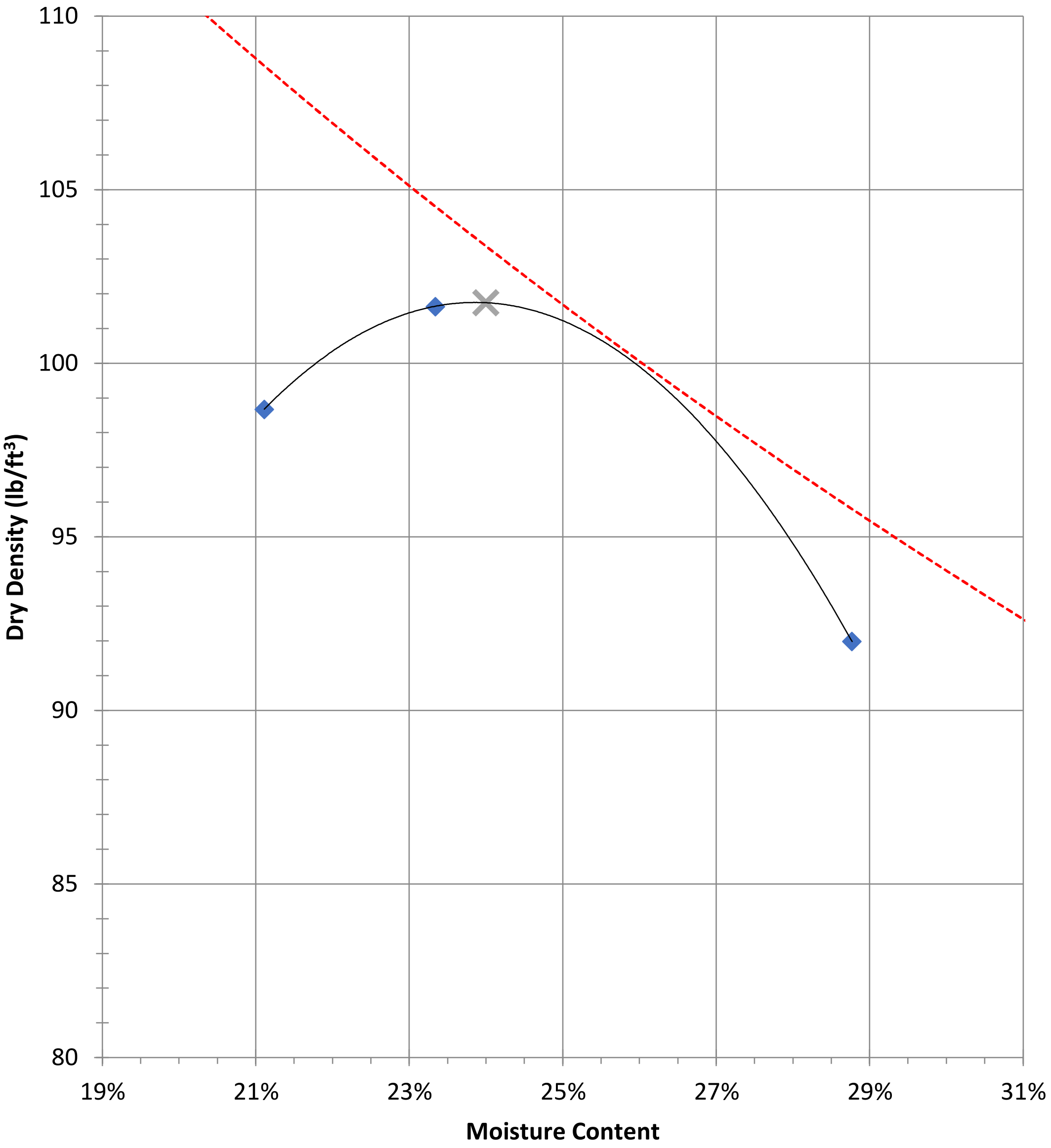
OCT. 2025 JOB NO. 7072-A FIG. 45A



Exploration No:	B-14	**Maximum Density w/ Oversize Correction (lb/ft^3):**	106.5
Sample Depth:	5-7’		
Maximum Density (lb/ft^3):	106.5	**Optimum Moisture w/ Oversized Correction (%):**	22.3
Optimum Moisture:	22.3	**Insitu Moisture Content (%):**	24.7
Project Name:	Happy Valley CC	**Date Tested:**	5/2/2025
Project No:	7072-A	**Tested By:**	ORE
Material Source:	On site	**Method:**	ASTM D698
Material Description:	Silty CLAY, trace sand; CL; brown, gray; medium to high plasticity; moist; stiff; fine to coarse sand	**Mold Size:**	4 in.

GRI

PROCTOR TEST



Exploration No:	B-22	**Maximum Density w/ Oversize Correction (lb/ft^3):**	101.8
Sample Depth:	3-5’		
Maximum Density (lb/ft^3):	101.75	**Optimum Moisture w/ Oversized Correction (%):**	24.0
Optimum Moisture:	24	**Insitu Moisture Content (%):**	28.1
Project Name:	Happy Valley CC	**Date Tested:**	5/2/2025
Project No:	7072-A	**Tested By:**	ORE
Material Source:	On site	**Method:**	ASTM D698
Material Description:	Silty CLAY, some sand, contains organics; CL; gray and brown; medium plasticity; moist; soft; fine to coarse sand; organics consist of fine roots	**Mold Size:**	4 in.

GRI

PROCTOR TEST

APPENDIX B

Cone Penetration Test Results

APPENDIX B

CONE PENETRATION TEST RESULTS

B.1 CONE PENETRATION TEST PROBES

Two cone penetration test (CPT) probes, designated CPT-1 and CPT-2, were advanced to depths of 25.9 feet and 19.7 feet, respectively, on April 21, 2025. The CPT probes were advanced using a track-mounted CPT rig provided and operated by Oregon Geotechnical Explorations, Inc., of Kaiser, Oregon. During a CPT, a steel cone is forced vertically into the soil at a constant rate of penetration. The force required to cause penetration at a constant rate can be related to the bearing capacity of the soil immediately surrounding the point of the penetrometer cone. This force is measured and recorded every 2 inches. In addition to the cone tip measurements, measurements are also obtained of the magnitude of force required to force a friction sleeve attached above the cone through the soil. The force required to move the friction sleeve can be related to the undrained shear strength of fine-grained soils. The dimensionless ratio of sleeve friction to point-bearing capacity provides an indicator of the type of soil penetrated. The cone penetration tip resistance and sleeve friction can be used to evaluate the relative consistency of cohesionless and cohesive soils, respectively. In addition, a vibrating-wire piezometer fitted between the cone and the sleeve measures changes in water pressure as the probe is advanced and can also be used to measure the depth to the top of the groundwater table. The probe was also operated using an accelerometer fitted to it, which allows measurement of the arrival time of shear waves from impulses generated at the ground surface. This allows the calculation of shear-wave velocities for the surrounding soil profile.

Logs of the two CPT probes and shear-wave velocity measurements recorded are provided in this appendix. The CPT logs present a graphical summary of the tip resistance, local (sleeve) friction, friction ratio, pore pressure, and soil behavior type index. The terms used to describe the soils encountered in the probe are defined in Table 1B.

Table 1B

SOIL CHARACTERIZATION BASED ON CONE PENETRATION TEST

Description of Relative Consistency for Cohesive (Fine-Grained) Soils

Cone Tip Resistance, tsf	Relative Consistency
<5	Very Soft
5 - 15	Soft to Medium Stiff
15 - 30	Stiff
30 - 60	Very Stiff
>60	Hard

Description of Relative Density for Cohesionless (Coarse-Grained) Soils

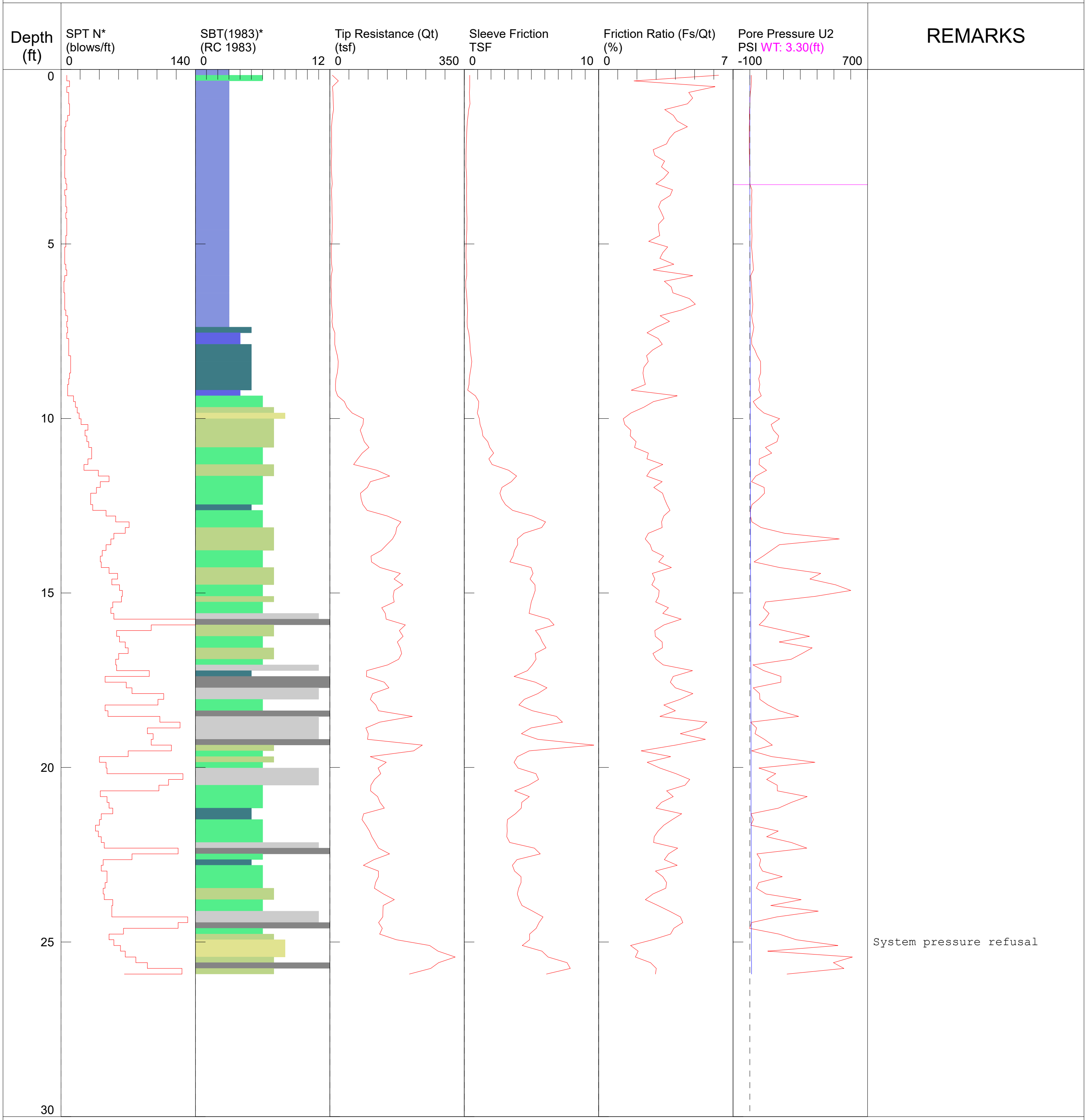
Cone Tip Resistance, tsf	Relative Density
<20	Very Loose
20 - 40	Loose
40 - 120	Medium Dense
120 - 200	Dense
>200	Very Dense

Reference

Kulhawy, F. H., and Mayne, P. W., 1990, Manual on Estimating Soil Properties for Foundation Design, Electric Power Research Institute, EL-6800.

GRI / CPT-1 / 19402 SE Foster Rd Happy Valley

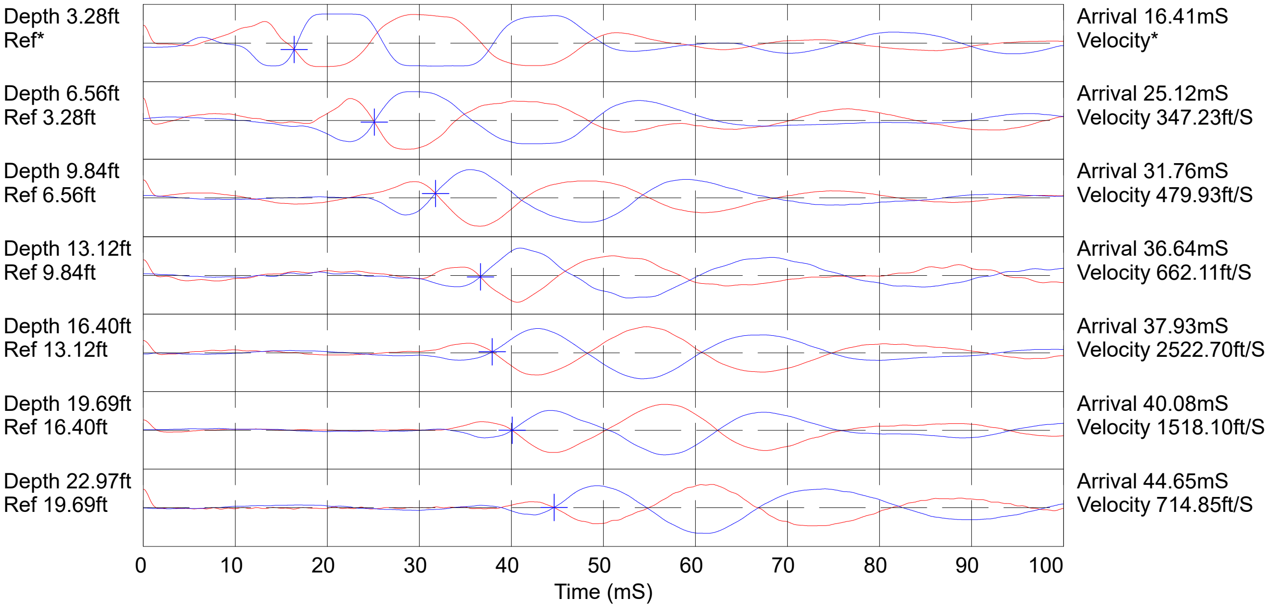
OPERATOR: OGE DMM
TEST DATE: 4/21/2025 11:45:51 AM
CONE ID: DDG1296
TOTAL DEPTH: 25.919 ft
HOLE NUMBER: CPT-1



🟥 1 sensitive fine grained
🟥 2 organic material
🟦 3 clay
🟦 4 silty clay to clay
🟦 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt
🟩 7 silty sand to sandy silt
🟨 8 sand to silty sand
🟫 9 sand
🟧 10 gravelly sand to sand
⬜ 11 very stiff fine grained (*)
⬛ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GRI / CPT-1 / 19402 SE Foster Rd Happy Valley

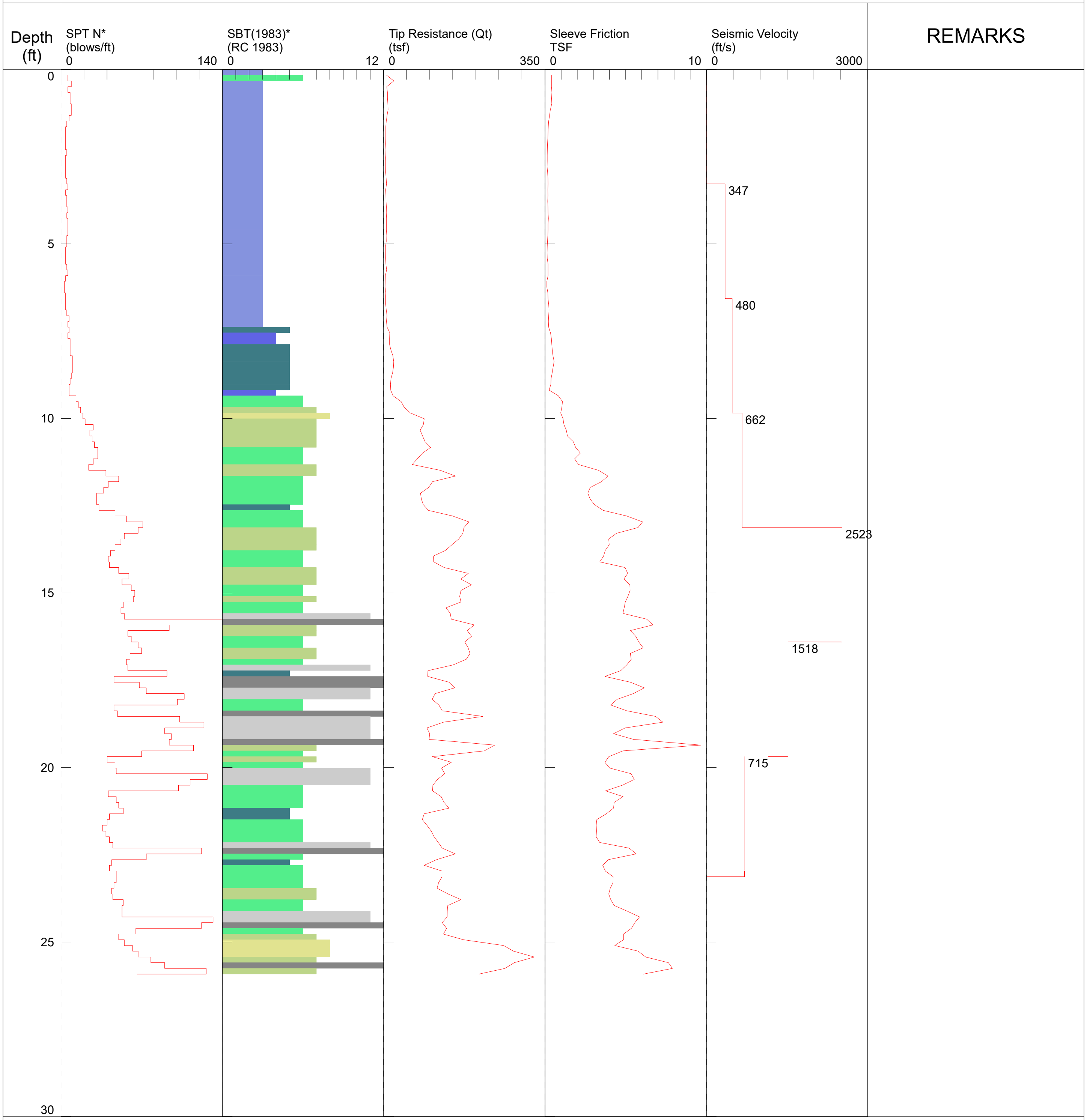


Hammer to Rod String Distance (ft): 1.97
* = Not Determined

COMMENT:

GRI / CPT-1 / 19402 SE Foster Rd Happy Valley

OPERATOR: OGE DMM
TEST DATE: 4/21/2025 11:45:51 AM
CONE ID: DDG1296
TOTAL DEPTH: 25.919 ft
HOLE NUMBER: CPT-1

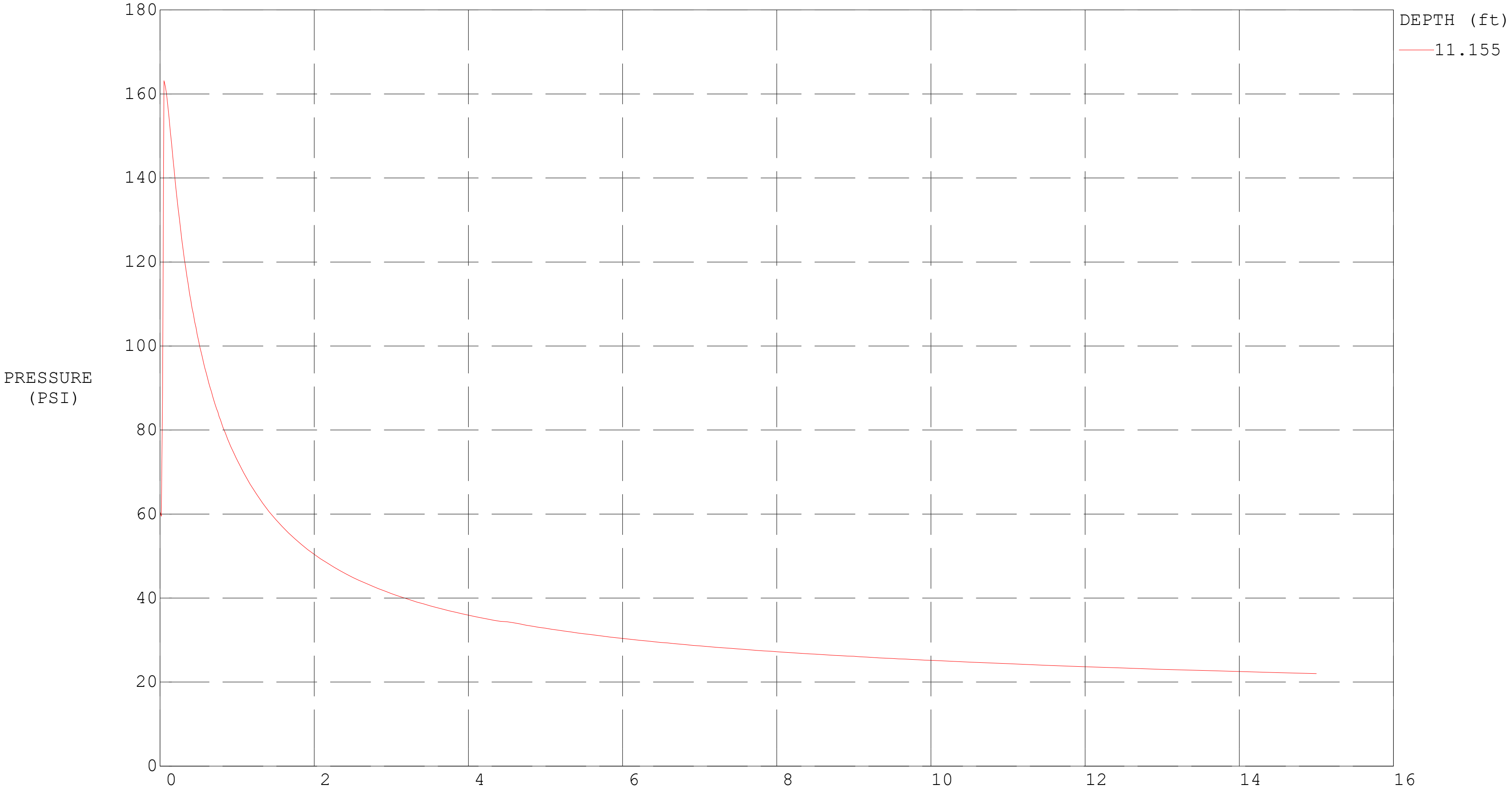


🟥 1 sensitive fine grained
🟥 2 organic material
🟦 3 clay
🟦 4 silty clay to clay
🟦 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt
🟩 7 silty sand to sandy silt
🟨 8 sand to silty sand
🟫 9 sand
🟧 10 gravelly sand to sand
⬜ 11 very stiff fine grained (*)
⬛ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GRI / CPT-1 / 19402 SE Foster Rd Happy Valley

OPERATOR: OGE DMM
CONE ID: DDG1296
TEST DATE: 4/21/2025 11:45:51 AM



MAXIMUM PRESSURE = 163.161 (PSI) TIME: (MINUTES)
HYDROSTATIC PRESSURE = 3.404 (PSI), WATER TABLE: 3.30 ft

GRI / CPT-1 / 19402 SE Foster Rd Happy Valley

OPERATOR: OGE DMM
TEST DATE: 4/21/2025 11:45:51 AM
CONE ID: DDG1296
TOTAL DEPTH: 25.919 ft
HOLE NUMBER: CPT-1

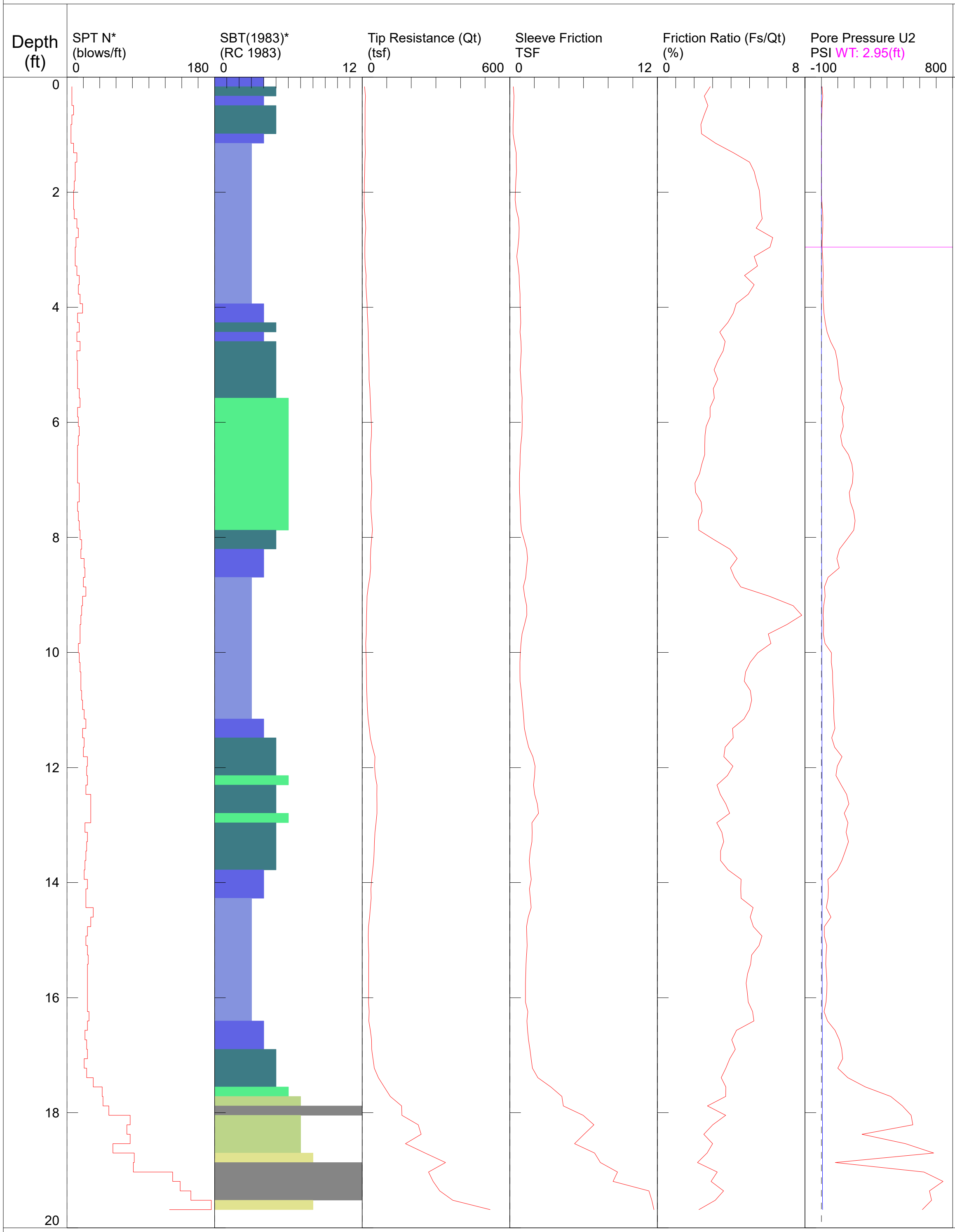
Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) TSF	Fr (Fs/Qt) (%)	Pressure (U2) PSI	SPT N* (blows/ft)	Zone	Soil Behavior Type UBC-1983
0.164	6.54	0.4086	6.250	8.653	6	3	clay
0.328	22.22	0.4091	1.842	8.204	9	6	sandy silt to clayey silt
0.492	6.73	0.4080	6.063	7.242	6	3	clay
0.656	8.19	0.3840	4.690	3.055	8	3	clay
0.820	8.28	0.4051	4.895	0.772	8	3	clay
0.984	9.16	0.4230	4.618	-0.719	9	3	clay
1.148	9.59	0.3301	3.442	-2.158	9	3	clay
1.312	7.36	0.2863	3.890	-3.195	7	3	clay
1.476	5.67	0.2317	4.089	-2.066	5	3	clay
1.640	4.45	0.2057	4.621	-3.487	4	3	clay
1.804	4.64	0.1835	3.957	-3.878	4	3	clay
1.969	4.65	0.1716	3.688	-3.460	4	3	clay
2.133	4.32	0.1525	3.532	-3.557	4	3	clay
2.297	4.94	0.1402	2.839	-3.234	5	3	clay
2.461	4.59	0.1341	2.924	-2.944	4	3	clay
2.625	3.94	0.1353	3.431	-2.300	4	3	clay
2.789	3.91	0.1275	3.266	-1.553	4	3	clay
2.953	4.43	0.1611	3.641	-0.856	4	3	clay
3.117	5.38	0.1820	3.386	-0.307	5	3	clay
3.281	6.25	0.1864	2.983	-0.535	6	3	clay
3.445	4.45	0.1709	3.846	12.068	4	3	clay
3.609	4.82	0.1796	3.728	11.020	5	3	clay
3.773	5.24	0.1698	3.243	11.789	5	3	clay
3.937	5.77	0.1802	3.124	10.496	6	3	clay
4.101	5.72	0.1869	3.269	10.526	5	3	clay
4.265	6.15	0.2089	3.399	10.855	6	3	clay
4.429	6.15	0.1918	3.120	10.443	6	3	clay
4.593	6.14	0.1919	3.127	11.098	6	3	clay
4.757	5.44	0.1731	3.183	12.511	5	3	clay
4.921	5.74	0.1494	2.605	11.393	5	3	clay
5.085	3.85	0.1383	3.595	9.503	4	3	clay
5.249	3.94	0.1317	3.341	13.261	4	3	clay
5.413	4.25	0.1355	3.187	15.672	4	3	clay
5.577	4.77	0.1866	3.912	18.934	5	3	clay
5.741	6.51	0.1847	2.836	21.630	6	3	clay
5.906	3.74	0.1832	4.903	4.898	4	3	clay
6.070	3.34	0.1141	3.418	8.003	3	3	clay
6.234	3.11	0.1175	3.782	10.289	3	3	clay
6.398	4.46	0.1722	3.863	12.090	4	3	clay
6.562	4.05	0.1911	4.720	13.604	4	3	clay
6.726	4.40	0.2216	5.034	17.646	4	3	clay
6.890	5.72	0.2467	4.317	15.631	5	3	clay
7.054	7.15	0.2287	3.200	10.674	7	3	clay
7.218	5.79	0.2134	3.689	15.536	6	3	clay
7.382	7.54	0.2275	3.018	23.138	7	3	clay
7.546	13.29	0.3342	2.516	17.618	6	5	clayey silt to silty clay
7.710	12.75	0.3953	3.101	9.539	8	4	silty clay to clay
7.874	12.74	0.4225	3.317	11.513	8	4	silty clay to clay
8.038	15.88	0.4459	2.808	29.734	8	5	clayey silt to silty clay
8.202	19.89	0.4968	2.498	43.837	10	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) TSF	Fr (Fs/Qt) (%)	Pressure (U2) PSI	SPT N* (blows/ft)	Zone	Soil Behavior Type UBC-1983
8.366	21.77	0.5619	2.581	63.364	10	5	clayey silt to silty clay
8.530	21.36	0.5033	2.357	64.301	10	5	clayey silt to silty clay
8.694	19.19	0.4428	2.308	64.039	9	5	clayey silt to silty clay
8.858	15.91	0.3744	2.354	52.718	8	5	clayey silt to silty clay
9.022	14.71	0.3571	2.428	58.954	7	5	clayey silt to silty clay
9.186	15.17	0.2574	1.697	53.552	7	5	clayey silt to silty clay
9.350	20.34	0.8325	4.093	67.791	13	4	silty clay to clay
9.514	37.91	1.0831	2.858	19.993	15	6	sandy silt to clayey silt
9.678	44.57	1.0465	2.348	42.487	17	6	sandy silt to clayey silt
9.843	57.99	0.9751	1.682	83.500	19	7	silty sand to sandy silt
10.007	87.85	1.1241	1.280	177.066	21	8	sand to silty sand
10.171	86.27	1.1742	1.361	126.511	28	7	silty sand to sandy silt
10.335	79.10	1.3221	1.672	140.338	25	7	silty sand to sandy silt
10.499	84.52	1.3878	1.642	171.717	27	7	silty sand to sandy silt
10.663	89.88	1.7499	1.947	161.363	29	7	silty sand to sandy silt
10.827	101.79	1.9014	1.868	93.042	32	7	silty sand to sandy silt
10.991	84.56	2.1894	2.590	129.854	32	6	sandy silt to clayey silt
11.155	73.02	1.8346	2.513	56.827	28	6	sandy silt to clayey silt
11.319	61.96	2.0698	3.341	54.603	24	6	sandy silt to clayey silt
11.483	122.51	3.3044	2.698	99.181	39	7	silty sand to sandy silt
11.647	155.77	3.8956	2.502	35.995	50	7	silty sand to sandy silt
11.811	105.92	3.4975	3.303	9.414	41	6	sandy silt to clayey silt
11.975	97.72	2.8063	2.872	84.788	37	6	sandy silt to clayey silt
12.139	79.86	2.6539	3.324	86.856	31	6	sandy silt to clayey silt
12.303	81.56	2.7945	3.427	52.950	31	6	sandy silt to clayey silt
12.467	86.07	3.0640	3.561	12.871	33	6	sandy silt to clayey silt
12.631	97.15	3.6054	3.712	2.567	47	5	clayey silt to silty clay
12.795	149.81	5.0708	3.386	3.044	57	6	sandy silt to clayey silt
12.959	184.92	6.0496	3.272	11.901	71	6	sandy silt to clayey silt
13.123	174.06	5.7633	3.312	66.637	67	6	sandy silt to clayey silt
13.287	171.43	4.4362	2.588	207.098	55	7	silty sand to sandy silt
13.451	163.06	3.9475	2.421	531.999	52	7	silty sand to sandy silt
13.615	148.17	3.9752	2.684	175.823	47	7	silty sand to sandy silt
13.780	134.06	3.7351	2.787	126.575	43	7	silty sand to sandy silt
13.944	107.37	3.6308	3.382	77.869	41	6	sandy silt to clayey silt
14.108	108.62	3.3947	3.126	24.652	42	6	sandy silt to clayey silt
14.272	131.16	4.9656	3.787	176.656	50	6	sandy silt to clayey silt
14.436	183.61	5.1238	2.791	421.982	59	7	silty sand to sandy silt
14.600	167.36	4.8903	2.923	356.931	53	7	silty sand to sandy silt
14.764	190.31	5.2618	2.766	509.973	61	7	silty sand to sandy silt
14.928	167.58	5.2759	3.149	600.910	64	6	sandy silt to clayey silt
15.092	164.79	5.1344	3.117	389.137	63	6	sandy silt to clayey silt
15.256	167.68	4.9779	2.969	93.354	54	7	silty sand to sandy silt
15.420	135.22	4.9093	3.632	79.667	52	6	sandy silt to clayey silt
15.584	144.69	4.8277	3.337	114.449	55	6	sandy silt to clayey silt
15.748	146.66	6.3004	4.297	89.817	140	11	very stiff fine grained (*)
15.912	196.75	6.6857	3.399	54.653	94	12	sand to clayey sand (*)
16.076	181.21	5.2985	2.925	202.381	58	7	silty sand to sandy silt
16.240	190.64	5.6230	2.950	354.801	61	7	silty sand to sandy silt
16.404	175.89	5.8062	3.302	174.984	67	6	sandy silt to clayey silt
16.568	183.26	6.0854	3.321	370.744	70	6	sandy silt to clayey silt
16.732	187.10	5.2817	2.824	310.326	60	7	silty sand to sandy silt
16.896	179.23	5.3492	2.985	245.705	57	7	silty sand to sandy silt
17.060	150.62	5.0639	3.363	18.823	58	6	sandy silt to clayey silt
17.224	96.03	4.6954	4.891	82.605	92	11	very stiff fine grained (*)
17.388	95.45	3.7071	3.885	184.941	46	5	clayey silt to silty clay
17.552	141.88	5.2898	3.729	184.420	68	12	sand to clayey sand (*)
17.717	154.06	6.1555	3.996	19.561	74	12	sand to clayey sand (*)
17.881	111.57	5.4794	4.912	57.622	107	11	very stiff fine grained (*)
18.045	105.28	4.4707	4.247	59.141	101	11	very stiff fine grained (*)
18.209	119.91	4.0677	3.393	108.135	46	6	sandy silt to clayey silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) TSF	Fr (Fs/Qt) (%)	Pressure (U2) PSI	SPT N* (blows/ft)	Zone	Soil Behavior Type UBC-1983
18.373	126.79	5.0702	4.000	174.853	49	6	sandy silt to clayey silt
18.537	215.07	6.8553	3.188	290.210	103	12	sand to clayey sand (*)
18.701	129.75	7.3099	5.635	4.112	124	11	very stiff fine grained (*)
18.865	93.85	4.9680	5.295	40.282	90	11	very stiff fine grained (*)
19.029	99.96	4.2512	4.254	30.746	96	11	very stiff fine grained (*)
19.193	98.35	5.4695	5.562	86.184	94	11	very stiff fine grained (*)
19.357	240.81	9.6409	4.005	133.322	115	12	sand to clayey sand (*)
19.521	218.42	4.8344	2.214	10.262	70	7	silty sand to sandy silt
19.685	105.56	3.9606	3.753	130.225	40	6	sandy silt to clayey silt
19.849	146.86	3.7099	2.527	387.560	47	7	silty sand to sandy silt
20.013	125.55	4.0065	3.192	54.578	48	6	sandy silt to clayey silt
20.177	132.58	5.3313	4.022	153.287	127	11	very stiff fine grained (*)
20.341	116.61	5.5344	4.747	100.262	112	11	very stiff fine grained (*)
20.505	106.74	4.7936	4.492	162.631	102	11	very stiff fine grained (*)
20.669	106.05	3.7532	3.540	163.326	41	6	sandy silt to clayey silt
20.833	124.69	4.8412	3.884	340.743	48	6	sandy silt to clayey silt
20.997	130.93	4.2781	3.268	247.352	50	6	sandy silt to clayey silt
21.161	142.01	4.2437	2.989	169.712	54	6	sandy silt to clayey silt
21.325	87.99	3.8012	4.321	6.994	42	5	clayey silt to silty clay
21.490	83.81	3.2134	3.835	21.186	40	5	clayey silt to silty clay
21.654	93.73	3.1773	3.391	5.422	36	6	sandy silt to clayey silt
21.818	103.08	3.1922	3.098	168.876	39	6	sandy silt to clayey silt
21.982	109.43	3.1632	2.891	100.388	42	6	sandy silt to clayey silt
22.146	118.57	3.3898	2.860	246.134	45	6	sandy silt to clayey silt
22.310	126.98	5.2176	4.110	338.139	122	11	very stiff fine grained (*)
22.474	155.48	5.6524	3.636	41.818	74	12	sand to clayey sand (*)
22.638	114.89	3.9253	3.418	63.824	44	6	sandy silt to clayey silt
22.802	87.64	3.5814	4.088	55.810	42	5	clayey silt to silty clay
22.966	126.60	3.7433	2.958	75.273	48	6	sandy silt to clayey silt
23.130	126.54	4.2215	3.337	193.324	48	6	sandy silt to clayey silt
23.294	119.18	4.2192	3.541	53.298	46	6	sandy silt to clayey silt
23.458	115.90	4.0468	3.493	39.170	44	6	sandy silt to clayey silt
23.622	139.92	3.9484	2.823	95.191	45	7	silty sand to sandy silt
23.786	167.88	4.0786	2.430	305.314	54	7	silty sand to sandy silt
23.950	138.92	4.2944	3.092	123.693	53	6	sandy silt to clayey silt
24.114	138.02	5.0914	3.690	407.101	53	6	sandy silt to clayey silt
24.278	137.74	5.8610	4.256	159.997	132	11	very stiff fine grained (*)
24.442	127.03	5.5700	4.386	9.420	122	11	very stiff fine grained (*)
24.606	136.67	5.3516	3.917	-1.759	65	12	sand to clayey sand (*)
24.770	129.71	4.8615	3.749	173.490	50	6	sandy silt to clayey silt
24.934	173.34	4.8613	2.805	275.759	55	7	silty sand to sandy silt
25.098	259.94	4.3195	1.662	523.580	62	8	sand to silty sand
25.262	281.63	5.7760	2.051	106.122	67	8	sand to silty sand
25.427	326.46	6.2379	1.911	609.834	78	8	sand to silty sand
25.591	283.33	7.6478	2.700	497.297	90	7	silty sand to sandy silt
25.755	262.77	7.8900	3.003	559.477	126	12	sand to clayey sand (*)
25.919	206.97	6.1100	2.953	220.677	66	7	silty sand to sandy silt

GRI / CPT-2 / 19402 SE Foster Rd Happy Valley

OPERATOR: OGE DMM
TEST DATE: 4/21/2025 8:35:24 AM
CONE ID: DDG1296
TOTAL DEPTH: 19.685 ft
HOLE NUMBER: CPT-2



REMARKS

Cone Tip sheared off

Lost tip and sleeve in the h-
ole

🟥 1 sensitive fine grained
■ 2 organic material
■ 3 clay

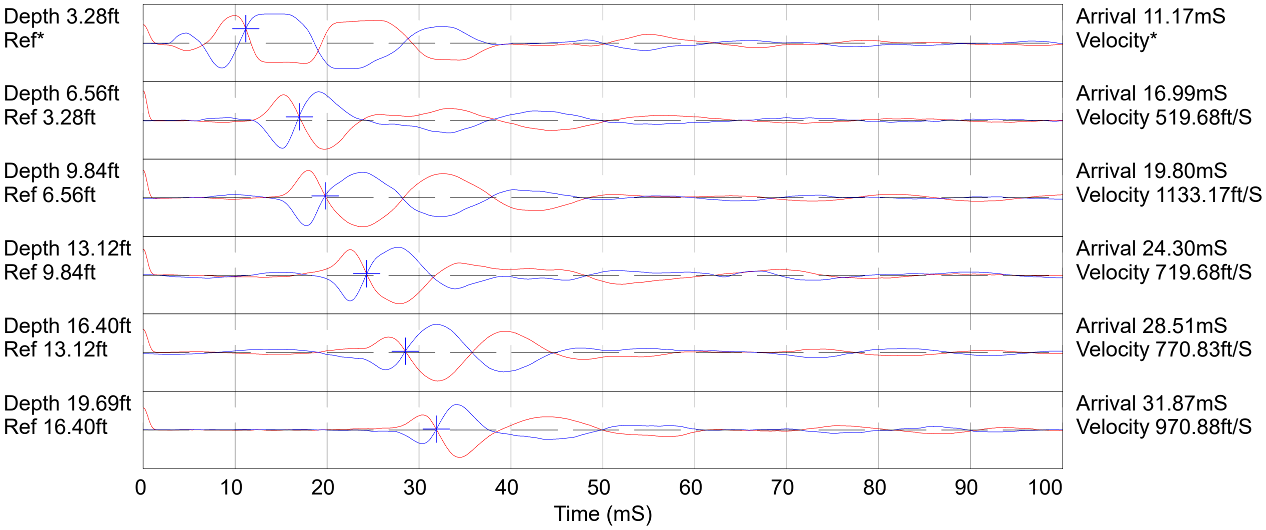
🟦 4 silty clay to clay
■ 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt

■ 7 silty sand to sandy silt
🟨 8 sand to silty sand
■ 9 sand

🟧 10 gravelly sand to sand
⬜ 11 very stiff fine grained (*)
⬛ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GRI / CPT-2 / 19402 SE Foster Rd Happy Valley

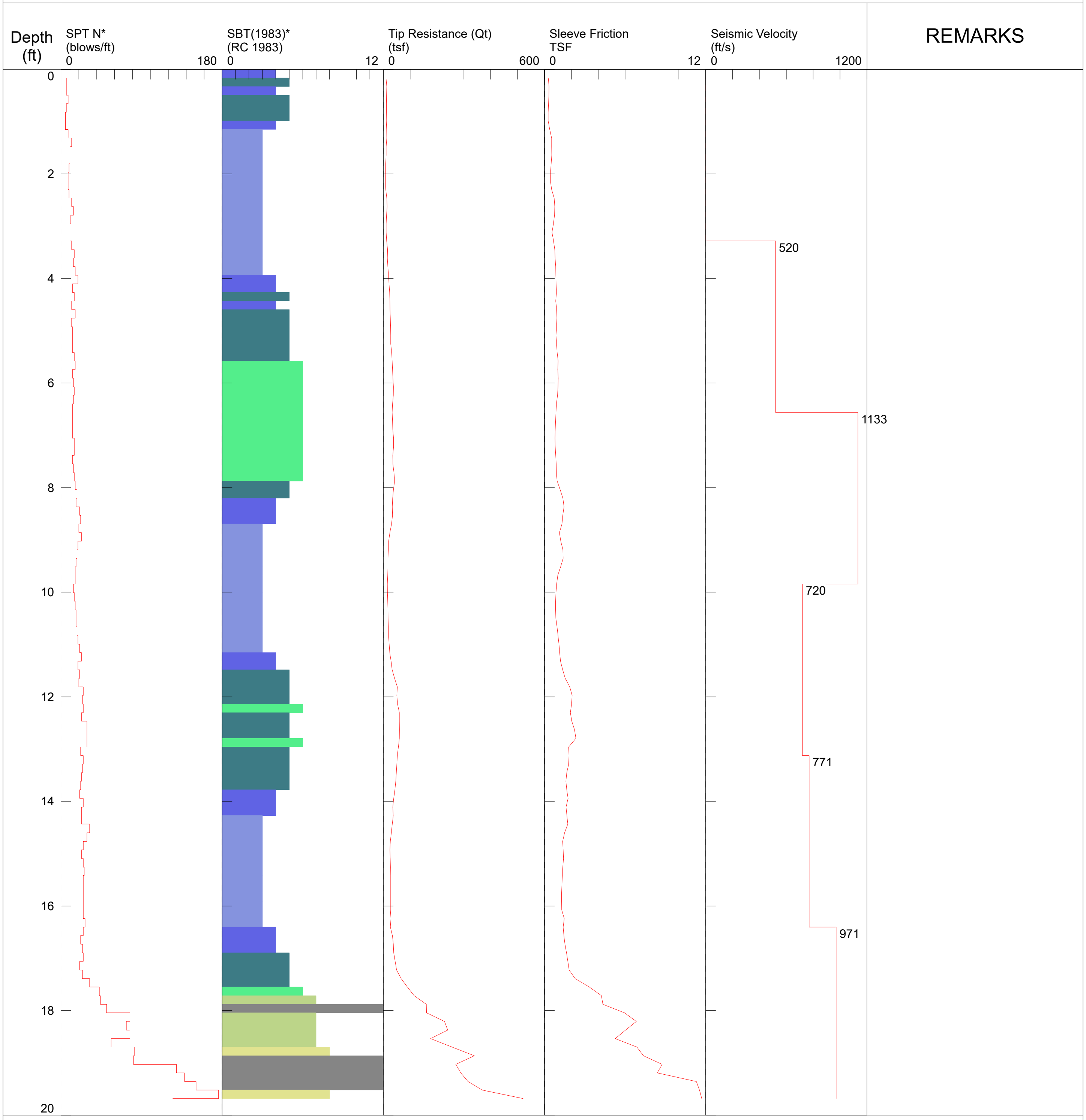


Hammer to Rod String Distance (ft): 1.97
* = Not Determined

COMMENT:

GRI / CPT-2 / 19402 SE Foster Rd Happy Valley

OPERATOR: OGE DMM
TEST DATE: 4/21/2025 8:35:24 AM
CONE ID: DDG1296
TOTAL DEPTH: 19.685 ft
HOLE NUMBER: CPT-2



■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay

■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt

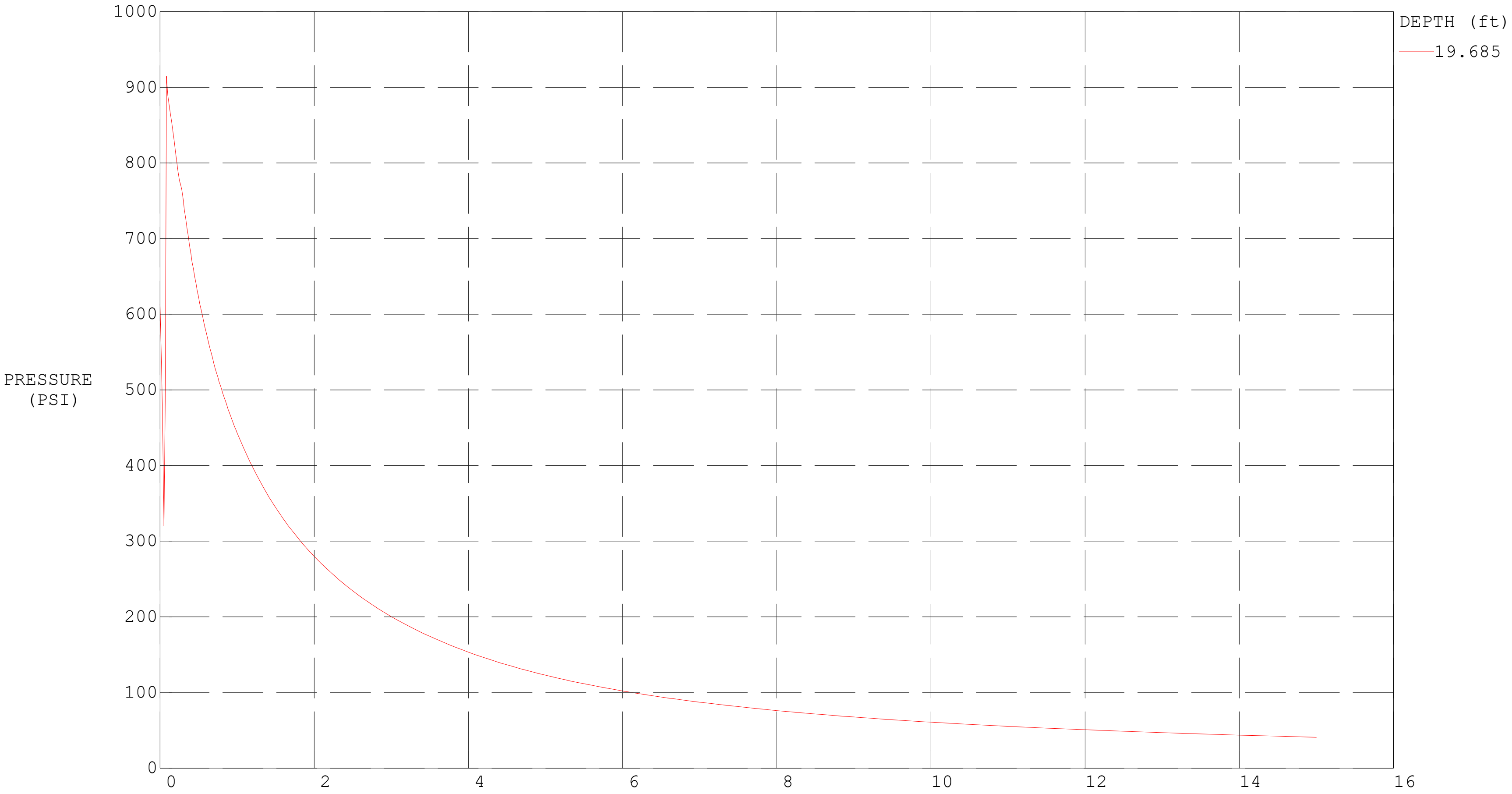
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand

■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GRI / CPT-2 / 19402 SE Foster Rd Happy Valley

OPERATOR: OGE DMM
CONE ID: DDG1296
TEST DATE: 4/21/2025 8:35:24 AM



MAXIMUM PRESSURE = 914.3 (PSI)
HYDROSTATIC PRESSURE = 7.252 (PSI), WATER TABLE: 2.95 ft

GRI / CPT-2 / 19402 SE Foster Rd Happy Valley

OPERATOR: OGE DMM
TEST DATE: 4/21/2025 8:35:24 AM
CONE ID: DDG1296
TOTAL DEPTH: 19.685 ft
HOLE NUMBER: CPT-2

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) TSF	Fr (Fs/Qt) (%)	Pressure (U2) PSI	SPT N* (blows/ft)	Zone	Soil Behavior Type UBC-1983
0.164	9.95	0.2851	2.865	4.605	6	4	silty clay to clay
0.328	13.20	0.3355	2.542	7.574	6	5	clayey silt to silty clay
0.492	11.85	0.3242	2.736	4.000	8	4	silty clay to clay
0.656	11.72	0.2972	2.535	2.754	6	5	clayey silt to silty clay
0.820	11.42	0.2692	2.358	1.519	5	5	clayey silt to silty clay
0.984	11.30	0.2706	2.395	1.199	5	5	clayey silt to silty clay
1.148	12.20	0.3854	3.159	0.931	8	4	silty clay to clay
1.312	12.78	0.5262	4.119	0.493	12	3	clay
1.476	10.63	0.5314	4.999	0.109	10	3	clay
1.640	10.36	0.5428	5.239	-0.301	10	3	clay
1.804	9.09	0.4882	5.373	0.025	9	3	clay
1.969	8.27	0.4562	5.521	0.204	8	3	clay
2.133	7.89	0.4400	5.575	2.074	8	3	clay
2.297	9.39	0.5257	5.599	6.624	9	3	clay
2.461	12.62	0.7167	5.681	9.325	12	3	clay
2.625	14.14	0.7571	5.355	7.613	14	3	clay
2.789	11.87	0.7419	6.253	5.553	11	3	clay
2.953	10.92	0.6655	6.097	4.722	10	3	clay
3.117	10.65	0.5586	5.246	7.150	10	3	clay
3.281	12.30	0.6665	5.418	9.882	12	3	clay
3.445	16.03	0.7576	4.726	11.558	15	3	clay
3.609	15.02	0.7871	5.243	9.556	14	3	clay
3.773	16.89	0.8308	4.922	9.587	16	3	clay
3.937	19.65	0.8390	4.272	12.207	19	3	clay
4.101	21.07	0.8670	4.116	15.104	13	4	silty clay to clay
4.265	23.30	0.8885	3.814	23.182	15	4	silty clay to clay
4.429	24.59	0.8310	3.380	34.888	12	5	clayey silt to silty clay
4.593	24.81	0.9098	3.668	55.445	16	4	silty clay to clay
4.757	26.03	0.9266	3.560	85.094	12	5	clayey silt to silty clay
4.921	27.14	0.8924	3.289	97.143	13	5	clayey silt to silty clay
5.085	27.64	0.8497	3.075	102.780	13	5	clayey silt to silty clay
5.249	27.58	0.9014	3.269	108.812	13	5	clayey silt to silty clay
5.413	31.17	0.9455	3.034	127.105	15	5	clayey silt to silty clay
5.577	32.71	1.0096	3.088	115.896	16	5	clayey silt to silty clay
5.741	34.40	0.9838	2.861	137.486	13	6	sandy silt to clayey silt
5.906	35.64	1.0182	2.857	125.959	14	6	sandy silt to clayey silt
6.070	38.02	1.0060	2.646	133.606	15	6	sandy silt to clayey silt
6.234	37.64	0.9695	2.577	117.022	14	6	sandy silt to clayey silt
6.398	34.39	0.8831	2.568	127.292	13	6	sandy silt to clayey silt
6.562	33.25	0.8510	2.560	164.909	13	6	sandy silt to clayey silt
6.726	34.38	0.8281	2.409	186.084	13	6	sandy silt to clayey silt
6.890	34.98	0.7995	2.286	192.566	13	6	sandy silt to clayey silt
7.054	38.37	0.7823	2.039	189.739	15	6	sandy silt to clayey silt
7.218	38.31	0.7950	2.076	170.287	15	6	sandy silt to clayey silt
7.382	35.14	0.8326	2.370	175.851	13	6	sandy silt to clayey silt
7.546	35.85	0.8690	2.424	196.271	14	6	sandy silt to clayey silt
7.710	39.50	0.8780	2.223	204.993	15	6	sandy silt to clayey silt
7.874	42.21	0.9459	2.241	196.817	16	6	sandy silt to clayey silt
8.038	38.09	1.1682	3.068	154.767	18	5	clayey silt to silty clay
8.202	34.76	1.3670	3.934	110.493	17	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) TSF	Fr (Fs/Qt) (%)	Pressure (U2) PSI	SPT N* (blows/ft)	Zone	Soil Behavior Type UBC-1983
8.366	33.53	1.4492	4.324	94.653	21	4	silty clay to clay
8.530	34.34	1.3590	3.959	108.113	22	4	silty clay to clay
8.694	31.07	1.2945	4.167	39.783	20	4	silty clay to clay
8.858	24.52	1.1059	4.512	18.494	23	3	clay
9.022	19.99	1.2135	6.071	22.611	19	3	clay
9.186	18.62	1.3701	7.360	13.607	18	3	clay
9.350	17.67	1.3821	7.825	10.284	17	3	clay
9.514	17.12	1.2017	7.020	13.690	16	3	clay
9.678	16.51	0.9914	6.008	12.224	16	3	clay
9.843	14.75	0.9071	6.153	22.204	14	3	clay
10.007	15.78	0.8574	5.436	61.212	15	3	clay
10.171	16.41	0.8261	5.036	60.384	16	3	clay
10.335	17.26	0.8245	4.778	67.278	17	3	clay
10.499	17.86	0.8412	4.710	69.026	17	3	clay
10.663	18.53	0.9337	5.039	71.446	18	3	clay
10.827	19.62	1.0028	5.111	75.496	19	3	clay
10.991	21.51	1.0722	4.986	74.139	21	3	clay
11.155	24.18	1.1340	4.692	76.539	23	3	clay
11.319	29.20	1.1902	4.077	82.000	19	4	silty clay to clay
11.483	32.93	1.3532	4.111	62.637	21	4	silty clay to clay
11.647	41.90	1.5391	3.674	80.408	20	5	clayey silt to silty clay
11.811	52.20	1.8805	3.603	125.982	25	5	clayey silt to silty clay
11.975	50.26	2.0564	4.092	96.053	24	5	clayey silt to silty clay
12.139	53.23	2.0204	3.796	88.228	25	5	clayey silt to silty clay
12.303	59.65	1.9271	3.232	121.413	23	6	sandy silt to clayey silt
12.467	59.64	2.0319	3.408	153.953	29	5	clayey silt to silty clay
12.631	60.47	2.2391	3.704	167.290	29	5	clayey silt to silty clay
12.795	59.78	2.3373	3.911	139.538	29	5	clayey silt to silty clay
12.959	56.14	1.8041	3.214	162.135	22	6	sandy silt to clayey silt
13.123	52.19	1.8208	3.489	150.605	25	5	clayey silt to silty clay
13.287	50.24	1.8041	3.592	164.494	24	5	clayey silt to silty clay
13.451	48.67	1.6633	3.418	146.092	23	5	clayey silt to silty clay
13.615	46.53	1.5903	3.419	124.401	22	5	clayey silt to silty clay
13.780	43.10	1.6505	3.830	96.535	21	5	clayey silt to silty clay
13.944	38.64	1.7535	4.539	39.270	25	4	silty clay to clay
14.108	35.46	1.5996	4.512	42.510	23	4	silty clay to clay
14.272	36.81	1.6700	4.538	40.427	23	4	silty clay to clay
14.436	33.42	1.7338	5.189	29.853	32	3	clay
14.600	29.84	1.4997	5.027	57.861	29	3	clay
14.764	26.05	1.3539	5.198	17.830	25	3	clay
14.928	24.53	1.3886	5.663	18.106	23	3	clay
15.092	25.62	1.4113	5.509	32.357	25	3	clay
15.256	26.64	1.3616	5.112	28.334	26	3	clay
15.420	26.26	1.3265	5.053	26.781	25	3	clay
15.584	26.49	1.2953	4.890	30.603	25	3	clay
15.748	26.57	1.2766	4.806	34.119	25	3	clay
15.912	26.17	1.2746	4.872	32.833	25	3	clay
16.076	26.02	1.2818	4.929	28.811	25	3	clay
16.240	28.41	1.4655	5.161	16.609	27	3	clay
16.404	26.60	1.3901	5.227	37.299	25	3	clay
16.568	33.69	1.4453	4.291	82.850	22	4	silty clay to clay
16.732	37.75	1.5203	4.028	109.754	24	4	silty clay to clay
16.896	38.80	1.6374	4.221	123.699	25	4	silty clay to clay
17.060	44.18	1.7347	3.928	129.647	21	5	clayey silt to silty clay
17.224	49.65	1.8419	3.711	99.858	24	5	clayey silt to silty clay
17.388	66.21	2.2862	3.454	160.655	32	5	clayey silt to silty clay
17.552	89.76	3.3326	3.714	268.210	43	5	clayey silt to silty clay
17.717	114.53	4.2325	3.697	421.762	44	6	sandy silt to clayey silt
17.881	160.33	4.3501	2.714	493.696	51	7	silty sand to sandy silt
18.045	161.14	5.9582	3.699	547.019	77	12	sand to clayey sand (*)
18.209	227.82	6.8404	3.003	557.104	73	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) TSF	Fr (Fs/Qt) (%)	Pressure (U2) PSI	SPT N* (blows/ft)	Zone	Soil Behavior Type UBC-1983
18.373	239.67	6.0302	2.517	247.631	77	7	silty sand to sandy silt
18.537	176.01	5.2657	2.993	512.680	56	7	silty sand to sandy silt
18.701	255.95	6.8891	2.692	684.867	82	7	silty sand to sandy silt
18.865	338.53	7.3603	2.175	85.036	81	8	sand to silty sand
19.029	269.98	8.7582	3.245	625.381	129	12	sand to clayey sand (*)
19.193	288.52	8.3860	2.907	741.812	138	12	sand to clayey sand (*)
19.357	315.43	11.3081	3.586	658.724	151	12	sand to clayey sand (*)
19.521	368.05	11.5400	3.136	672.027	176	12	sand to clayey sand (*)
19.685	520.16	11.7100	2.252	615.755	125	8	sand to silty sand

APPENDIX C

Earth Dynamics, LLC Geophysical Report

APPENDIX C

EARTH DYNAMICS, LLC GEOPHYSICAL REPORT

C.1 GENERAL

Earth Dynamics, LLC of Portland, Oregon, performed geophysical testing at the proposed building and Rock Creek crossing locations as part of our field exploration program for this project. The geophysical testing consisted of collecting data from two refraction microtremor (ReMi) arrays that were designated ReMi Array 1 and ReMi Array 2 and had lengths of 345 feet and geophone spacing of 15 feet. The geophysical report that was prepared for this project is provided in this appendix.

Report on

Shear Wave Refraction Microtremor Analysis (ReMi)
Happy Valley Community Center
Happy Valley, Oregon

Data Acquisition Date: April 9, 2025
Report Date: April 24, 2025

Prepared for:

GRI
16950 SW Upper Boones Fry
Tigard, OR 97224

Prepared by:

EARTH DYNAMICS LLC
2284 N.W. Thurman St.
Portland, OR 97210
(503) 227-7659
Project No. 25204

1.0 INTRODUCTION

GRI engaged Earth Dynamics LLC to conduct a geophysical exploration at the proposed Community Center site in Happy Valley, Oregon. This study was requested and authorized by Mr. Ryan Lawrence of GRI. The geophysical field work was completed by Mr. Daniel Lauer of Earth Dynamics LLC on April 9, 2025. This report describes the methodology and results of the geophysical investigation.

2.0 SCOPE OF WORK

The purpose of this study is to characterize the subsurface shear wave velocity at the site. These data are needed to help determine the seismic response of the site to earthquake loading. The exploration consisted of two twenty-four channel refraction microtremor (ReMi) arrays.

3.0 METHOD

The ReMi technique provides a simplified characterization of relatively large volumes of the subsurface. The method can be used to estimate one-dimensional shear wave velocity profiles and provide site-specific soil classification data as described in ASCE/SEI 7-16 (2017). In a ReMi survey, geophones are deployed at designated intervals along a linear array. The resolution and depth of investigation depends upon the geophone cut-off frequency, spacing of the geophones, the total array length, and the frequency characteristics of the Rayleigh waves at the site. For “rule of thumb” survey planning, the nominal depth of investigation is assumed to be approximately one-third of the geophone array length.

The theoretical basis of the ReMi method is the same as Spectral Analysis of Surface Waves (SASW) and Multi-channel Analysis of Surface Waves (MASW) as first described to the earthquake engineering community by Nazarian and Stokoe (1984). However, ReMi does not require a frequency-controlled source and the field equipment is much more compact and economical. A complete description of the theoretical basis for ReMi is described by Louie (2001). In ReMi analysis all interpretation is done in the frequency domain, and the method assumes that the most energetic arrivals recorded are Rayleigh waves. By applying a time-domain velocity analysis, Rayleigh waves can be separated from body waves, air waves, and other coherent noise. Transforming the time-domain velocity results into the frequency domain allows combination of many arrivals over a long time period and yields recognition of dispersive surface waves.

Data reduction is completed in two steps. First, the time versus amplitude seismic records are transformed into spectral energy shear wave frequency versus shear wave velocity (or slowness). The data are graphically presented in what is commonly termed a p-f plot. The interpreter determines a dispersion curve from the p-f plot by selecting the lower bound of the spectral energy shear wave velocity versus frequency trend. The second phase of the analysis consists of fitting the measured dispersion curve with a theoretical dispersion curve that is based upon a model of multiple layers with various shear wave velocities. The model velocities and layer thicknesses are adjusted until a 'best fit' to the measured data is obtained. This type of interpretation does not provide a unique model. Interpreter experience and knowledge of the existing geology are important to provide a realistic solution. The data are presented as one-dimensional velocity profiles that represent the average shear wave velocities of the subsurface layers over the length of the geophone array.

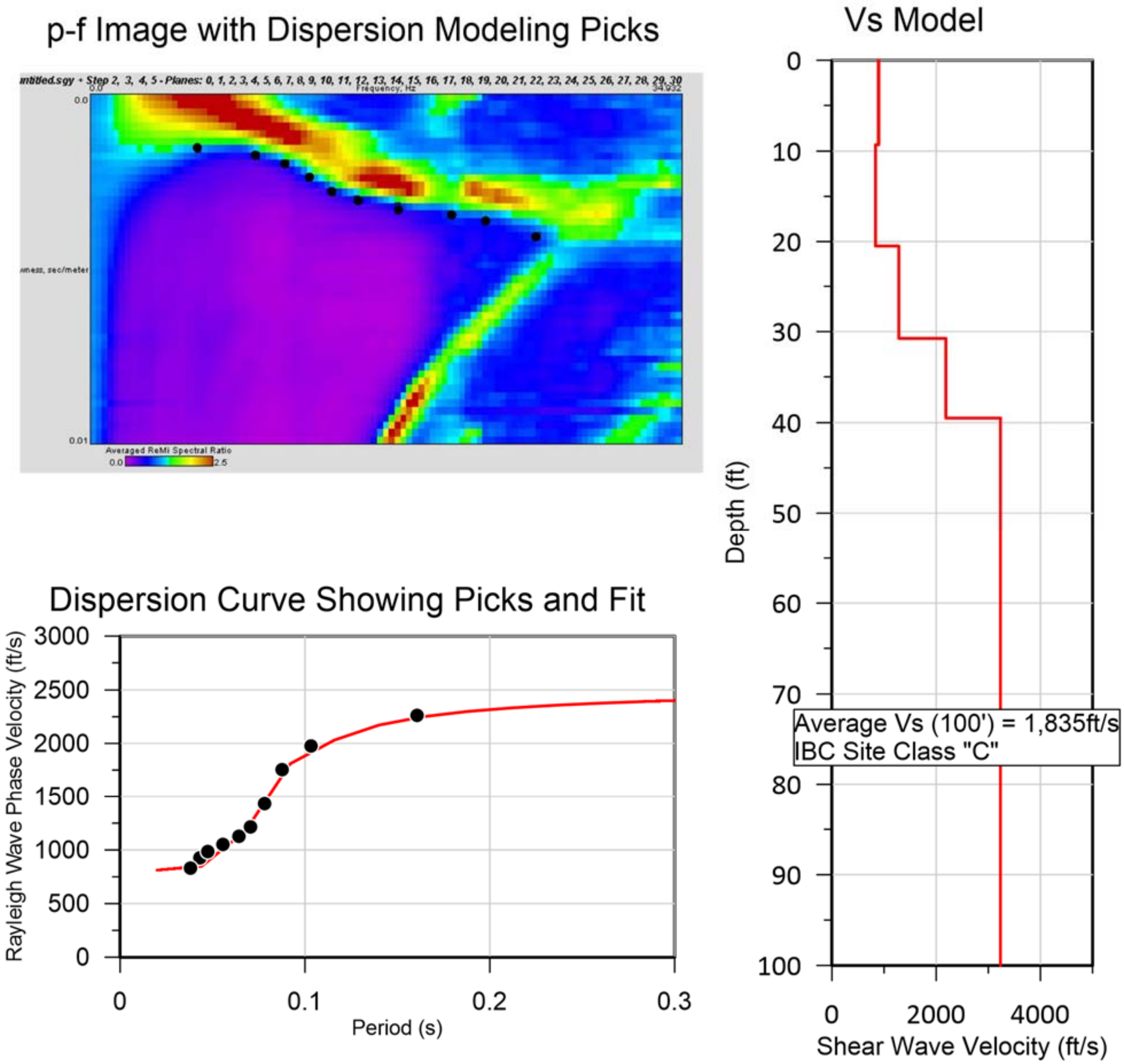
For this project, data were acquired along two ReMi arrays. Each array consists of twenty-four 4.5 Hz vertical geophones spiked in firm soil with a geophone spacing of fifteen feet and a total array length of 345 feet. More than thirty 30-second-long seismic records of ambient and active seismic noise were recorded for each array. Data were acquired when vehicles, and people were moving on and near the site.

4.0 RESULTS

The approximate locations of the ReMi arrays are shown on the Google Earth image in Figure 4-1. The ReMi analysis and results for ReMi Array 1 are contained in Figure 4-2. The ReMi analysis and results for ReMi Array 2 are contained in Figure 4-3. Figures 4-2 and 4-3 include the p-f plot, the dispersion curve, the derived velocity versus depth model that best fits the data and expected geology of the site and a table containing the shear wave velocity with depth for the array.

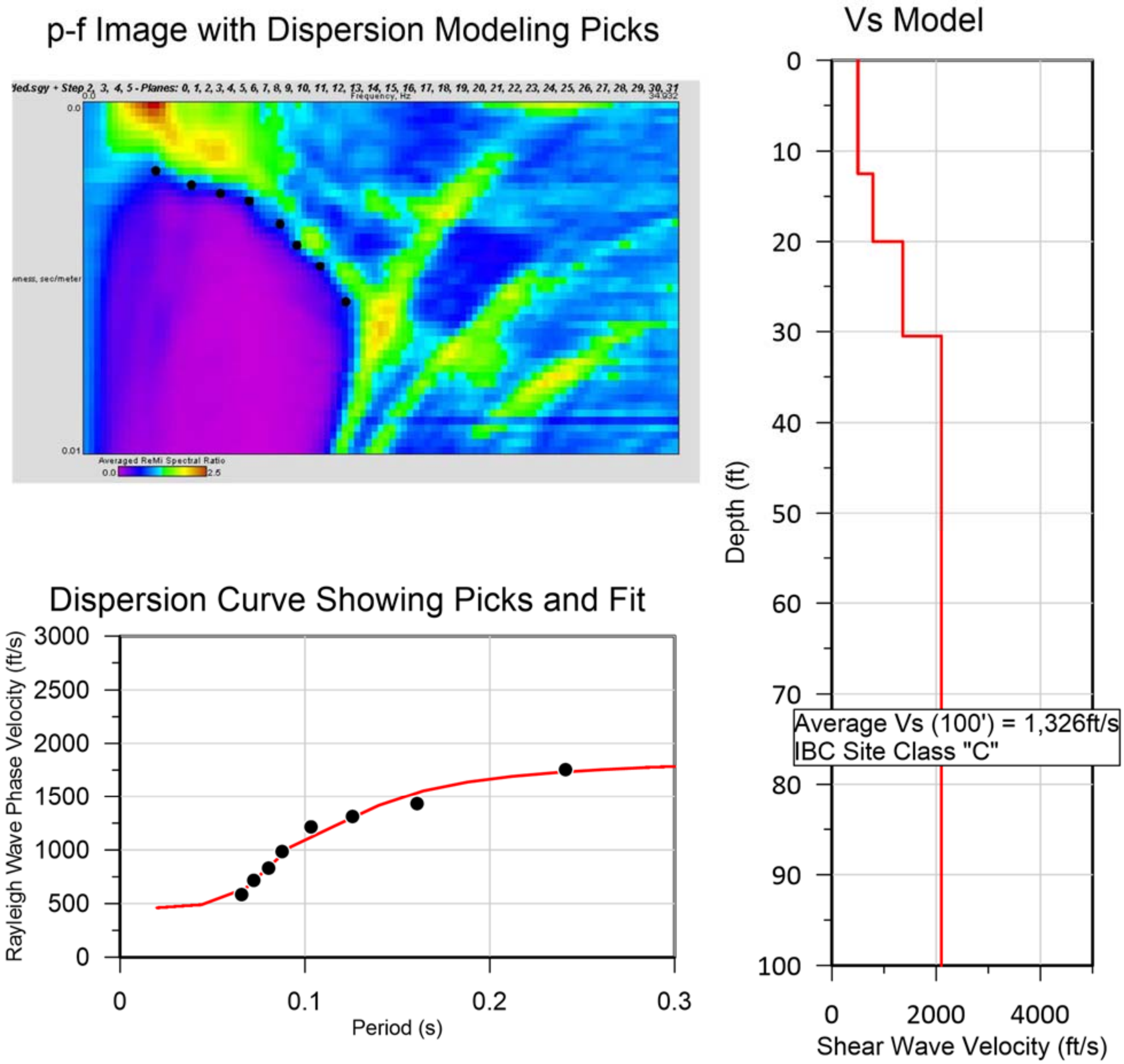
The dispersion curve for each array is well defined and choosing the lower energy bound is distinct. The RMS error of the model fit to the data is less than 100 ft/s. The dispersion curve data suggest that the depth of investigation for each array is at least 100 feet bgs.

Figure 4-1. Site layout showing location of the ReMi arrays.



Depth Interval (ft)	Shear-wave velocity (ft/s)
0 – 9.3	894
9.3 – 20.5	836
20.5 – 30.75	1,282
30.75 – 39.5	2,185
39.5 - 100	3,237

Figure 4-2. ReMi Array 1 Results



p-f Image with Dispersion Modeling Picks

Dispersion Curve Showing Picks and Fit

Vs Model

Depth Interval (ft)	Shear-wave velocity (ft/s)
0 – 12.5	500
12.5 – 20.0	786
20.0 – 30.5	1,359
30.5 – 100	2,100

Figure 4-3. ReMi Array 2 Results

5.0 DISCUSSION

5.1 Site Geology

Boring logs near ReMi Array 1 indicate that the site is underlain by very stiff Clay to a depth of approximately 25 feet below the ground surface (bgs) and Sandy Silt to a depth of approximately 70 feet bgs. Boring logs from B-3 near ReMi Array 2 indicate that the site is underlain by stiff Silt to a depth of approximately 20 feet bgs, hard Silt to a depth of approximately 30 feet bgs, and basalt to the bottom of the boring at 67 feet bgs. The ReMi models appear to correlate with the information from the boring logs.

5.2 ASCE Classifications

ASCE/SEI 7-16 (2017) defines five site classes based upon the average shear-wave velocity of the soil to a depth of 30 Meters (100 feet). The ASCE classification is summarized in Table 5-1. The classifications in Table 5-1 are incorporated into the International Building Code (IBC 2021). Earthquake shaking is expected to be stronger where shear-wave velocity is lower. Average shear wave velocity to a depth of 100 ft (V_{s100}) is calculated using Equation 5-1.

$$Vs(100) = \frac{100}{\sum_{i=1}^{i=n}\left(\frac{d_i}{Vs_i}\right)} \qquad \text{Equation 5-1}$$

Where:

n = the number of intervals
i = the interval number
d_i = the thickness of the ith interval in feet
Vs_i = the velocity of the ith interval

Using Equation 5-1 and the data in Figure 4-2, the average shear wave velocity to a depth of 100 ft is calculated to be 1,835 ft/s for ReMi Array 1 and 1,326 ft/s for ReMi Array 2. The modelled velocity for each array is in the range for IBC seismic design classification of “C”.

Table 5-1. Summary of ASCE soil classification.

Class	Average S-wave Velocity (ft/sec)	Description
A	> 5,000	Hard rock
B	2,500 – 5,000	Rock
C	1,200 – 2,500	Very dense soil and soft rock
D	600 – 1,200	Stiff soil
E	<600	Soil

6.0 LIMITATIONS

The geophysical method used in this study involves the inversion of measured data. Theoretically, the inversion process yields an infinite number of models which will fit the data. Further, many geologic materials have the same seismic velocity. We have presented models and interpretations which we believe to be the best fit given the geology and known conditions at the site. However, no warranty is made or intended by this report or by oral or written presentation of this work. Earth Dynamics accepts no responsibility for damages because of decisions made or actions taken based upon this report.

7.0 REFERENCES

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

Louie, J.N. (2001). “Faster, better: shear-wave velocity to 100 meters depth from refraction microtremor arrays”, Bull. Seism. Soc. Am., 91, 347-364.

Nazarian, S., and Stokoe II, K.H., (1984), “In situ shear-wave velocities from spectral analysis of surface waves”, Proceedings for the World Conference on Earthquake Engineering Vol. 8, San Francisco, Calif., July 21-28, v.3, 31-38.

IBC (2021) 2021 International Building Code , International Code Council, Washington D.C.

RESPECTFULLY SUBMITTED
EARTH DYNAMICS LLC

Daniel Lauer
Partner - Senior Geophysicist

APPENDIX D

Previous Field Explorations by Others

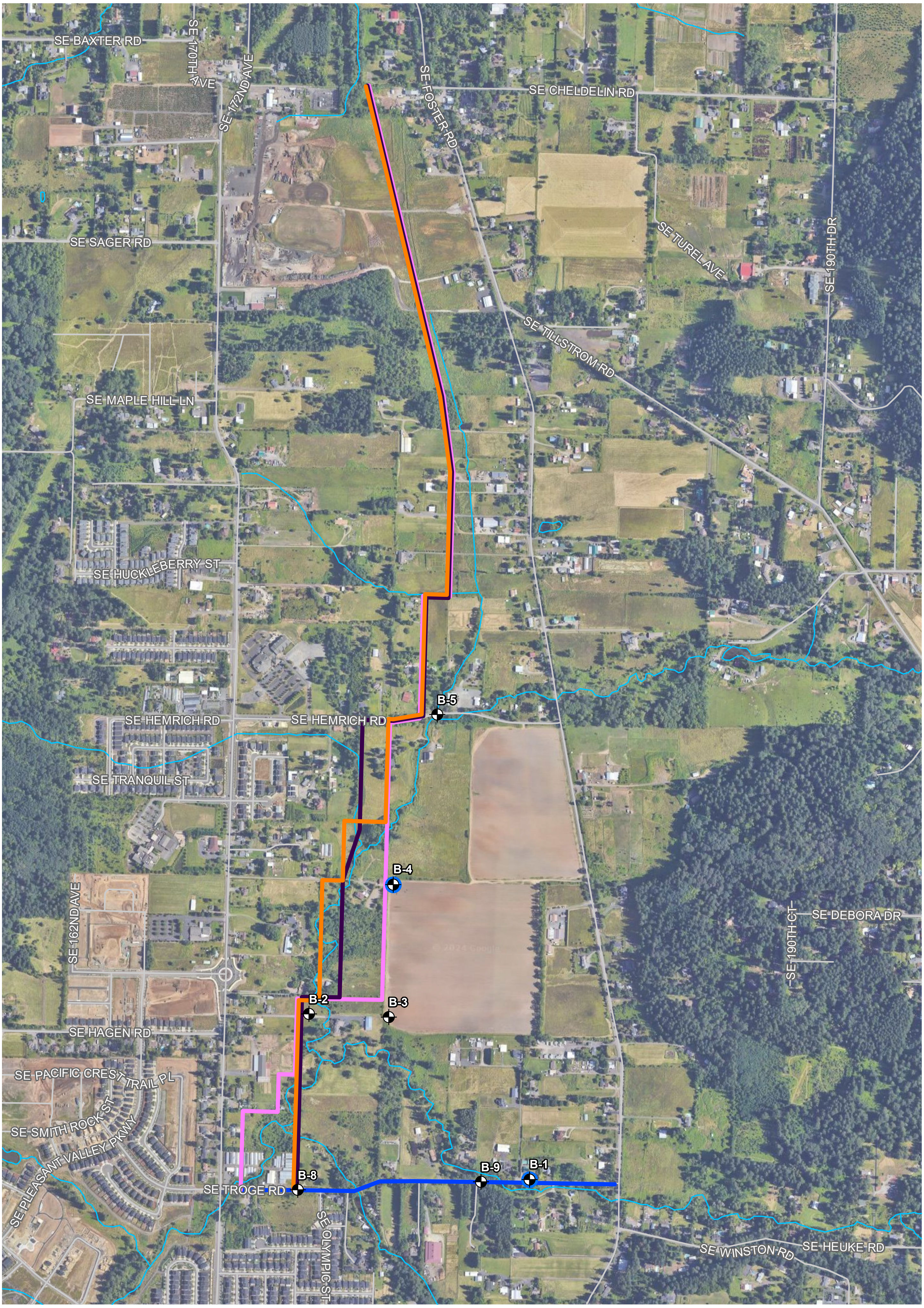
APPENDIX D

PREVIOUS FIELD EXPLORATIONS BY OTHERS

D.1 GENERAL

Shannon & Wilson, Inc. previously performed seven geotechnical borings at or near the project site in 2024. Shannon & Wilson, Inc. also performed laboratory testing on selected soil samples and collected a groundwater reading in one of the borings at the project site. Figures, logs, and laboratory test results from Shannon & Wilson, Inc.'s 2024 geotechnical report are provided in this appendix.

Filename: P:\GIS\PDX\112000s\112335_Rock Creek Inte\Avmxd\SitePlan_11x17.mxd Date: 8/14/2024 Login: KJW



LEGEND

B-2 ⊕	Designation and Location of Completed Boring
B-1 ⊕	Designation and Location of Completed Boring with Vibrating Wire Piezometer
—	Streams, creeks, and drainages
—	Alternative Alignment 1East
—	Alternative Alignment 2
—	Alternative Alignment 5
—	Alternative Alignment 3

0 400 800 1,600

Scale in Feet

NOTES

1. Aerial imagery obtained through Google Maps Satellite.
2. Taxlots, streams, and streets obtained through Metro RLIS.

Rock Creek Interceptor Extension
Happy Valley, Oregon

SITE AND EXPLORATION PLAN

October 2024 112335

SHANNON & WILSON

FIG. 2

FIG. 2

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

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

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

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

MOISTURE CONTENT TERMS

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

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

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

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

PARTICLE SIZE DEFINITIONS

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

RELATIVE DENSITY / CONSISTENCY

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

WELL AND BACKFILL SYMBOLS

▧	Bentonite Cement Grout	▩	Surface Cement Seal
▨	Bentonite Grout	■	Asphalt or Cap
▦	Bentonite Chips	◩	Slough
░	Silica Sand	▥	Inclinometer or Non-perforated Casing
▓	Gravel	◫	Vibrating Wire Piezometer
▤	Perforated or Screened Casing		

PERCENTAGES TERMS [1, 2]

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

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

[2] Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

Rock Creek Interceptor Sewer
Clackamas County, OR

SOIL DESCRIPTION AND LOG KEY

October 2024
112335

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A1
Sheet 1 of 3

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

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

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

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

NOTES

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

Rock Creek Interceptor Sewer
Clackamas County, OR

SOIL DESCRIPTION AND LOG KEY

October 2024
112335

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A1
Sheet 2 of 3

GRADATION TERMS

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

CEMENTATION TERMS[1]

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

PLASTICITY[2]

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

ADDITIONAL TERMS

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

PARTICLE ANGULARITY AND SHAPE TERMS[1]

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

ACRONYMS AND ABBREVIATIONS

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

STRUCTURE TERMS[1]

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

Rock Creek Interceptor Sewer
Clackamas County, OR

SOIL DESCRIPTION AND LOG KEY

October 2024
112335

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A1
Sheet 3 of 3

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[1] Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

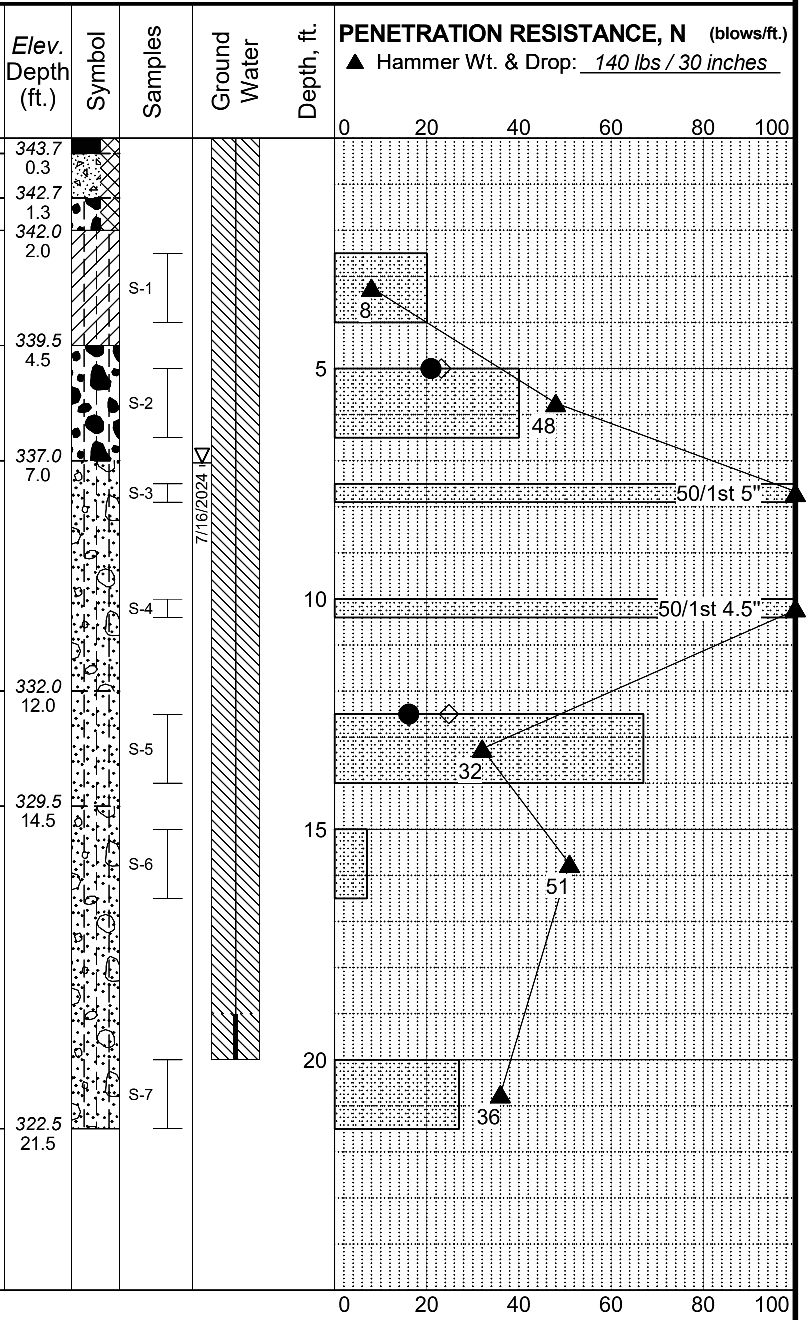
[2] Adapted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

2013 BORING CLASS3 108413 GINT.GPJ SW2013LIBRARYPDX.GLB SWNEW.GDT 3/27/23

Total Depth:	21.5 ft.	Northing:	~	Drilling Method:	Mud Rotary
Top Elevation:	~ 344 ft.	Easting:	~	Drilling Company:	Western States
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-75 Truck Rig #4
Horiz. Datum:		Offset:	~	Other Comments:	

Hole Diam.: 12 in.
Rod Type: NWJ
Hammer Type: Automatic

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)
Asphalt Concrete: 4-inches thick	*343.7* 0.3
Aggregate Cement Concrete: 12-inches thick, contains asphalt and concrete fragments	*342.7* 1.3
Base Aggregate: 9-inches thick	*342.0* 2.0
Medium stiff to stiff, brown, *Lean Clay to Lean Clay with Sand (CL)*; moist; fine to medium sand; medium to high plasticity. **CATASTROPHIC FLOOD DEPOSITS**	*339.5* 4.5
Dense, brown and gray, *Silty Gravel with Sand (GM)*; moist; fine to coarse, subangular to subrounded gravel; fine to coarse sand; nonplastic to low plasticity fines. **SPRINGWATER FORMATION**	*337.0* 7.0
Very dense, brown and gray, *Silty Sand with Gravel (SM)*; moist; fine to coarse, angular to subrounded gravel; fine to coarse sand; nonplastic fines; weakly cemented.	*332.0* 12.0
Dense, brown and gray, *Silty Sand (SM)*; moist; trace fine subangular to subrounded gravel; fine to coarse sand; low plasticity fines.	*329.5* 14.5
Dense to very dense, brown and gray, *Silty Sand with Gravel (SM)*; moist; fine to coarse, angular to subrounded gravel; fine to coarse sand; nonplastic to low plasticity fines. *RST Vibrating Wire Piezometer installed 20 feet below ground surface*	*322.5* 21.5
Completed: July 8, 2024	



LEGEND

⌶ Standard Penetration Test

∇ Groundwater Level on Date Shown

▨ Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

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

Rock Creek Interceptor Sewer
Clackamas County, Oregon

LOG OF BORING B-1

October 2024 112335

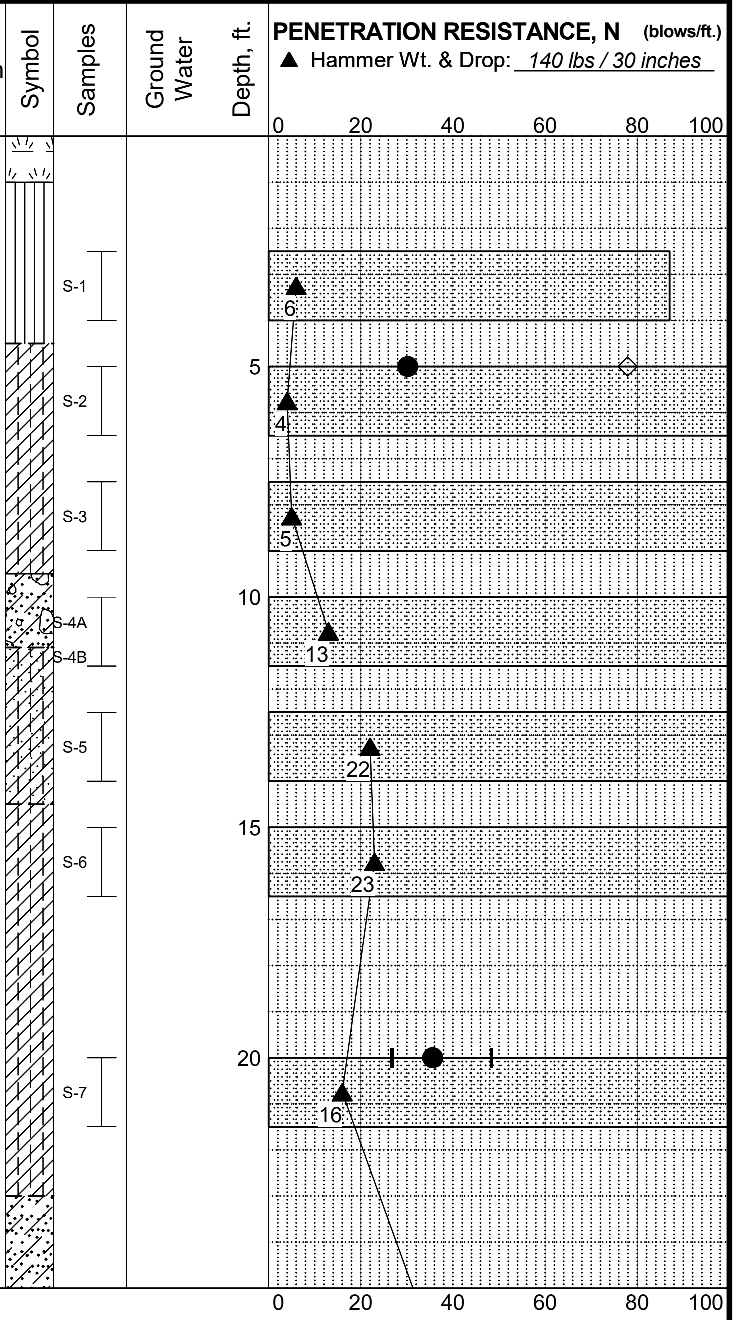
SHANNON & WILSON

FIG. A2

REV 2

Total Depth: 26.5 ft.
Top Elevation: ~ 318 ft.
Vert. Datum:
Horiz. Datum:
Northing: ~
Easting: ~
Station: ~
Offset: ~
Drilling Method: Mud Rotary
Drilling Company: Western States
Drill Rig Equipment: CME-75 Truck Rig #4
Other Comments:
Hole Diam.: 6 in.
Rod Type: NWJ
Hammer Type: Automatic

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)
Topsoil: 12-inches thick	
Medium stiff, brown, *Silt with Sand (ML)*; moist; fine to medium sand; low plasticity; trace organics; micaceous; iron oxidation and staining. **CATASTROPHIC FLOOD DEPOSITS**	*317.0* 1.0
Medium stiff to stiff, brown and gray, *Lean Clay with Sand (CL)*; moist; fine sand; low to medium plasticity; micaceous.	*313.5* 4.5
Medium dense, red-brown, *Clayey Sand with Gravel (SC)*; moist; fine to coarse, subangular to subrounded gravel; fine to coarse sand; low to medium plasticity fines; iron oxidation and staining. **SPRINGWATER FORMATION**	*308.5* 9.5
Medium stiff, red-brown, *Lean Clay with Sand (CL)*; moist; trace fine subangular to subrounded gravel; fine to medium sand; medium plasticity; iron oxidation and staining.	*306.9* 11.1
Very stiff, brown and red-brown, *Lean Clay (CL)*; moist; fine sand; high plasticity; iron oxidation and staining.	*303.5* 14.5
Dense, brown and dark brown, *Clayey Sand with Gravel (SC)*.	*295.0* 23.0



CONTINUED NEXT SHEET

LEGEND

⌶ Standard Penetration Test

▨ Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├──┤ Liquid Limit

Log: NMB Rev: DSJ Typ: DSJ

MASTER_LOG_E 112335.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 8/5/24

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

Rock Creek Interceptor Sewer
Clackamas County, Oregon

LOG OF BORING B-2

October 2024
112335

SHANNON & WILSON

FIG. A3
Sheet 1 of 2

REV 2

Total Depth: 26.5 ft.
Top Elevation: ~ 318 ft.
Vert. Datum:
Horiz. Datum:

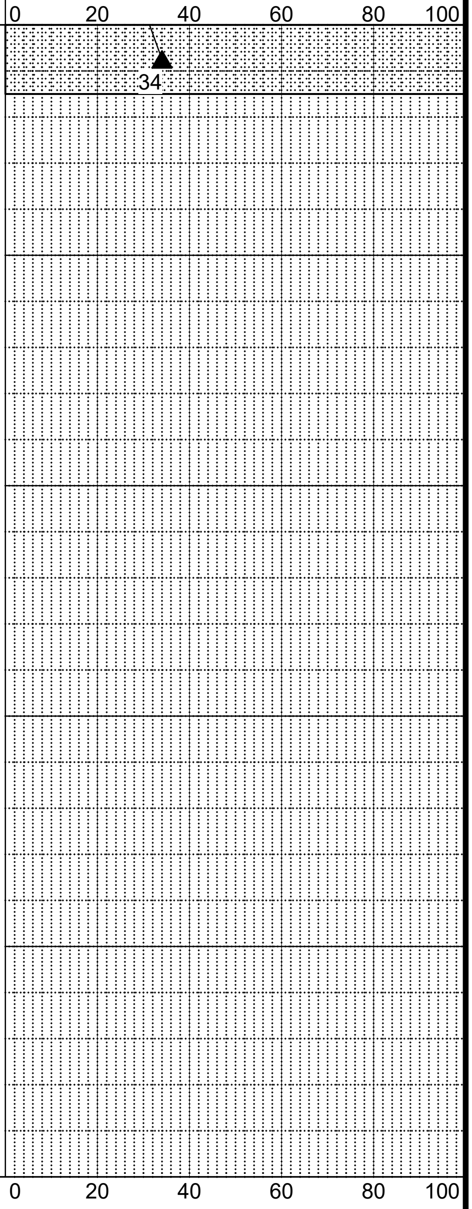
Northing: ~
Easting: ~
Station: ~
Offset: ~

Drilling Method: Mud Rotary
Drilling Company: Western States
Drill Rig Equipment: CME-75 Truck Rig #4
Other Comments:

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

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft.
Continued: Dense, brown and dark brown, *Clayey Sand with Gravel (SC)*; moist; fine to coarse, subangular to subrounded gravel; fine to coarse sand; low to medium plasticity fines; iron oxidation and staining	*291.5* 26.5	▨	S-8		
Completed: July 10, 2024					30 35 40 45

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND
⌶ Standard Penetration Test

⛶ Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├──┤ Liquid Limit

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

Rock Creek Interceptor Sewer
Clackamas County, Oregon

LOG OF BORING B-2

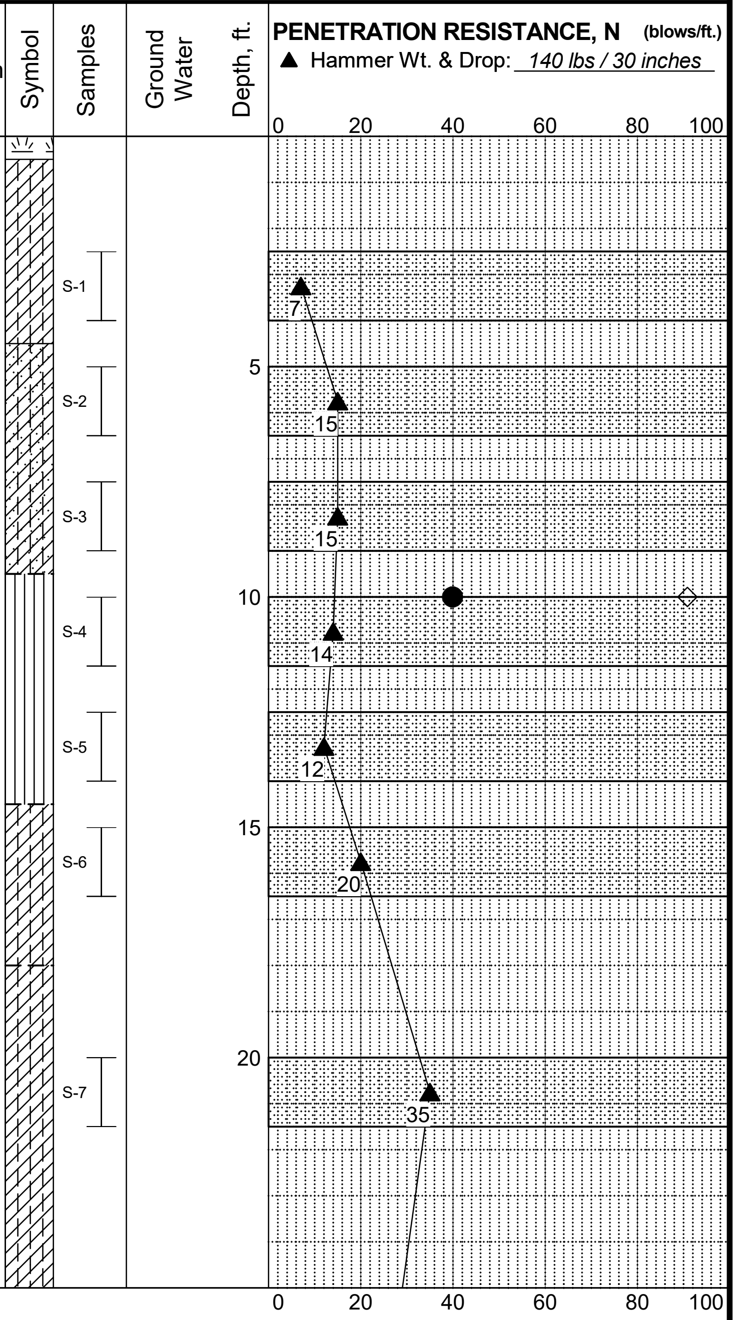
October 2024 112335

SHANNON & WILSON

FIG. A3
Sheet 2 of 2

Total Depth: 40.2 ft.
Top Elevation: ~ 343 ft.
Vert. Datum:
Horiz. Datum:
Northing: ~
Easting: ~
Station: ~
Offset: ~
Drilling Method: Mud Rotary
Drilling Company: Western States
Drill Rig Equipment: CME-75 Truck Rig #4
Other Comments:
Hole Diam.: 6 in.
Rod Type: NWJ
Hammer Type: Automatic

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)
Topsoil: 6-inches thick	*342.5* 0.5
Medium stiff, gray, *Lean Clay (CL)*; moist; fine sand; medium to high plasticity; trace organics; micaceous. **CATASTROPHIC FLOOD DEPOSITS**	*338.5* 4.5
Stiff to very stiff, red-brown and gray, *Lean Clay with Sand (CL)*; moist; fine to medium sand; medium plasticity; iron oxidation and staining. **SPRINGWATER FORMATION**	*333.5* 9.5
Stiff, red-brown and gray, *Silt to Lean Clay (ML/CL)*; moist; fine sand; low plasticity; iron oxidation and staining.	*328.5* 14.5
Very stiff, brown, *Lean Clay with Sand (CL)*; moist; fine sand; medium plasticity; iron oxidation and staining.	*325.0* 18.0
Very stiff to hard, brown and red-brown, *Lean Clay to Lean Clay with Sand (CL)*.	



CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered
⌶ Standard Penetration Test

▨ Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├──┤ Liquid Limit

Rock Creek Interceptor Sewer
Clackamas County, Oregon

LOG OF BORING B-3

October 2024
112335

SHANNON & WILSON

FIG. A4
Sheet 1 of 2

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

MASTER_LOG_E 112335.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 8/5/24
Log: NMB Rev: DSJ Typ: DSJ

REV 2

Total Depth: 40.2 ft.
Top Elevation: ~ 343 ft.
Vert. Datum:
Horiz. Datum:
Northing: ~
Easting: ~
Station: ~
Offset: ~
Drilling Method: Mud Rotary
Drilling Company: Western States
Drill Rig Equipment: CME-75 Truck Rig #4
Other Comments:
Hole Diam.: 6 in.
Rod Type: NWJ
Hammer Type: Automatic

SOIL DESCRIPTION

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

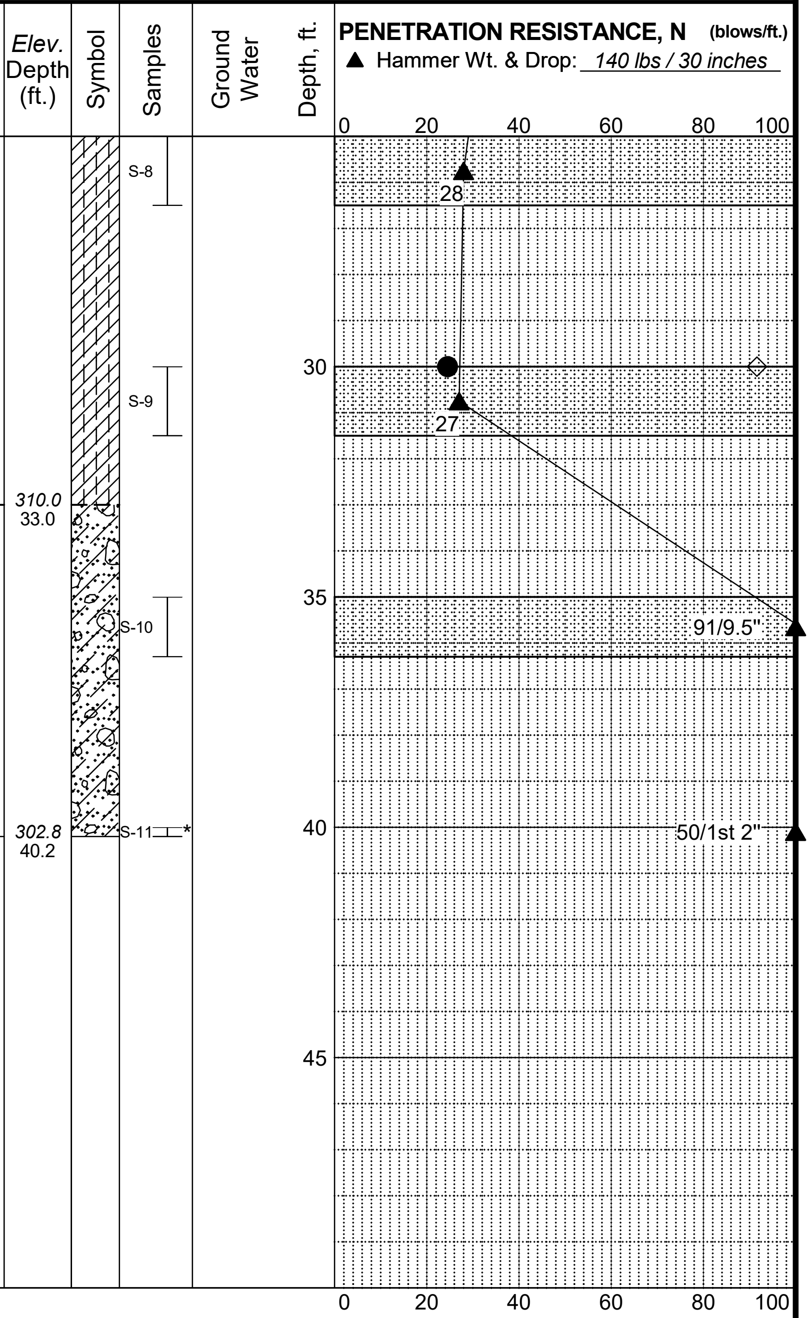
Continued:

Very stiff to hard, brown and red-brown, *Lean Clay to Lean Clay with Sand (CL)*;moist; trace fine subangular gravel; fine to medium sand; medium plasticity; trace organics; iron oxidation and staining

SPRINGWATER FORMATION

Very dense, dark gray and red-brown, *Clayey Sand with Gravel (SC)*; moist; fine to coarse, subangular to subrounded gravel; fine to medium sand; medium plasticity fines; iron oxidation and staining.

Completed: July 11, 2024



▨ Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ⊢—⊣ Liquid Limit

LEGEND

* Sample Not Recovered
⌶ Standard Penetration Test

NOTES

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

Rock Creek Interceptor Sewer
Clackamas County, Oregon

LOG OF BORING B-3

October 2024 112335

SHANNON & WILSON

FIG. A4
Sheet 2 of 2

REV 2

Total Depth:	41.5 ft.	Northing:	~	Drilling Method:	Mud Rotary
Top Elevation:	~ 341 ft.	Easting:	~	Drilling Company:	Western States
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-75 Truck Rig #4
Horiz. Datum:		Offset:	~	Other Comments:	

Hole Diam.:	12 in.
Rod Type:	NWJ
Hammer Type:	Automatic

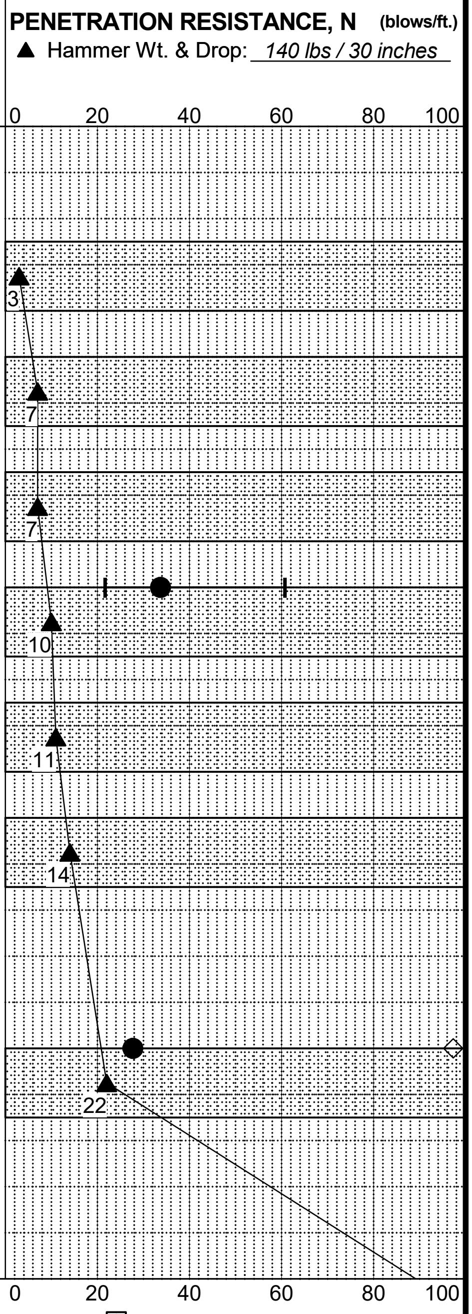
SOIL DESCRIPTION

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

SOIL DESCRIPTION	Elev. Depth (ft.)	Samples	Ground Water
Topsoil: 12-inches thick	340.0 1.0		
Soft, gray, *Lean Clay (CL)*; moist; fine sand; medium to high plasticity; trace organics and rootlets; slight iron oxidation and staining; micaceous. **CATASTROPHIC FLOOD DEPOSITS**	336.5 4.5	S-1	
Medium stiff, gray, *Fat Clay (CH)*; moist; fine sand; high plasticity; trace organics and rootlets; micaceous.	334.0 7.0	S-2	
Loose, yellow-brown, *Silty Sand (SM)*; moist; fine to medium sand; nonplastic to low plasticity fines; micaceous.	331.5 9.5	S-3	
Stiff to very stiff, gray, *Lean Clay to Fat Clay (CL/CH)*; moist; fine sand; medium to high plasticity; iron oxidation and staining. **SPRINGWATER FORMATION**	318.0 23.0	S-4 S-5 S-6 S-7	⊽ 7/16/2024
Hard/Very dense, gray-brown, *Silt with Sand to Sandy Silt (ML)*.			

PENETRATION RESISTANCE, N (blows/ft.)

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



CONTINUED NEXT SHEET

LEGEND

⌶ Standard Penetration Test

⊽ Groundwater Level on Date Shown

⛆ Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

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

Rock Creek Interceptor Sewer
Clackamas County, Oregon

LOG OF BORING B-4

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SHANNON & WILSON

FIG. A5
Sheet 1 of 2

REV 2

MASTER_LOG_E 112335.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 8/5/24 Log: NMB Rev: DSJ Typ: DSJ

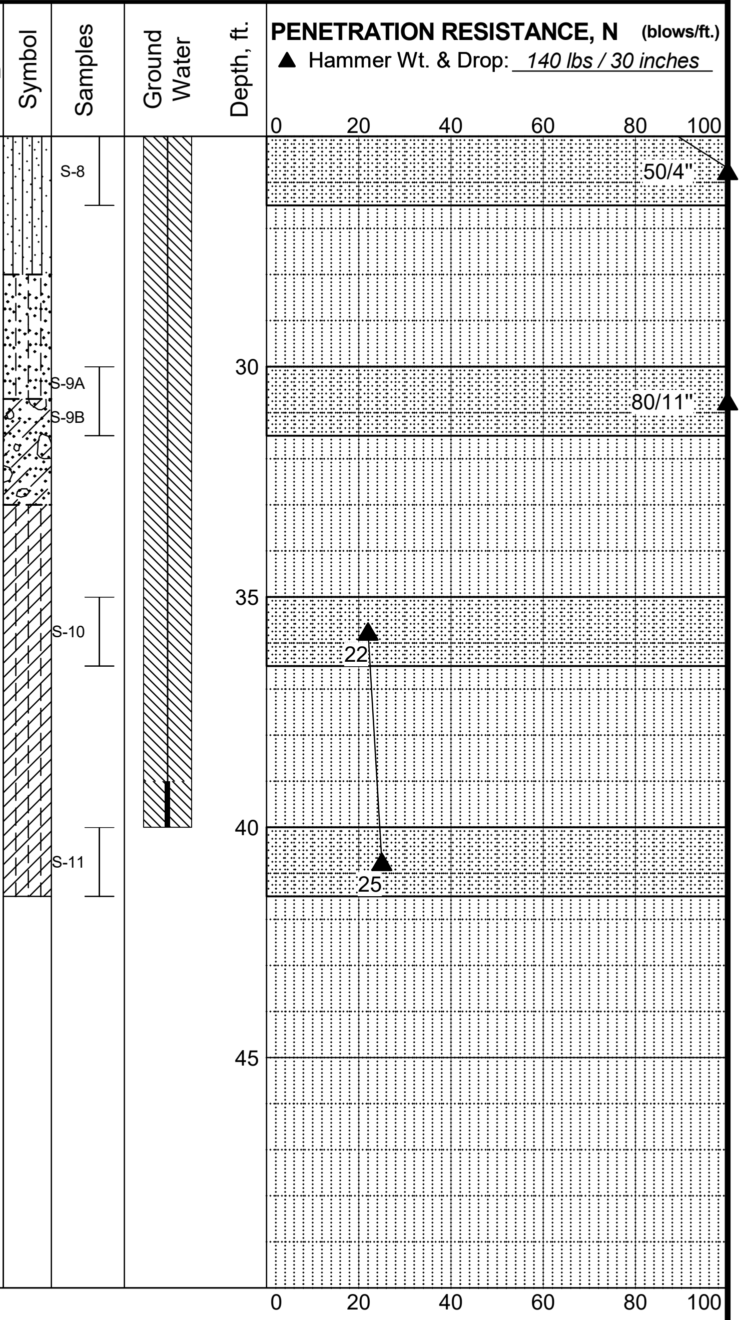
Total Depth:	41.5 ft.	Northing:	~	Drilling Method:	Mud Rotary	Hole Diam.:	12 in.
Top Elevation:	~ 341 ft.	Easting:	~	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-75 Truck Rig #4	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~	Other Comments:			

SOIL DESCRIPTION

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

SOIL DESCRIPTION	Elev. Depth (ft.)
Continued: Hard/Very dense, gray-brown, *Silt with Sand to Sandy Silt (ML)*; moist; fine sand; nonplastic to low plasticity; weakly cemented; slight iron oxidation and staining **SPRINGWATER FORMATION**	*313.0* 28.0
Very dense, red-brown, *Silty Sand (SM)*; moist; trace fine subangular to subrounded gravel; fine to medium sand; low plasticity fines.	*310.3* 30.7
Very dense, red-brown, *Clayey Sand with Gravel (SC)*; moist; fine to coarse, subangular to subrounded gravel; fine to coarse sand; low plasticity fines.	*308.0* 33.0
Very stiff, brown, *Lean Clay with Sand (CL)*; moist; fine to medium sand; medium plasticity. *RST Vibrating Wire Piezometer (S/N: VW188252) installed 40 feet below ground surface*	*299.5* 41.5

Completed: July 11, 2024



▨ Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

LEGEND

⌶ Standard Penetration Test

∇ Groundwater Level on Date Shown

NOTES

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

Rock Creek Interceptor Sewer
Clackamas County, Oregon

LOG OF BORING B-4

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SHANNON & WILSON

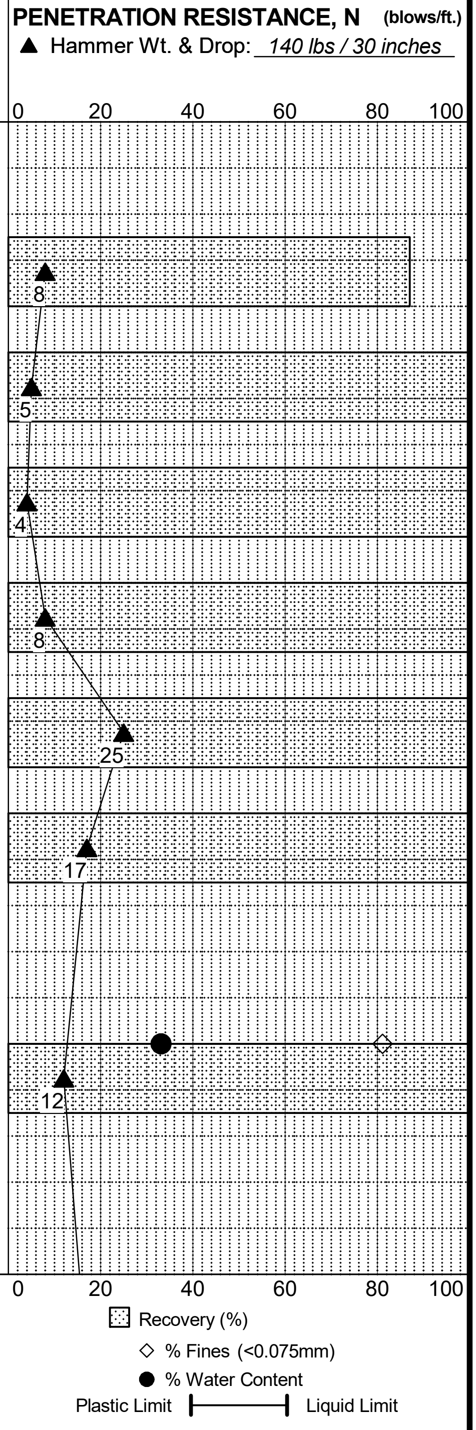
FIG. A5
Sheet 2 of 2

REV 2

MASTER_LOG_E 112335.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 8/5/24 *Log: NMB Rev: DSJ Typ: DSJ*

Total Depth: 41.5 ft.	Northing: ~	Drilling Method: Mud Rotary	Hole Diam.: 6 in.
Top Elevation: ~ 338 ft.	Easting: ~	Drilling Company: Western States	Rod Type: NWJ
Vert. Datum:	Station: ~	Drill Rig Equipment: CME-75 Truck Rig #4	Hammer Type: Automatic
Horiz. Datum:	Offset: ~	Other Comments:	

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)	Samples
Asphalt Concrete: 4-inches thick	*337.7* 0.3	
Aggregate Cement Concrete: 12-inches thick, contains asphalt and concrete fragments	*336.7* 1.3	
Base Aggregate: 4-inches thick	*336.3* 1.7	
Medium stiff to stiff, red-yellow and gray, *Lean Clay with Sand (CL)*; moist; fine to coarse sand; medium plasticity; trace organics; slight iron oxidation and staining. **FILL**	*333.5* 4.5	S-1
Loose, gray and brown, *Silty Sand (SM)*; moist; fine to medium sand; nonplastic to low plasticity fines; micaceous. **CATASTROPHIC FLOOD DEPOSITS**	*331.0* 7.0	S-2
Soft to medium stiff, gray, *Silt with Sand to Sandy Silt (ML)*; moist to wet; trace fine subangular to subrounded gravel; fine to coarse sand; low plasticity; trace organics; organic odor; micaceous. **ALLUVIUM**	*326.0* 12.0	S-3 S-4
Very stiff, brown and gray, *Lean Clay with Sand (CL)*; moist; fine sand; medium plasticity. **SPRINGWATER FORMATION**	*320.0* 18.0	S-5 S-6
Medium dense/Stiff, brown, *Silt with Sand (ML)*; moist; fine sand; nonplastic to low plasticity; iron oxidation and staining.	*315.0* 23.0	S-7
Hard, red-yellow, *Lean Clay with Sand (CL)*.		



CONTINUED NEXT SHEET

LEGEND

Ⅰ Standard Penetration Test

NOTES

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

Rock Creek Interceptor Sewer
Clackamas County, Oregon

LOG OF BORING B-5

October 2024 112335

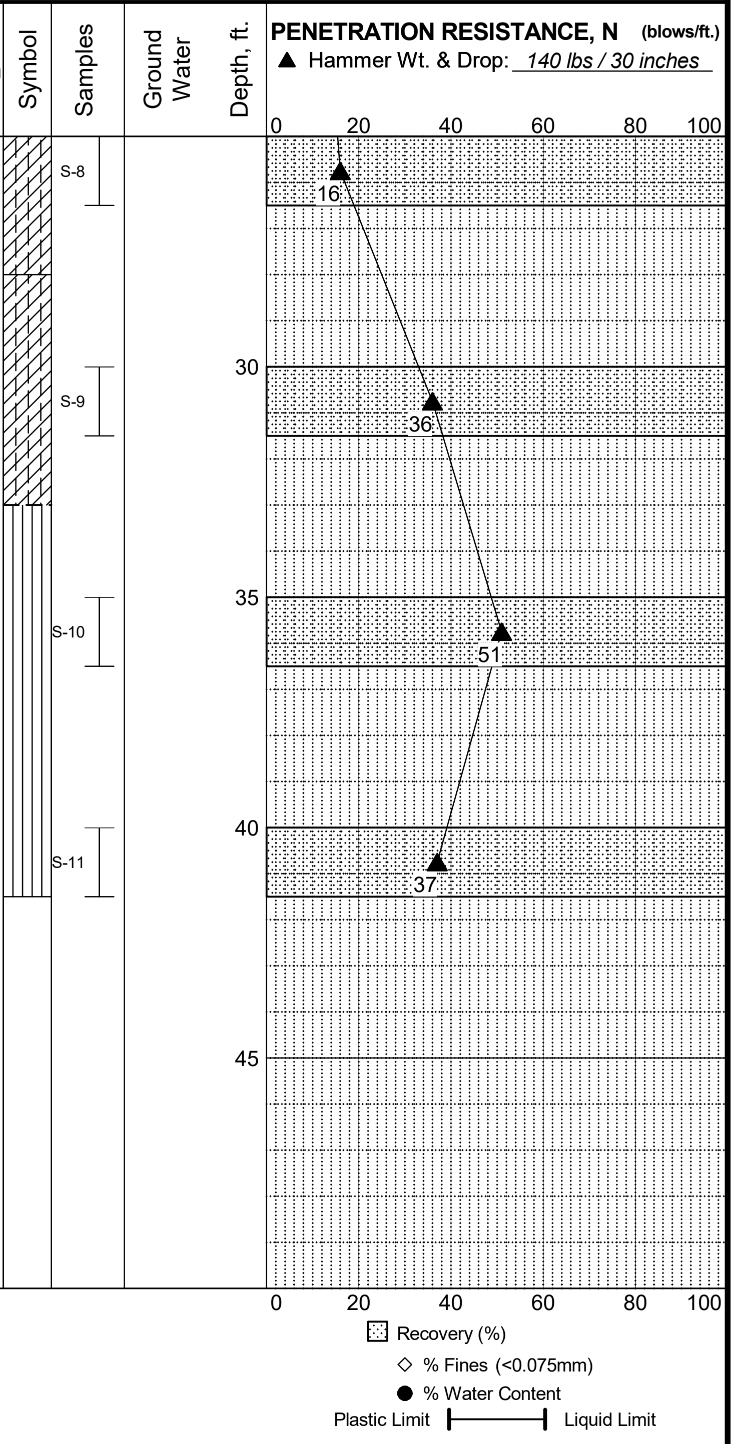
SHANNON & WILSON

FIG. A6
Sheet 1 of 2

REV 2

Total Depth: 41.5 ft.
Top Elevation: ~ 338 ft.
Vert. Datum:
Horiz. Datum:
Northing: ~
Easting: ~
Station: ~
Offset: ~
Drilling Method: Mud Rotary
Drilling Company: Western States
Drill Rig Equipment: CME-75 Truck Rig #4
Other Comments:
Hole Diam.: 6 in.
Rod Type: NWJ
Hammer Type: Automatic

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)
Continued: Hard, red-yellow and gray, *Lean Clay with Sand (CL)*; moist; trace fine subangular gravel; fine sand; medium plasticity; iron oxidation and staining	310.0 28.0
SPRINGWATER FORMATION Hard, brown, *Lean Clay to Lean Clay with Sand (CL)*; moist; fine sand; medium plasticity.	305.0 33.0
Hard, brown and yellow-brown, *Silt with Sand (ML)*; moist; fine sand; low plasticity.	296.5 41.5
Completed: July 10, 2024	



LEGEND

⌶ Standard Penetration Test

NOTES

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

Rock Creek Interceptor Sewer
Clackamas County, Oregon

LOG OF BORING B-5

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FIG. A6
Sheet 2 of 2

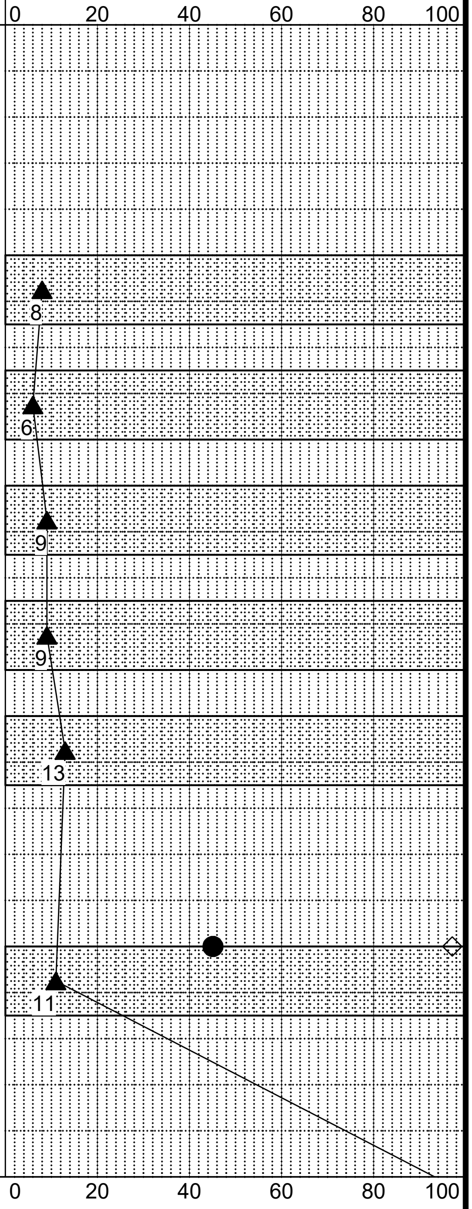
REV 2

MASTER_LOG_E 112335.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 8/5/24 Log: NMB Rev: DSJ Typ: DSJ

Total Depth: 30.3 ft.
Top Elevation: ~ 329 ft.
Vert. Datum:
Horiz. Datum:
Northing: ~
Easting: ~
Station: ~
Offset: ~
Drilling Method: Mud Rotary
Drilling Company: Western States
Drill Rig Equipment: CME-75 Truck Rig #4
Other Comments:
Hole Diam.: 6 in.
Rod Type: NWJ
Hammer Type: Automatic

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)	Samples
Asphalt Concrete: 4-inches thick	*328.7* 0.3	
Boring vacuum-excavated to 5-feet below ground surface		
Aggregate Cement Concrete: 12-inches thick, contains asphalt and concrete fragments	*327.7* 1.3	
Base Aggregate: 11-inches thick	*326.7* 2.3	
Medium stiff to stiff, gray-brown, *Lean Clay to Lean Clay with Sand (CL)*; moist; fine sand; medium plasticity; iron oxidation and staining. **CATASTROPHIC FLOOD DEPOSITS**		S-1 S-2
Stiff, red-brown and gray, *Lean Clay (CL)*; moist; fine sand; medium to high plasicity; iron oxidation and staining. **SPRINGWATER FORMATION**	*319.5* 9.5	S-3 S-4 S-5
Stiff, red, *Lean Clay (CL)*; moist; fine sand; medium plasticity.	*312.0* 17.0	S-6
Hard, red-brown and gray, *Sandy Lean Clay with Gravel (CL)*.	*306.0* 23.0	

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



CONTINUED NEXT SHEET

LEGEND

⌶ Standard Penetration Test

▨ Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├──┤ Liquid Limit

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

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Clackamas County, Oregon

LOG OF BORING B-8

October 2024
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FIG. A7
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REV 2

MASTER_LOG_E 112335.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 8/5/24 Log: NMB Rev: DSJ Typ: DSJ

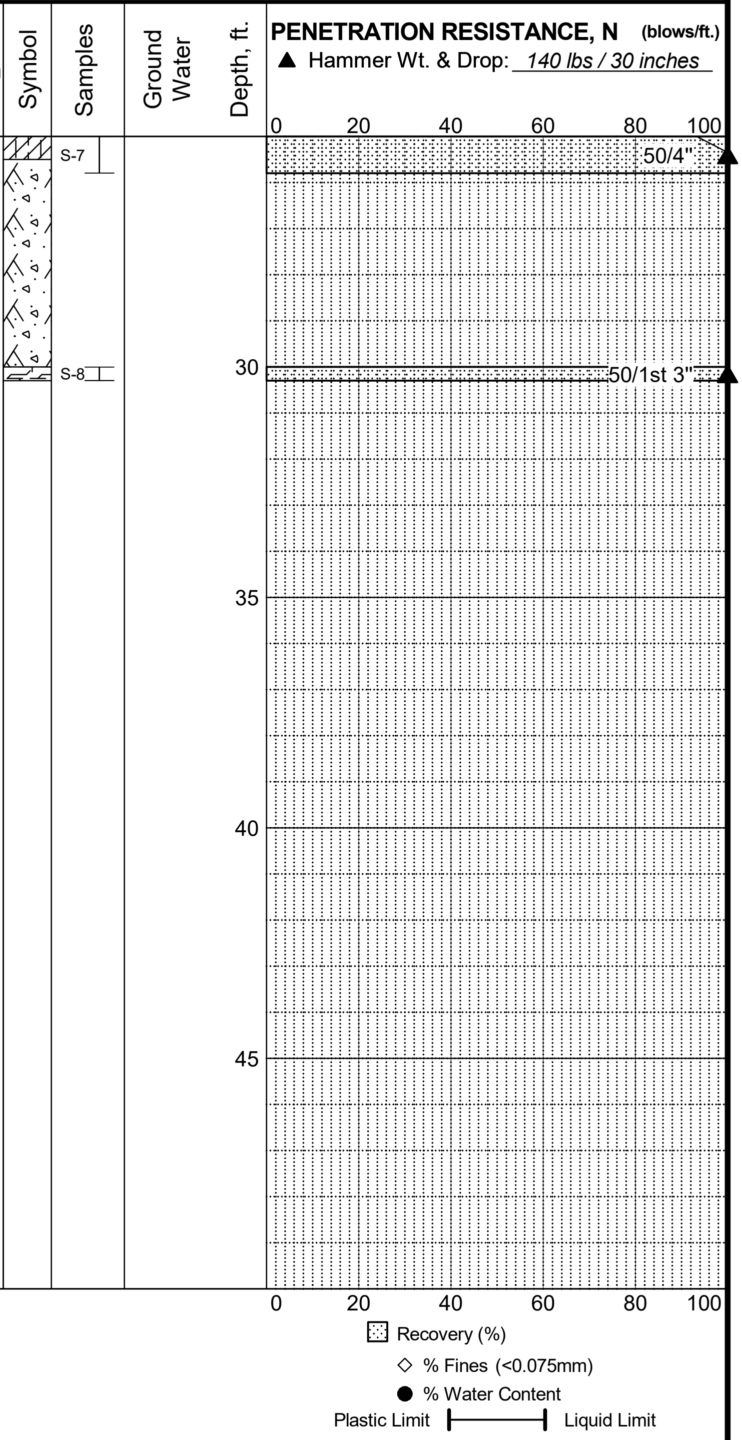
Total Depth:	30.3 ft.	Northing:	~	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 329 ft.	Easting:	~	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME-75 Truck Rig #4	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~	Other Comments:			

SOIL DESCRIPTION

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

SOIL DESCRIPTION	*Elev.* Depth (ft.)
Continued: Hard, red-brown and gray; *Sandy Lean Clay with Gravel (CL)*; moist; fine to coarse, subangular to subrounded gravel; fine to coarse sand; medium plasticity; iron oxidation and staining **SPRINGWATER FORMATION**	*303.5* 25.5
Very dense, red-brown and gray, *Clayey Gravel with Sand (GC)*; moist; fine to coarse, subangular to subrounded gravel; fine to coarse sand; medium plasticity fines. **DECOMPOSED BORING BASALT**	*299.0* 30.0
BASALT: medium strong to strong (R3-R4), gray, fine-grained, slightly vesicular; joints indiscernible in SPT sample; slightly weathered. **BORING BASALT**	*298.7* 30.3

Completed: July 10, 2024



LEGEND

⌶ Standard Penetration Test

NOTES

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

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Clackamas County, Oregon

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FIG. A7
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REV 2

MASTER_LOG_E 112335.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 8/5/24 *Log: NMB Rev: DSJ Typ: DSJ*

Total Depth: 36.3 ft.
Top Elevation: ~ 353 ft.
Vert. Datum:
Horiz. Datum:
Northing: ~
Easting: ~
Station: ~
Offset: ~
Drilling Method: Mud Rotary
Drilling Company: Western States
Drill Rig Equipment: CME-75 Truck Rig #4
Other Comments:
Hole Diam.: 6 in.
Rod Type: NWJ
Hammer Type: Automatic

SOIL DESCRIPTION

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

Asphalt Concrete: 4-inches thick

Aggregate Cement Concrete: 14-inches thick, contains asphalt and concrete fragments

Base Aggregate: 5-inches thick

Medium stiff to stiff, brown, *Lean Clay with Sand (CL)*; moist; fine sand; medium plasticity; micaceous; iron oxidation and staining.

CATASTROPHIC FLOOD DEPOSITS

Trace gravel at 5 feet

Very dense, brown and gray, *Clayey Sand (SC)*; moist; trace fine to coarse, subangular to subrounded gravel; fine to coarse sand; low to medium plasticity fines.

SPRINGWATER FORMATION

Very dense, gray, *Silty Sand (SM)*; moist; fine to medium sand; nonplastic to low plasticity fines; weakly cemented.

Very dense, gray, *Silty Gravel with Sand (GM)*; moist; fine to coarse, subangular to subrounded gravel; fine to coarse sand; nonplastic to low plasticity fines; weakly cemented.

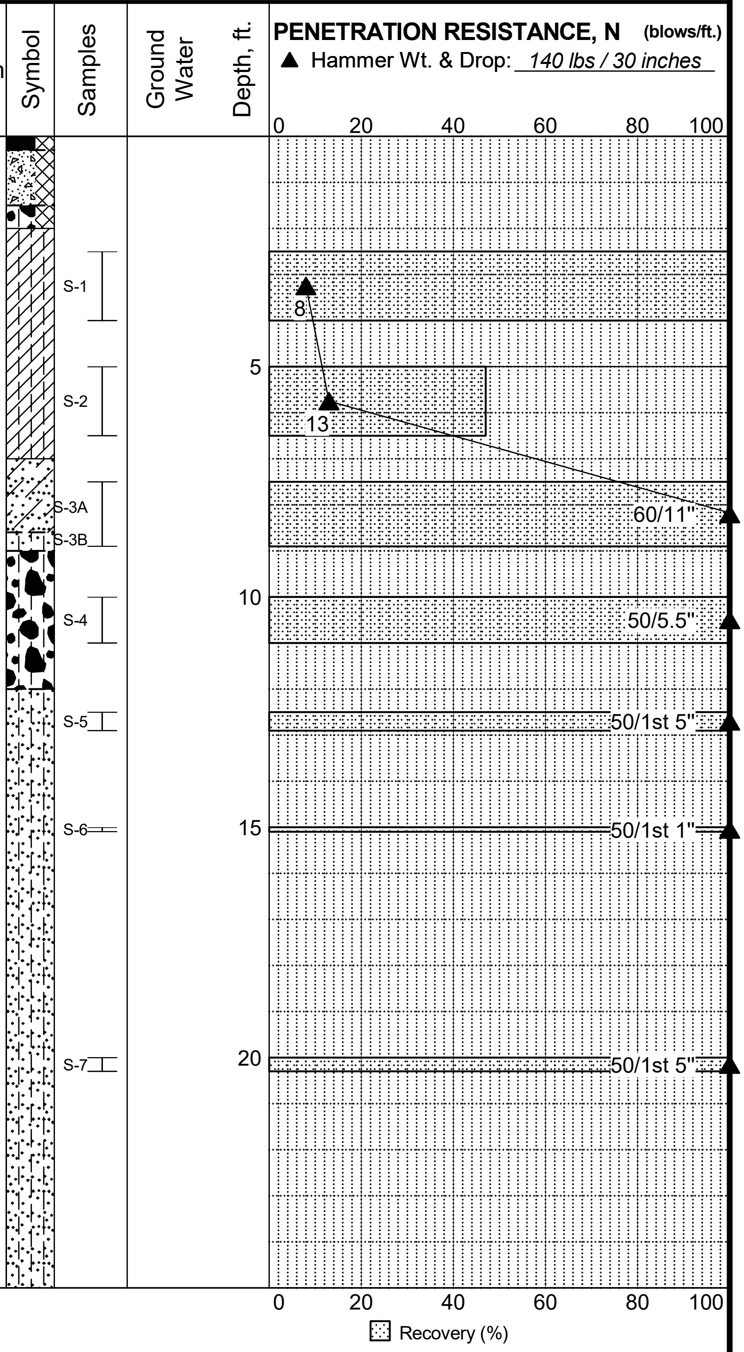
Very dense, gray, *Silty Sand with Gravel (SM)*; moist; fine subangular to subrounded gravel; fine to coarse sand; nonplastic fines; weakly cemented.

Elev. Depth (ft.)

352.7 0.3
351.5 1.5
351.0 2.0
346.0 7.0
344.4 8.6
344.0 9.0
341.0 12.0

PENETRATION RESISTANCE, N (blows/ft.)

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



CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered

⌶ Standard Penetration Test

● % Water Content

Plastic Limit ├───┤ Liquid Limit

NOTES

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

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FIG. A8
Sheet 1 of 2

REV 2

MASTER_LOG_E 112335.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 8/5/24 Log: NMB Rev: DSJ Typ: DSJ

Total Depth: 36.3 ft.
Top Elevation: ~ 353 ft.
Vert. Datum:
Horiz. Datum:

Northing: ~
Easting: ~
Station: ~
Offset: ~

Drilling Method: Mud Rotary
Drilling Company: Western States
Drill Rig Equipment: CME-75 Truck Rig #4
Other Comments:

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

SOIL DESCRIPTION

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

Elev. Depth (ft.)

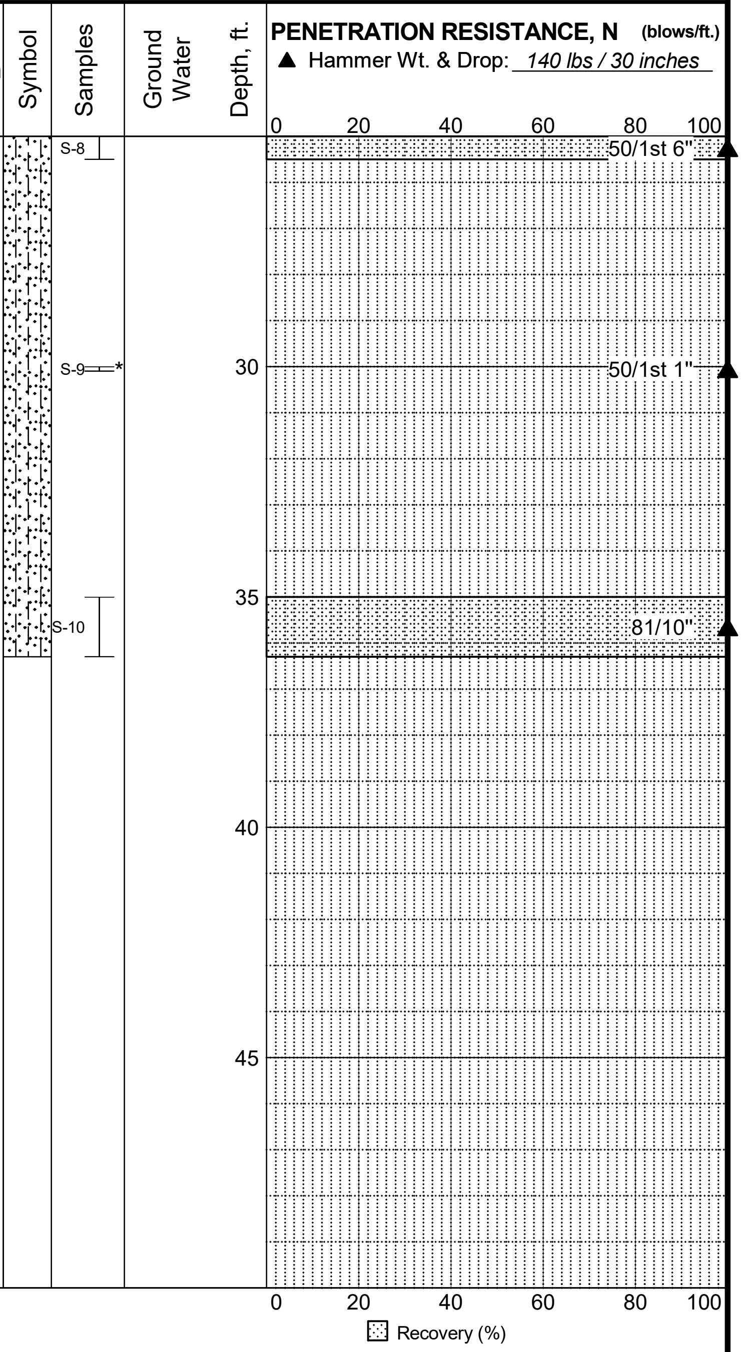
Continued:

Very dense, gray, *Silty Sand with Gravel (SM)*; moist; fine subangular to subrounded gravel; fine to coarse sand; nonplastic fines; weakly cemented

SPRINGWATER FORMATION

316.7
36.3

Completed: July 8, 2024



LEGEND
- * Sample Not Recovered
- ⌶ Standard Penetration Test

● % Water Content
Plastic Limit ├──┤ Liquid Limit

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

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FIG. A8
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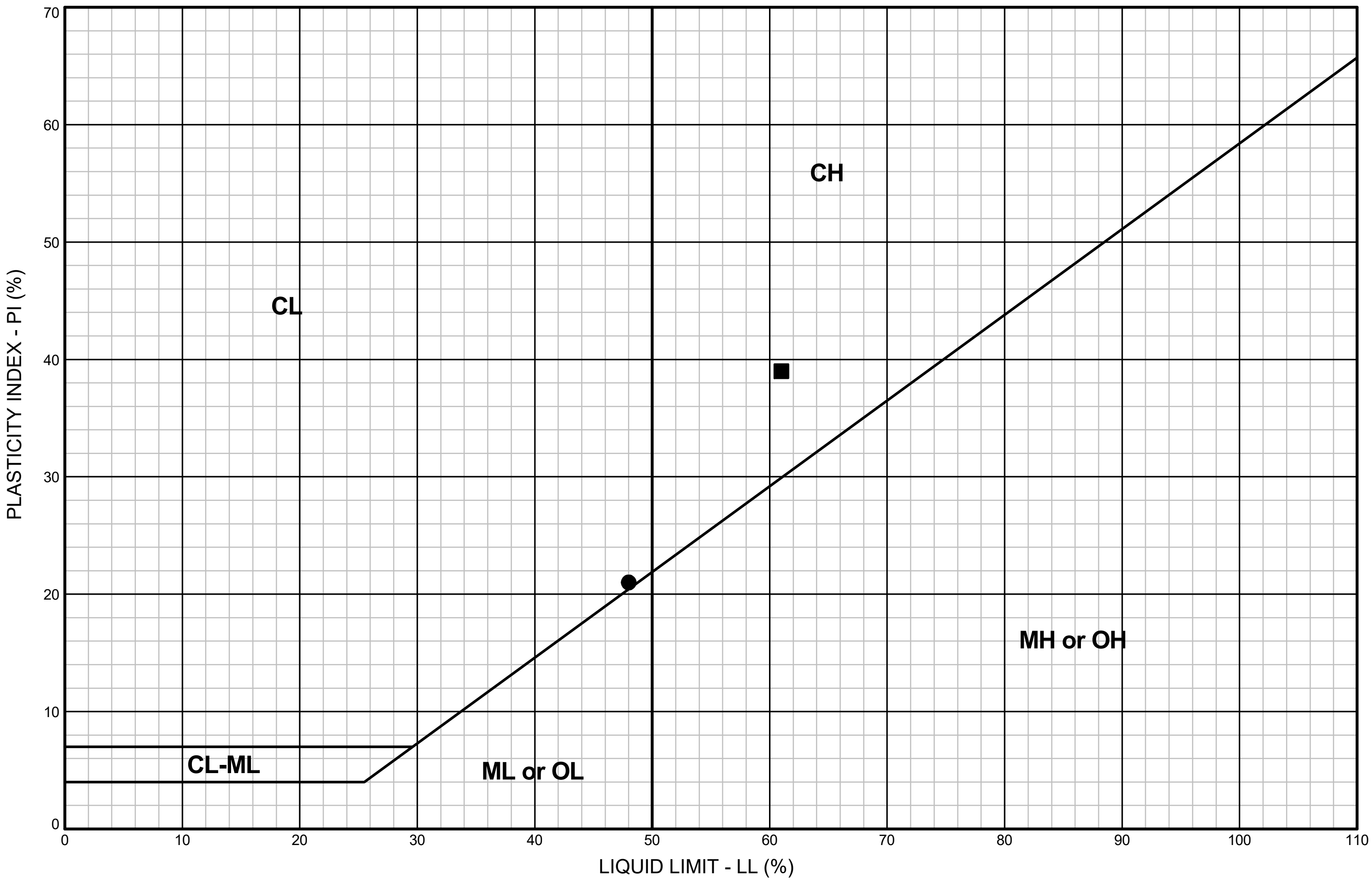
REV 2

MASTER_LOG_E 112335.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 8/5/24 Log: NMB Rev: DSJ Typ: DSJ



NOTES

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

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

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

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	LL %	PL %	PI %[3]	NAT. W.C. %	FINES %
● B-2, S-7	20.0	*CL*	*Lean Clay*	48	27	21	36	
■ B-4, S-4	10.0	*CH*	*Fat Clay*	61	22	39	34	

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ATTERBERG LIMITS RESULTS

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FIG. B1

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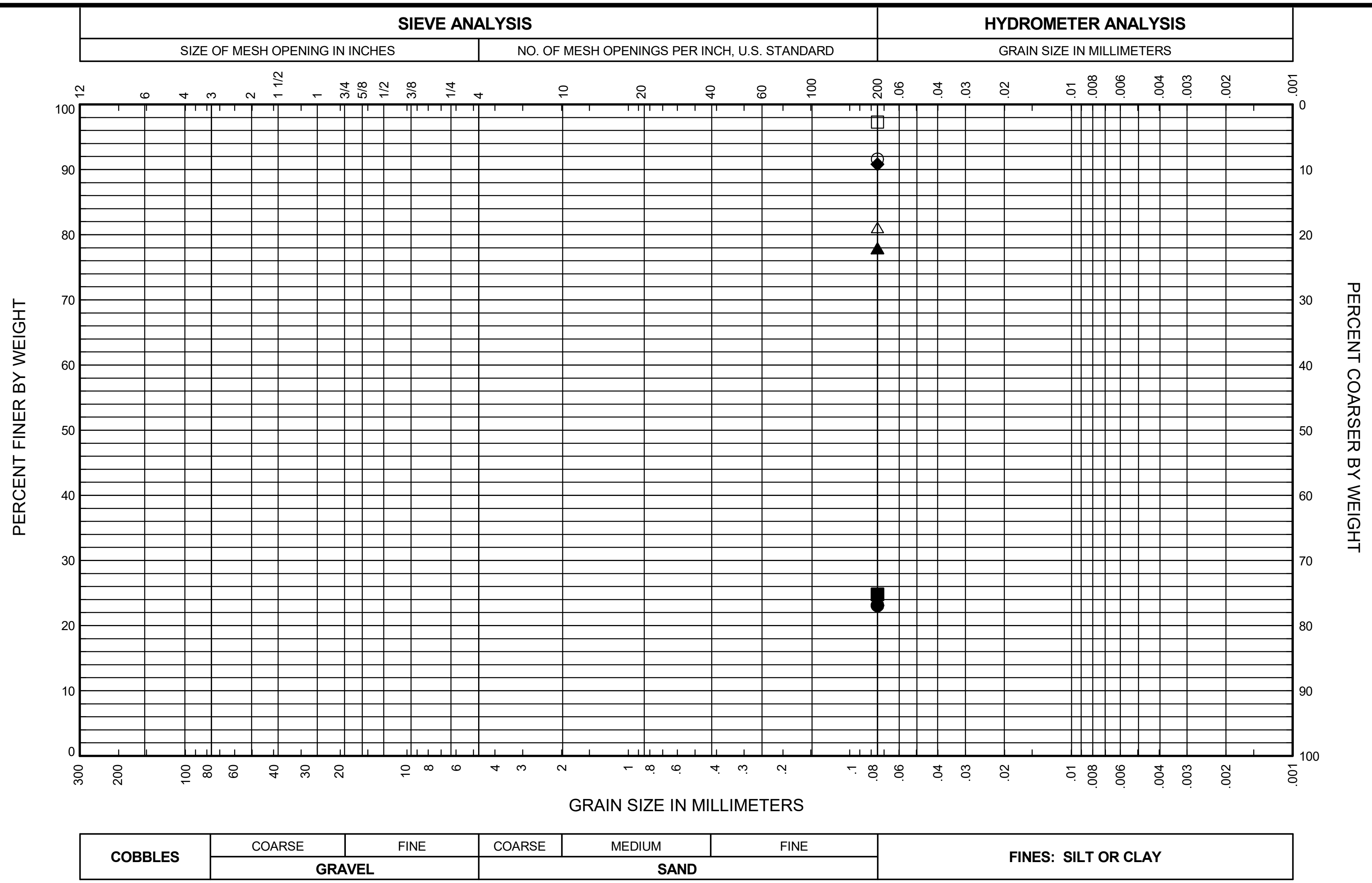
FIG. B1



NOTES:

1) Sieve analyses were performed in general accordance with ASTM D6913, sieve with hydrometer analyses were performed in general accordance with ASTM D422, and amount finer than #200 sieve analyses were performed in general accordance with ASTM D1140 unless otherwise noted in the report.

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

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-1, S-2	5.0	*GM*	*Silty Gravel with Sand*	-	-	23	21	
■ B-1, S-5	12.5	*SM*	*Silty Sand*	-	-	25	16	
▲ B-2, S-2	5.0	*CL*	*Lean Clay with Sand*	-	-	78	30	
◆ B-3, S-4	10.0	*ML/CL*	*Silt to Lean Clay*	-	-	91	40	
○ B-3, S-9	30.0	*CL*	*Lean Clay*	-	-	92	25	
□ B-4, S-7	20.0	*CL/CH*	*Lean Clay to Fat Clay*	-	-	97	28	
△ B-5, S-7	20.0	*ML*	*Silt with Sand*	-	-	81	33	

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GRAIN SIZE DISTRIBUTION

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FIG. B2
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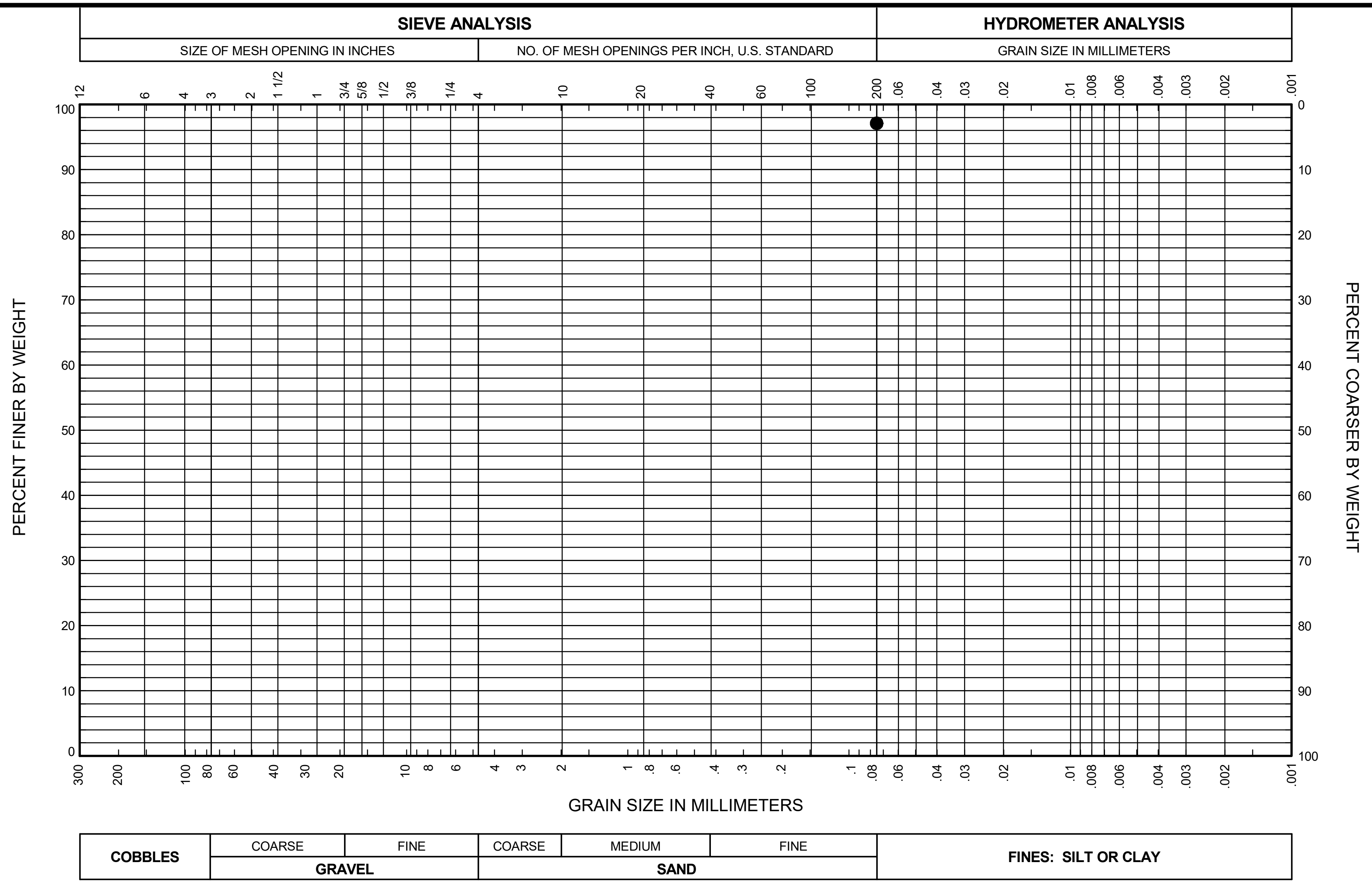

NOTES:

1) Sieve analyses were performed in general accordance with ASTM D6913, sieve with hydrometer analyses were performed in general accordance with ASTM D422, and amount finer than #200 sieve analyses were performed in general accordance with ASTM D1140 unless otherwise noted in the report.

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

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL^2	GROUP NAME^2	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-8, S-6	20.0	*CL*	*Lean Clay*	-	-	97	45	

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GRAIN SIZE DISTRIBUTION

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FIG. B2
Sheet 2 of 2

FIG. B2

APPENDIX E

Site-Specific Seismic Hazard Evaluation

APPENDIX E

SITE-SPECIFIC SEISMIC HAZARD EVALUATION

E.1 GENERAL

GRI completed a site-specific seismic hazard evaluation for the proposed City of Happy Valley Community Recreation Center project located in Happy Valley, Oregon. The proposed project includes construction of a two-story community recreation center building and associated improvements. The primary purpose of this work was to review the potential seismic hazards associated with regional and local seismicity. We understand the project will be designed in accordance with the upcoming 2025 *Oregon Structural Specialty Code* (OSSC) and the American Society of Civil Engineers (ASCE) 7-22 Document, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-22). ASCE 7-22 requires evaluation of seismic hazards based on the Risk-Targeted Maximum Considered Earthquake (MCE_R), which is defined in Chapter 21 of ASCE 7-22 as the response spectrum expected to achieve a 1% probability of building collapse within a 50-year period. We understand that the proposed building is considered a risk category of III in accordance with Section 1604.5 of the 2025 OSSC. As a large public assembly structure, the proposed building meets the criteria for special occupancy and therefore requires a site-specific seismic hazard evaluation under the 2025 OSSC.

Our site-specific seismic hazard study was based on the potential for regional and local seismic activity, as described in the existing scientific literature, and the subsurface conditions at the site, as disclosed by the geotechnical explorations completed for the project. Specifically, our work included the following tasks:

1. A review of available literature, including published papers, maps, open-file reports, seismic histories and catalogs, and other sources of information regarding the tectonic setting, regional and local geology, and historical seismic activity that might have a significant effect on the site.

2. Compilation, examination, and evaluation of existing subsurface data gathered at the site, including classification and laboratory analyses of soil samples. This information was used to prepare a generalized subsurface profile for the site.

3. Identification of potential seismic sources appropriate for the site and characterization of those sources in terms of magnitude, distance, and acceleration response spectra.

4. Office studies based on the generalized subsurface profile and controlling seismic sources resulting in conclusions and recommendations concerning the following:

a. Specific seismic events and characteristic earthquakes that might have a significant effect on the project site.

b. The potential for ground motion amplification and liquefaction or soil-strength loss at the site.

c. Site-specific acceleration response spectra for design of structures at the site.

This appendix describes the work accomplished and summarizes our conclusions and recommendations.

E.2 TECTONIC AND GEOLOGIC SETTING

On a regional scale, the site lies at the northern end of the Willamette Valley, a broad, gently deformed, north-south-trending topographic feature separating the Coast Range to the west from the Cascade Mountains to the east. The site lies approximately 95 kilometers (km) inland from the down-dip edge of the seismogenic extent of the Cascadia Subduction Zone (CSZ), an active convergent-plate boundary along which remnants of the Farallon Plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American continent. The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs of the Gorda, Juan de Fuca, and Explorer plates and the overriding North American Plate, as shown on the Tectonic Setting Summary, Figure 1E.

On a local scale, the site lies within the Portland BasinAr, a large, well-defined, northwest-trending structure characterized as a right-lateral pull-apart basin in the forearc of the CSZ. The local surface geology in close proximity to the site is shown on the Local Geologic Map, Figure 2E.

The site is generally mantled with a layer of Pleistocene fine-grained facies of catastrophic flood deposits, referred to as Willamette Silt. These deposits consist of stratified clay, silt, sand and smaller amounts of gravel. These deposits are underlain by a thick sequence of basalt lava flows of Boring Lava. Mapped nearby is the Pliocene to Pleistocene Basalt of Boring Lava and Springwater Formation. Cross-sections show the Boring Lava basalts interfingering with the slightly older Springwater Formation. The Boring Lava originates from a series of local vents and is separated into several different chemically distinct basalt flows, typically gray basalt and basaltic andesite flows and associated scoria (Madin, 1994). The Springwater Formation is mapped as a fluvial conglomerate, volcaniclastic sandstone, siltstone, and debris flows derived from the Cascade Range (Madin, 1994).

The Portland Basin is bounded by high-angle, northwest-trending, right-lateral strike-slip faults considered to be seismogenic; however, the relationship between specific earthquakes and individual faults in the area is not well understood because few of these faults are expressed clearly at the ground surface. The distribution of nearby Quaternary faults is shown on the Local Fault Map, Figure 3E.

E.3 SEISMIC SETTING

E.3.1 General

Because of the proximity of the site to the CSZ and its location within the Portland Basin, three seismic sources contribute to the potential for damaging earthquake motions at the site. Two of these sources are associated with tectonic activity related to the CSZ, including interface subduction-zone events related to sudden slip between the upper surface of the Juan de Fuca Plate and the lower surface of the North American Plate and subcrustal (Benioff zone) events related to deformation and volume changes within the deeper portion of the subducted Juan de Fuca Plate. The third source is associated with movement on relatively shallow faults within and adjacent to the Portland Basin. Each of these sources is considered capable of producing damaging earthquakes in the Pacific Northwest; however, there are no historical records of significant subcrustal earthquakes (M_W >6.0) in northwest Oregon and southwest Washington. Wong (2005) hypothesizes that, due to subduction-zone geometry, geophysical conditions, and local geology, southwest Washington and northwest Oregon may not be subject to subcrustal earthquakes of significant magnitude.

Based on review of historical records and evaluation of U.S. Geological Survey (USGS) national seismic hazard maps (NSHMs), the two primary types of seismic sources at the site are the CSZ interface and local crustal faults.

E.3.2 Cascadia Subduction Zone

Coastal paleoseismic evidence, offshore geological studies, and historical tsunami accounts indicate the CSZ is capable of producing large-magnitude megathrust earthquakes (M_W 8 to M_W 9) at the interface between the Juan de Fuca and North American plates (Atwater et al., 1995; Goldfinger et al., 2012). Geological studies indicate these megathrust earthquakes have occurred repeatedly in the past 10,000 years (Walton et al., 2021). A combination of paleoseismic and geologic studies (Kelsey et al., 2005) and geodetic studies (Savage et al., 2000) indicate a rate of strain accumulation consistent with the assumption that the CSZ is locked beneath offshore northern California, Oregon, Washington, and southern British Columbia (Fluck et al., 1997; Wang et al., 2001). Numerous geological and geophysical studies suggest the CSZ may be segmented (Hughes and Carr, 1980; Weaver and Michaelson, 1985; Guffanti and Weaver, 1988; Goldfinger, 1994; Kelsey and Bockheim, 1994; Mitchell et al., 1994; Personius, 1995; Nelson and Personius, 1996; Witter, 1999), but the most recent studies suggest that, for the last great earthquake in 1700, most of the subduction zone ruptured in a single M_W 9.0

earthquake (Satake et al., 1996; Atwater and Hemphill-Haley, 1997; Clague et al., 2000). There is consensus within the scientific community that the most recent great earthquake occurred along the CSZ in January 1700 (Atwater et al., 2015), based on paleoseismic evidence and historical records of an orphan tsunami in Japan. Tsunami modeling completed for the 1700 orphan tsunami indicated the 1700 earthquake ruptured the whole length of the CSZ and had a moment magnitude of about M_W 9.0 (Satake et al. 2003).

The average recurrence interval for a CSZ megathrust event is estimated to be around 350 years to 600 years based on prehistoric geologic evidence (Atwater and Hemphill-Haley, 1997; Kelsey et al., 2002; Witter et al., 2003). Tsunami inundation in buried marshes along the Washington and Oregon coasts and stratigraphic evidence from the Cascadia margin support these recurrence intervals (Kelsey et al., 2005; Goldfinger et al., 2003). Goldfinger et al. (2003, 2012, 2017) evaluated turbidite evidence at the heads of Cascadia submarine canyons, the results of which indicated the occurrence of more than 40 great earthquakes over the past 10,000 years with partial or entire length rupture of the CSZ. About 20 of the earthquake events are associated with partial ruptures concentrated in the southern part of the margin and have estimated recurrence intervals of about 220 years to 320 years. About 19 of the events are associated with a rupture of the full CSZ, characterized by a moment magnitude (M_W) of about 8.5 to 9.1 or greater. Considering a combination of recent paleoseismic, geodetic, and geologic research, the average recurrence interval for a full-rupture CSZ earthquake is estimated to be about 500 years to 540 years (Walton et al., 2021).

The USGS probabilistic analysis assumes four potential locations (three alternative down-dip edge options and one up-dip edge option) for the eastern edge of the earthquake rupture zone for the CSZ, as shown on the Location of Surface Traces for Up-Dip Edge & Three Down-Dip Edge Options Used in 2014 NSHMs, Figure 4E. As discussed in Petersen et al. (2014), the 2014 USGS mapping effort represents the 2014 CSZ source model with full-CSZ ruptures with moment magnitudes from M_W 8.6 to M_W 9.3, supplemented by partial ruptures with smaller magnitudes (M_W 8.0 to M_W 9.1). There is also a possibility of serial M_W 8 earthquakes that rupture the entire CSZ over a period of a few decades or less; however, this is not implemented in the current NSHMs. The partial ruptures were accounted for using a segmented model and an unsegmented model. The magnitude-frequency distribution showing the contributions to the earthquake rates from each of the models and how the estimated rates vary along the fault is presented on the Variation of Earthquake Rates Cascadia Subduction Zone, Figure 5E.

E.3.3 Local Crustal Event

Sudden crustal movements along relatively shallow, local faults in the project area, although rare, have been responsible for local crustal earthquakes. The precise relationship between specific earthquakes and individual faults is not well understood because few of the faults in the area are expressed at the ground surface and there is a limited history of crustal events in the region. The history of local seismic activity is commonly used as a basis for determining the size and frequency to be expected of local crustal events. Although the historical record of local earthquakes is relatively short (the earliest reported seismic event in the area occurred in 1920), it can serve as a guide for estimating the potential for seismic activity in the area.

The locations of and general information regarding Quaternary faults (i.e., those that have experienced movement during the last 1.6 million years and are considered potentially active) are available through the USGS Earthquake Hazards Program. Based on fault mapping conducted by the USGS, the Grant Butte fault, located approximately 4.5 km away with a characteristic earthquake magnitude of Mw 6.2, and the Portland Hills fault, located about 10.5 km from the site with a characteristic magnitude of Mw 7.0, are the two crustal faults that contribute significantly to the site's seismic hazard. Although not included in the 2018 USGS NSHM due to a lack of evidence for movement since the late Pleistocene, the Damascus-Tickle Creek fault zone is situated within approximately 1 km of the site. One of its inferred fault traces is mapped as crossing the northeastern portion of the site.

E.4 SITE-SPECIFIC GROUND MOTIONS

E.4.1 General

As previously stated, the seismic evaluation for the proposed community recreation center building is being completed in accordance with ASCE 7-22 and the 2025 OSSC. The proposed building is considered a risk category of III in accordance with Section 1604.5 of the 2025 OSSC. A ground motion hazard analysis was completed in accordance with Section 21.2 of ASCE 7-22 to evaluate the seismic response of the soils at the site and develop a recommended MCE_R response spectrum for the project. The recommended MCE_R response spectra at the ground surface are generally developed by comparing site-specific and code-based spectral values at the ground surface.

The site-specific MCE_R spectral values are defined as the lesser of probabilistic and deterministic ground motions as described in Section 21.2 of ASCE 7-22. The ground motion associated with the probabilistic MCE_R represents a targeted risk level of 1% in the 50-year probability of collapse in the direction of maximum horizontal response with 5% damping. The deterministic MCE_R is calculated as an 84th-percentile, 5%-damped spectral response in the direction of maximum horizontal response, based on scenario earthquakes associated with known active faults in the region. The highest spectral acceleration from all such scenario earthquakes is used, with the scenarios derived from deaggregation analyses identifying sources that contribute more than 10% to the probabilistic spectral response at each period.

The code-based spectral response values were obtained from the USGS Seismic Design Geodatabase using the web-based ASCE 7 Hazard Tool. This tool provides both two-period (short-period and 1-second) and multi-period response spectra in accordance with provisions in Chapter 11 of ASCE 7-22. The spectral values are derived for the site's specific latitude and longitude, incorporating site class effects and seismic design category based on the mapped seismic hazard data.

E.4.2 Site-Specific MCE_R Response Spectrum

As previously mentioned, the site-specific MCE_R spectral response acceleration is defined by lesser spectral response accelerations from probabilistic ground motions and deterministic ground motions. The site-specific probabilistic seismic hazard analysis (PSHA) was conducted using the USGS NSHM Hazard Tool. In accordance with ASCE 7-22, the probabilistic MCE_R response spectral accelerations correspond to the direction of maximum horizontal response and are represented by a 5%-damped acceleration response spectrum that targets a uniform 1% probability of collapse within a 50-year period. The PSHA results are initially expressed as geometric mean spectral accelerations with a 2,475-year return period (i.e., 2% probability of exceedance in 50 years). To obtain the probabilistic MCE_R values, the geometric mean results are adjusted using directionality factors and risk-targeted collapse fragility coefficients. The directionality factors convert the geometric mean values to the maximum-direction response, while the fragility curves calibrate the ground motions to meet the targeted collapse risk of 1% in 50 years.

The site-specific PSHA requires an average shear wave velocity in the upper 100 feet (V_{S30}) as an input to the NSHM Hazard Tool. The V_{s30} for the site was estimated based on shear wave data obtained from a seismic cone penetration test probe and a shear wave refraction microtremor analysis test completed at the site. The average shear wave velocity in the upper 100 feet was estimated to be approximately 1,760 feet per second (ft/s) representing a Site Class C condition. The resulting site-specific probabilistic spectral values are summarized in Table 1E, below.

Table 1E: SITE-SPECIFIC PROBABILISTIC MCE_R AND DETERMINISTIC LOWER LIMIT VALUES

Period, second	Probabilistic MCE_R Values, g	Deterministic Lower Limit
PGA	0.44	0.73
0.05	0.56	0.96
0.1	0.87	1.37
0.2	1.06	1.71
0.3	0.97	1.66
0.5	0.72	1.38
0.75	0.55	1.07

Period, second	Probabilistic MCE_R Values, g	Deterministic Lower Limit
1	0.44	0.86
2	0.24	0.45
3	0.15	0.31
4	0.11	0.24
5	0.09	0.19

Abbreviations: MCE_R = Risk-Targeted Maximum Considered Earthquake; PGA = peak ground acceleration

A deterministic seismic hazard analysis can be completed concurrently with the PSHA to evaluate ground motions in accordance with Section 21.2.2 of ASCE 7-22. However, calculation of the deterministic ground motion response spectrum is not required when the probabilistic MCE_R spectral response values at all periods are less than the deterministic lower limit spectral values specified for the site class. Table 1E summarizes the deterministic lower limit spectral values for Site Class C conditions. As shown, the probabilistic MCE_R spectral values are below the deterministic lower limit values across all periods. Therefore, the site-specific MCE_R response spectrum is governed entirely by the probabilistic MCE_R values.

E.4.3 Recommended Design Acceleration Parameters

The recommended response spectrum for structural design is typically developed by comparing the site-specific spectrum based on ground motion hazard analysis with the code-based spectral values based on Site Class. The project site is designated Site Class C based on available shear wave velocity data at the site. ASCE 7-22 requires the site-specific spectral accelerations at the ground surface to not be less than 80% of the spectral values determined for Site Class C.

Comparisons of the site-specific and code-based MCE_R ground-surface spectra for Site Class C are shown on the MCE_R Response Spectra Comparison (5% Damping), Figure 6E. As shown in the figure, the site-specific MCE_R spectral values were generally observed to be greater than 80% of the code-based MCE_R spectral values at all periods. Therefore, the site-specific MCE_R spectral values summarized in Table 1E above represent the recommended multi-period MCE_R spectrum for dynamic seismic analysis of the building using the modal response-spectrum analysis or nonlinear response history analysis procedures. The design response spectrum is developed by taking two-thirds of the MCE_R response spectrum. Our recommended MCE_R and design response spectral values for design of the project are summarized in Table 2E. The table presents both multi-period and two-period spectral values. The two-period spectral values are derived in accordance with the guidelines provided in Section 21.4 of ASCE 7-22. In accordance with Section 21.4, the 0.2-second MCE_R spectral value can be taken as 90% of the maximum spectral acceleration obtained from the site-specific response spectrum at any period within the range of 0.2 seconds to 5.0 seconds. The 1.0-second MCE$_R$ spectral value can be derived based on 90% of the maximum value of the product of spectral acceleration and corresponding periods for periods ranging from 1.0 seconds to 2 seconds for sites with a V$_{S30}$ value greater than 1,450 ft/s but not less than 100% of the spectral value at 1 second.

Table 2E: RECOMMENDED MCE$_R$ AND DESIGN RESPONSE SPECTRAL VALUES AT GROUND SURFACE, 5% DAMPING

Period, seconds	Recommended Multi-Period Spectral Values	
	MCE$_R$-Level Response Spectral Values, g	Design-Level Response Spectral Values, g
PGA	0.44	0.30
0.05	0.56	0.38
0.1	0.87	0.58
0.2	1.06	0.71
0.3	0.97	0.64
0.4	0.83	0.55
0.5	0.72	0.48
0.75	0.55	0.37
1	0.44	0.29
1.5	0.31	0.21
2	0.24	0.16
3	0.15	0.10
4	0.11	0.07
5	0.09	0.06
Parameter	**Recommended Two-Period Spectral Values**	
S_{MS} / S_{DS}	0.96	0.64
S_{M1} / S_{D1}	0.44	0.29

Abbreviations: MCE$_R$ = Risk-Targeted Maximum Considered Earthquake; PGA = peak ground acceleration

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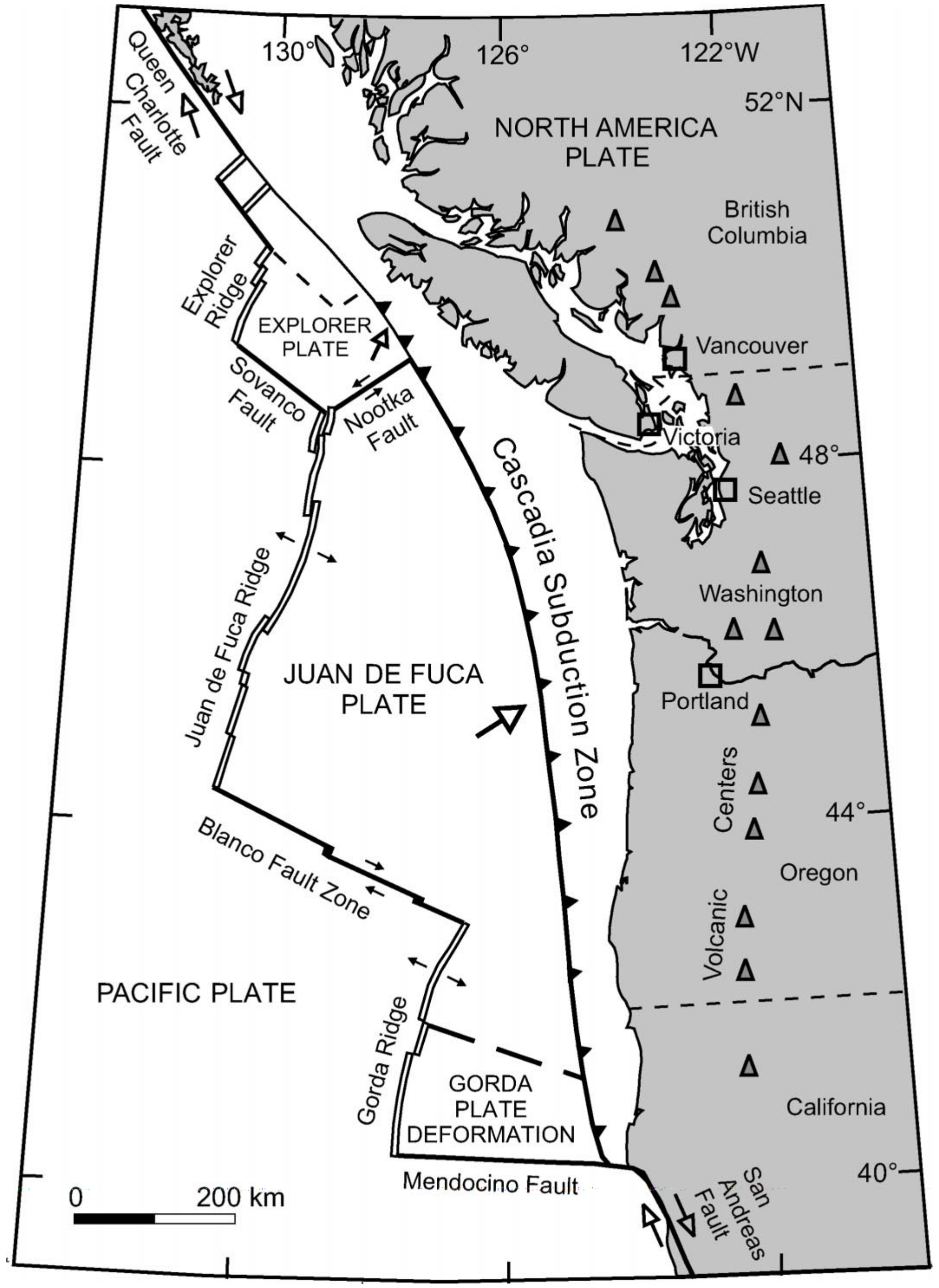
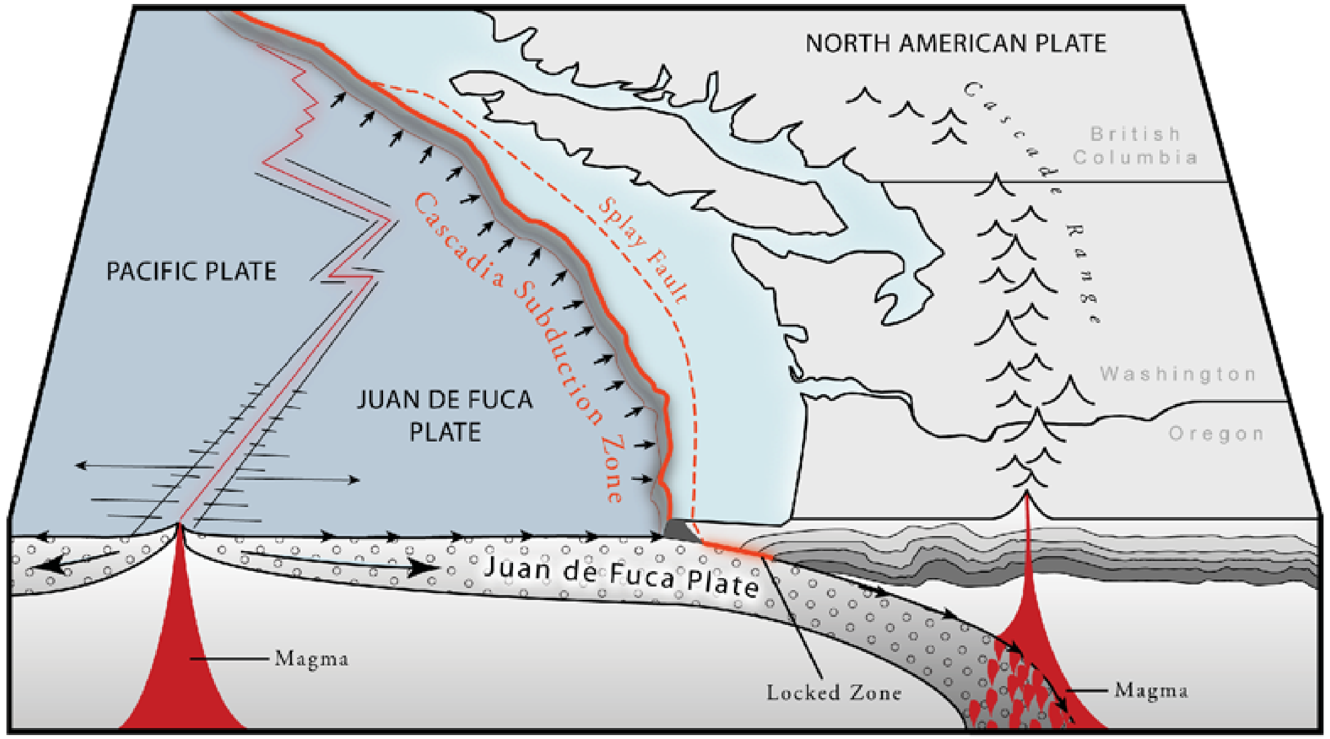


A) TECTONIC MAP OF PACIFIC NORTHWEST, SHOWING ORIENTATION AND EXTENT OF CASCADIA SUBDUCTION ZONE (MODIFIED FROM DRAGERT ET AL., 1994)

Cascadia Subduction Zone Setting

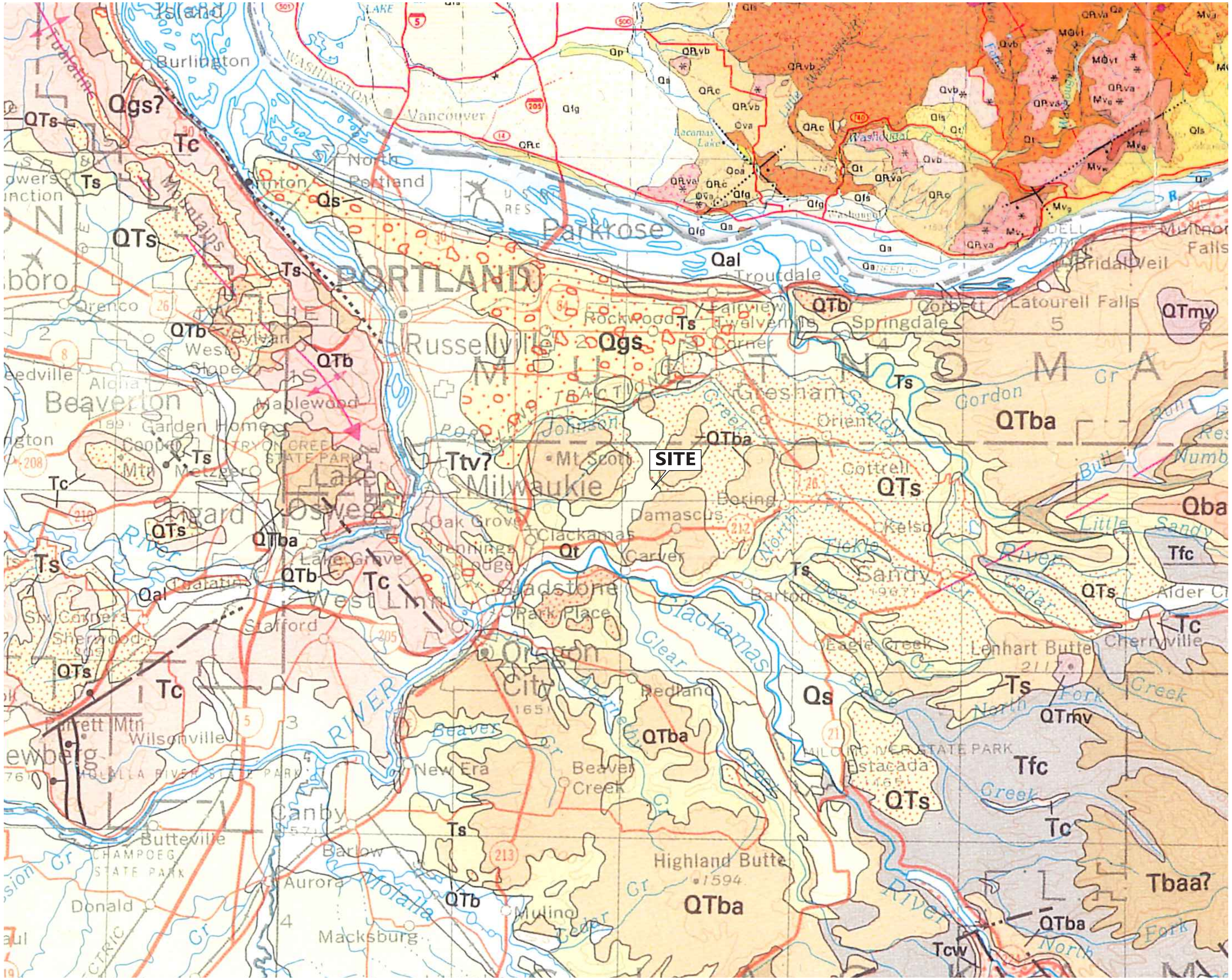


CASCADIA SUBDUCTION ZONE SETTING, TSUNAMI INUNDATION MAPS (OREGON DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRY, 2013)

GRI

TECTONIC SETTING SUMMARY

OCT. 2025 JOB NO. 7072-A FIG. 1E



—— **Contact** — Approximately located

—•—?— • • • **Fault** — Dashed where inferred; dotted where concealed; queried where doubtful; ball and bar on downthrown side

—▲—?▲ ▲ **Thrust fault** — Dashed where inferred; dotted where concealed; queried where doubtful; sawteeth on upper plate

⊥7 Strike and dip of bed

FROM:

WALSH, T. J., KOROSEC, M. A., PHILLIPS, W. M., LOGAN, R. L., AND SCHASSE, H. W., 1987, GEOLOGIC MAP OF WASHINGTON-SOUTHWEST QUADRANT; 1:250,000: WASHINGTON DIVISION OF GEOLOGY AND EARTH RESOURCES, 6M-34

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N

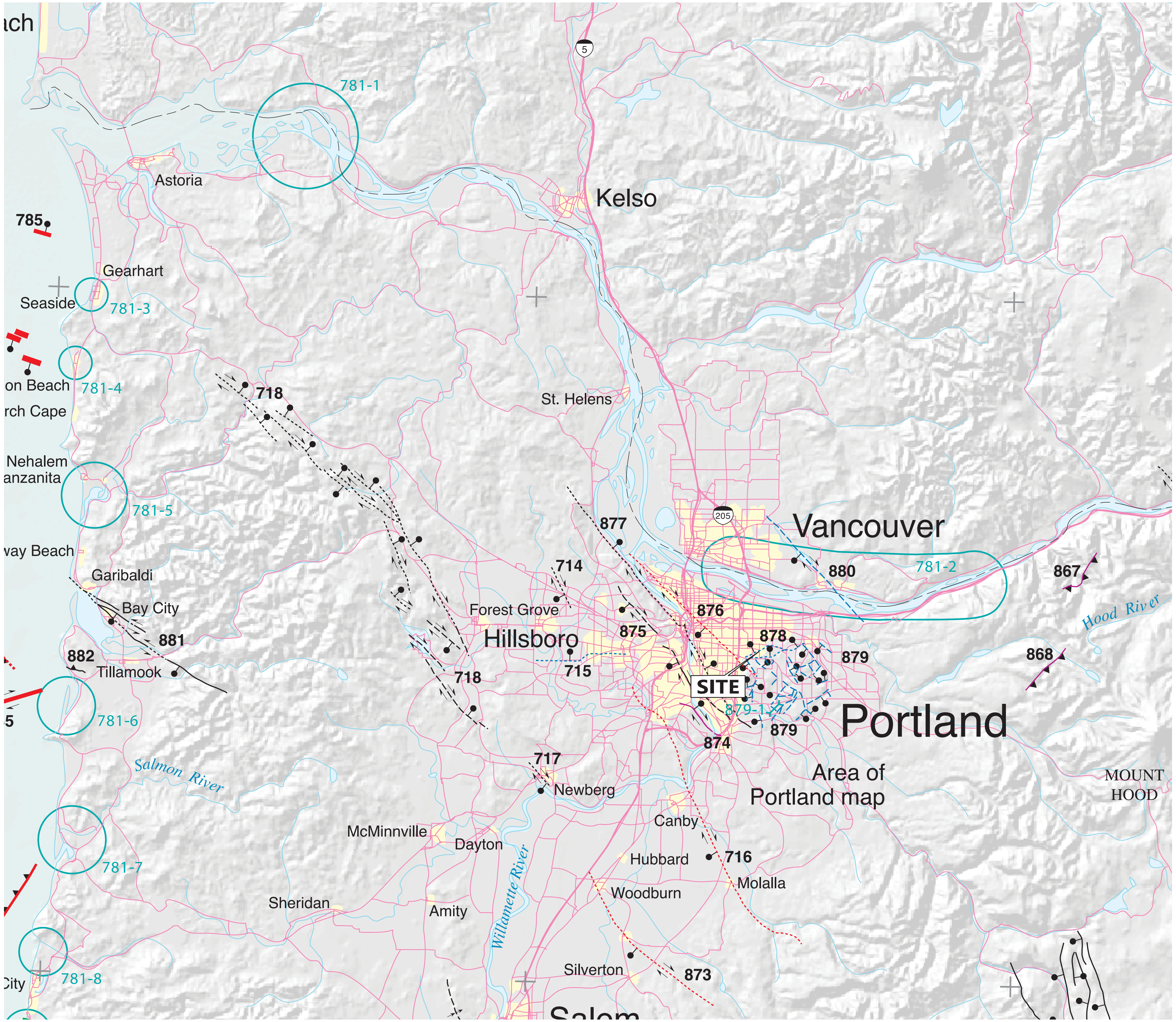
0 5 10 MILES

0 5 10 KILOMETERS

GRI

REGIONAL GEOLOGIC MAP

OCT. 2025 JOB NO. 7072-A FIG. 2E



TIME OF MOST RECENT SURFACE RUPTURE

- Holocene (<10,000 years) or post last glaciation (<15,000 years; 15 ka); no historic ruptures in Oregon to date
- Late Quaternary (<130,000; post penultimate glaciation)
- Late and middle Quaternary (<750,000 years; 750 ka)
- Quaternary, undifferentiated (<1,600,000 years; <1.6 Ma)
- Class B structure (age or origin uncertain)

SLIP RATE

- >5 mm/year
- 1.0-5.0 mm/year
- 0.2-1.0 mm/year
- <0.2 mm/year

TRACE

- Mostly continuous at map scale
- Mostly discontinuous at map scale
- Inferred or concealed

STRUCTURE TYPE AND RELATED FEATU

- Normal or high-angle reverse fault
- Strike-slip fault
- Thrust fault
- Anticlinal fold
- Synclinal fold
- Monoclinal fold
- Plunge direction of fold
- Fault section marker

DETAILED STUDY SITES

- 731-2 Trench site
- 781-2 Subduction zone study site

CULTURAL AND GEOGRAPHIC FEATURES

- Divided highway
- Primary or secondary road
- Permanent river or stream
- Intermittent river or stream
- Permanent or intermittent lake

FAULT NUMBER	NAME OF STRUCTURE
714	HELVETIA FAULT
715	BEAVERTON FAULT
716	CANBY-MOLALLA FAULT
717	NEWBERG FAULT
718	GALES CREEK FAULT ZONE
719	SALEM-EOLA HILLS HOMOCLINE
864	CLACKAMAS RIVER FAULT ZONE
867	EAGLE CREEK THRUST FAULT
868	BULL RUN THRUST FAULT
872	WALDO HILLS FAULT
873	MOUNT ANGEL FAULT
874	BOLTON FAULT
875	OATFIELD FAULT
876	EAST BANK FAULT
877	PORTLAND HILLS FAULT
878	GRANT BUTTE FAULT
879	DAMASCUS-TICKLE CREEK FAULT ZONE
880	LACAMAS LAKE FAULT
881	TILLAMOOK BAY FAULT ZONE

NOTE: NOT ALL QUATERNARY FAULTS ARE SHOWN.

FROM: PERSONIUS, S. F., ET AL., 2003, MAP OF QUATERNARY FAULTS AND FOLDS IN OREGON, USGS OPEN FILE REPORT OFR-03-095.

N

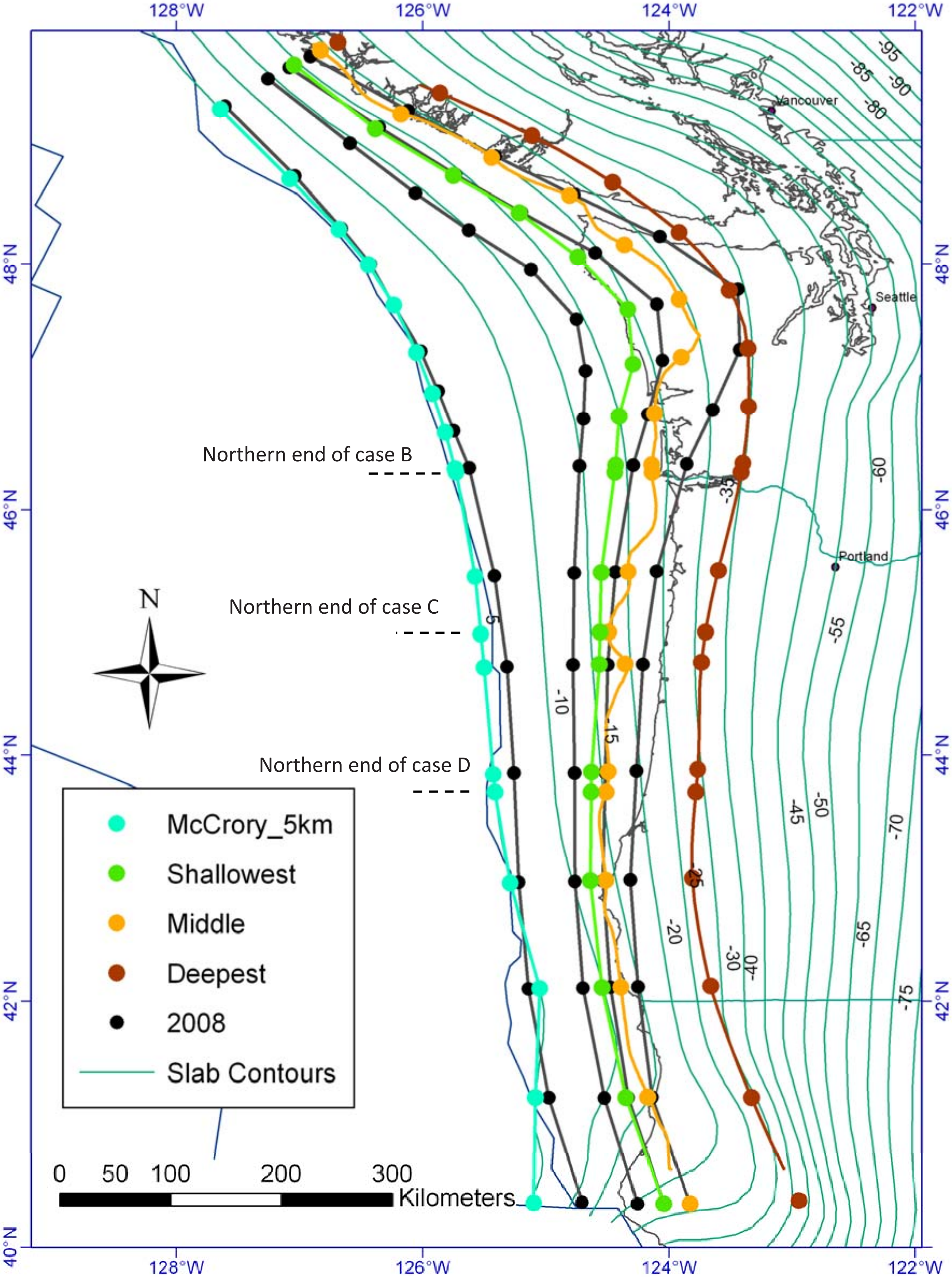
0 10 20 MILES

0 20 40 KILOMETERS

GRI

LOCAL FAULT MAP

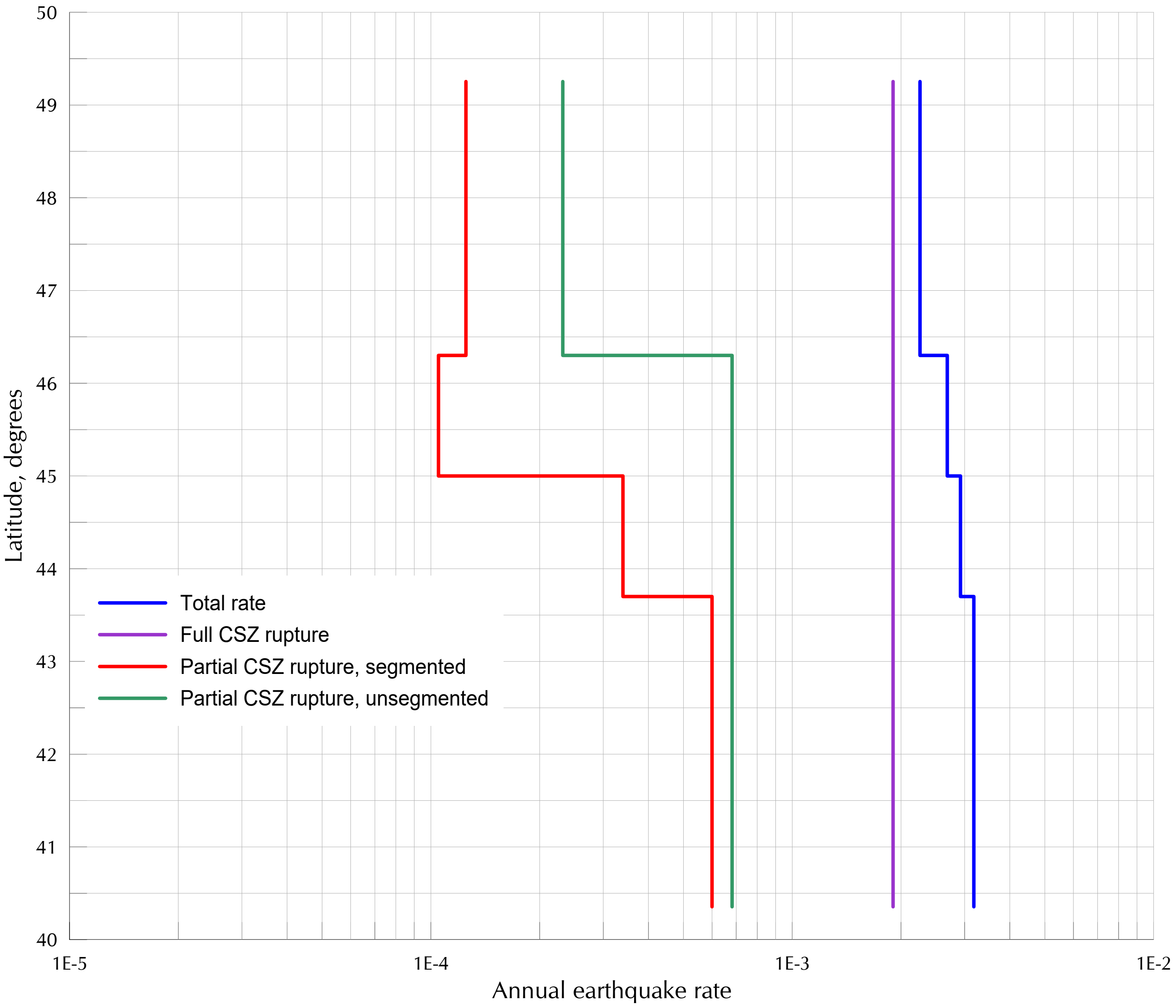
OCT. 2025 JOB NO. 7072-A FIG. 3E



GRI

LOCATION OF SURFACE TRACES FOR
UP-DIP EDGE & THREE DOWN-DIP EDGE
OPTIONS USED IN 2014 NSHMS
(CHEN ET AL., 2014)

OCT. 2025 JOB NO. 7072-A FIG. 4E

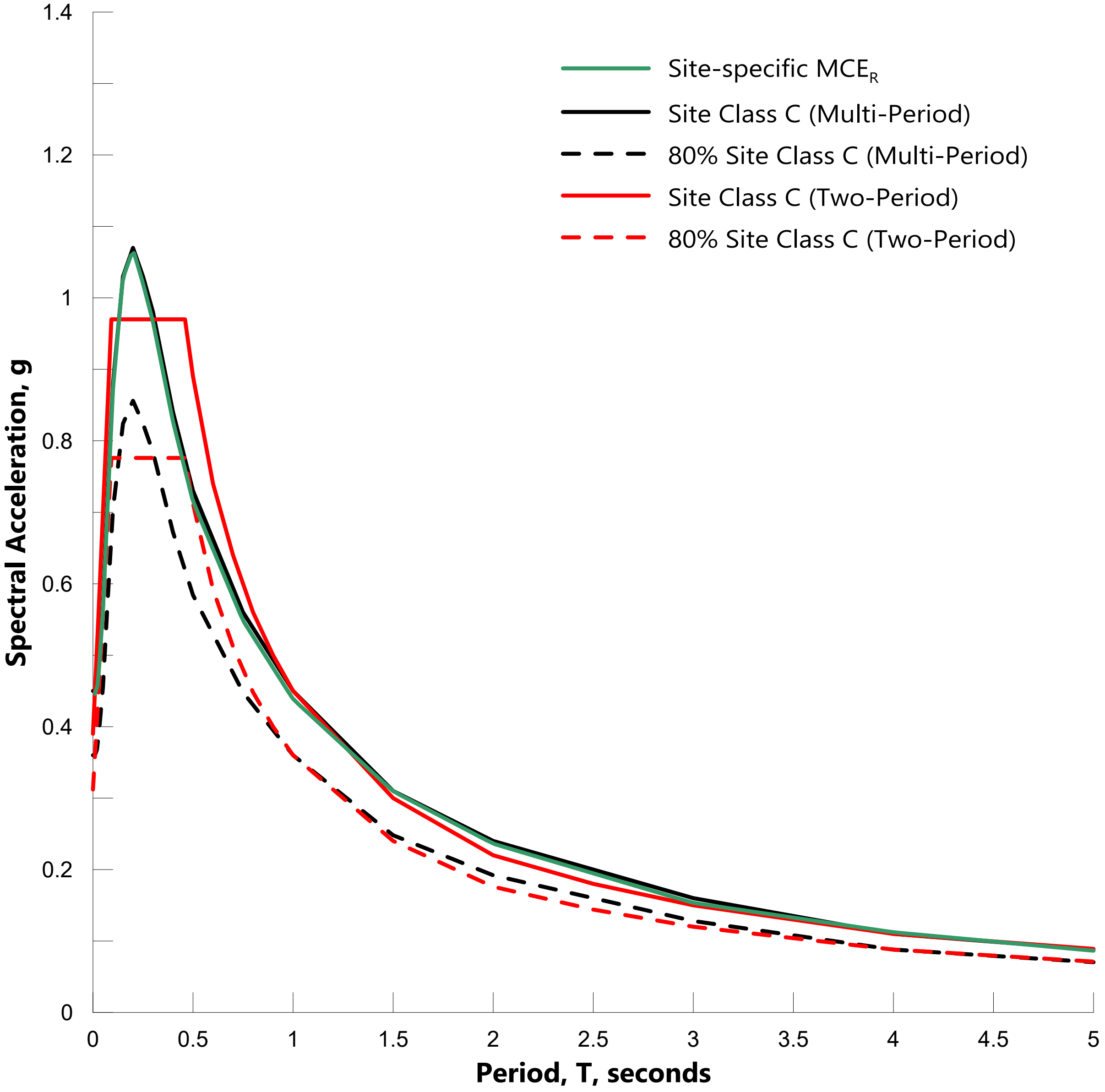


REFERENCE:
PETERSEN, M. D., MOSCHETTI, M. P., POWERS, P. M., MUELLER, C. S., HALLER, K. M., FRANKEL, A. D., ZENG, Y., REZAEIAN, S., HARMSEN, S. C., BOYD, O. S., FIELD, N., CHEN, R., RUKSTALES, K. S., NICO, L., WHEELER, R. L., WILLIAMS, R. A., AND OLSEN, A. H., 2014, DOCUMENTATION FOR THE 2014 UPDATE OF THE UNITED STATES NATIONAL SEISMIC HAZARD MAPS: U.S. GEOLOGICAL SURVEY OPEN-FILE REPORT 2014–1091, 243 PP.

GRI

VARIATION OF EARTHQUAKE RATES
CASCADIA SUBDUCTION ZONE

OCT. 2025 JOB NO. 7072-A FIG. 5E



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MCE_R RESPONSE SPECTRA COMPARISON

(5% DAMPING)

OCT. 2025 JOB NO. 7072-A FIG. 6E

APPENDIX F

Traffic Analysis and Pavement Design Calculation Worksheets

APPENDIX F

TRAFFIC ANALYSIS AND PAVEMENT DESIGN CALCULATION WORKSHEETS

F.1 GENERAL

We used the methodology presented in the Oregon Department of Transportation *Pavement Design Guide* (ODOT Guide) to approximate cumulative 18-kip Equivalent Single Axle Load (ESAL) repetitions (traffic loading) over 25-year design periods for flexible pavement designs. We based our traffic loading approximations on vehicle class and frequency information provided by the design team. We used these data to forecast traffic over a 25-year period, assuming construction will occur in the year 2025. Additional details of our analysis methodology and our approximations for future traffic loading are presented below.

F.2 24-HOUR TRAFFIC VOLUME ESTIMATE

Based on the information provided to us by the design team, it our understanding that the parking lot will see an estimated 2,730 vehicles per average weekday. In terms of heavy truck activity, the anticipated traffic will consist of two garbage trucks per week, two recycling trucks per week, two delivery trucks per day, and one to two school buses per week during the school year. As requested by the City, we did not design the parking lot pavement to accommodate construction traffic. Construction traffic should be limited to haul roads. If construction traffic is allowed to operate on the new pavement, the design-life of the pavement could be reduced and it may be necessary to repair some of the pavement that becomes damaged.

F.3 ANNUAL EQUIVALENT SINGLE AXLE LOAD REPETITIONS

We used the ODOT ESAL conversion factors for standard vehicles and calculated Load Equivalency Factor based on the 1993 American Association of State Highway and Transportation Officials *AASHTO Guide for Design of Pavement Structures* (AASHTO Guide) for a fire apparatus to estimate the annual ESAL repetitions.

F.4 TRAFFIC GROWTH

We did not assume any growth rate for the traffic.

F.5 DESIGN EQUIVALENT SINGLE AXLE LOAD

Our approximation of cumulative ESAL repetitions (traffic loading) for the 25-year design period for flexible pavements is summarized in Table 1F in this appendix.

F.6 FLEXIBLE PAVEMENT DESIGN

We used the guidance and methodologies presented in the 2019 ODOT Guide, 2025 City of Happy Valley Engineering Design and Standards Details Manual, and the 1993 AASHTO

Guide to develop pavement designs for new construction. Design parameters used in the analyses, along with pavement design worksheets, are shown in Tables 2F and 3F in this appendix.

F.7 REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), 1993, AASHTO guide for design of pavement structures: Washington, D.C.

City of Happy Valley, 2025, Engineering design and standards details mannual: Engineering Division, Happy Valley, Oregon.

Oregon Department of Transportation (ODOT), 2019, ODOT pavement design guide: Pavement Services Unit, Salem, Oregon.

Table 1F: TRAFFIC ANALYSIS WORKSHEET
CITY OF HAPPY VALLEY COMMUNITY RECREATION CENTER PARKING LOT

Project: **City of Happy Valley Community Recreation Center**
Effective Date: **6/1/2025**
Source of Traffic Volume Data: **Based on the information provided by the design team**
One-Way or Two-Way Volumes? **One-Way**
Year of Traffic Volume Count Data: **2025**
Project Construction Year: **2025**
Years between Count & Construction: **0**
Directional Factor: **1.00**
Lane Distribution Factor: **1.00**
Annual Compound Growth Rate: **0.00%**
Pavement Type: **Flexible**
Agency for ESAL Conversion Factors: **Oregon Department of Transportation**

	FHWA Vehicle Class and Corresponding Oregon Department of Transportation Flexible Pavement Two-Way ESAL Conversion Factor														Total 24-hour Volume	Percent Trucks	Annual ESALs During Count Year	Annual ESALs During Construction Year
FHWA Vehicle Class	1	2	3	4T	4	5	6	7	8	9	10	11	12	13				
Two-Way ESAL Conversion Factors	0	0	0	0	246	104	284	757	253	466	561	603	546	1037				
Vehicle Classification Description	**Motor-cycles**	**Cars**	**Light Pickups**	**Transit Buses (single)**	**Other Buses (single)**	**2-axle, 6-tire (single)**	**3-axle (single)**	**4-axle (single)**	**<5-axle (double)**	**5-axle (double)**	**>6-axle (double)**	**<6-axle (multi)**	**6-axle (multi)**	**>6-axle (multi)**				
	Annual Average Daily Traffic Volume																	
Count Year Data	0	2,727	0	0	0.3	2	1	0	0	0	0	0	0	0	2,730	0.1%	671	671

Annual and Cumulative ESALs

Year	Annual ESALs	Cumulative ESALs	Year	Annual ESALs	Cumulative ESALs	Year	Annual ESALs	Cumulative ESALs	Year	Annual ESALs	Cumulative ESALs	Year	Annual ESALs	Cumulative ESALs
2026 (1)	671	671	2036 (11)	671	7,379	2046 (21)	671	14,088	2056 (31)	671	20,796	2066 (41)	671	27,504
2027 (2)	671	1,342	2037 (12)	671	8,050	2047 (22)	671	14,758	2057 (32)	671	21,467	2067 (42)	671	28,175
2028 (3)	671	2,013	2038 (13)	671	8,721	2048 (23)	671	15,429	2058 (33)	671	22,138	2068 (43)	671	28,846
2029 (4)	671	2,683	2039 (14)	671	9,392	2049 (24)	671	16,100	2059 (34)	671	22,809	2069 (44)	671	29,517
2030 (5)	671	3,354	2040 (15)	671	10,063	2050 (25)	671	16,771	2060 (35)	671	23,479	2070 (45)	671	30,188
2031 (6)	671	4,025	2041 (16)	671	10,733	2051 (26)	671	17,442	2061 (36)	671	24,150	2071 (46)	671	30,859
2032 (7)	671	4,696	2042 (17)	671	11,404	2052 (27)	671	18,113	2062 (37)	671	24,821	2072 (47)	671	31,529
2033 (8)	671	5,367	2043 (18)	671	12,075	2053 (28)	671	18,784	2063 (38)	671	25,492	2073 (48)	671	32,200
2034 (9)	671	6,038	2044 (19)	671	12,746	2054 (29)	671	19,454	2064 (39)	671	26,163	2074 (49)	671	32,871
2035 (10)	671	6,708	2045 (20)	671	13,417	2055 (30)	671	20,125	2065 (40)	671	26,834	2075 (50)	671	33,542

Design (Cumulative) ESALs (Rounded up to the next 1,000 ESALs)

1-Year ESALs	2-Year ESALs	5-Year ESALs	8-Year ESALs	10-Year ESALs	15-Year ESALs	20-Year ESALs	25-Year ESALs	40-Year ESALs	50-Year ESALs
1,000	**2,000**	**4,000**	**6,000**	**7,000**	**11,000**	**14,000**	**17,000**	**27,000**	**34,000**

Abbreviations: FHWA = Federal Highway Administration; ESAL = Equivalent Single Axle Load (ESALs is plural); 4T = Class 4 Transit Buses

Table 2F: ASPHALT CONCRETE PAVEMENT DESIGN WORKSHEET

CITY OF HAPPY VALLEY COMMUNITY RECREATION CENTER PARKING LOT: AGGREGATE REINFORCEMENT WITH GEOTEXTILE FOR 25-YEAR DESIGN PERIOD

AASHTO Design Parameters & Input Values		Notes
Functional Classification	Private	per information provided by the design team
Design Period, years	25	per City of Happy Valley Engineering Design Manual (City EDM)
Cumulative Equivalent Single Axle Load (ESAL) Repetitions	17,000	see Table 1F
Design Reliability, %	75	per Oregon Department of Transportation (ODOT) Pavement Design Guide (PDG)
Overall Standard Deviation, S_o	0.49	per ODOT PDG
Initial Serviceability, p_o	4.2	per ODOT PDG
Terminal Serviceability, p_t	2.5	per ODOT PDG
Effective Subgrade Resilient Modulus (M_R), pounds per square inch (psi)	4,000	approximated based on Dynamic Cone Penetration (DCP) testing
New Aggregate Base (AB) Course Modulus, psi	20,000	per ODOT PDG
New Asphalt Concrete (AC) Layer Coefficient	0.42	per ODOT PDG
New AB Layer Coefficient	0.10	per ODOT PDG
New Aggregate Subbase (ASB) Layer Coefficient	0.08	per ODOT PDG
New AB Drainage Coefficient	1.00	per ODOT PDG
New ASB Drainage Coefficient	0.80	per ODOT PDG
Minimum AB thickness on geotextile for support of construction with 1.5-inch allowable rut depth, inches	12.0	per Giroud & Han procedure on CBR 2.7 subgrade with CBR 80 AB on geotextile
Minimum AB thickness, inches	10.0	per City EDM
Minimum AC thickness, inches	3.0	per City EDM
Structural Number (SN) required above AB	1.02	
SN required above ASB	1.43	
SN required above subgrade	2.05	

Pavement Section

Layer Description	Thickness, inches	Layer Coefficient	SN	SN Subtotals	Notes
Level 2, ½-inch Dense Asphalt Concrete Pavement, PG 64-22	3.00	0.42	1.26	1.26	>=1.02 required above aggregate base - OK
¾-inch-0 Crushed Aggregate Leveling Course	2.00	0.10	0.20	1.46	>=1.43 required above aggregate subbase - OK
1½-inch-0 Crushed Aggregate Base Rock Course	8.00	0.08	0.64	2.10	>=2.05 required above subgrade - OK
Geotextile	NA				
Total Depth	13.00				

Abbreviations: PG = Perfomance Grade**;** AASHTO = American Association of State Highway and Transportation Officials**;** NA = not applicable; CBR = California bearing ratio

Notes:

Denotes user defined value

Denotes calculated value

References:

Giroud, J. P. and Han, J., 2004, Design method for geogrid-reinforced unpaved roads. I. Development of design method, *Journal of Geotechnical and Geoenvironmental Engineering,* vol 130, iss. 8, pp. 775-786.

Giroud, J. P. and Han, J., 2004, Design method for geogrid-reinforced unpaved roads. II. Development of design method, Journal of Geotechnical and Geoenvironmental Engineering, vol 130, iss. 8, pp. 787-797.

Table 3F: ASPHALT CONCRETE PAVEMENT DESIGN WORKSHEET

CITY OF HAPPY VALLEY COMMUNITY RECREATION CENTER PARKING LOT: SOIL STABILIZATION WITH CEMENT FOR 25-YEAR DESIGN PERIOD

AASHTO Design Parameters & Input Values		Notes
Functional Classification	Private	per information provided by the design team
Design Period, years	25	per City of Happy Valley Engineering Design Manual (City EDM)
Cumulative Equivalent Single Axle Load (ESAL) Repetitions	17,000	see Table 1F
Design Reliability, %	75	per Oregon Department of Transportation (ODOT) Pavement Design Guide (PDG)
Overall Standard Deviation, S_o	0.49	per ODOT PDG
Initial Serviceability, p_o	4.2	per ODOT PDG
Terminal Serviceability, p_t	2.5	per ODOT PDG
Effective Subgrade Resilient Modulus (M_R), pounds per square inch (psi)	4,000	approximated based on Dynamic Cone Penetration (DCP) testing
New Aggregate Base (AB) Modulus, psi	20,000	per ODOT PDG
New Asphalt Concrete (AC) Layer Coefficient	0.42	per ODOT PDG
New AB Layer Coefficient	0.10	per ODOT PDG
Cement-Stabilized Soil (CSS) Layer Coefficient	0.14	per AASHTO Design Guide
New AB Drainage Coefficient	1.00	per ODOT PDG
CSS Compressive Strength, psi	300	
CSS Resilient Modulus, psi	360,000	per Thompson Equation
Structural Number (SN) required above AB	1.02	
SN required above CSS	0.00	
SN required above subgrade	2.05	

Pavement Section

Layer Description	Thickness, inches	Layer Coefficient	SN	SN Subtotals	Notes
Level 2, ½-inch Dense Asphalt Concrete Pavement, PG 64-22	3.00	0.42	1.26	1.26	>=1.02 required above aggregate base - OK
¾-inch-0 Crushed Aggregate Leveling Course	0.00	0.10	0.00	1.26	>=0.00 required above cement-stabilized base - OK
Cement-Stabilized Soil	12.00	0.14	1.68	2.94	>=2.05 required above subgrade - OK
Total Depth	15.00				

Abbreviations: PG = Perfomance Grade**;** AASHTO = American Association of State Highway and Transportation Officials**;** CBR = California bearing ratio

Notes: Denotes user defined value

Denotes calculated value

Reference: Thomson, M.R., July 1986, Mechanistic design concept for stabilized base pavements, Civil Engineering Studies, Transportation Engineering Series No. 46, Illinois Cooperative Highway and Transportation Series No. 214, University of Illinois, Urbana, Illinois.

APPENDIX G

Geoprofessional Business Association Guidance Document

Important Information about This

Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are <u>Not</u> Covered

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

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

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