Geotechnical Investigation and Site-Specific Seismic-Hazard Evaluation

Oak Lodge Community Project

3811 SE Concord Road

Milwaukie, Oregon

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Prepared for

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As requested, GRI completed a geotechnical investigation for the North Clackamas Parks and Recreation District (NCPRD) Oak Lodge Community Project located in Milwaukie, Oregon. The Vicinity Map, Figure 1, shows the general location of the site. The purpose of our investigation was to evaluate subsurface conditions at the site and develop geotechnical recommendations for use in the design and construction of the proposed improvements to the site. The investigation included a review of existing geotechnical information for the site and surrounding area, subsurface explorations, laboratory testing, and engineering analyses. This report describes the work accomplished and provides preliminary conclusions and recommendations for use in the design and construction of the proposed project.

1 PROJECT DESCRIPTION

We understand the project will consist of a renovation and new addition to the existing NCPRD building to construct a new community center for Clackamas County. A new library on the property is also being considered with the project. We understand Opsis Architecture, LLP (Opsis) is the architect for the project and Catena Consulting Engineers, Inc. (Catena), is the structural engineer for the project. We anticipate the project will be constructed in accordance with the 2018 International Building Code (IBC) with modifications by the 2019 Oregon Structural Specialty Code (OSSC). The structural loads for the buildings are unknown at this time. However, we anticipate the new structures will be supported on conventional column- and wall-type spread footings. Based on our understanding of the project, we anticipate the maximum column and wall loads will be on the order of 100 kips to 200 kips and 3 kips/foot to 4 kips/foot, respectively.

We anticipate minor cuts and fills will be necessary to reach final site grades for the associated site improvements. We understand the parking lots and drive lanes will be reconfigured and paved with asphalt concrete (AC) pavement. The vehicle traffic loading is unknown at this time.

2 SITE DESCRIPTION

2.1 General

The Site Plan, Figure 2, shows the existing site conditions and proposed improvement areas for the project in Milwaukie, Oregon. The proposed community center site is currently occupied by NCPRD located at 3811 SE Concord Road in unincorporated Oak Grove, Oregon, and is bordered by commercial properties to the west, SE Concord Road to the south, and residential housing to the north and east. The site is partially bisected by SE Spaulding Avenue. Based on our observations on site, an existing building occupies the southern portion of the site, grass fields occupy the northern portion, and play areas occupy the western portion. Mature trees surround the building and site perimeter, and an approximately 4- to 6-foot-high retaining wall surrounds the north and east sides of the existing building. Portland cement concrete (PCC) sidewalks and AC pavement roadways and parking are located along the south, central, and west portions of the property.

2.2 Geology

The project site is located in the northern Willamette Valley in the southwestern portion of the Portland Basin, a northwest-trending structural depression encompassing approximately 770 square miles of northern Oregon and southern Washington (Evarts et al., 2009). The north-flowing Willamette River is located approximately 1 mile west-southwest of the project site. Published geologic mapping indicates the site is mantled by Pleistocene-age Missoula flood deposits (which include silt, clay, sand, and gravel), with the Basalt of Sand Hollow member of the Wanapum Basalt (Columbia River Basalt Group of Miocene age) mapped at the ground surface adjacent to the eastern portion of the site (Wells et al., 2018).

No mapped or historic landslides were identified within the limits of the project site or in the general vicinity on the Oregon Department of Geology and Mineral Industries (DOGAMI) statewide landslide hazard database (SLIDO Version 4.0). DOGAMI is the state agency responsible for geologic hazard mapping in Oregon. Regional landslide susceptibility at the site is mapped as low to moderate, while susceptibility to shallow (<15 feet below ground surface) landslides is mapped as low to high (Franczyk et al., 2019). In general, areas of greater landslide susceptibility generally correspond to areas of greater relief.

2.3 Oatfield Fault

The Portland Hills fault zone includes the Oatfield Fault, as well as the Portland Hills Fault, and is interpreted as potentially seismogenic (capable of generating earthquakes) (Wong et al., 2001). The Oatfield Fault strikes northwest, has an approximately 70° northeast dip, and is estimated to be on the order of 40 kilometers in length (Horst, 2019). The inferred trace of the Oatfield Fault crosses the southwestern portion of the site, see Figure 2. Southeast of the site, the fault location is depicted as moderately well constrained, whereas within and adjacent to the project site, the fault location is shown as “inferred,” meaning the fault location is approximately located (USGS, 2020). The U.S. Geological Survey (USGS) Quaternary Fold and Fault Database characterizes the Oatfield Fault’s age of most recent activity as undifferentiated Quaternary, which indicates evidence of activity within the last 1.6 million years exists, but timing of fault activity such as recurrence and the most recent earthquake event is unconstrained (Personius et al., 2003). Fault motion is interpreted to vary over the total length of the fault as normal, reverse, and right-lateral strike-slip (Wells et al., 2018; and Personius et al., 2003). The Oatfield Fault is believed to be capable of generating up to moment magnitude (M_W) 7.0 (reverse) to M_W 7.1 (strike-slip) earthquakes (Horst, 2019).

In addition to the poorly constrained timing of fault activity, uncertainty exists about the precise location of the Oatfield Fault. Heavy vegetation, urbanization, and thick alluvial sediments mantling the ground surface associated with the Missoula floods (12,000 years to 15,000 years ago) obscure potential surface exposures of potentially active faults in the Portland area (Wong et al., 2001).

3 SUBSURFACE CONDITIONS

3.1 General

Subsurface materials and conditions at the site were investigated on August 5 through September 1, 2020, with two seismic-refraction arrays designated S-1 and S-2, one cone penetration test (CPT) probe designated CPT-1, three test-pit excavations designated TP-1 through TP-3, and three drilled borings designated B-1 through B-3. The drilled borings were advanced to depths of about 31.5 feet to 71.5 feet. The CPT probe was advanced to practical refusal at a depth of about 28.5 feet. The test pits were excavated to the practical limits of the equipment to depths of about 16 feet to 19 feet using a Case 580 backhoe. Logs of the borings are provided on Figures 1A through 3A, logs of the CPT probe are provided on Figures 4A and 5A, and logs of the test pits are provided on Figures 6A through 8A. The field- and laboratory-testing programs conducted to evaluate the physical engineering properties of the materials encountered in the explorations are described in Appendix A. The terms and symbols used to describe the soil and rock encountered in the explorations are defined in Tables 1A and 2A and on the attached legend. A seismic-refraction geophysical exploration survey and analysis was completed at the site to assist in evaluating the potential presence of subsurface geologic structures, the results of which are provided in Appendix C.

3.2 Sampling

Disturbed soil samples were obtained from the test-pit excavations at generally 2-foot intervals within the upper 4 feet of the excavation. Disturbed and undisturbed soil samples were obtained from the drilled borings at generally 2.5-foot intervals of depth in the upper 15 feet and 5-foot intervals below 15 feet. Disturbed soil samples were obtained using a 2-inch-outside-diameter standard split-spoon sampler. Standard penetration tests (SPTs) were conducted by driving the sampler into the soil a distance of 18 inch 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, or SPT N-value. SPT N-values provide a measure of the relative density of granular soils and the relative consistency of cohesive soils. 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 soil in the Shelby tubes was extruded in our laboratory and further classified using laboratory testing described in Appendix A.

3.3 Soils

For the purpose of discussion, the soils disclosed by our investigation have been grouped into the following categories based on their physical characteristics and engineering properties:

a. Asphalt Concrete PAVEMENT
b. Sandy, Silty GRAVEL (FILL)
c. SILT, SAND, and CLAY (Alluvium)
d. GRAVEL and COBBLES (Alluvium)
e. BASALT (Columbia River Basalt)

The following paragraphs provide a description of the soil units encountered in the explorations completed by GRI for this investigation and a discussion of the groundwater conditions at the site.

a. Asphalt Concrete PAVEMENT

Borings B-2 and B-3 were advanced in existing paved areas and encountered approximately between 3 inches to 6 inches of AC pavement at the ground surface. The AC pavement is underlain by about 4 inches to 12 inches of crushed-rock base (CRB) course. The thicknesses of AC pavement and CRB encountered during our explorations are tabulated below.

Table 3-1: ASPHALT CONCRETE AND CRUSHED-ROCK BASE SECTIONS

Boring	AC Thickness, in.	CRB Thickness, in.
B-2	3	4
B-3	6	12

b. Sandy, Silty GRAVEL (FILL)

Sandy, silty gravel fill was encountered beneath the pavement in boring B-3 and extends to a depth of about 5 feet. The sand is fine to coarse grained and the gravel is angular. Based on an SPT N-value, we estimate the relative density of the sandy, silty gravel fill is loose.

c. SILT, SAND, and CLAY (Alluvium)

Silt, sand, and clay interpreted to be alluvium were encountered at the ground surface in boring B-1 and test pits TP-1 through TP-3, beneath the pavement in boring B-2, and beneath the fill in boring B-3. The silt, sand, and clay extend to depths ranging from about 14 feet to 63 feet. Boring B-2 was terminated in the silt, sand, and clay at a depth of about

31.5 feet, and test-pit excavation TP-1 was terminated in the silt, sand, and clay at a depth of about 16 feet.

The silt ranges in color from light brown to orange-brown or dark brown, rust, or gray and may contain rust, dark brown, light gray, or gray mottling. The silt contains a trace of clay to clayey and a trace of sand to sandy. A 3- to 4-inch-thick heavily rooted zone was observed at the ground surface in boring B-1 and test pits TP-1 through TP-3. Based on SPT N-values and Torvane shear-strength values, the relative consistency of the silt ranges from very soft to stiff. The natural moisture content of the silt ranges from 6% to 59%. Atterberg-limits testing completed on samples of the silt are provided on Figure 9A and indicate the silt has a low to high plasticity. One-dimensional consolidation testing was completed on select samples of silt obtained from boring B-1 at depths of about 2.8 feet and 8 feet. Test results indicate the silt soil is heavily overconsolidated and has a relatively low compressibility in the preconsolidated range of pressures and a moderate to high compressibility in the normally consolidated ranges of pressures, see Figures 10A and 11A.

The sand ranges in color from light brown to brown and may be mottled rust. The sand is silty and may contain a trace of subrounded gravel. Based on SPT N-values, the relative density of the sand ranges from loose to medium dense. The natural moisture content of the sand ranges from 24% to 46%.

The clay ranges in color from light brown to orange-brown or dark brown, blue-gray or gray and may contain light gray to dark gray mottling. The clay contains a trace of silt to silty and a trace of sand to sandy. The clay may contain gravel or cobbles. We estimate the relative consistency of the clay is stiff. The natural moisture content of the clay ranges from 29% to 47%. Atterberg-limits testing completed on a sample of the clay are provided on Figure 9A and indicate the clay has medium plasticity.

d. GRAVEL and COBBLES (Alluvium)

An interbedded layer of sandy, silty gravel was encountered in boring B-3 between depths of about 7.5 feet and 10 feet, and sandy gravel to gravelly sand was encountered in test pit TP-2 at a depth of about 14 feet. Gravel and cobbles were encountered in test pit TP-2 at a depth of about 16 feet. The gravel is sandy with fine- to coarse-grained sand and contains a trace of silt to silty. The gravel is subrounded to subangular. The gravel and cobbles contain some sand and a trace of silt. Based on an SPT N-value, the relative density of the gravel is medium dense. It should be noted that the relative density of very coarse, granular material such as gravel tends to be overestimated using the standard split-spoon sampler.

e. BASALT (Columbia River Basalt)

Basalt interpreted to be Columbia River Basalt was encountered at depths of about 63 feet in boring B-1, 17 feet in boring B-3, and 16 feet in test pit TP-3. Borings B-1 and B-3 were terminated in the basalt at depths of about 71.5 feet and 31.5 feet, respectively. Test pit TP-3 was terminated in the basalt at the maximum depth of the excavation equipment at a depth of about 19 feet. The basalt is typically brown to gray and may be mottled brown and black or rust, contains some vesicles, and is predominantly decomposed to decomposed. The weathering of the basalt may decrease with depth. The relative hardness of the basalt ranges from extremely soft (R0) to soft (R2). Atterberg-limits testing completed on a decomposed sample of basalt is provided on Figure 9A and indicate the decomposed basalt has a medium to high plasticity.

3.4 Groundwater

The borings were advanced using mud-rotary drilling techniques, which do not allow for the direct measurement of groundwater levels. Moist to wet soils were encountered within test-pit excavations TP-1 and TP-2 at depths of about 12 feet and 11 feet, respectively. Groundwater was not encountered in test pit TP-3 within the depths of excavation. Based on nearby well logs and published USGS groundwater studies in the vicinity of the project area, the estimated depth to groundwater is about 30 feet below the ground surface at the project site (USGS, 2020). Perched-groundwater conditions may develop in the near-surface soil and approach the ground surface during intense or prolonged precipitation.

3.5 Geologic Structure Evaluation

Based on the proposed public use of new improvements at the subject property, a field investigation was conducted to evaluate possible evidence of subsurface structures consistent with a Quaternary fault. This included geophysical explorations followed by test pits, which were sited based on existing published mapping and results of the geophysical survey.

Earth Dynamics LLC of Portland, Oregon, conducted a geophysical exploration at the project site on August 5, 2020, consisting of two seismic-refraction profiles in the southwestern portion of the project site. The goal of this survey was to help identify the top of basalt bedrock, which is interpreted to underly more recent surficial sedimentary units. Seismic-refraction surveys are performed using geophones arranged in an array with a minimum length of 115 feet and an energy source to provide a small seismic wave. The refraction of the seismic waves on geologic layers and rock/soil units can be used to characterize the subsurface geologic conditions and assist with providing the potential locations of subsurface geologic structures. The profiles were oriented perpendicular to the inferred trace of the Oatfield Fault per USGS 2020.

The geophysical survey indicated the presence of a distinct vertical offset at depth in material interpreted as basalt, which, in our opinion, correlates with the approximate mapped trace of the Oatfield Fault. Details of the seismic-refraction survey, including specific methodology and results, are included in Earth Dynamics LLC's "Report on Geophysical Exploration, 3811 SE Concord Rd., Milwaukie, OR," provided in Appendix C.

Based on the results of the geophysical survey, GRI identified three locations to excavate test pits to visually evaluate continuously exposed geologic units and soils, which can help identify geologic structures. GRI submitted a request for utility locates to the Utility Notification Center and subsequently used a private utility locator to further identify potential utility conflicts with the proposed test pit locations. The locations of the test pits are shown on the Site Plan, Figure 2.

On August 25, 2020, a representative of Dan Fischer Excavating, Inc., of Forest Grove, Oregon, excavated three test pits (TP-1 through TP-3) under the direct supervision of GRI. A log of the materials and conditions encountered was generated and selected samples were collected for subsequent laboratory testing.

Test pit TP-2 was located in a strip of grass-covered level ground between a playground and paved area. This location spans an area interpreted from the results of the geophysical survey to be underlain by vertically offset basalt. An initial test pit approximately 35 feet long and up to 5 feet deep was uncovered. Representatives from GRI subsequently used hand scrapers and brushes to clean the northern pit wall of smeared or loose debris and closely examined the exposure for indications of fault-related features such as offset geologic units, contacts, or laminations (bedding); tectonically disturbed or deformed clay layers; clay gouge, soil- or clay-filled fractures or fissures; or slickensides. Stratigraphic offsets of horizontally bedded surficial sedimentary units that would be an indicator of fault rupture were not observed. Test pit TP-2 was terminated at a depth of approximately 17 feet, the limit of the excavation machinery. Basalt bedrock was not encountered. In our opinion, no evidence of faulted Pleistocene-age alluvial deposits was observed. An annotated mosaic of photographs of the northern wall of TP-2 is included as Figure 3. Test-pit details are described in Appendix A.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 General

Subsurface explorations completed for this investigation indicate the site is typically mantled with silt, sand, and clay overlying gravel and cobbles over moderately weathered to decomposed basalt. Fill was also encountered in boring B-3 to a depth of about 5 feet. Moist to wet soils were encountered within the test-pit excavations TP-1 and TP-2 at depths of about 12 feet and 11 feet, respectively. We anticipate perched groundwater can approach the ground surface during periods of heavy or prolonged precipitation.

In our opinion, foundation support for new structural loads can be provided by conventional column- and wall-type spread footings established in firm, undisturbed native soil or compacted structural fill. The primary geotechnical considerations associated with construction of the proposed improvements include the potential presence of fill soils within the footprint of the proposed improvements, the presence of fine-grained soils that are sensitive to moisture content, and the potential for shallow, perched groundwater conditions. The following sections of this report provide our conclusions and recommendations for use in the design and construction of the project.

4.2 Seismic Considerations

4.2.1 General

We understand the project will be designed in accordance with the 2019 Oregon Structural Specialty Code (OSSC), which references American Society of Civil Engineers (ASCE) Document 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16) for seismic design. We understand the proposed improvements at the NCPRD Oak Lodge Community Project will be considered a special-occupancy structure as defined by Oregon Revised Statute 455.447 and will therefore require a site-specific seismic-hazard evaluation. A site-specific seismic-hazard evaluation was completed for the project to fulfill the requirements of amended Section 1803 of the 2019 OSSC for special-occupancy structures. Details of the site-specific seismic-hazard evaluation and the development of the recommended response spectrum are provided in Appendix B.

4.2.2 Mapped Acceleration Parameters

The S_S and S_1 mapped spectral response acceleration parameters for the site located at the approximate latitude and longitude coordinates of 45.4104° N and 122.6243° W are 0.87 g and 0.39 g, respectively, for Site Class B/C, or bedrock conditions.

4.2.3 Site Class

In accordance with Section 20.4 of ASCE 7-16, the site is classified as Site Class D, or a stiff-soil site, based on an average shear-wave velocity (field-measured shear-wave velocity [V_S] to a depth of about 28 feet and estimated V_s below 28 feet) of about 1,060 feet/second (about 325 meters/second) in the upper 100 feet of the soil profile. However, our analysis identified a potential risk of seismically induced settlement at the site. In accordance with Section 20.3.1 of ASCE 7-16, sites with soils vulnerable to failure or collapse under seismic loading should be classified as Site Class F, which requires a site-specific site-response analysis unless the structure has a fundamental period of vibration less than or equal to 0.5 second. The design response spectrum for sites with structures having a fundamental period of less than or equal to 0.5 second can be derived using the non-liquefied subsurface profile and code-tabulated site coefficients. We anticipate the new structure will have a fundamental period of less than 0.5 second; therefore, the code-based Site Class D conditions are appropriate for design of the structure.

4.2.4 *Site Coefficients*

Due to the S_1 acceleration parameter being greater than or equal to 0.2 g, Section 11.4.8 of ASCE 7-16 requires a ground-motion hazard analysis unless the seismic response coefficient C_s is determined in accordance with Exception 2 of Section 11.4.8 of ASCE 7-16. Assuming the seismic response coefficient C_s is determined in accordance with Exception 2 of Section 11.4.8 of ASCE 7-16, the site coefficients F_a and F_v were determined from code-tabulated values to be 1.15 and 1.92, respectively, in accordance with Section 11.4 of ASCE 7-16. The site coefficients F_a and F_v were used to develop the Site Class D, Risk-Targeted Maximum Considered Earthquake (MCE_R)-level spectrum in accordance with Section 11.4 of ASCE 7-16.

4.2.5 *Recommended Seismic Design Parameters*

The design-level response spectrum is calculated as two thirds of the ground-surface MCE_R spectra. The recommended MCE_R- and design-level spectral-response parameters for Site Class D conditions are provided below in Table 4-1.

Table 4-1: RECOMMENDED SEISMIC DESIGN PARAMETERS (2019 OSSC/ASCE 7-16)

Seismic Parameter	Recommended Values*
Site Class	D
MCE_R 0.2-Sec Period Spectral Response Acceleration, S_{MS}	1.00 g
MCE_R 1.0-Sec Period Spectral Response Acceleration, S_{M1}	0.74 g
Design-Level 0.2-Sec Period Spectral Response Acceleration, S_{DS}	0.67 g
Design-Level 1.0-Sec Period Spectral Response Acceleration, S_{D1}	0.49 g

Note: *Exception 2 of Section 11.4.8 should be considered when evaluating base shear calculations in Section 12.8.

4.2.6 *Liquefaction/Cyclic Softening*

The potential for liquefaction and/or cyclic softening at the site was evaluated using the simplified method based on procedures recommended by Idriss and Boulanger (2008) with subsequent revisions (2014). This method utilizes the peak ground acceleration (PGA) to predict the cyclic shear stresses induced by the earthquake. The USGS National Seismic Hazard Mapping Project (NSHMP) was used to determine the contributing earthquake magnitudes that represent the seismic exposure of the site for the Maximum Considered Earthquake Geometric Mean (MCE_G) hazard level. A crustal event on the Portland Hills Fault and an event on the Cascadia Subduction Zone (CSZ) were determined to represent the sources of seismic shaking.

The results of our evaluation indicate there is a potential that the interbedded layers of sand below the groundwater surface at the site could experience limited liquefaction, and zones of the low-plasticity sandy silt below the groundwater surface at the site could experience limited cyclic softening. Our analysis indicates the potential for up to about 1 inch to 2 inches of seismically induced settlement may occur during the earthquake and after earthquake shaking has ceased. Additional details regarding our liquefaction and/or cyclic softening evaluation are provided in Appendix B. Discussion of seismically induced building-foundation settlement is presented in the Foundation Support section later in this report.

4.2.7 Other Seismic Hazards

Based on subsurface conditions and site topography, the risk of earthquake-induced slope instability and/or lateral spreading is low. The risk of damage by tsunami and/or seiche at the site is absent. The USGS considers the Portland Hills Fault, located about 1.2 miles (1.9 kilometers) east of the project site, and the Bolton Fault, located approximately 1.8 miles (3 kilometers) southwest of the site, to be the closest crustal fault sources contributing to the overall seismic hazard at the site. The CSZ is mapped approximately 100 kilometers west of the site (Petersen et al., 2014).

4.2.8 Local Fault Structures

The Oatfield Fault is considered potentially Quaternary active and is mapped by the USGS in the southwestern portion of the project site. Results of geophysical explorations indicate basalt bedrock underlying the project site is vertically offset in the general alignment with the mapped trace of the Oatfield Fault. Clear stratigraphic offset of surficial geologic units in the upper 5 feet of a test pit excavated perpendicular to the inferred area of bedrock offset was not observed.

GRI understands the current proposed development plan includes a parking lot in the southwestern portion of the project site. As a best management practice and consistent with U.S. West Coast jurisdictions with fault management regulations, we recommend no structural development (i.e., buildings intended for public occupancy or use) occur within at least 50 feet of the mapped Oatfield Fault trace. Additional geotechnical investigations may be required for future development scenarios that may encroach on the mapped trace of the Oatfield Fault.

4.3 Earthwork

4.3.1 *General*

The silt and clay soils that mantle the site are moisture sensitive, and perched groundwater may approach the ground surface during the wet winter months and following periods of sustained precipitation. Therefore, it is our opinion earthwork can be completed most economically during the dry summer months, typically extending 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, the moisture content of the soil below this depth 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 prevent disturbance and softening of the subgrade soils. To minimize disturbance of the moisture-sensitive, fine-grained soils, site grading can be completed using track-mounted hydraulic excavators. The excavation 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 excavation to minimize subgrade disturbance. If the subgrade is disturbed during construction, soft, disturbed soils should be overexcavated to firm soil and backfilled with structural fill.

4.3.2 *Site Preparation*

The ground surface within all building areas, paved areas, walkways, and areas to receive structural fill should be stripped of existing vegetation, surface organics, and loose surface soils. We anticipate stripping up to a depth of about 6 inches to 8 inches will likely be required in the grass field areas; however, deeper grubbing may be required to remove brush and tree roots. All trees, brush, and surficial organic material should be removed from within the limits of the proposed improvements. Excavations required to remove existing improvements, brush, and trees should be backfilled with structural fill. Organic strippings should be disposed of off site or stockpiled on site for use in landscaped areas.

Following stripping or excavation to subgrade level, the exposed subgrade should be evaluated by a qualified geotechnical engineer or engineering geologist. Proof rolling with a loaded dump truck may be part of this evaluation. Any soft areas or areas of unsuitable material disclosed by the evaluation should be overexcavated to firm material and backfilled with structural fill. We recommend the contract documents provide unit costs for subgrade overexcavation and structural backfill.

4.3.3 *Granular Work Pads*

If construction occurs during wet-ground conditions, granular work pads will be required to protect the underlying silt 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 prevent disturbance of the subgrade by lighter construction equipment and limited traffic by dump trucks. Haul roads and other high-density traffic areas, including the use of Gradalls and forklifts, will require a minimum of 18 inches to 24 inches of fragmental rock up to 6-inch nominal size to reduce the risk of subgrade deterioration. The use of a geotextile fabric over the subgrade may reduce the need for maintenance during construction. Haul roads can also be constructed by placing a thickened section of pavement base course and subsequently spreading and grading the excess CRB after earthwork is complete.

4.3.4 Prior Site Development

Due to previous development in the project vicinity and the potential to encounter fill soils, it should be anticipated that some overexcavation of subgrade may be required. In addition, site improvements within previously developed areas include a risk of encountering undocumented or poorly documented improvements and infrastructure. Although not encountered within the subsurface explorations completed at the site, the possibility does exist to encounter existing underground improvements.

4.3.5 Site Grading

Final grading across the project should provide for positive drainage of surface water away from exposed slopes to reduce the potential for erosion. Prior to placing pavement base course aggregate, subgrade should be sloped to a minimum-0.5% slope to aid in drainage. Permanent cut and fill slopes should be no steeper than 2H:1V (Horizontal to Vertical) and should be protected with vegetation to reduce the risk of surface erosion due to rainfall. Seeps or springs emerging on cut slopes may require drainage provisions depending on the actual conditions observed during construction. These provisions could include French drains, drainage blankets, and subdrains (possibly placed in utility trenches) to collect and remove water.

4.3.6 Cement Amendment

Cement amendment may be an option to stabilize subgrade soils during periods when the soil cannot be suitably moisture conditioned. Typically, 6% to 8% cement (by dry weight of soil) mixed to a depth of 12 inches to 14 inches below subgrade is sufficient to provide a stable platform for construction. Additional cement may be necessary for drying if the in-situ moisture content of the subgrade is well above its optimum moisture content for compaction. Cement amendment may allow the contractor to extend the construction season to the typically wet winter to spring months; however, installation of the cement amendment should be accomplished during the drier months. The installation timeline may be extended slightly (i.e., into shoulder season) with the understanding of a likely higher failure rate, but installation should not be conducted during the winter or prolonged periods of wet weather. The cement-amended soil should be compacted with a sheepsfoot or segmented-pad roller to achieve compaction of about 95% of the maximum dry density as determined by ASTM International (ASTM) D558. After the cement-amended area is graded, a smooth-drum roller should be used to produce a smooth, compacted surface. All compaction and grading operations should be completed within four hours of soil mixing and tilling with the cement. Final cement-amended grades should be sloped to a minimum-0.5% slope to aid in drainage and avoid water ponding. Cement-amended soils should be cured for a minimum of five days to increase their strength gain prior to being trafficked by any equipment or placement of the granular base course. After curing, the smooth, compacted surface should be evaluated to determine suitability. Proof rolling with a fully loaded dump truck may be part of this evaluation. Soft areas or areas of insufficient cement should be overexcavated and/or retreated. To support construction equipment, the cement-amended subgrade should be capped with an approximately 6- to 12-inch-thick section of relatively clean, crushed rock that has less than 5% passing the No. 200 sieve (washed analysis). If the cement amendment option is selected, we recommend additional testing be considered to define the proper cement content of the soil that will achieve a minimum compressive strength of 100 pounds per square inch (psi).

4.4 Structural Fill

4.4.1 General

We anticipate minor amounts of structural fill may be required to achieve the finished floor elevation for the structure and finished grades for the associated improvements. In general, structural fills should consist of imported or on-site, organic-free soils and should extend a minimum horizontal distance of 5 feet beyond the edge of new foundations and 1 foot beyond the limits of ancillary improvements, such as the edge of new pavements.

4.4.2 On-Site Fill

The use of on-site, fine-grained soils for structural fill material should be limited to the dry summer months, when the moisture content of these soils can be controlled to within about 3% of optimum. However, the natural moisture content of the on-site fine-grained soils will probably exceed the optimum moisture content throughout the majority of the year; therefore, some aeration and drying will be required to meet the requirements for proper compaction. The required drying can best be accomplished by spreading the material in thin lifts and tilling. Drying rates are dependent on weather factors such as wind, temperature, and relative humidity. On-site soils used as structural fill should have a maximum size of 2 inches and should be placed in 8-inch-thick lifts (loose) and compacted with a segmented-pad or sheepsfoot roller to at least 95% of the maximum dry density as determined by ASTM D698. If fine-grained soils are not compacted at a moisture content within about 3% of optimum, the specified density cannot be achieved and the fill material will be relatively weak and possibly compressible.

On-site, fine-grained soils and site strippings free of debris may be used as fill in non-structural landscaped areas where overlying hardscapes such as sidewalks will not be constructed. These materials should be placed at about 90% of the maximum dry density determined by ASTM D698. The moisture contents of soils placed in landscaped areas are not as critical as the moisture contents of soils placed in structural areas, provided construction equipment can effectively handle the materials. However, it should be understood that fine-grained soils compacted to less than 95% of the maximum dry density determined by ASTM D698 or at a moisture content 3% outside the optimum may result in excessive settlement of fill soils.

4.4.3 Imported Granular Fill

During wet conditions, imported granular material would be most suitable for construction of the structural fills. Granular material, such as sand, sandy gravel, or crushed rock, with a maximum size of 2 inches and less than 5% passing the No. 200 sieve (washed analysis) would be suitable structural fill material. Granular fill should be placed in lifts and compacted with vibratory equipment to at least 95% of the maximum dry density determined in accordance with ASTM D698. 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 of up to 2 feet may be acceptable.

4.4.4 Utility Trench Backfill

All utility trench excavations within building, hardscape, and pavement areas should be backfilled with relatively clean, granular material, such as sand, sandy gravel, or crushed rock, of up to 1½-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, and the utilities should be underlain by a minimum-6-inch thickness of bedding material. The granular backfill material should be compacted to at least 95% of the maximum dry density as determined by ASTM D698 in the upper 5 feet of the trench and at least 92% of this density below a depth of 5 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.

4.5 Excavation

4.5.1 General

We anticipate the maximum depth of excavations to establish finished site grades will generally be less than 5 feet, and the depth of utility excavations may be on the order of 10 feet to 15 feet. The method of excavation and the design of excavation support are the responsibilities of the contractor and are subject to applicable local, state, and federal regulations, including the current Occupational Safety and Health Administration (OSHA) excavation and trench safety standards. The means, methods, and sequencing of construction operations and site safety are also the responsibility of the contractor. The information provided below is for the use of our client and should not be interpreted to mean we are assuming responsibility for the contractor's actions or site safety.

4.5.2 Groundwater Management

Perched groundwater may be encountered in the excavations. 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 perched groundwater levels are lowest, and by limiting the depths of the excavations.

We anticipate perched groundwater inflow, if encountered, can generally be controlled by pumping from sumps. To facilitate dewatering, it will likely 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. 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 evaluated based on field observations during construction. We recommend the use of relatively clean, free-draining material, such as 2- to 4-inch-minus crushed rock, for this purpose. The use of a geotextile fabric over the excavation base will assist in subgrade stability and dewatering.

4.5.3 Temporary Excavations

The inclination of temporary excavation slopes will depend on the groundwater conditions and variable soil conditions. In this regard, we anticipate temporary excavation slopes in native soils can be cut at 1H:1V to a maximum depth of 10 feet if groundwater levels are maintained at least 2 feet below the bottom of the excavation and there are no existing improvements or surcharge loading within a horizontal distance from the crest of the slope equal to the height of the slope. Flatter slopes may also be necessary if significant seepage conditions are encountered. Some sloughing, slumping, or running of temporary slopes should be anticipated where significant groundwater seepage occurs. A blanket of relatively clean, well-graded crushed rock placed on the slopes may be required to reduce the risk of raveling-soil conditions if temporary excavation slopes encounter perched groundwater. 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 in the range of 12 inches to 24 inches.

Other measures that should be implemented to reduce the risk of localized failures of temporary slopes include 1) using 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; and 5) periodically monitoring the area around the top of the excavation for evidence of ground cracking. It must be emphasized that following these recommendations will not guarantee 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. It should also be realized that blocks of ground and/or localized slumps may tend to move into the excavation during construction.

4.5.4 Utility Excavations

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

1) Provide stable excavation side slopes or support for trench sidewalls to minimize loss of ground.

2) Provide a safe working environment during construction.

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

According to current OSHA regulations, the majority of the fine-grained soils encountered in the explorations may be classified as Type B. In our opinion, trenches less 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. Excavations that encounter groundwater or are more than 4 feet deep should be laterally supported or alternatively provided with sideslopes of 1H:1V or flatter to depths less than 15 feet. In our opinion, adequate lateral support may be provided by common methods, such as the use of a trench shield or hydraulic shoring systems. If deeper excavations are required, we should be contacted to reevaluate our temporary slope recommendations.

4.6 Foundation Support

4.6.1 General

Although the proposed structural loads are currently unavailable, we anticipate the maximum column loads and wall loads will be on the order of 100 kips to 200 kips and 3 kips/foot to 4 kips/foot, respectively. In our opinion, the proposed structural loads can be supported on conventional spread and wall footings in accordance with the following design criteria. Excavations for footings will encounter variable subsurface conditions consisting of fill, silt, sand, and clay. Fill soils were encountered in boring B-3 to a depth of about 5 feet; however, fill thicknesses are likely variable across the site and may extend deeper in other areas. Where encountered, fill soils will need to be removed in foundation excavations and replaced with compacted crushed-rock structural fill. Replacing with controlled-density fill or designing the foundations to extend below the fill are alternatives to overexcavation and backfill with structural fill.

The base of all new footings should be established at a minimum depth of 18 inches below the lowest adjacent finished grade. The footing width should not be less than 24 inches for isolated column footings and 18 inches for wall footings.

4.6.2 Footing Subgrade Preparation

All footings should be established in the firm, native soils that mantles the site or well-compacted structural fill. Soft or otherwise unsuitable material encountered at foundation subgrade level should be overexcavated and backfilled with granular structural fill. Local areas of softer subgrade may require deeper overexcavation and should be evaluated by a member of GRI's geotechnical engineering staff. We recommend the contract documents provide unit costs for subgrade overexcavation and structural backfill. Final excavations for all foundations should be made with a smooth-edged bucket, and all footing subgrades should be observed by a member of GRI's geotechnical engineering staff. Our experience indicates the subgrade soils are easily disturbed by excavation and construction activities. Due to these considerations, we recommend installing a minimum 3-inch-thick layer of compacted crushed rock in the bottom of all footing excavations. Relatively clean, ¾-inch-minus crushed rock having less than about 5% fines passing the No. 200 sieve (washed analysis) is suitable for this purpose.

4.6.3 Allowable Bearing Pressure and Settlement

Our allowable bearing pressures are based on our assumed column and wall loads, the proposed building locations provided, and the field investigations. Footings established in accordance with the above criteria can be designed based on an allowable soil bearing pressure of 3,000 pounds per square foot (psf). These values apply to the total of dead load and/or frequently applied live loads and can be increased by one third for the total of all loads: dead, live, and wind or seismic.

We estimate the total static settlement of spread and wall footings designed in accordance with the recommendations presented above will be less than 1 inch for footings supporting column and wall loads of up to 200 kips and 4 kips/foot, respectively. Differential static settlements between adjacent, comparably loaded and similarly supported footings should be less than half the total settlement. Differential static settlements between footings supported on differing subsurface conditions may approach total settlements.

As discussed earlier, our analysis indicates up to about 1 inch to 2 inches of dynamic settlement could occur following a code-based seismic event. Based on the thicknesses of the non-liquefiable soils that mantle the site and the discrete, thin soil lenses subject to liquefaction and/or cyclic softening, we estimate the potential for significant ground manifestation of the seismically induced settlement is generally low. For foundation design purposes, we recommend assuming differential seismic settlement will approach 50% of the calculated total seismic settlement over the length of the building.

Subsection 12.13.9.2 of ASCE 7-16 provides guidance for acceptable limits of seismic differential settlement for different types of structures and different risk categories. In our opinion, based on review of Table 12.13 3 of ASCE 7-16 and our experience with similar Risk Category III structures, 0.5 inch to 1 inch of seismic differential settlement over the building dimension is consistent with current standards of practice for a life safety performance level. However, the structural engineer should determine if the structure can accommodate the estimated total and differential seismic settlements. Tying the foundations together with a network of grade beams, as identified in Subsection 12.13.9.2.1.1 of ASCE 7-16, will reduce the potential adverse effects associated with differential movement.

4.6.4 Horizontal Forces

Horizontal shear forces can be resisted partially or completely by frictional forces developed between the base of the footings and the underlying soil and by soil passive resistance. The total frictional resistance between the footing and the soil is the normal force times the coefficient of friction between the soil and the base of the footing. We recommend allowable values of 0.35 and 0.40 for the coefficient of friction for footings cast on firm, native soil and granular structural fill, respectively. The normal force is the sum of the vertical forces (dead load plus real live load). If additional lateral resistance is required, passive earth pressures against embedded footings can be computed based on an equivalent fluid having a unit weight of 300 pounds per cubic foot (pcf). This design passive earth pressure would be applicable only if the footing is cast neat against undisturbed soil or backfill for the footings is placed as granular structural fill and assumes up to 0.01H inch of lateral movement of the structure will occur in order for the soil to develop this resistance, where H is the depth of embedment to the bottom of the footing. This value also assumes the ground surface in front of the foundation is horizontal, i.e., does not slope downward away from the toe of the footing.

4.7 Subdrainage/Floor Support

To provide a capillary break and reduce the risk of damp floors, slab-on-grade floors established at or near adjacent final site grades should be underlain by a minimum of 8 inches of free-draining, clean, angular rock. This material should consist of angular rock such as 1½ - to ¾-inch crushed rock with less than 2% passing the No. 200 sieve (washed analysis) and should be capped with a 2-inch-thick layer of compacted, ¾-inch-minus crushed rock to improve workability. The slab base course section should be placed in one lift and compacted to at least 95% of the maximum dry density (ASTM D698) or until well keyed. In areas where floor coverings will be provided or moisture-sensitive materials stored, it would be appropriate to also install a vapor-retarding membrane. The membrane should be installed as recommended by the manufacturer. In addition, a foundation drain should be installed around the building perimeter to collect water that could potentially infiltrate beneath the foundations and should drain by gravity or pumped from sumps and discharge to an approved storm drain. The perimeter foundation drain should be placed at the base of the footing and embedded within free-draining, clean angular rock, such as 1½- to ¾-inch crushed rock with less than 2% passing the No. 200 sieve (washed analysis).

We anticipate the finished floor elevation for the new buildings will be established near or above the adjacent surrounding site grades. If structures such as floors are established below final site grades, the structures should be provided with a subdrainage system. A subdrainage system will reduce the buildup of hydrostatic pressures on the floor slab and the risk of groundwater entering through embedded walls and floor slabs. GRI should be contacted if embedded structures are being considered.

In our opinion, it is appropriate to assume a coefficient of subgrade reaction, k, of 150 pci to characterize the subgrade support for point loading with 10 inches of compacted crushed rock beneath the floor slab.

4.8 Retaining Walls

We anticipate portions of the improvements may be partially embedded and may require embedded walls. For this report, we assumed any site retaining walls will consist of conventional cast-in-place walls supported on spread foundations. Foundation design and subgrade preparation should conform to the recommendations provided above for foundation support.

Design lateral earth pressures for retaining walls depend on the type of construction, i.e., the ability of the wall to yield. Possible conditions are 1) a wall 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 cantilever or gravity wall, that yields to the active state by tilting about its base. A conventional basement wall and cantilever retaining wall are examples of non-yielding and yielding walls, respectively. For completely drained, horizontal backfill, yielding and non-yielding walls may be designed based on equivalent fluid unit weights of 35 pcf and 55 pcf, respectively. To account for seismic loading, the earth pressures should be increased by 8 pcf and 18 pcf for yielding and non-yielding walls, respectively. 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. Additional lateral loading due to surcharge loads can be evaluated using the criteria shown on Figure 4.

The lateral earth pressure design criteria presented above are appropriate if the retaining walls are fully drained. Perched groundwater may occur within the shallow, fine-grained soils and existing utility trenches during periods of prolonged or intense precipitation. Based on these considerations, we recommend installation of a permanent drainage system behind all the retaining walls. 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. The drainage blanket should have a minimum width of 12 inches and consist of crushed drain rock that contains less than 2% fines content (washed analysis). A nonwoven geotextile fabric should separate the drain rock and wall backfill. A typical drainage system for retaining walls constructed with a drainage blanket is shown on Figure 5. The drainage blanket or drainage panels should extend to the base of the wall, where water should be collected in a perforated pipe and discharged to a suitable outlet, such as a sump or approved storm drain. In addition, the wall design should include positive drainage measures to prevent ponding of surface water behind the top of the wall.

Overcompaction of backfill behind walls should be avoided. Heavy compactors and large pieces of construction equipment should not operate within 5 feet of any embedded wall to avoid the buildup of excessive lateral earth pressures. Compaction close to the walls should be accomplished with hand-operated, vibratory-plate compactors. Overcompaction of backfill could significantly increase lateral earth pressures behind walls.

4.9 Pavement Design

4.9.1 Recommended Design

We anticipate the access roads and parking areas at the site will be subjected to automobile, light, and occasional heavy truck traffic. We anticipate the majority of the site will be paved with AC pavement; however, areas subjected to repeated heavy-truck traffic, such as trash-enclosure and service areas, may be paved with PCC pavement. Traffic estimates for the access roads and parking areas are currently unknown.

Based on our experience with similar projects and subgrade soil conditions, we recommend the following pavement sections provided in Table 4-2.

Table 4-2: RECOMMENDED PAVEMENT SECTIONS

Pavement Type	Traffic Loading	CRB Thickness, in.	AC Thickness, in.
AC	Areas Subject to All Traffic (Access Roads)	14	5
AC	Areas Subject to Primarily Automobile Traffic (Parking Lot Drive Aisles, Occasional Truck Traffic)	12	4
AC	Areas Subject to Automobile Parking (Parking Areas)	8	3
PCC	Areas Subject to Repeated Heavy Truck Traffic (Trash Enclosure and Service Areas)	6	6

Note:

The recommended pavement sections should be considered minimum thicknesses and underlain by a nonwoven geotextile fabric.

It should be assumed that some maintenance will be required over the life of the pavement (15 years to 20 years). The recommended pavement sections are based on the assumption that pavement construction will be accomplished during the dry season and after construction of the building has been completed. If wet-weather pavement construction is considered, it will likely be necessary to increase the thickness of CRB course to support construction equipment and protect the subgrade from disturbance. The indicated sections are not intended to support construction traffic such as forklifts, dump trucks, and concrete trucks. Pavements subject to construction traffic may require repair.

For the above-indicated sections, drainage is an essential aspect of pavement performance. We recommend all paved areas be provided positive drainage to remove surface water and water within the base course. This will be particularly important in cut sections or at low points within the paved areas, such as at catch basins. Effective methods to prevent saturation of the base-course materials include providing weepholes in the sidewalls of catch basins, subdrains in conjunction with utility excavations, and separate trench-drain systems. To help ensure quality materials and construction practices, we recommend the pavement work conform to Oregon Department of Transportation (ODOT) standards.

Prior to placing base-course materials, all pavement area subgrade should be proof rolled with a fully loaded dump truck. Any soft areas detected by the proof rolling should be overexcavated to firm ground and backfilled with compacted structural fill.

Provided the pavement section is installed in accordance with the recommendations provided above, it is our opinion the site-access areas will support infrequent traffic by an emergency vehicle having a gross vehicle weight of up to 80,000 pounds. For the purposes of this evaluation, "infrequent" can be defined as once a month or less.

4.9.2 *Standard Specifications*

Construction materials and procedures should comply with the applicable sections of the current ODOT *Oregon Standard Specifications for Construction* given in Table 4-3.

Table 4-3: ODOT SPECIFICATIONS FOR CONSTRUCTION

Materials/Activity	Specification
Asphalt Concrete New Construction	Special Provision 00745. Place the AC section using a minimum lift thickness of 2 in. and a maximum lift thickness of 3 in. Lime or latex treatment of aggregate is not required.
Asphalt Binder	Use Performance Grade (PG) 64-22 Asphalt Cement in Level 2.
Aggregate Base	Section 00641 (¾ in. – 0 or 1 in. – 0).
Non-Woven Geotextile	Sections 00350 and 02320.

5 LIMITATIONS

This report has been prepared to aid the owner, architect, and engineer in the planning and preparation of design and associated cost estimates. 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 construction of foundations, retaining walls, and pavements. Depending on the final layout of the facility, additional subsurface explorations, laboratory testing, and engineering studies may be required to provide criteria that are suitable for final design.

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, it is acknowledged 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, during construction, subsurface conditions differ from those encountered in the explorations, we should be advised at once so we can observe and review these conditions and reconsider our recommendations where necessary.

Please contact the undersigned if you have any questions.

Submitted for GRI,

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WESLEY SPANG

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Expires 06-2022

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This document has been submitted electronically.

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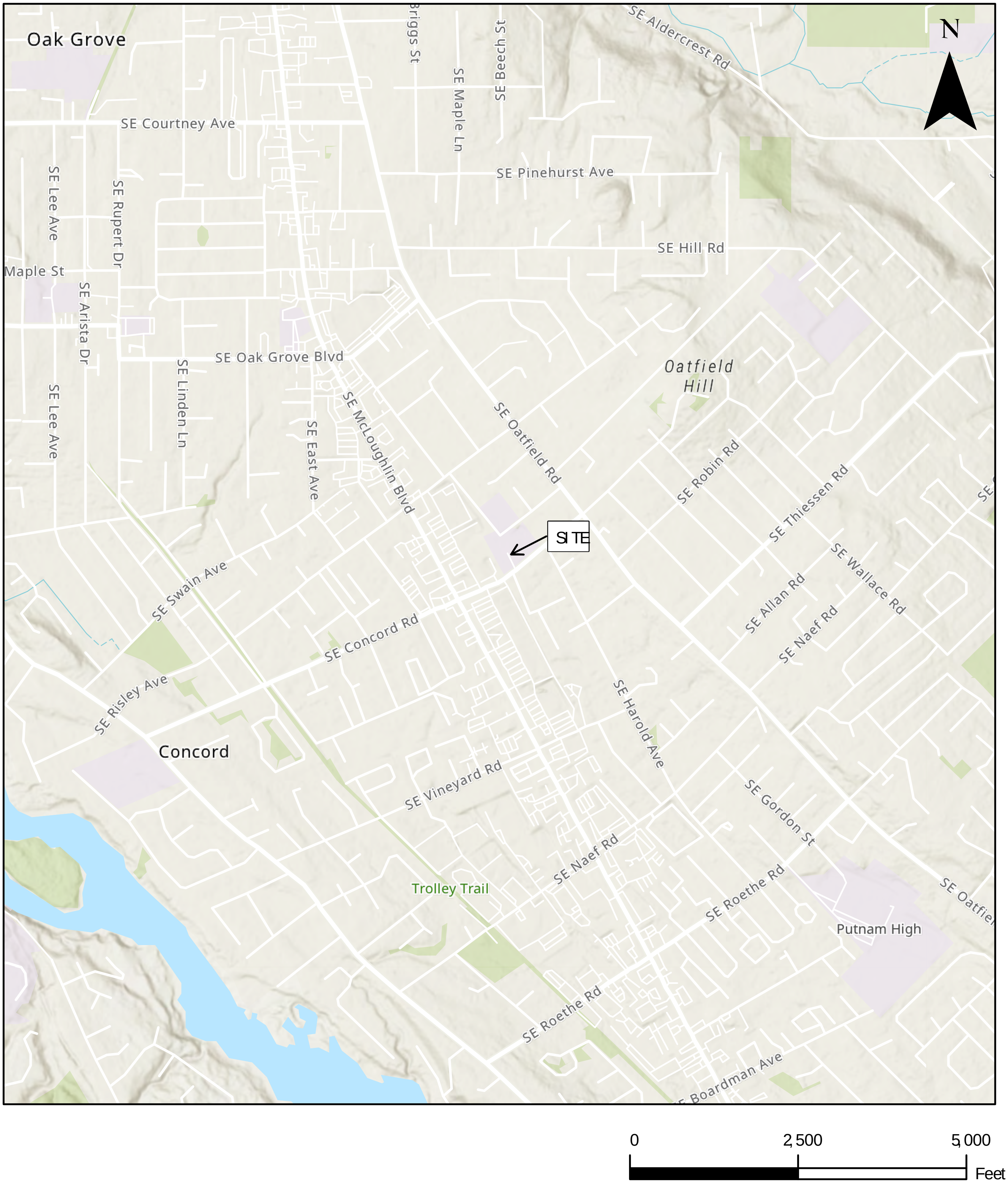

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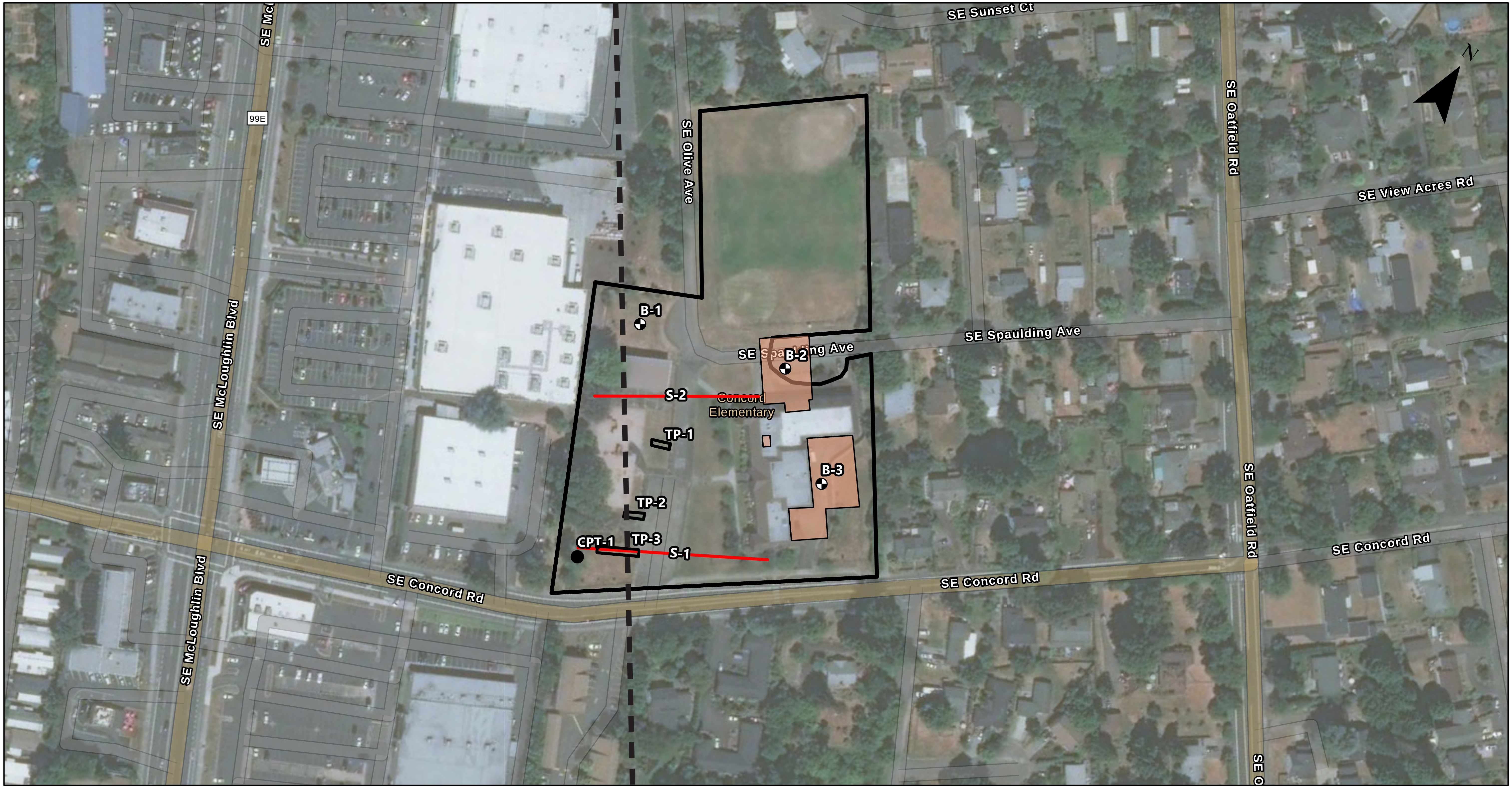
VICINITY MAP

OCT. 2020 JOB NO. 6334-A FIG. 1



⊕ BORING COMPLETED BY GRI
(SEPTEMBER 1, 2020)

▨ PROPOSED IMPROVEMENT AREAS

— SEISMIC REFRACTION LINES COMPLETED BY EARTH DYNAMICS
(AUGUST 5, 2020)

● CONE PENETRATION TEST COMPLETED BY GRI
(AUGUST 25, 2020)

▭ TEST PITS COMPLETED BY GRI
(AUGUST 25, 2020)

– – INFERRED OATFIELD FAULT PER USGS 2020

0 150 300
Feet

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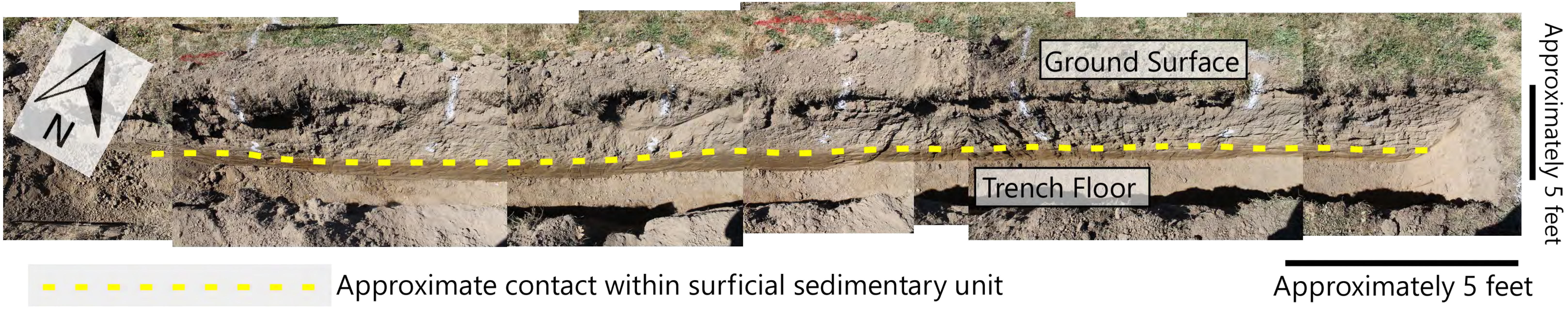
SITE PLAN

OCT. 2020 JOB NO. 6334-A FIG. 2



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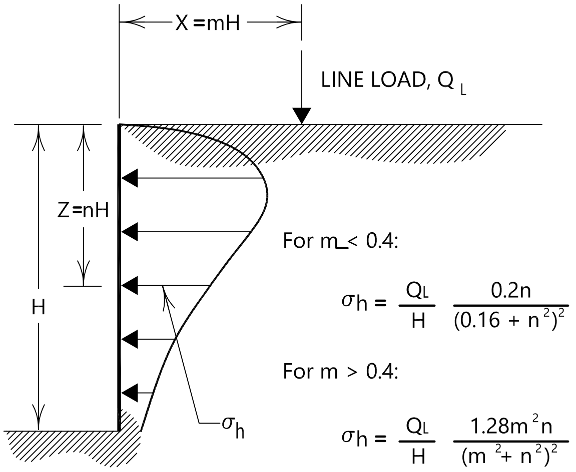
ANNOTATED TEST PIT PHOTOGRAPH

OCT. 2020 JOB NO. 6334-A FIG. 3



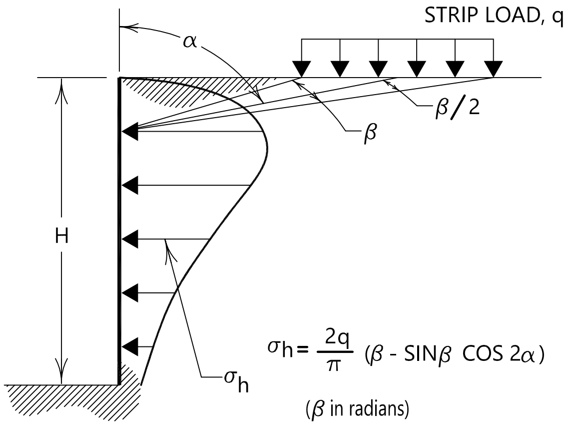
For $m \leq 0.4$:

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

For $m > 0.4$:

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

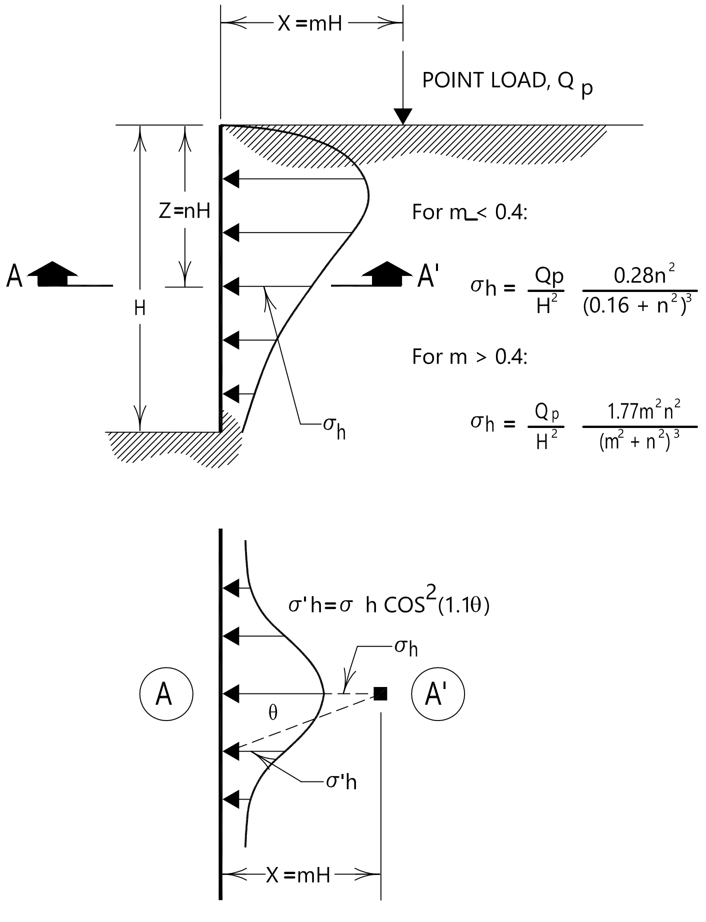
LINE LOAD PARALLEL TO WALL



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

(β in radians)

STRIP LOAD PARALLEL TO WALL



For $m \leq 0.4$:

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

For $m > 0.4$:

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

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

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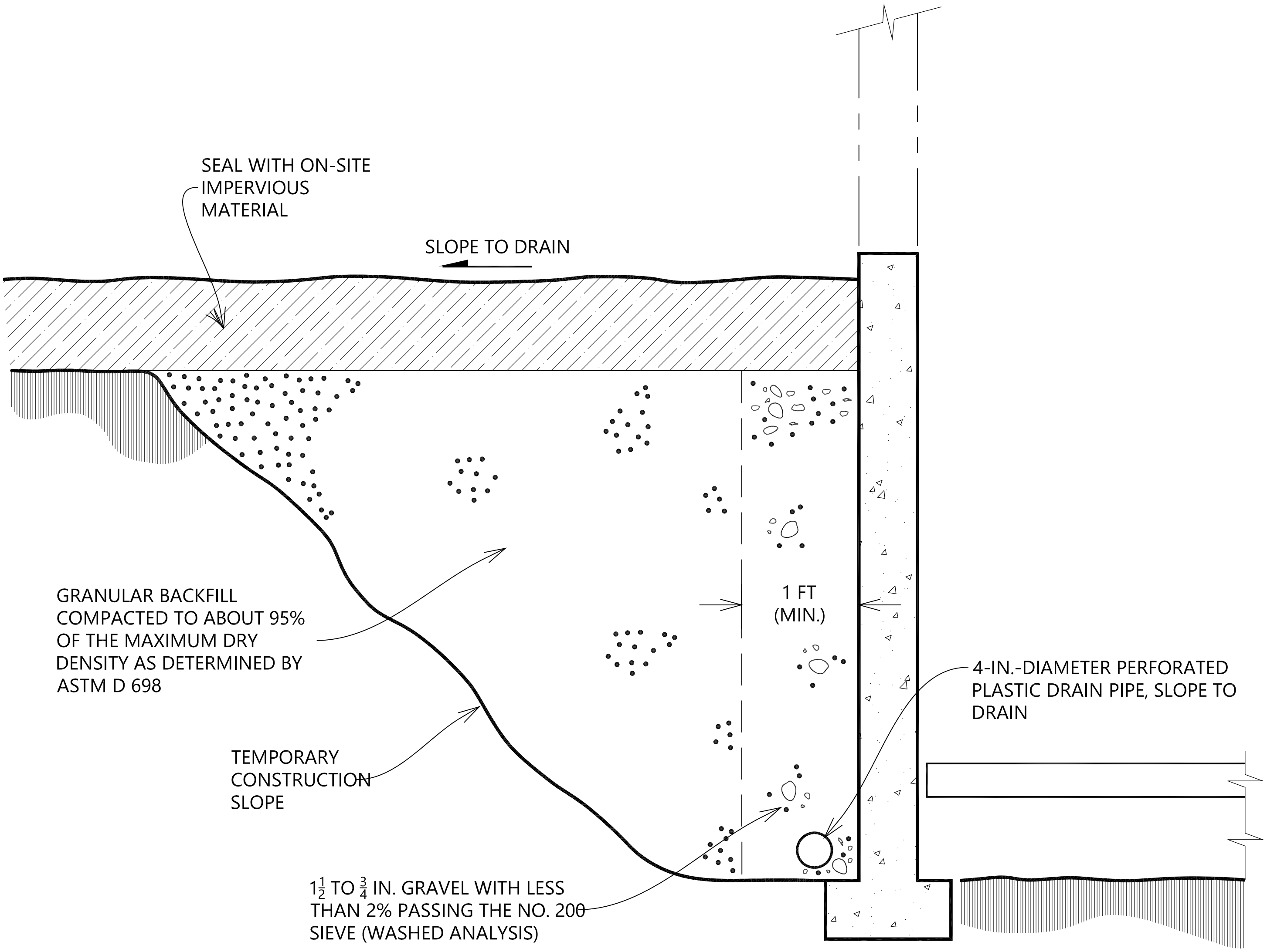
SURCHARGE-INDUCED LATERAL PRESSURE

OCT. 2020	JOB NO. 6334-A	FIG. 4



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WALL SUBDRAINAGE DETAIL

OCT. 2020 JOB NO. 6334-A FIG. 5

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

A.1 FIELD EXPLORATIONS

Subsurface materials and conditions at the site were investigated on August 5 through September 1, 2020, with two seismic-refraction arrays designated S-1 and S-2, one cone penetration test (CPT) probe designated CPT-1, three test-pit excavations designated TP-1 through TP-3, and three drilled borings designated B-1 through B-3. The approximate locations of the explorations completed for this investigation are shown on Figure 2.

A.1.1 Drilled Borings

The drilled borings were completed using mud-rotary, open-hole drilling techniques using a truck-mounted Geoprobe 7720 DT drill rig provided and operated by Western States Soil Conservation, Inc., of Hubbard, Oregon. The drilled borings were advanced to depths of about 31.5 feet to 71.5 feet below existing site grades. The field-exploration work was coordinated and documented by an experienced member of GRI's geotechnical engineering team, who maintained a log of the materials and conditions disclosed during the course of work.

Disturbed and undisturbed soil samples were generally obtained from the borings at 2.5-foot intervals of depth in the upper 15 feet and typically 5-foot intervals below this depth. Disturbed soil samples were obtained using a 2-inch-outside-diameter standard split-spoon sampler. The standard penetration test (SPT) was 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, or SPT N value. The SPT N-value provides a measure of relative density of granular soils and the relative consistency of cohesive soils. Samples obtained from the borings were placed in airtight jars and returned to our laboratory for further classification and testing. In addition, 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 soil exposed in the ends of the Shelby tubes 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 classification.

Logs of the drilled borings are provided on Figures 1A through 3A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth at which the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples are indicated. Farther to the right, SPT N-values are shown graphically, along with the natural moisture contents, dry unit weights, Torvane shear-strength values, Atterberg limits, and percentages passing the No. 200 sieve where applicable. The terms and symbols used to describe the materials encountered in the borings are defined in Table 1A, 2A, and the attached legend.

A.1.2 Cone Penetration Test

One CPT probe, designated CPT-1, was advanced to a depth of about 28.5 feet using a track-mounted rig provided and operated by Oregon Geotechnical Explorations, Inc., of Keizer, 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 measurements, measurements are 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 resistance and sleeve friction can be used to evaluate the relative consistency of cohesionless and cohesive soils, respectively. In addition, a 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 groundwater depth. The probe is also operated using an accelerometer fitted to the probe, which allows measurement of the arrival time of shear waves from impulses generated at the ground surface and calculation of shear-wave velocities for the surrounding soil profile.

A log of the CPT probe is provided on Figure 4A, which presents 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 3A. Shear-wave velocity measurements were recorded for the CPT-1 probe and are shown on Figure 5A.

A.1.3 Test Pits

The test pits were excavated using a Case 580 backhoe provided and operated by Dan Fischer Excavating, Inc., of Forest Grove, Oregon. The test pits were excavated to the practical limits of the equipment to depths of about 16 feet to 19 feet. The field-exploration work was coordinated and documented by an experienced member of GRI's geology team, who maintained a log of the materials and conditions disclosed during the course of work.

Disturbed soil samples were generally obtained from the test pit excavations at 2-foot intervals of depth or as subsurface conditions changed. The soil samples were classified in the field. After classification, the samples were sealed in jars and returned to our laboratory for further examination and classification.

Logs of the test pits are provided on Figures 6A through 8A. Each log presents a descriptive summary of the various types of materials encountered in the excavation and notes the depth at which the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples are indicated. Farther to the right, Torvane shear-strength values, Atterberg limits, and percentages passing the No. 200 sieve are shown where applicable. The terms and symbols used to describe the materials encountered in the excavations are defined in Tables 1A, 2A, and the attached legend.

A.1.4 Seismic Refraction

The seismic-refraction geophysical exploration surveys and analysis were completed at the site by Earth Dynamics LLC of Portland, Oregon. The seismic-refraction surveys were used to assist in evaluating the potential presence of subsurface geologic structures. The report on the surveys is provided in Appendix C.

A.2 LABORATORY TESTING

A.2.1 General

The samples obtained from the borings and test pits were examined in 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 one-dimensional consolidation, Atterberg-limits determination, and a grain-size analysis. A summary of the laboratory test results has been provided in Table 4A. The following sections describe the testing program in more detail.

A.2.2 Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM International (ASTM) D2216. The results are summarized on Figures 1A through 3A, 6A through 8A, and in Table 4A.

A.2.3 Grain-Size Analysis

A.2.3.1 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 was oven-dried and weighed. The percentage of material passing the No. 200 sieve was then calculated. The results are summarized on Figures 1A through 3A, and 6A through 8A, where applicable, and in Table 4A.

A.2.4 Atterberg Limits

Atterberg-limits determinations were performed on samples obtained from the borings in conformance with ASTM D4318. The results of the tests are shown graphically on Figures 1A through 3A, 6A through 8A, where applicable, the Plasticity Chart, Figure 9A, and in Table 4A.

A.2.5 Torvane Shear Strength

The approximate undrained shear strength of the fine-grained soils was determined using the Torvane shear device. The Torvane is a hand-held 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 shear-strength testing are shown on Figures 1A through 3A and 6A through 8A, where applicable.

A2.6 One-Dimensional Consolidation

One-dimensional consolidation testing was performed in accordance with ASTM D2435 on relatively undisturbed soil samples obtained from boring B-1 at depths of about 2.8 feet and 8 feet. The test provides data on the compressibility of underlying fine-grained soils. Test results are summarized on Figures 10A and 11A in the form of a curve showing effective stress versus percent strain. The initial dry unit weight and moisture content of the samples are also shown on the figures.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

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

Description of Consistency for Fine-Grained (Cohesive) Soils

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

Grain-Size Classification

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

Modifier for Subclassification

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

Table 2A

GUIDELINES FOR CLASSIFICATION OF ROCK

Relative Rock Weathering Scale

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

Relative Rock Hardness Scale

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

RQD and Rock Quality

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

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

Table 3A

CONE PENETRATION TEST (CPT) CORRELATIONS

Cohesive Soils

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

Cohesionless Soils

Cone Tip Resistance, tsf	Relative Density
<20	Very Loose
20 to 40	Loose
40 to 120	Medium
120 to 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.

Table 4A

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	3.0	215.0	33	--	--	--	92	SILT
	S-1	3.5	214.5	32	88	--	--	--	SILT
	S-2	4.0	214.0	35	--	39	12	85	SILT
	S-3	7.5	210.5	36	--	--	--	93	SILT
	S-4	10.0	208.0	42	--	--	--	--	Clayey SILT
	S-5	12.5	205.5	50	--	--	--	68	Sandy SILT
	S-6	15.0	203.0	48	--	--	--	--	Sandy SILT
	S-7	20.0	198.0	59	--	--	--	68	Sandy SILT
	S-8	22.0	196.0	44	--	--	--	38	Silty SAND
	S-8	23.0	195.0	46	76	--	--	--	Silty SAND
	S-9	23.5	194.5	50	--	--	--	58	Sandy SILT
	S-10	25.0	193.0	46	--	--	--	--	Sandy SILT
	S-11	30.0	188.0	46	--	--	--	54	Sandy SILT
	S-12	55.0	163.0	39	--	50	26	79	CLAY
	S-13	70.0	148.0	44	--	--	--	--	BASALT
B-2	S-1	2.5	228.0	30	--	--	--	--	SILT
	S-2	5.0	225.5	37	--	--	--	--	SILT
	S-3	8.0	222.5	37	--	--	--	76	SILT
	S-3	9.0	221.5	39	84	--	--	--	SILT
	S-4	9.5	221.0	36	--	--	--	84	SILT
	S-5	13.0	217.5	39	--	--	--	61	Sandy SILT
	S-6	15.0	215.5	24	--	--	--	18	Silty SAND
	S-7	20.0	210.5	46	--	--	--	56	Sandy SILT
	S-8	25.0	205.5	42	--	--	--	39	Sandy SILT
	S-9	30.0	200.5	46	--	--	--	--	Sandy SILT
B-3	S-2	5.0	229.3	28	--	--	--	75	SILT
	S-3	7.5	226.8	28	--	--	--	42	Sandy GRAVEL
	S-4	10.0	224.3	53	--	64	30	81	SILT
	S-5	12.5	221.8	53	--	--	--	--	SILT
	S-6	15.3	219.0	51	--	--	--	82	SILT
	S-6	15.8	218.5	47	75	--	--	--	SILT
	S-6	16.5	217.8	40	82	--	--	--	SILT
	S-7	17.0	217.3	43	--	--	--	--	BASALT
	S-8	20.0	214.3	37	--	--	--	--	BASALT
	S-9	25.0	209.3	53	--	73	30	75	BASALT
	S-10	30.0	204.3	52	--	--	--	--	BASALT
TP-1	S-1	1.0	209.0	9	--	--	--	--	Clayey SILT
	S-2	3.5	206.5	19	--	34	13	87	Clayey SILT
	S-4	15.5	194.5	47	--	--	--	69	Sandy CLAY
TP-2	S-1	1.0	211.0	6	--	--	--	--	SILT

Table 4A

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
TP-2	S-2	2.0	210.0	11	--	--	--	--	Clayey SILT
	S-3	3.5	208.5	29	--	--	--	97	Silty CLAY
TP-3	S-1	1.0	215.0	9	--	--	--	--	SILT
	S-2	4.0	212.0	28	--	--	--	97	Clayey SILT

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
I	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

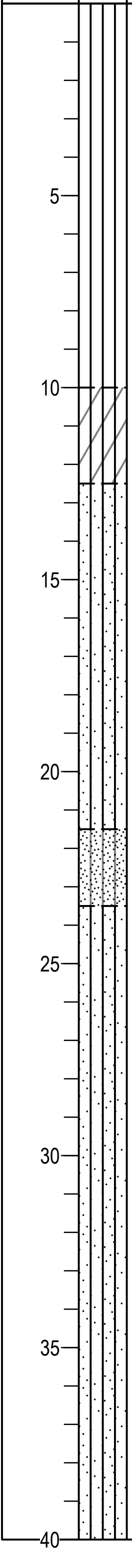
Symbol	Symbol Description
▥	Flush-mount monument set in concrete
▥	Concrete, well casing shown where applicable
▨	Bentonite seal, well casing shown if applicable
░	Filter pack, machine-slotted well casing shown where applicable
▥	Grout, vibrating-wire transducer cable shown where applicable
P	Vibrating-wire 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, %)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS

Surface Elevation: 218.0 ft [±] (NAVD 88)



SILT, some clay, trace to some fine-grained sand, brown mottled rust, stiff, 3-in.-thick heavily rooted zone at ground surface (Alluvium)

---medium stiff to stiff at 4 ft

---trace clay below 7.5 ft

---clayey, some fine- to coarse-grained sand, trace subangular gravel, gray, rust and dark brown, very soft to soft below 10 ft

---sandy, some clay, very soft below 12.5 ft

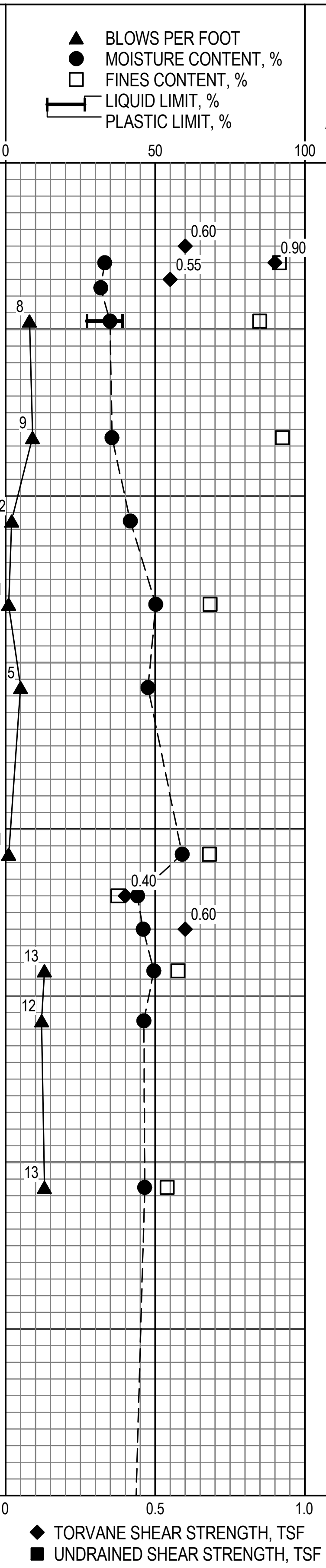
---light brown mottled rust, medium stiff below 15 ft

---light brown to brown, very soft below 20 ft

---interbedded with silty SAND, some clay at 21.5 ft

---sandy to silty SAND, trace clay, brown mottled rust, stiff or medium dense, fine- to medium-grained sand below 23.5 ft

SAMPLE NO.	BLOW COUNT
S-1	
S-2	2 4 4
S-3	1 4 5
S-4	0 0 2
S-5	1 0 1
S-6	3 2 3
S-7	1 1 0
S-8	
S-9	3 5 8
S-10	3 5 7
S-11	7 5 8



Dry Density = 88 pcf

Dry Density = 76 pcf

Drilling proceeds without sampling to notable material change below 31.5 ft

(CONTINUED NEXT PAGE)

Logged By: B. Cook	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 9/1/20	**GPS Coordinates:** 45.4107° N -122.6251° W (WGS 84)
Drilling Method: Mud Rotary **Equipment:** Geoprobe 7720 DT **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.8
Note: See Legend for Explanation of Symbols	

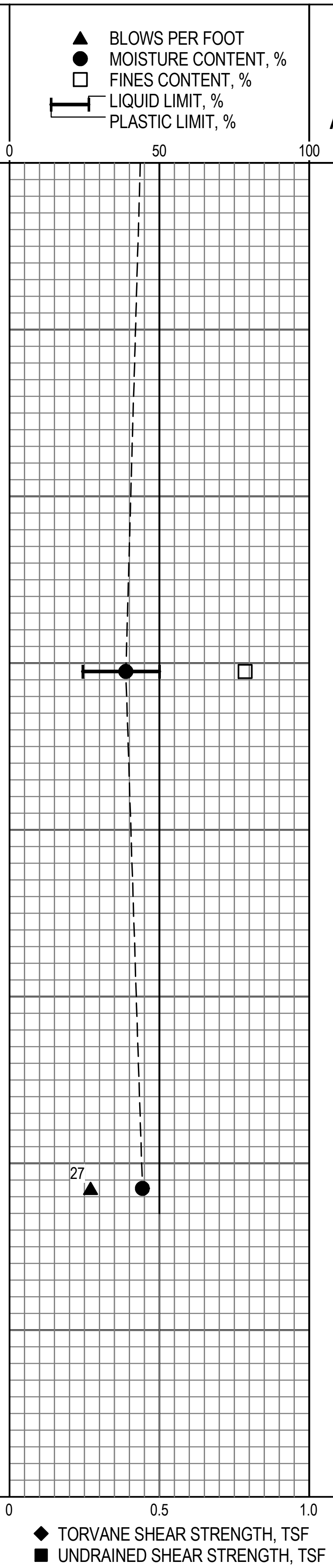
GRI

BORING B-1

OCT. 2020 JOB NO. 6334-A FIG. 1A

Surface Elevation: 218.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Sandy SILT to silty SAND, trace clay, brown mottled rust, stiff or medium dense, fine- to medium-grained sand (Alluvium)						Drilling proceeds without sampling to notable material change below 31.5 ft
50							Driller notes stiffer materials, sandy clay in cuttings below 50 ft
55	CLAY, some silt and fine- to medium-grained sand, blue-gray, stiff (Alluvium)	163.0 / 55.0		S-12	☒		
63	BASALT, some vesicles, gray mottled brown and black, predominantly decomposed, extremely soft to very soft (R0 to R1) (Columbia River Basalt)	155.0 / 63.0					
70				S-13	I	7 11 16	
71.5	(9/1/2020)	146.5 / 71.5					



GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 10/8/20

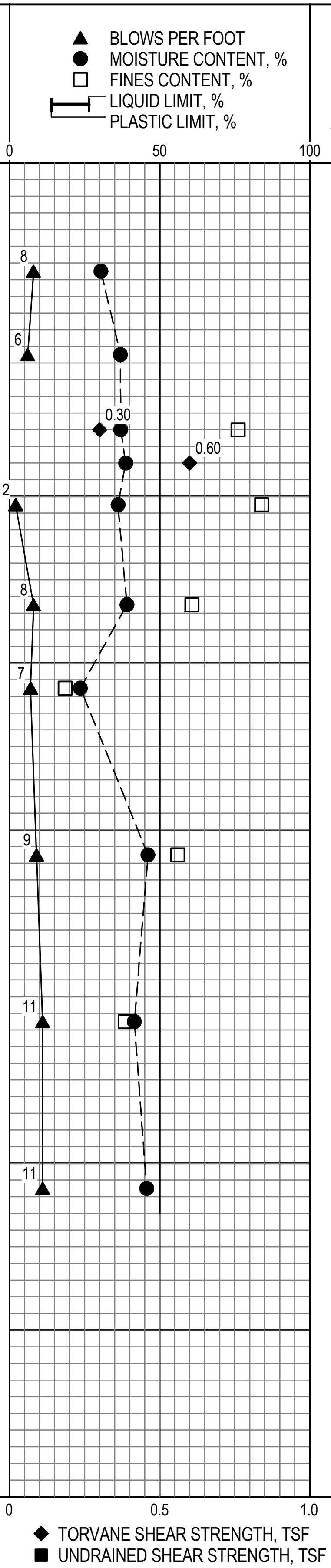
GRI

BORING B-1

OCT. 2020 JOB NO. 6334-A FIG. 1A

Surface Elevation: 230.5 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Asphalt concrete PAVEMENT (3 in.) over crushed rock BASE COURSE (4 in.)	229.9 0.6				
	SILT, some clay, trace fine-grained sand, brown mottled rust, medium stiff to stiff (Alluvium)					
				S-1	1 3 5	
5				S-2	1 3 3	
	---some sand, trace clay below 7 ft					
				S-3		Dry Density = 84 pcf
	---very soft to soft below 9.5 ft					
10				S-4	1 1 1	
	---sandy, medium stiff to stiff below 13 ft			S-5	0 3 5	Heavy drill chatter at 13.5 ft
15	Silty SAND, trace subrounded gravel, brown, loose, fine to coarse grained (Alluvium)	215.5 15.0		S-6	4 3 4	
20	Sandy SILT to silty SAND, trace clay, gray to brown mottled rust, stiff or loose to medium dense, fine-grained sand (Alluvium)	210.5 20.0		S-7	3 5 4	
25				S-8	4 5 6	
30				S-9	4 6 5	
	(9/1/2020)	199.0 31.5				
35						
40						



Logged By: B. Cook
Drilled by: Western States Soil Conservation, Inc.
Date Started: 9/1/20
GPS Coordinates: 45.4109° N -122.6243° W (WGS 84)
Drilling Method: Mud Rotary
Equipment: Geoprobe 7720 DT
Hole Diameter: 5 in.
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: 0.8
Note: See Legend for Explanation of Symbols

GRI

BORING B-2

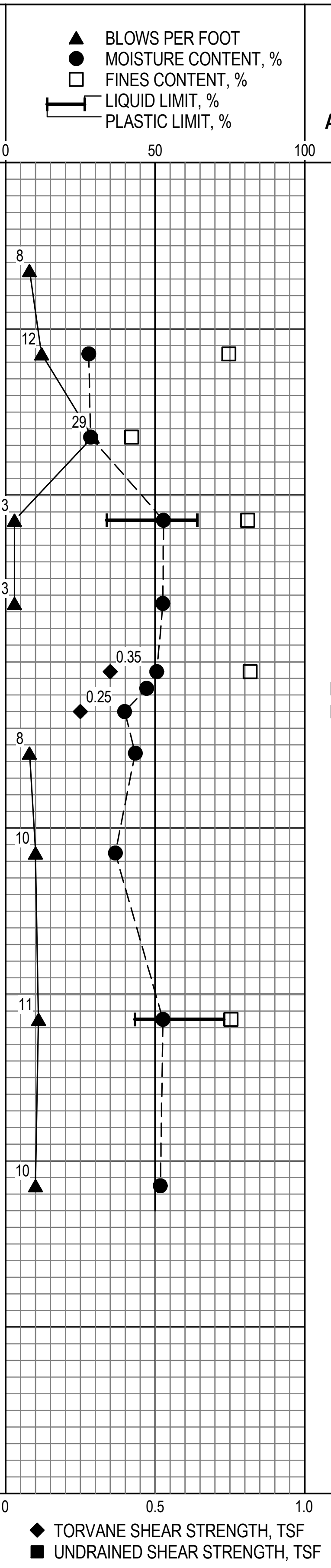
OCT. 2020 JOB NO. 6334-A FIG. 2A

CLASSIFICATION OF MATERIAL

Surface Elevation: 234.3 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT
	Asphalt concrete PAVEMENT (6 in.) over crushed rock BASE COURSE (12 in.)	
	Sandy, silty GRAVEL, loose, fine- to coarse-grained sand, angular gravel (Fill)	232.8 1.5
5	SILT, some clay and fine-grained sand, trace subrounded gravel, brown mottled rust, stiff (Alluvium)	229.3 5.0
	Sandy, silty GRAVEL, medium dense, fine- to coarse-grained sand, subrounded gravel (Alluvium)	226.8 7.5
10	SILT, some clay to clayey, some fine-grained sand, brown mottled rust, soft (Alluvium)	224.3 10.0
15	---medium stiff, interbedded with thin lenses of sand below 15 ft	
	BASALT, brown mottled rust, decomposed, extremely soft to very soft (R0 to R1) (Columbia River Basalt)	217.3 17.0
20		
25		
30		
	(9/1/2020)	202.8 31.5
35		
40		

SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
S-1	2 3 5	
S-2	4 5 7	
S-3	2 10 19	
S-4	1 2 1	
S-5	1 1 2	
S-6		Dry Density = 75 pcf Dry Density = 82 pcf
S-7	3 4 4	
S-8	3 4 6	
S-9	3 5 6	
S-10	3 4 6	



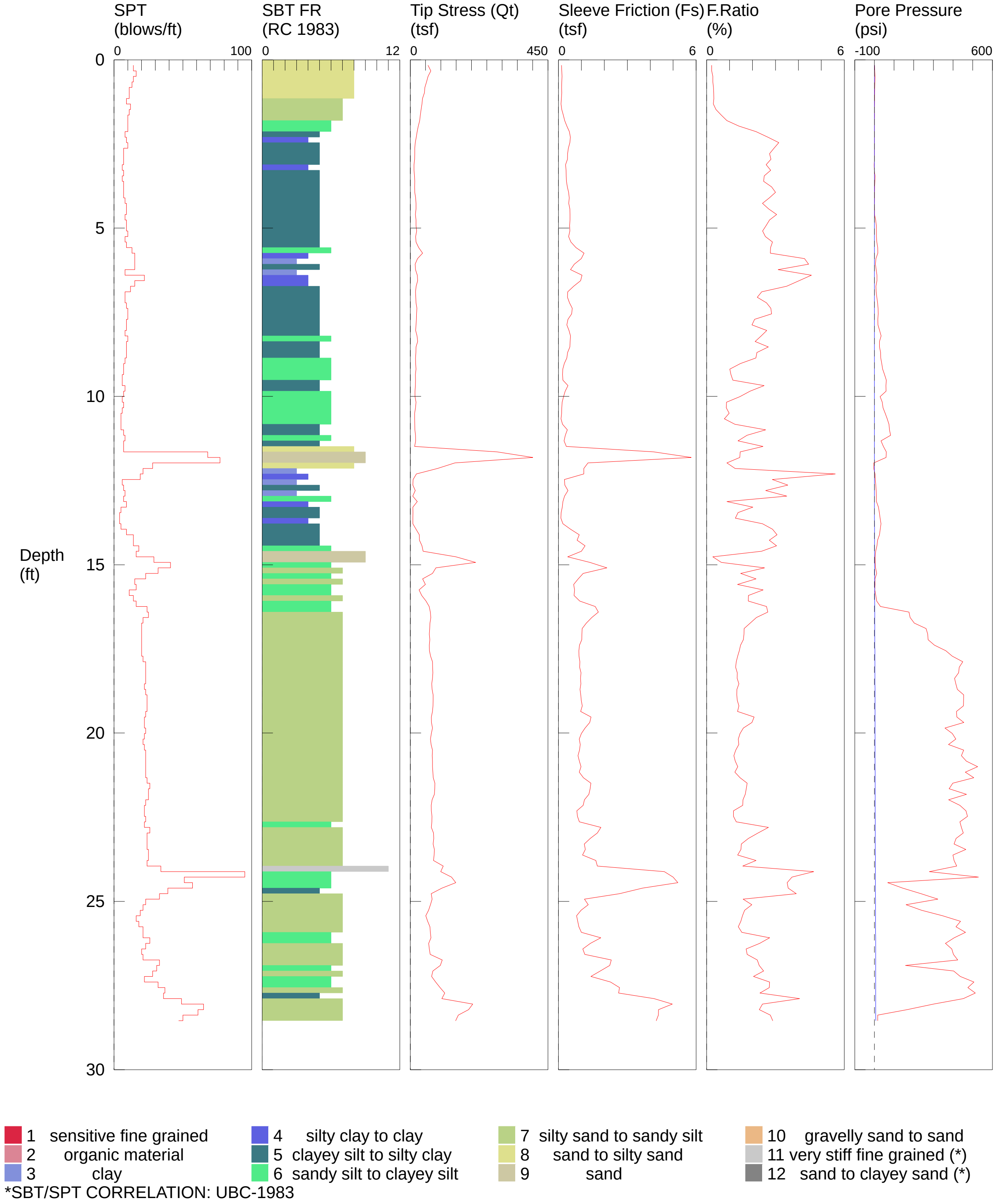
Logged By: B. Cook	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 9/1/20	**GPS Coordinates:** 45.4106° N -122.6237° W (WGS 84)
Drilling Method: Mud Rotary **Equipment:** Geoprobe 7720 DT **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.8
Note: See Legend for Explanation of Symbols	

GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 10/8/20

GRI

BORING B-3

OCT. 2020 JOB NO. 6334-A FIG. 3A

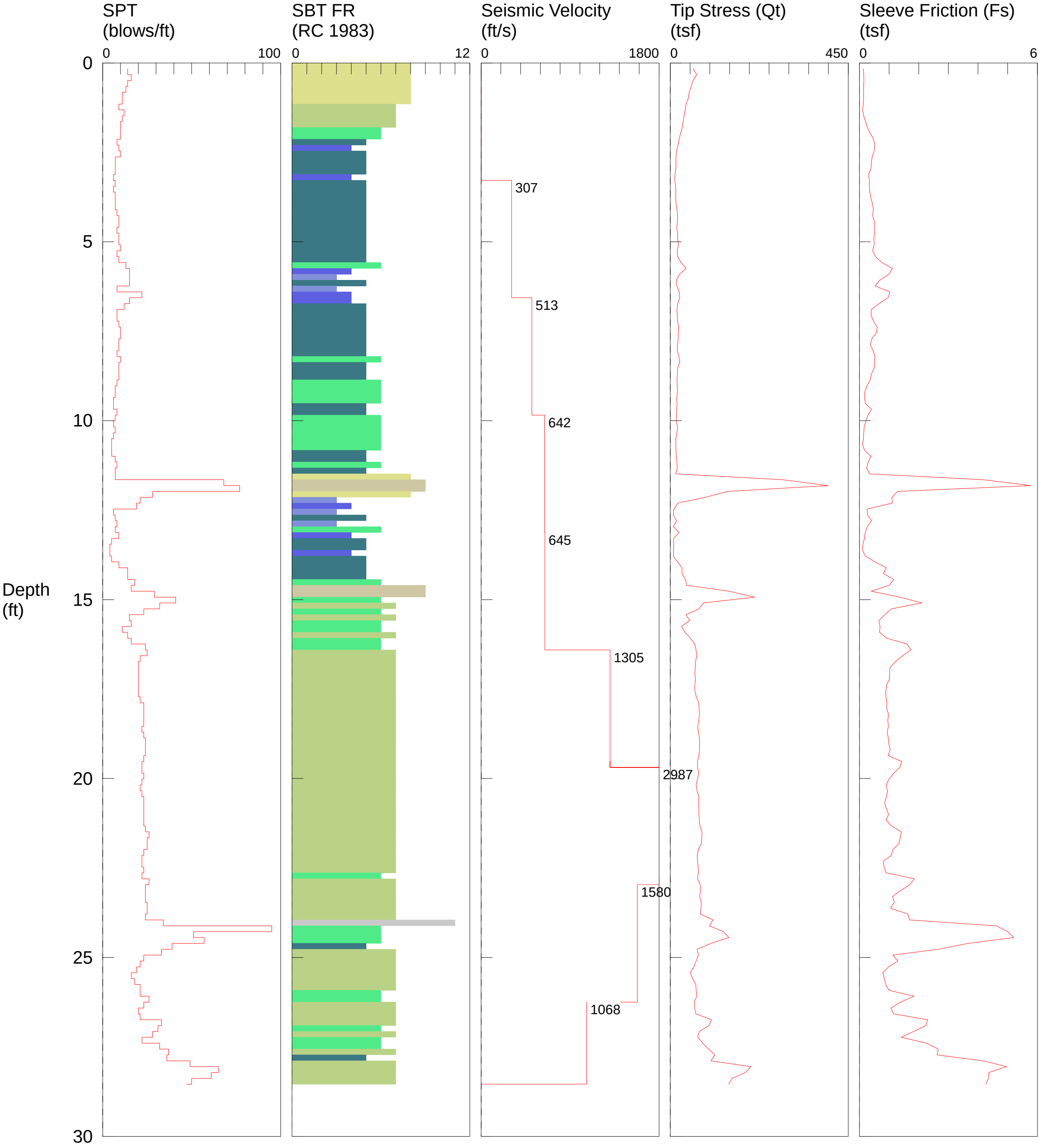


GRI

Observed By: B. Cook
Advanced By: Oregon Geotechnical Explorations, Inc.
Date Started: 08/25/20
Ground Surface Elevation: Not Available
Coordinates: Not Available

CONE PENETRATION TEST CPT-1

OCT. 2020 JOB NO. 6334-A FIG. 4A



🟥 1 sensitive fine grained
🟥 2 organic material
🟦 3 clay
*SBT/SPT CORRELATION: UBC-1983

🟦 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 (*)

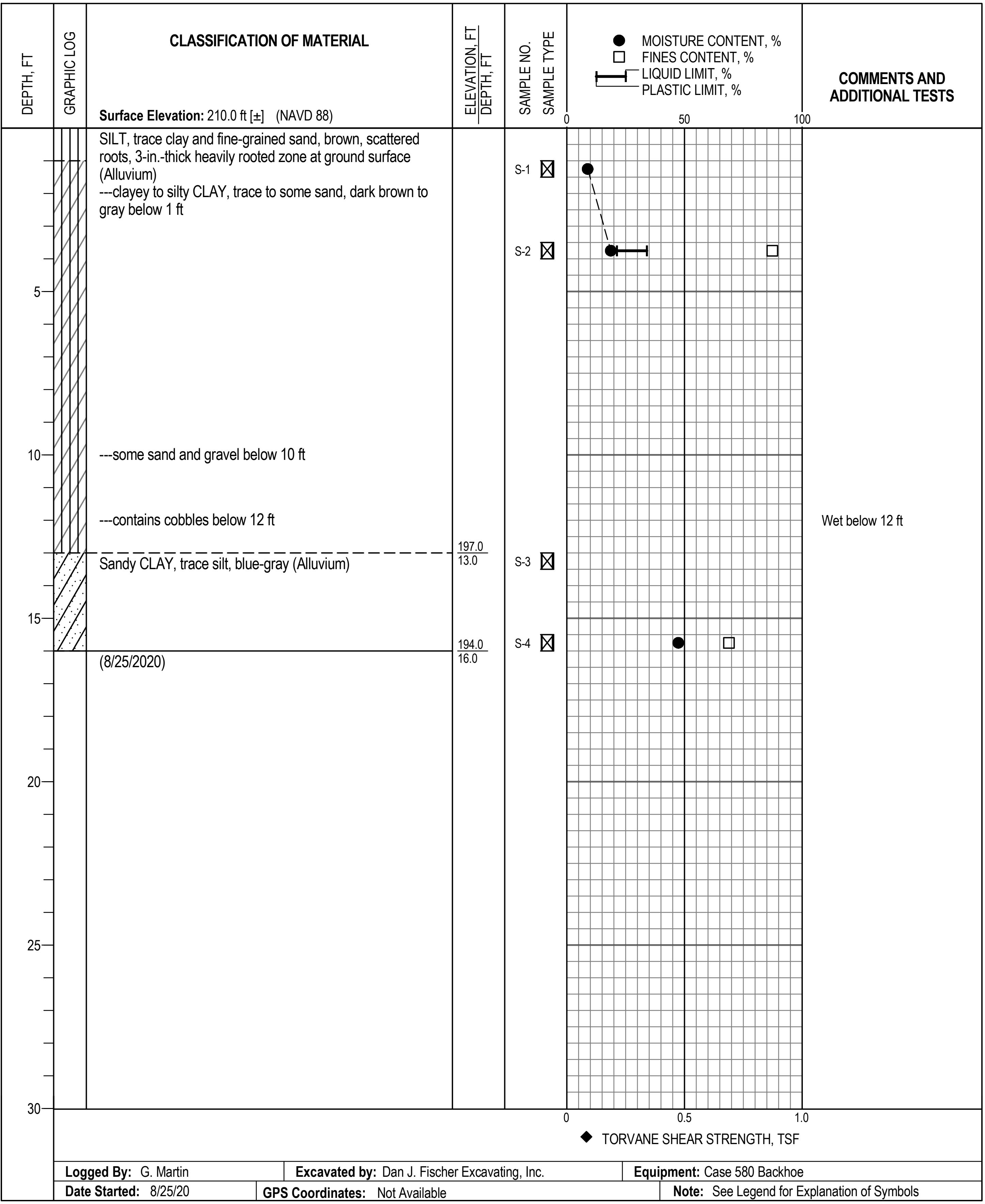
Observed By: B. Cook
Advanced By: Oregon Geotechnical Explorations, Inc.
Date Started: 08/25/20
Ground Surface Elevation: Not Available
Coordinates: Not Available

GRI

CONE PENETRATION TEST CPT-1

(SEISMIC VELOCITY PROFILE)

OCT. 2020 JOB NO. 6334-A FIG. 5A



CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	SAMPLE NO.	COMMENTS AND ADDITIONAL TESTS
Surface Elevation: 210.0 ft [±] (NAVD 88)			
SILT, trace clay and fine-grained sand, brown, scattered roots, 3-in.-thick heavily rooted zone at ground surface (Alluvium) ---clayey to silty CLAY, trace to some sand, dark brown to gray below 1 ft ---some sand and gravel below 10 ft ---contains cobbles below 12 ft		S-1 ☒ S-2 ☒	Wet below 12 ft
Sandy CLAY, trace silt, blue-gray (Alluvium)	197.0 / 13.0	S-3 ☒ S-4 ☒	
(8/25/2020)	194.0 / 16.0		

Logged By: G. Martin
Excavated by: Dan J. Fischer Excavating, Inc.
Equipment: Case 580 Backhoe
Date Started: 8/25/20
GPS Coordinates: Not Available
Note: See Legend for Explanation of Symbols

GRI TP LOG (GPS) GRI DATA TEMPLATE.GDT 10/8/20

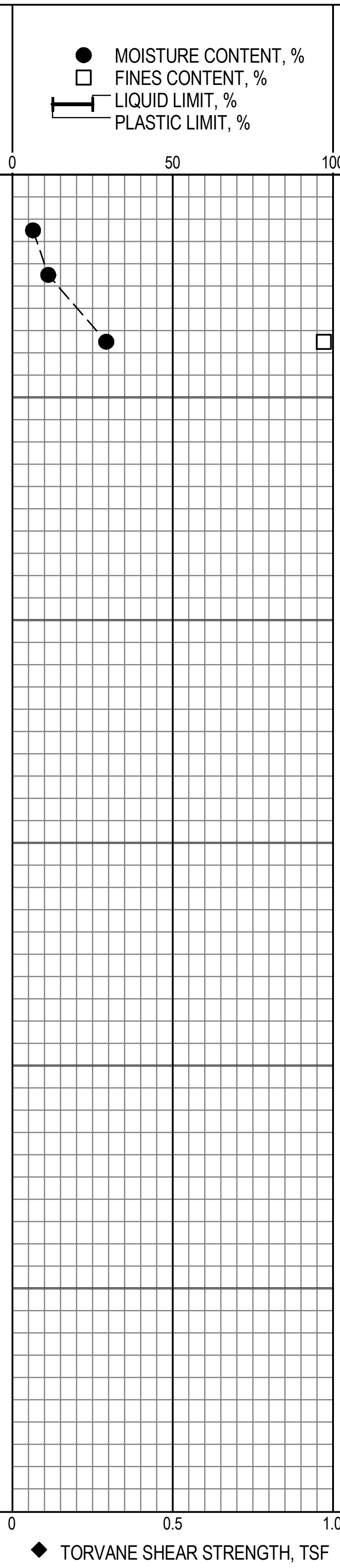
GRI

TEST PIT TP-1

OCT. 2020 JOB NO. 6334-A FIG. 6A

Surface Elevation: 212.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT DEPTH, FT	SAMPLE NO.	SAMPLE TYPE	COMMENTS AND ADDITIONAL TESTS
	SILT, trace fine-grained sand, light brown-gray, scattered roots, 3- to 4-in.-thick heavily rooted zone at ground surface (Alluvium) ---clayey to silty CLAY, light gray to rust below 1.5 ft		S-1 S-2	☒ ☒	
5	Silty CLAY, trace fine-grained sand, orange-brown mottled gray to dark gray, trace roots and organics (Alluvium)	209.0 3.0	S-3	☒	
10	Sandy SILT, trace clay, brown (Alluvium) ---gray mottled brown below 11 ft ---scattered gravel below 13 ft	203.0 9.0			Moist to wet below 11 ft
15	Sandy GRAVEL to gravelly SAND, trace to some silt and clay, brown to gray, fine- to coarse-grained sand, subangular gravel (Alluvium) ---contains cobbles below 15 ft	198.0 14.0			
	GRAVEL and COBBLES, some fine- to coarse-grained sand, trace silt, brown (Alluvium) (8/25/2020)	196.0 16.0 195.0 17.0			
20					
25					
30					



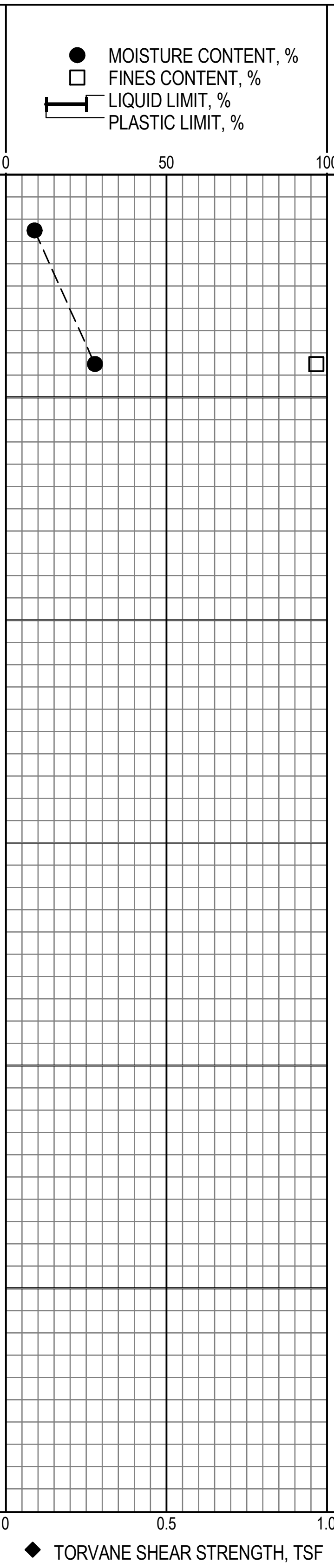
Logged By: G. Martin	**Excavated by:** Dan J. Fischer Excavating, Inc.	**Equipment:** Case 580 Backhoe
Date Started: 8/25/20	**GPS Coordinates:** Not Available	**Note:** See Legend for Explanation of Symbols

GRI

TEST PIT TP-2

OCT. 2020 JOB NO. 6334-A FIG. 7A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** 216.0 ft [±] (NAVD 88)	ELEVATION, FT DEPTH, FT	SAMPLE NO. SAMPLE TYPE	COMMENTS AND ADDITIONAL TESTS
		SILT, trace fine-grained sand, brown, 3-in.-thick heavily rooted zone at ground surface (Alluvium)		S-1 ☒	
		---clayey to silty CLAY, orange-brown mottled light gray below 2 ft		S-2 ☒	
5					
10		---some sand, brown below 11.5 ft ---contains cobbles, dark gray to black below 12 ft			
15					
		BASALT, some vesicles, gray mottled brown and black, moderately weathered to predominantly decomposed, very soft to soft (R1 to R2) (Columbia River Basalt)	200.0 16.0		Weathering in basalt decreases with depth
		(8/25/2020)	197.0 19.0		
20					
25					
30					



Logged By: G. Martin
Excavated by: Dan J. Fischer Excavating, Inc.
Equipment: Case 580 Backhoe
Date Started: 8/25/20
GPS Coordinates: Not Available
Note: See Legend for Explanation of Symbols

GRI TP LOG (GPS) GRI DATA TEMPLATE.GDT 10/8/20

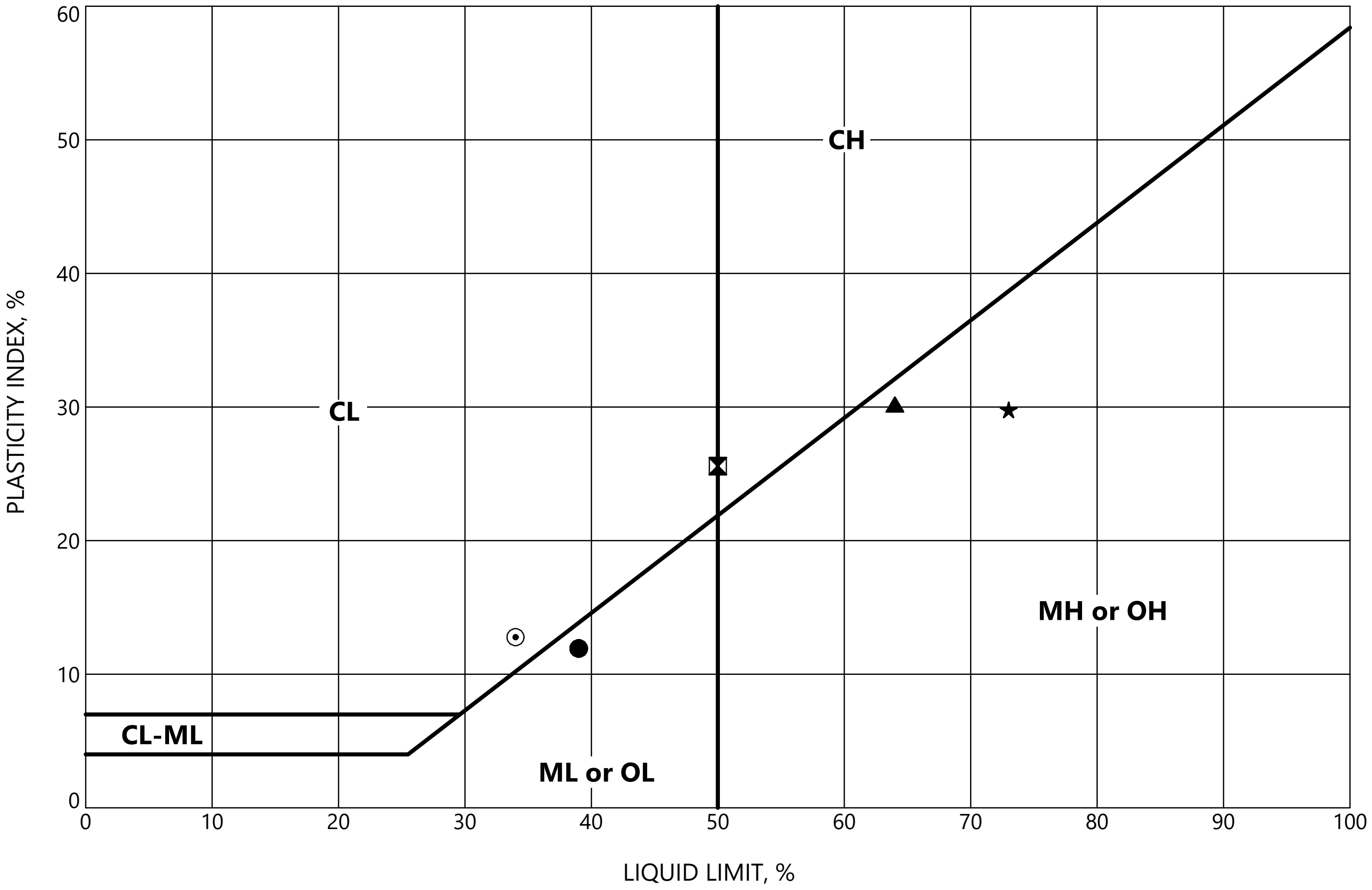
GRI

TEST PIT TP-3

OCT. 2020 JOB NO. 6334-A FIG. 8A

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

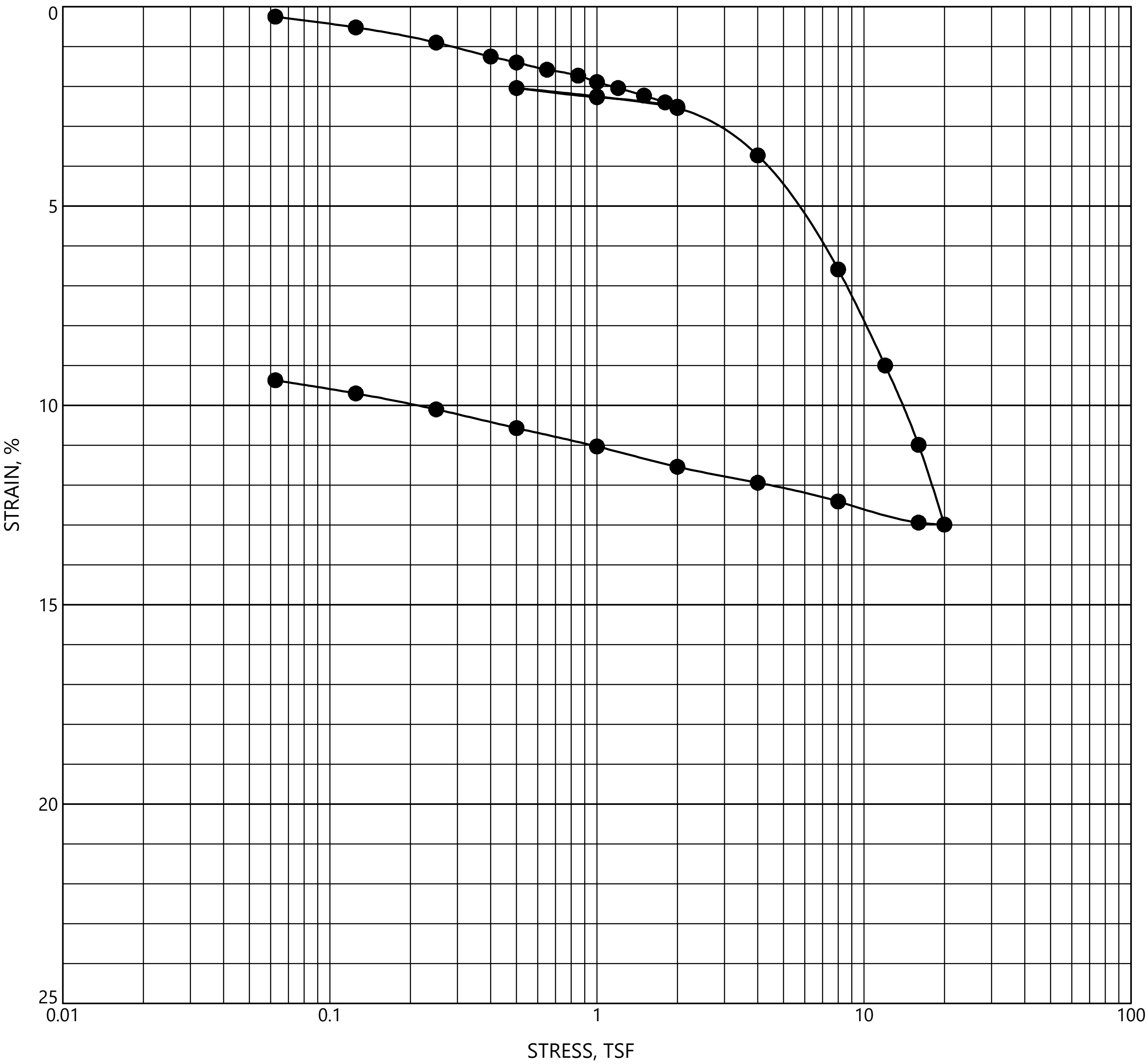


	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-1	S-2	4.0	SILT, some clay, trace to some fine-grained sand, brown mottled rust (Alluvium)	39	27	12	35
⧗	B-1	S-12	55.0	CLAY, some silt and fine- to medium-grained sand, blue-gray (Alluvium)	50	24	26	39
▲	B-3	S-4	10.0	SILT, some clay to clayey, some fine-grained sand, brown mottled rust (Alluvium)	64	34	30	53
★	B-3	S-9	25.0	BASALT, brown mottled rust, decomposed, extremely soft to very soft (R0 to R1) (Columbia River Basalt)	73	43	30	53
⊙	TP-1	S-2	3.5	Silty CLAY, trace to some fine-grained sand, dark brown to gray (Alluvium)	34	21	13	19

GRI

PLASTICITY CHART

OCT. 2020 JOB NO. 6334-A FIG. 9A



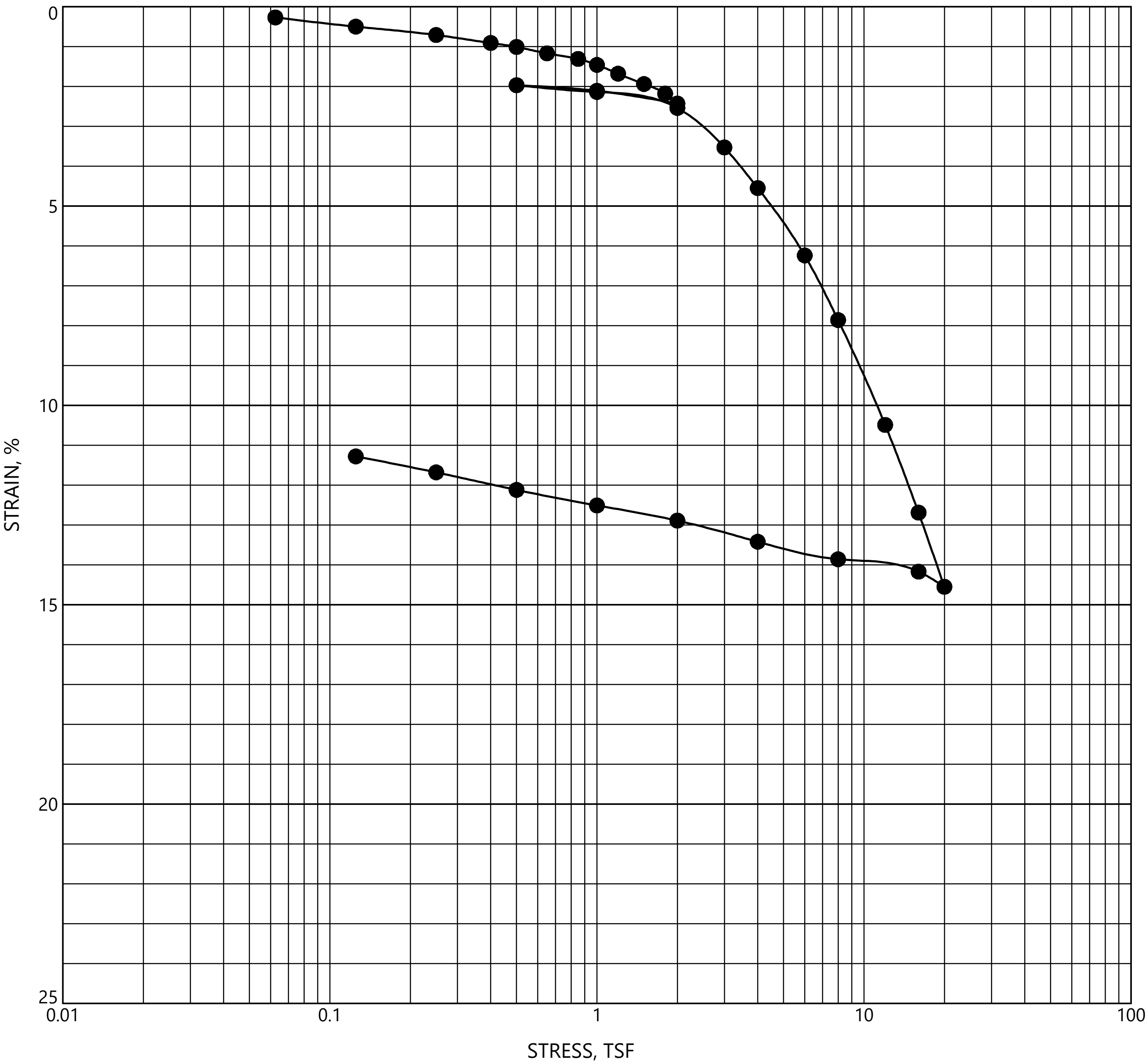
					Initial	
	Location	**Sample**	**Depth, ft**	**Classification**	**γ_d, pcf**	**MC, %**
●	B-1	S-1	2.8	SILT, some clay, trace to some fine-grained sand, brown mottled rust, stiff (Alluvium)	86	33

GRI

CONSOLIDATION TEST

OCT. 2020 JOB NO. 6334-A FIG. 10A

CONSOL STRAIN GRI-0 TO 25-1 PER PAGE GRI DATA TEMPLATE.GDT 10/8/20



	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-2	S-3	8.0	SILT, some fine-grained sand, trace clay, brown mottled rust, medium stiff to stiff (Alluvium)	84	37

GRI

CONSOLIDATION TEST

OCT. 2020 JOB NO. 6334-A FIG. 11A

CONSOL STRAIN GRI-0 TO 25-1 PER PAGE GRI DATA TEMPLATE.GDT 10/8/20

APPENDIX B

Site-Specific Seismic-Hazard Evaluation

APPENDIX B

SITE-SPECIFIC SEISMIC-HAZARD EVALUATION

B.1 INTRODUCTION

GRI completed a site-specific seismic-hazard evaluation for the proposed improvements to the North Clackamas Parks and Recreation District (NCPRD) Oak Lodge Community Project located in Milwaukie, Oregon. The purpose of the evaluation was to review the potential seismic hazards associated with regional and local seismicity. The site-specific seismic-hazard evaluation is intended to fulfill the requirements of amended Section 1803 of the 2019 Oregon Structural Specialty Code (OSSC) for special-occupancy structures, which references 2016 American Society of Civil Engineers (ASCE) 7-16 document, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16), for seismic design. Our site-specific seismic-hazard evaluation 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 detailed 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 and evaluation of subsurface data collected at and in the vicinity of 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) Engineering analyses based on the generalized subsurface profile and potential seismic sources resulting in conclusions and recommendations concerning:

    a) specific seismic events and characteristic earthquakes that might have a significant effect on the project site;

    b) the potential for seismic energy amplification and liquefaction or soil-strength loss at the site; and

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.

B.2 GEOLOGIC SETTTING

B.2.1 General

On a regional scale, the site is located in the northern end of the Willamette River Valley of the Puget-Willamette lowland trough of the Cascadia convergent tectonic system (Blakely et al., 2000). The lowland areas consist of broad, north-south-trending basins in the underlying geologic structure between the Coast Range to the west and the Cascade Mountains to the east. The lowland trough is characterized by alluvial plains with areas of buttes and terraces. The site is located approximately 100 kilometers inland from the rupture zone 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 and the overriding North American Plate, as shown on Figure 1B.

On a local scale, the site is located in the Portland Basin, a large, south-trending structural basin bounded by high-angle, northwest-trending, right-lateral, strike-slip faults considered to be seismogenic. The inferred trace of the Oatfield Fault crosses the southwestern portion of the site, see Figure 2. The geologic units in the area are shown on the Regional Geologic Map, Figure 2B. The distribution of nearby Quaternary faults is shown on the Local Fault Map, Figure 3B. Information regarding the continuity and potential activity of these faults is lacking due largely to the scale at which geologic mapping in the area has been conducted and the presence of thick, relatively young, basin-filling sediments that obscure underlying structural features. Active faults may be present within the basin, but clear stratigraphic and/or geophysical evidence regarding their location and extent is not presently available. Additional discussion regarding crustal faults is provided in the Local Crustal Event section below.

Because of the proximity of the site to the CSZ and its location within the Portland Basin, three distinctly different seismic sources contribute to the potential for damaging earthquake motions at the site. Two of these sources are associated with deep-seated tectonic activity related to the CSZ; the third is associated with movement on relatively shallow faults within and adjacent to the Portland Basin.

B.2.2 Subsurface and Geologic Conditions

Published geologic mapping indicates the site is mantled by Pleistocene-age Missoula flood deposits (which include silt, clay, sand, and gravel), with the Basalt of Sand Hollow member of the Wanapum Basalt (Columbia River Basalt Group of Miocene age) mapped at the ground surface depicted immediately adjacent to the east portion of the site (Wells et al., 2018).

B.3 SEISMICITY

B.3.1 General

The available information indicates the potential seismic sources that may affect the site can be grouped into three independent categories: *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, *subcrustal events* related to deformation and volume changes within the subducted mass of the Juan de Fuca Plate, and *local crustal events* associated with movement on shallow, local faults within and adjacent to the Portland Basin. Each of these sources is considered capable of producing damaging earthquakes in the Pacific Northwest. Based on our review of currently available information, we developed generalized design earthquakes for each of these categories. The design earthquakes are characterized by three important properties: size, location relative to the subject site, and the peak horizontal bedrock accelerations produced by the event. In this study, earthquake size is generally expressed by moment magnitude (M_W); location is expressed as the closest distance to the fault rupture, measured in kilometers; and peak horizontal bedrock accelerations are expressed in units of gravity (1 g = 32.2 feet/second^2 = 981 centimeters/second^2).

B.3.2 Cascadia Subduction Zone (CSZ) Event

Written Japanese tsunami records suggest a great CSZ earthquake occurred in January 1700 (Atwater et al., 2015). Geological studies suggest great megathrust earthquakes have occurred repeatedly in the past 7,000 years (Atwater et al., 1995; Clague, 1997; Goldfinger et al., 2003; and Kelsey et al., 2005), and geodetic studies (Hyndman and Wang, 1995; and Savage et al., 2000) indicate 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; and 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; and 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 earthquake (Satake et al., 1996; Atwater and Hemphill-Haley, 1997; and Clague et al., 2000). Published estimates of the probable maximum size of subduction-zone events range from M_W 8.3 to >M_W 9.

Numerous detailed studies of coastal subsidence, tsunamis, and turbidites yield a wide range of recurrence intervals, but the most complete records (>4,000 years) indicate intervals of about 350 years to 600 years between great earthquakes on the CSZ (Adams, 1990; Atwater and Hemphill-Haley, 1997; Witter, 1999; Clague et al., 2000; Kelsey et al., 2002; Kelsey et al., 2005; and Witter et al., 2003). Tsunami inundation in buried marshes along the Washington and Oregon coast and stratigraphic evidence from the Cascadia margin support these recurrence intervals (Kelsey et al., 2005; and Goldfinger et al., 2003). Goldfinger et al. (2003, 2012, and 2016) evaluated turbidite evidence for 20 earthquakes that ruptured the entire CSZ over the past 10,000 years and about 20 M_W 8 earthquakes that only ruptured along the southern portion of the CSZ and developed a model for recurrence of CSZ M_W 8 to M_W 9 earthquakes.

The U.S. Geological Survey (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 Figure 4B. As discussed in Petersen et al. (2014), the 2014 USGS mapping effort represents the 2014 CSZ source model with the full CSZ ruptures with moment magnitudes from M_W 8.6 to M_W 9.3 supplemented by partial ruptures with smaller magnitudes from M_W 8.0 to M_W 9.1. The partial ruptures were accounted for using a segmented model and unsegmented model. The magnitude-frequency distribution showing the contributions to the earthquake rates from each of the models and how the rates vary along the fault is presented on Figure 5B. In general, the earthquake rates along the CSZ are dominated by the full-characteristic ruptures, with one event in 526 years (M_W 8.6 to M_W 9.3 earthquakes likely occur more often than the smaller, segmented ruptures). Therefore, in our opinion, the CSZ event should be represented by an earthquake of M_W 9.0 at a focal depth of 30 kilometers and rupture distance of about 100 kilometers.

B.3.3 Subcrustal Event

There is no historical earthquake record of significant (i.e., >M_W 6.0) subcrustal, intraslab earthquakes in Oregon. Although both the Puget Sound and northern California regions have experienced many of these earthquakes in historical times, Wong (2005) hypothesizes that due to subduction-zone geometry, geophysical conditions, and local geology, Oregon may not be subject to intraslab earthquakes. In the Puget Sound area, these moderate to large earthquakes are deep (40 kilometers to 60 kilometers) and more than 200 kilometers from the deformation front of the subduction zone. Offshore along the northern California coast, the earthquakes are shallower (up to 40 kilometers) and located along the deformation front. Estimates of the probable size, location, and frequency of subcrustal events in Oregon are generally based on comparisons of the CSZ with active convergent-plate margins in other parts of the world and the historical seismic record for the region surrounding Puget Sound, where significant events known to have occurred within the subducting Juan de Fuca Plate have been recorded. The 1949, 1965, and 2001 documented subcrustal earthquakes in the Puget Sound area correspond to M_W 7.1, M_W 6.5, and M_W 6.8, respectively. Published estimates of the probable maximum size of these events range from M_W 7.0 to M_W 7.5. Published information regarding the location and geometry of the subducting zone indicates a focal depth of 50 kilometers is probable (Weaver and Shedlock, 1989). In our opinion, it is appropriate to represent the subcrustal event by a design earthquake of M_W 7.0 at a focal depth of 50 kilometers and rupture distance of 60 kilometers.

B.3.4 Local Crustal Event

Sudden crustal movements along relatively shallow, local faults in the Portland area, although rare, have been responsible for local crustal earthquakes. The precise relationship between specific earthquakes and individual faults is not well understood since few of the faults in the area are expressed at the ground surface and the foci of the observed earthquakes have not been located with precision. 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.

Based on fault mapping conducted by the USGS (2014 National Seismic Hazard Maps [NSHMs]), there are about five faults within 25 kilometers of the site the USGS identifies as contributing to the crustal seismic hazard: the Portland Hills Fault at about 1.9 kilometers, Bolton Fault at about 3.0 kilometers, Grant Butte Fault at about 9.6 kilometers, Sandy River fault zone at about 24.2 kilometers, and Helvetia Fault at about 24.7 kilometers. The USGS does not consider the Oatfield Fault to be a crustal fault source contributing to the overall seismic hazard at the site. Based on our review of the faults that contribute to the overall seismicity of the site, the Portland Hills Fault is the closest dominant crustal fault identified as a hazard to the site, with a magnitude of M_W 6.9.

B.4 CODE BACKGROUND AND DESIGN RESPONSE SPECTRUM

B.4.1 General

We understand the project will be designed in accordance with the 2019 OSSC, which references ASCE 7-16, for seismic design. A site-specific seismic-hazard evaluation was completed for the project to fulfill the requirements of amended Section 1803 of the 2019 OSSC for special-occupancy structures.

B.4.2 Code Background

The ASCE 7-16 seismic-hazard levels are based on a Risk-Targeted Maximum Considered Earthquake (MCE_R) with the intent of including the probability of structural collapse. Based on generalized building fragility curves, seismic design of a structure using the probabilistic MCE_R represents a targeted risk level of 1% in 50 years probability of collapse in the direction of maximum horizontal response. In general, these risk-targeted ground motions are developed by applying adjustment factors of directivity and risk coefficients to the 2% probability of exceedance in 50 years (2,475-year return-period hazard level) ground motions developed from the recently updated 2014 USGS probabilistic seismic-hazard maps. The risk-targeted probabilistic values are also subject to a deterministic check, which is computed from the models of earthquake sources and ground-motion propagation that form the basis of the 2014 USGS NSHMs. ASCE 7-16 defines the site-specific deterministic MCE_R ground motions in terms of 84th-percentile, 5%-damped response spectral acceleration in the direction of maximum horizontal response. The MCE_R ground motions are taken as the lesser of the probabilistic and deterministic spectral accelerations.

The ASCE methodology uses two bedrock spectral response mapped acceleration parameters, S_S and S_1, corresponding to periods around 0.2 second and 1.0 second to develop the MCE_R response spectrum. To establish the ground-surface MCE_R spectrum, these mapped bedrock spectral parameters are adjusted for site class using the short- and long-period site coefficients, F_a and F_v, in accordance with Section 11.4.3 of ASCE 7-16, which includes new seismic site coefficients to adjust the mapped values for soil properties.

B.4.3 Probabilistic and Deterministic Seismic Hazard Considerations

A Probabilistic Seismic Hazard Analysis (PSHA) estimates the seismic hazard at a specific location using a statistical evaluation of the potential earthquake sources in consideration and implicitly incorporates uncertainties in fault parameters such as location and geometry, slip rate and activity, probable magnitude, and potential ground motions. The potential variations in input parameters are considered with different assumptions and assigned relative weighting in a logic-tree format. The output from a PSHA includes a seismic-hazard curve showing the variation of a selected ground-motion parameter, such as peak ground acceleration (PGA), as a function of the annual frequency of exceedance (i.e., reciprocal of the average return period). The USGS provides probabilistic seismic-hazard maps for various probabilities of exceedance or hazard levels (i.e., specified probabilities of being exceeded over a given time period), which are updated about every six years. The results of a PSHA for a given hazard level are commonly referred to as a Uniform Hazard Spectrum (UHS) because all spectral ordinates have a uniform probability of exceedance in a given period of time.

The site-specific PSHA was derived based on the 2014 USGS Probabilistic NSHMs, using the interactive USGS Unified Hazard Tool (USGS, 2014). The 2014 NSHM for the Pacific Northwest includes significant changes to the seismic-source and ground-motion models from the previous 2008 version. A detailed discussion of the updated seismic-source models implemented in the 2014 NSHMs are provided in Petersen et al. (2014). The site-specific PSHA provides base ground motions for use in site response modeling or for consideration at the ground surface with the use of V_{s30} values at the site. The base motions for site response are commonly developed for Site Class B/C boundary conditions, which correspond to a shear-wave velocity of 2,500 feet/second. Table 1B summarizes the site-specific UHS values with a 2,475-year (2% in 50 years) obtained for the project site. These PSHA values represent the "geomean" spectral response accelerations.

Table 1B: 2014 USGS 2,475-YEAR
UHS SPECTRAL VALUES (B/C BOUNDARY CONDITION)

Spectral Acceleration, g	
Period, sec	2,475-Year Return Period
PGA	0.41
0.10	0.89
0.20	0.92
0.30	0.75
0.50	0.54
0.75	0.41
1.00	0.32
2.00	0.18
3.00	0.11
4.00	0.09
5.00	0.06

A Deterministic Seismic Hazard Analysis (DSHA) can be computed concurrently with the PSHA to evaluate the ground motions in accordance with Section 21.2.2 of ASCE 7-16. However, an exception was included in Section 21.2.2 of ASCE 7-16 allowing the deterministic analysis to be disregarded when the largest spectral response acceleration from the probabilistic ground motion is less than 1.2 F_a (i.e., F_a=1.0 for B/C boundary condition). Therefore, the deterministic analysis was not completed for the project since largest spectral response acceleration from the probabilistic ground motion, S_a = 0.92 g, is less than 1.2 g. Thus, the controlling target bedrock spectrum for design is defined by the probabilistic response spectral acceleration.

B.4.3 Mapped Acceleration Parameters

The S_S and S_1 mapped spectral response acceleration parameters for the site located at the approximate latitude and longitude coordinates of 45.4104° N and 122.6243° W are 0.87 g and 0.39 g, respectively, for Site Class B/C, or bedrock conditions.

B.4.4 Site Class

In accordance with Section 20.4 of ASCE 7-16, the site is classified as Site Class D, or a stiff-soil site, based on an average shear-wave velocity (field-measured shear-wave velocity [V_S] to a depth of about 28 feet and estimated V_s below 28 feet) of about 1,060 ft/second (about 325 m/second) in the upper 100 feet of the soil profile. However, our analysis identified a potential risk of seismically induced settlement at the site. In accordance with Section 20.3.1 of ASCE 7-16, sites with soils vulnerable to failure or collapse under seismic loading should be classified as Site Class F, which requires a site-specific site-response analysis unless the structure has a fundamental period of vibration less than or equal to 0.5 second. The design response spectrum for sites with structures having a fundamental period of less than or equal to 0.5 second can be derived using the non-liquefied subsurface profile and code-tabulated site coefficients. We anticipate the new structure will have a fundamental period of less than 0.5 second; therefore, the code-based Site Class D conditions are appropriate for design of the structure.

B.4.5 Site Coefficients

Due to the S1 acceleration parameter being greater than or equal to 0.2 g, Section 11.4.8 of ASCE 7-16 requires a ground-motion hazard analysis unless the seismic response coefficient C_s is determined in accordance with Exception 2 of Section 11.4.8 of ASCE 7 16. Assuming the seismic response coefficient C_s is determined in accordance with Exception 2 of Section 11.4.8 of ASCE 7-16, the site coefficients F_a and F_v were determined from code-tabulated values to be 1.15 and 1.92, respectively, in accordance with Section 11.4 of ASCE 7-16.

B.4.6 Recommended Seismic Design Parameters

The design-level response spectrum is calculated as two thirds of the ground-surface MCE_R spectrum. The recommended MCE_R- and design-level spectral-response parameters for Site Class D conditions are provided below in Table 2B.

Table 2B: RECOMMENDED SEISMIC DESIGN PARAMETERS (2019 OSSC/ASCE 7-16)

Seismic Parameter	Recommended Values*
Site Class	D
MCE_R 0.2-Sec Period Spectral Response Acceleration, S_{MS}	1.00 g

Seismic Parameter	Recommended Values*
MCE_R 1.0-Sec Period Spectral Response Acceleration, S_{M1}	0.74 g
Design-Level 0.2-Sec Period Spectral Response Acceleration, S_{DS}	0.67 g
Design-Level 1.0-Sec Period Spectral Response Acceleration, S_{D1}	0.49 g

Note: *Exception 2 of Section 11.4.8 should be considered when evaluating base shear calculations in Section 12.8.

B.4.7 Liquefaction/Cyclic Softening

Liquefaction is the process by which loose, saturated granular materials, such as clean sand and, to a somewhat lesser degree, non-plastic 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.

"Cyclic softening" is a term that describes a relatively gradual and progressive increase in shear strain with load cycles and is more common within fine-grained soils. Excess pore pressures may increase due to cyclic loading but will generally not approach the total overburden stress. Shear strains accumulate with additional loading cycles, but an abrupt or sudden decrease in shear stiffness is not typically expected. Settlement due to post-seismic consolidation can occur, particularly in lower-plasticity silts. Large shear strains can develop, and strength loss related to soil sensitivity may be a concern.

The potential for liquefaction and/or cyclic softening is typically estimated using a simplified method that compares the cyclic shear stresses induced by the earthquake (demand) to the cyclic shear strength of the soil available to resist these stresses (resistance). Estimates of seismically induced stresses are based on earthquake magnitude (M_W) and PGA. The cyclic resistance of soils is dependent on several factors, including the number of loading cycles, relative density, confining stress, plasticity, natural water content, stress history, age, depositional environment (fabric), and composition. The cyclic resistance of soils is evaluated using in-situ testing in conjunction with laboratory index testing but may also include monotonic and cyclic laboratory strength tests. For sand-like soils, the cyclic resistance is typically evaluated using SPT N-values or CPT tip-resistance values normalized for overburden pressures and corrected for factors that influence cyclic resistance, such as fines content. For clay-like soils, the cyclic resistance is typically evaluated using estimates of the undrained shear strength, overconsolidation ratio, and sensitivity or directly from cyclic laboratory tests.

The potential for liquefaction and/or cyclic softening at the site was evaluated using the simplified method based on procedures recommended by Idriss and Boulanger (2008) with subsequent revisions (2014). This method utilizes the PGA to predict the cyclic shear stresses induced by the earthquake. The USGS National Seismic Hazard Mapping Project (NSHMP) was used to determine the contributing earthquake magnitudes that represent the seismic exposure of the site for the Maximum Considered Earthquake Geometric Mean (MCE_G) hazard level. A crustal event on the Portland Hills Fault and an event on the CSZ were determined to represent the sources of seismic shaking.

For our evaluation, we considered an M_W 7.0 crustal earthquake at a distance of about 3 kilometers and M_W 9.0 CSZ earthquake at a distance of about 100 kilometers with code-level PGAs (PGA_M) of 0.48 g and 0.39 g, respectively. We assumed a groundwater depth of about 15 feet below the ground surface, which corresponds to the anticipated year-round sustained groundwater level at the site. The results of our evaluation indicate there is a potential that the interbedded layers of sand below the groundwater surface at the site could experience limited liquefaction, and zones of the low-plasticity sand and sandy silt below the groundwater surface at the site could experience limited cyclic softening. Our analysis indicates the potential for up to about 1 inch to 2 inches of seismically induced settlement that may occur during the earthquake and after earthquake shaking has ceased.

B.4.8 Other Seismic Hazards

Based on subsurface conditions and site topography, the risk of earthquake-induced liquefaction, cyclic softening, slope instability, and/or lateral spreading is low. The risk of damage by tsunami and/or seiche at the site is absent. The northwest-trending Oatfield fault, listed in the USGS Quaternary Fault and Fold Database, is mapped across the southwestern portion of the project site (USGS, 2020). The USGS does not consider the Oatfield fault to be an active contributing source in their current Probabilistic Seismic Hazard Analysis (PSHA). The USGS considers the Portland Hills Fault, located about 1.2 miles (1.9 kilometers) east of the project site, and the Bolton Fault, located approximately 1.8 miles (3 kilometers) southwest of the site, to be the closest crustal fault sources contributing to the overall seismic hazard at the site. The CSZ is mapped approximately 100 kilometers west of the site (Petersen et al., 2014).

B.5 CONCLUSIONS

Based on our review of the ASCE 7-16 design methodology we recommend for the project site at the approximate latitude and longitude coordinates of 45.4104° N and 122.6243° W , be designed using the mapped spectral acceleration parameters of S_S and S_1, are 0.87 g and 0.39 g, respectively. We recommend using the Site Class D design spectrum and tabulated code values for design of the proposed improvements.

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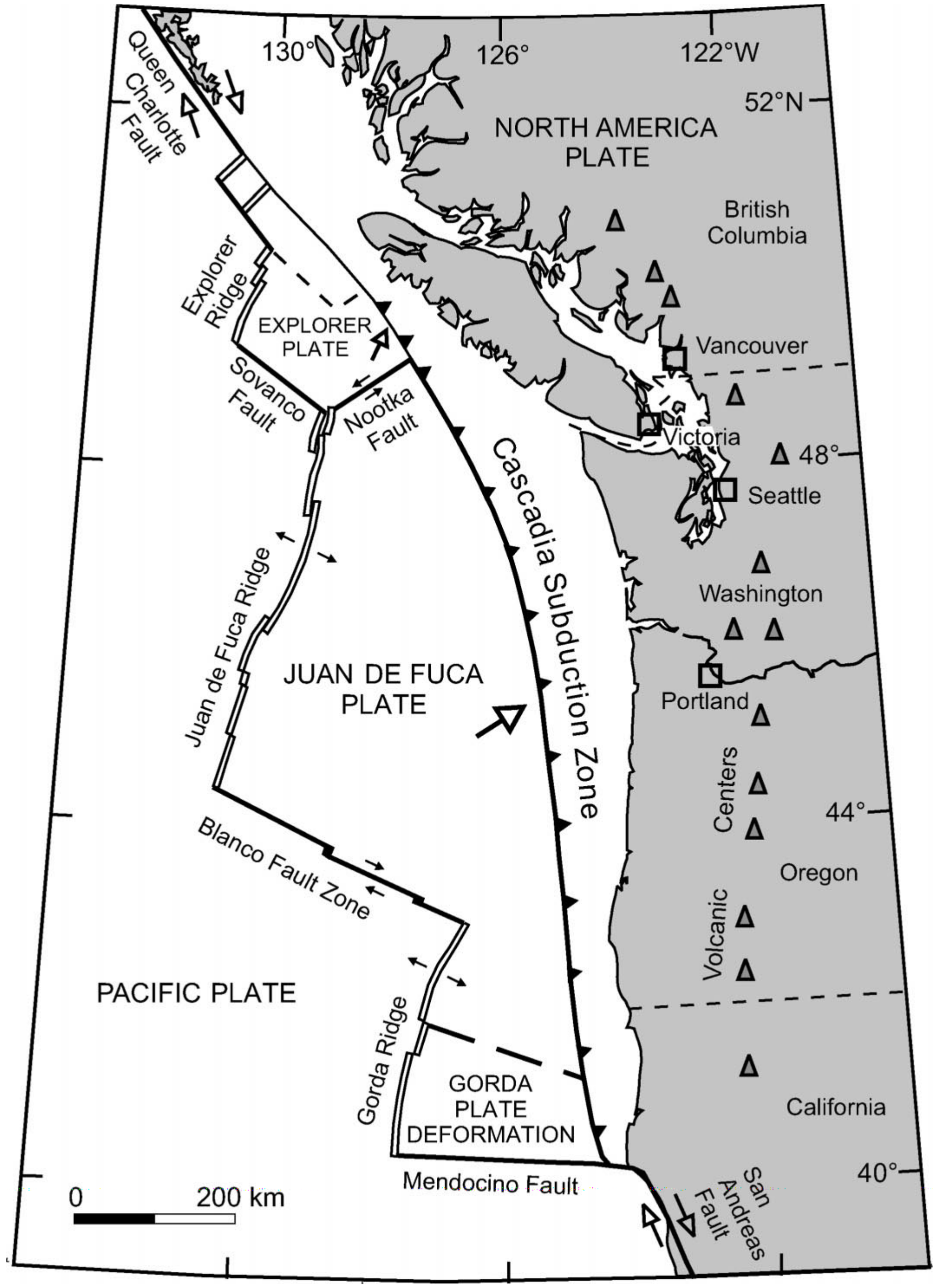
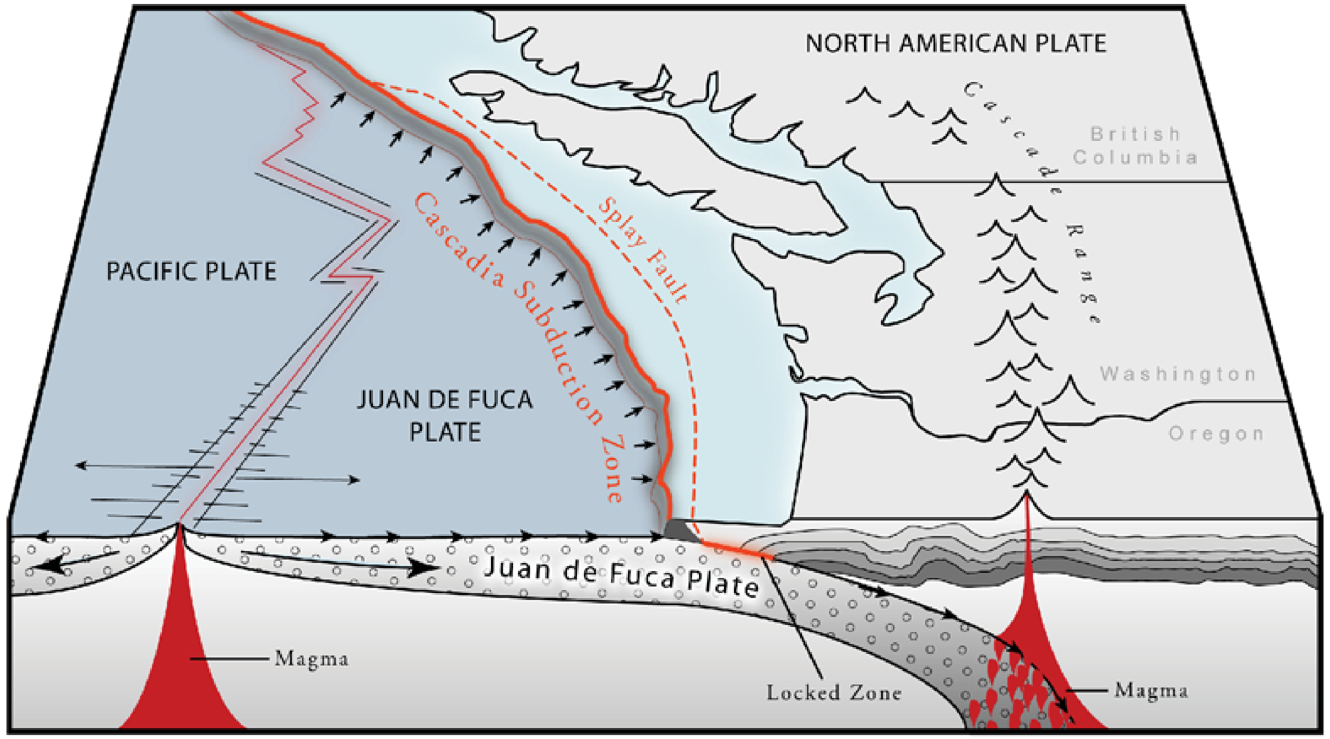


A) TECTONIC MAP OF PACIFIC NORTHWEST, SHOWING ORIENTATION AND EXTENT OF CASCADIA SUBDUCTION ZONE (MODIFIED FROM DRAGERT AND OTHERS, 1994)

Cascadia Subduction Zone Setting

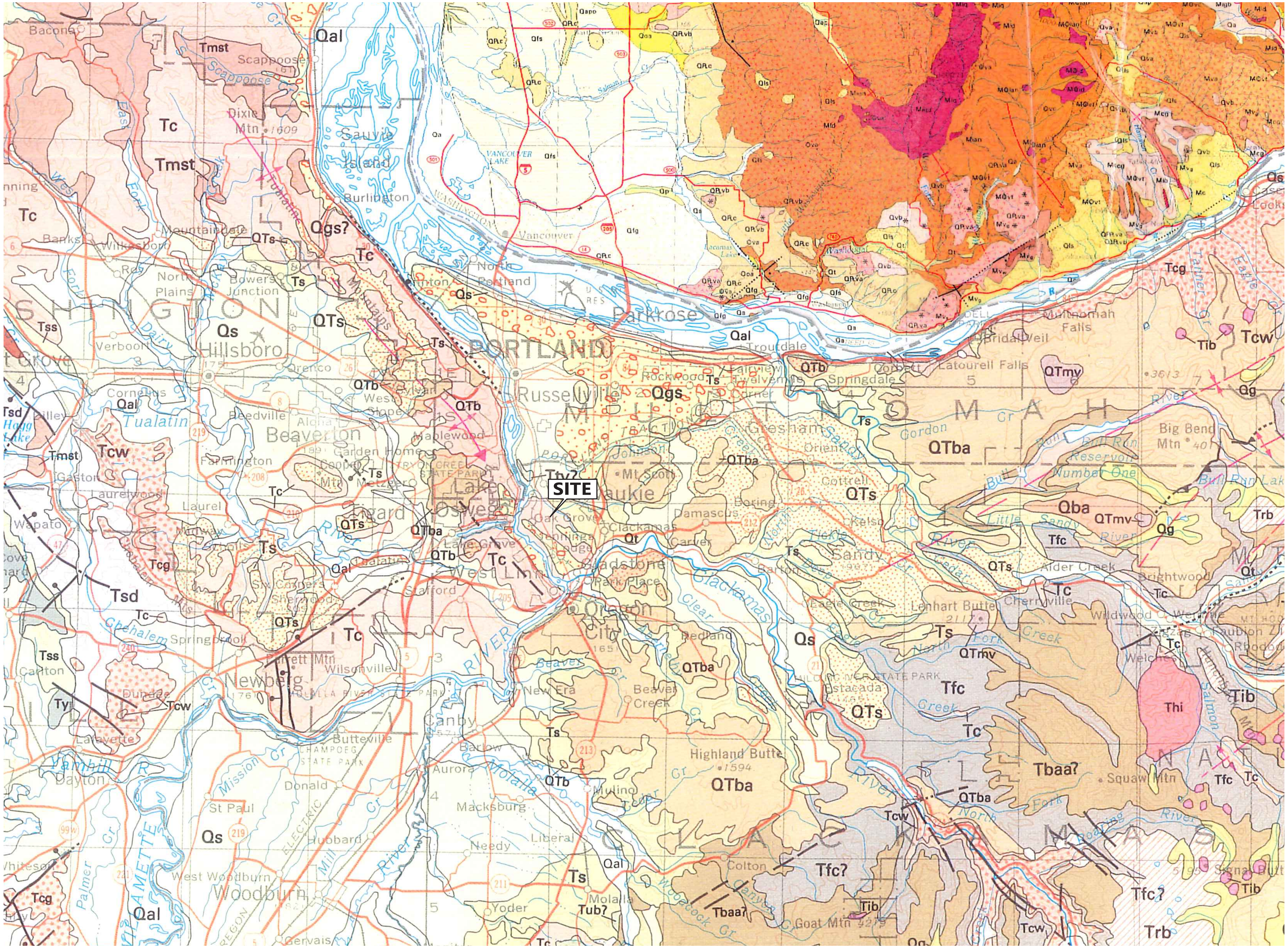


CASCADIA SUBDUCTION ZONE SETTING, TSUNAMI INUNDATION MAPS, OREGON DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRY, 2013

GRI

TECTONIC SETTING
SUMMARY

OCT. 2020 JOB NO. 6334-A FIG. 1B



- 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
- 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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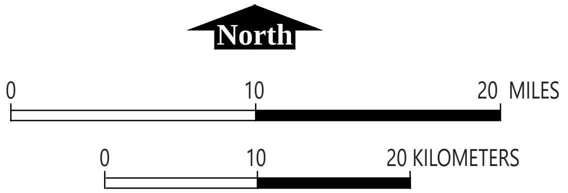

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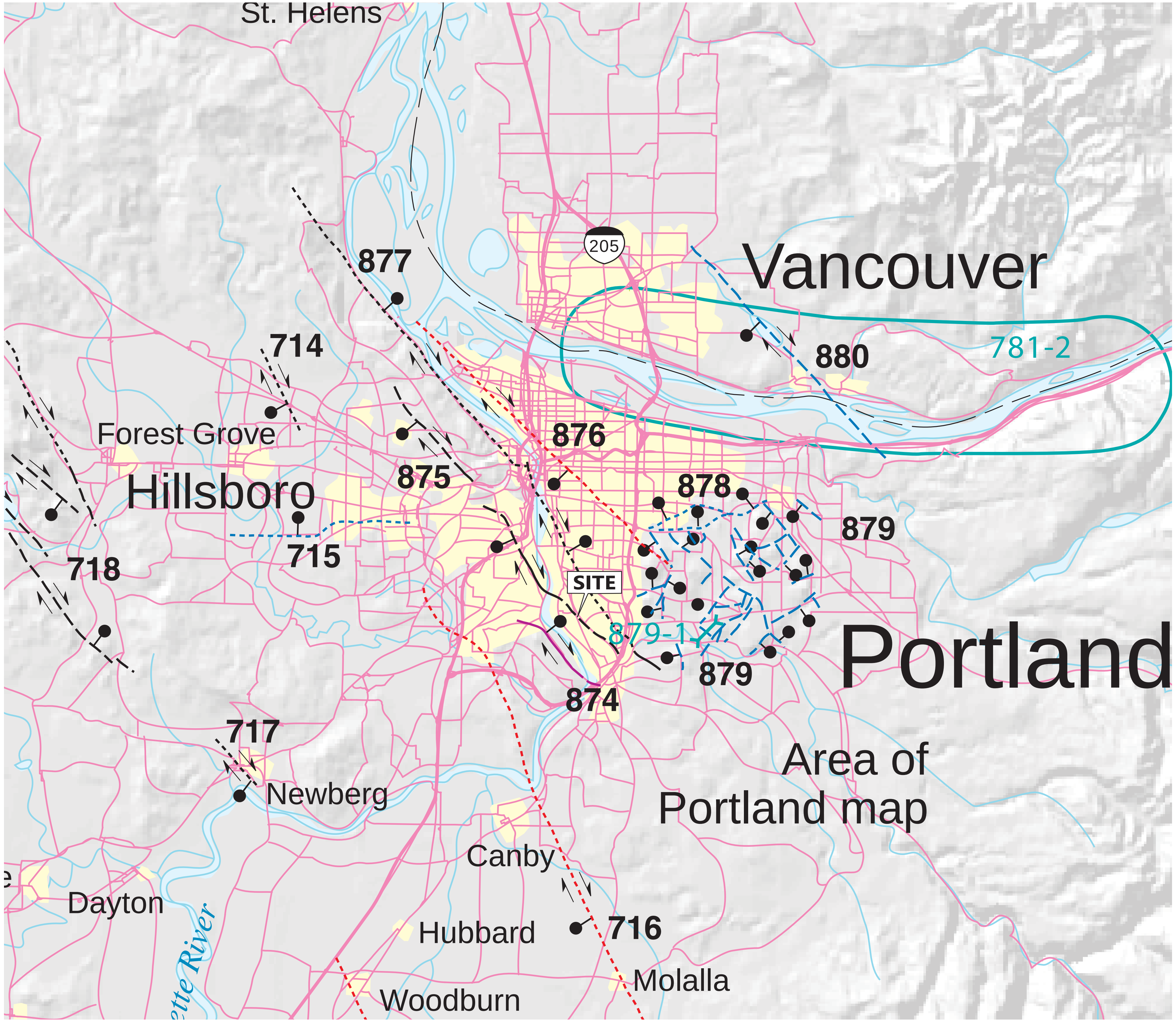
REGIONAL GEOLOGIC MAP

OCT. 2020 JOB NO. 6334-A FIG. 2B



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 FEATURES

- 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., AND OTHERS, 2003, MAP OF QUATERNARY FAULTS AND FOLDS IN OREGON, USGS OPEN FILE REPORT OFR-03-095.

North

0 5 10 MILES

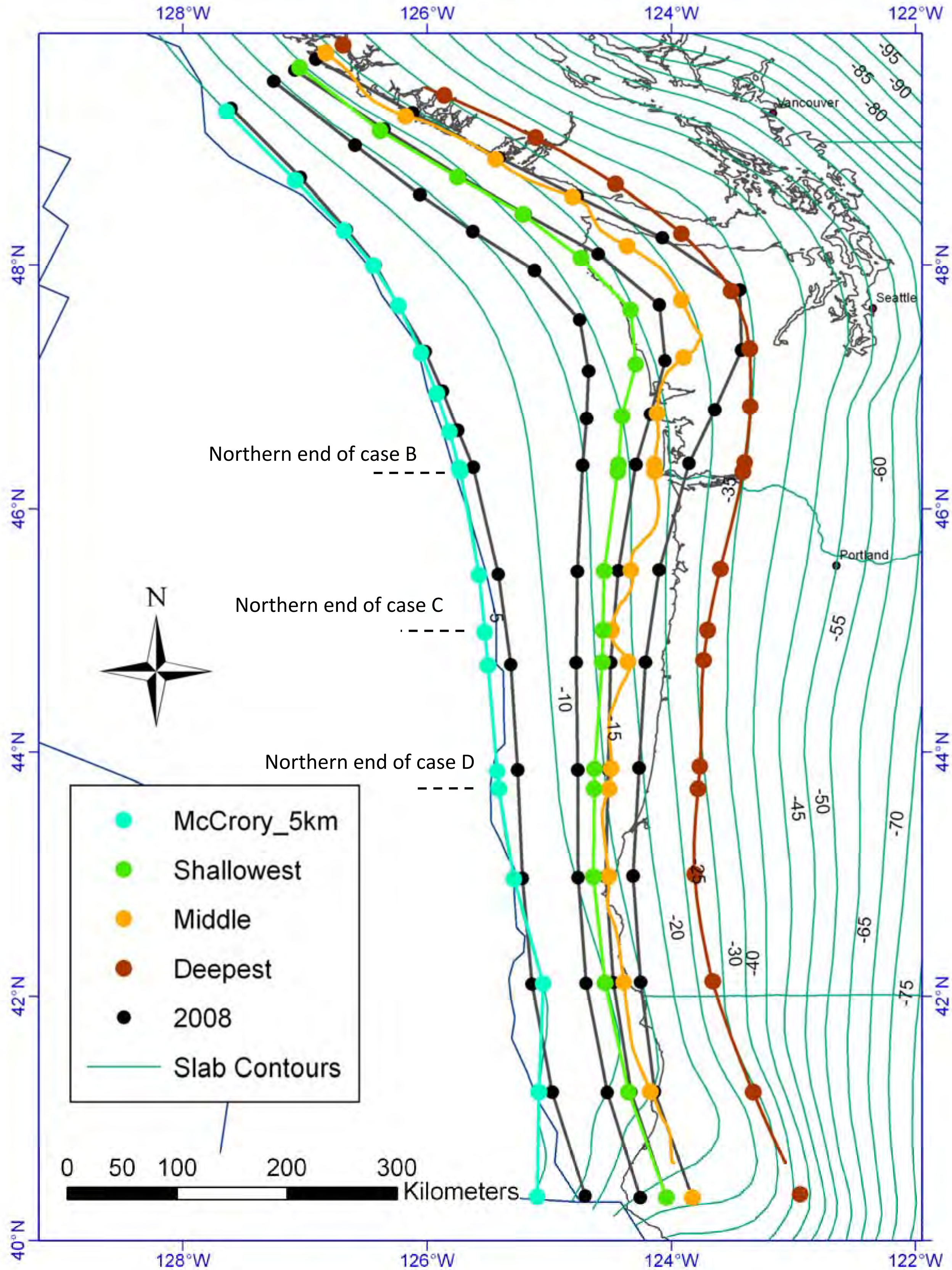
0 10 20 KILOMETERS

GRI

NORTH CLACKAMAS PARKS AND RECREATION DISTRICT
OAK LODGE COMMUNITY PROJECT

LOCAL FAULT MAP

OCT. 2020 JOB NO. 6334-A FIG. 3B



REFERENCE:
CHEN ET AL, 2014

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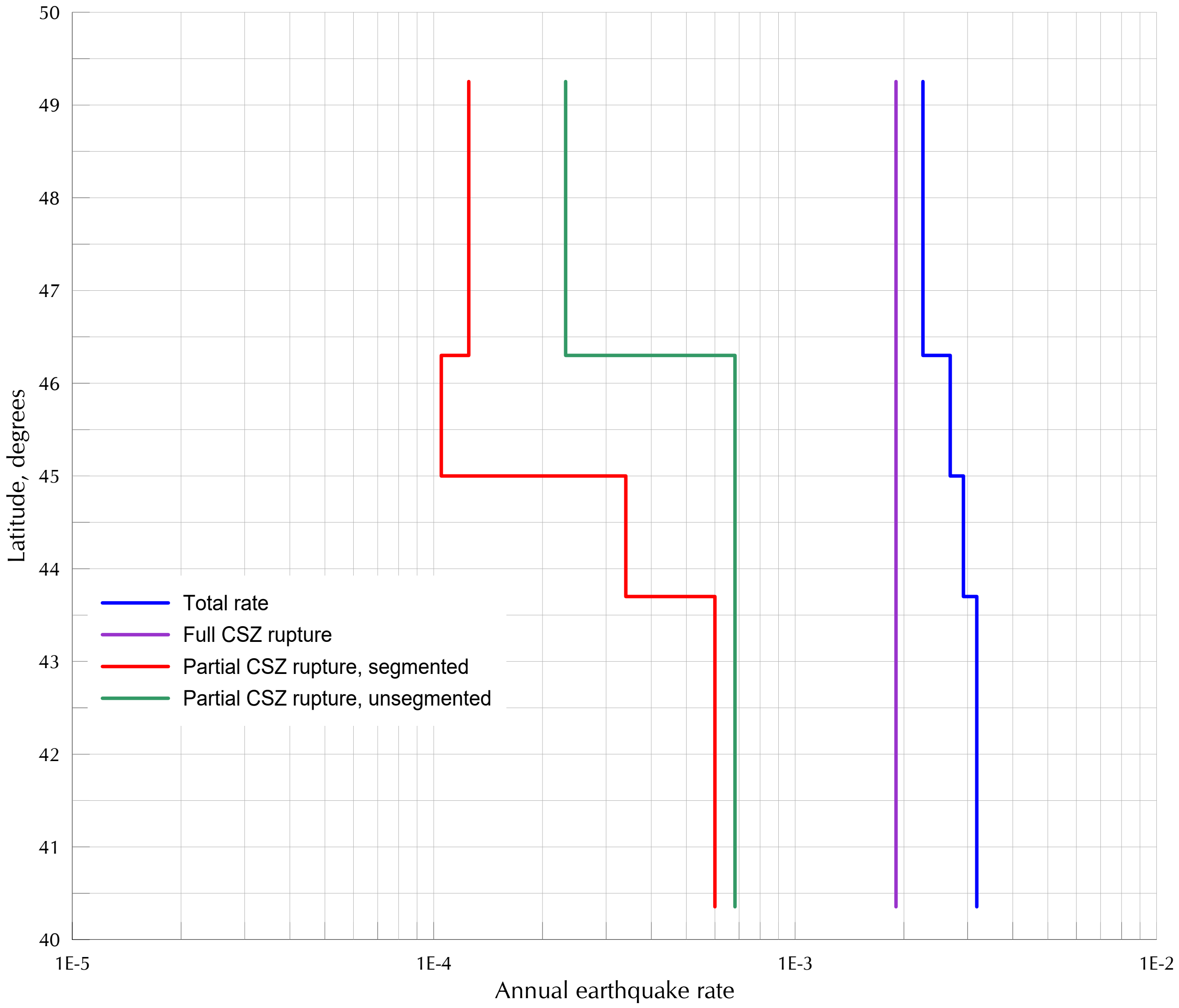
ASSUMED RUPTURE LOCATIONS
(CASCADIA SUBDUCTION ZONE)



REFERENCE:
CHEN ET AL, 2014
PETERSEN ET AL, 2014

GRI

VARIATION OF EARTHQUAKE RATES
(CASCADIA SUBDUCTION ZONE)

OCT. 2020 JOB NO. 6334-A FIG. 5B

APPENDIX C

Shear-Wave Refraction Microtremor (ReMi) Analysis Report

Report on

Geophysical Exploration
3811 SE Concord Rd.
Milwaukie, OR

Report Date: August 21, 2020

Prepared for:

GRI
9750 SW Nimbus Ave
Beaverton, OR 97008

Prepared by:

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Project No. 20212

1.0 INTRODUCTION

GRI is conducting a geotechnical investigation at the former site of the Concord Elementary School located at 3811 SE Concord Road in Milwaukie, Oregon. The site is currently occupied by North Clackamas Parks and Recreation. Geotechnical data are needed to help design new buildings for a community center and library that are proposed to be built at the site.

Earth Dynamics LLC completed geophysical explorations at the site at the request of Mr. Brian Cook of GRI. The primary goal of the geophysical explorations is to help map the subsurface geology at the site.

The geophysical exploration consists of two seismic refraction profiles. Seismic refraction data were acquired under the supervision of Mr. Daniel Lauer of Earth Dynamics LLC on August 5, 2020.

2.0 METHOD

2.1 Seismic Refraction

A seismic refraction exploration consists of measuring the time required for a seismic wave to travel from a seismic source to a receiving transducer. A sledgehammer, large weight dropped, or explosive device is typically used for the seismic source and vertical geophones are used as receiving transducers. A seismograph records signals from the geophones. By analyzing the arrival time of the seismic wave as a function of distance from the seismic source, the seismic velocities of the underlying soil/rock units and the depth to geologic contacts can be determined. The seismic refraction method requires that seismic sources be placed at each end of the geophone array. Intermediate and off end sources are also often used to increase resolution and penetration. Application of the method is limited to areas where seismic velocity increases or is constant with depth. The depth of penetration is typically one-quarter to one-third of the geophone array length, and lateral resolution is typically one-half of the geophone spacing.

The seismic refraction survey for this study was conducted using a Seismic Source 24-channel DAQ Link III seismograph. Data were acquired on two profiles. These Profiles are designated S1 and S2. Profile S1 is 276 feet-long with a geophone spacing of twelve feet. Profile S2 is 230 feet long with a geophone spacing of ten feet. A three-hundred-pound weight drop was used as the seismic source. The weight was dropped from approximately six to ten feet above the ground surface using a mini-excavator. The weight drop arrangement is shown in Photo 1.

Photo 1. Seismic Refraction using 300 # weight drop.

The seismic data are analyzed using SeisOpt@2D Ver. 5.0 by Optim Software to create two-dimensional profiles representing the seismic velocity of the subsurface materials. SeisOpt@2D uses a forward modeling global optimization technique. The technique consists of creating a finite element velocity model through which travel times are computed. The computed times are compared with the observed data. Thousands of iterations are completed to find the velocity model with the minimum travel time error. Comparison of the computed travel times to the measured values provides an indication of the validity of the model. Several velocity models are run using different grid resolution and depth values to obtain the best result for each data set. SeisOpt generates xyz data files that are input to Surfer® 16 for contouring, scaling, and data presentation. The SeisOpt modeling technique is generally superior to discrete layer modeling because lateral, as well as vertical variations can be resolved, and gradual increases in seismic velocity with depth can be quantified.

2.2 Location and Elevation Survey

The profiles were laid out using tape measures draped on the ground surface. Horizontal and vertical position data were obtained along each profile using a Trimble GEOXH 6000 GPS receiver. The position data were post-processed to increase the accuracy of the GPS positions. The reported horizontal and vertical accuracy of the post-processed position data is better than plus or minus one foot. Location data were recorded at the ends of each profile and at selected locations at the project site. Recorded GPS data for the profile end points are summarized in Table 1. The GPS data are displayed in decimal degrees Latitude and Longitude using the WGS 1984 datum.

Table 1. GPS Position Data for Seismic Refraction Profile endpoints. (WGS 1984).

Profile Location	Latitude	Longitude
S1 0’	45° 24.5893’N	122° 37.4793’W
S1 276’	45° 24.6128’N	122° 37.4245’W
S2 0’	45° 24.6226’N	122° 37.5037’W
S2 230’	45° 24.6454’N	122° 37.4609’W

3.0 RESULTS

A Google Earth image showing the approximate locations of the seismic refraction profiles is contained in Figure 1. The computed seismic velocity models with interpreted geology for the Seismic Refraction profiles are contained in Appendix A.

The acquired data for the seismic arrays are generally of very good quality. The data from several weight drops were stacked for each shot point to help increase the signal to noise ratio of the seismic waveform. Distinct first arrivals of compressional seismic waves were observed for all shot points.

Figure 1. Site image showing locations of geophysical profiles and mapped Oatfield Fault. (USGS)

4.0 DISCUSSION

The seismic velocity of soil and rock is a function of the density and elastic properties of the material. Therefore, variations in subsurface materials can be determined from analysis of the seismic velocity. Typical seismic velocities for various soil and rock types are listed in Table 4-1.

Table 4-1. Typical seismic velocities for geologic materials.

Description	Velocity (ft/sec)
Top Soils:	
Loose and dry	<1,000
Moist loamy or silty	1,000 – 1,300
Clayey	1,300 – 2,000
Wet Loam	2,400 – 2,600
Clay	2,900 – 8,800
Loose rock, talus (dry)	1,100 – 3,000
Sand (wet)	2,000 – 8,200
Gravel and cobble alluvium	4,900 – 7,400
Sandstone	4,600 – 14,000
Weathered and fractured rock	1,500 – 8,000
Intact Basalt or Diabase	7,000 – 16,000

The Seismic Refraction Profiles contained in Appendix A have been annotated with dashed lines showing possible geologic contacts. These contacts are based on steep gradients in the modelled seismic velocity.

Earth Dynamics LLC has completed numerous seismic refraction studies of basalt in the Pacific Northwest. In many cases the minimum velocity of un-weathered and fractured basalt is approximately 5,000 feet per second. The seismic velocity models in Appendix A indicate that a seismic interface is present at approximately 5,000 ft/s. This contact is indicated on each profile with a solid black line. The material with a seismic velocity greater than 5,000 ft/s may be basalt bedrock or dense sediment. Material with a seismic velocity range of 3,000 ft/s to 5,000 ft/s may be weathered (residual) basalt or sediments. Material with a seismic velocity less than 3,000 ft/s is likely sediment.

5.0 LIMITATIONS

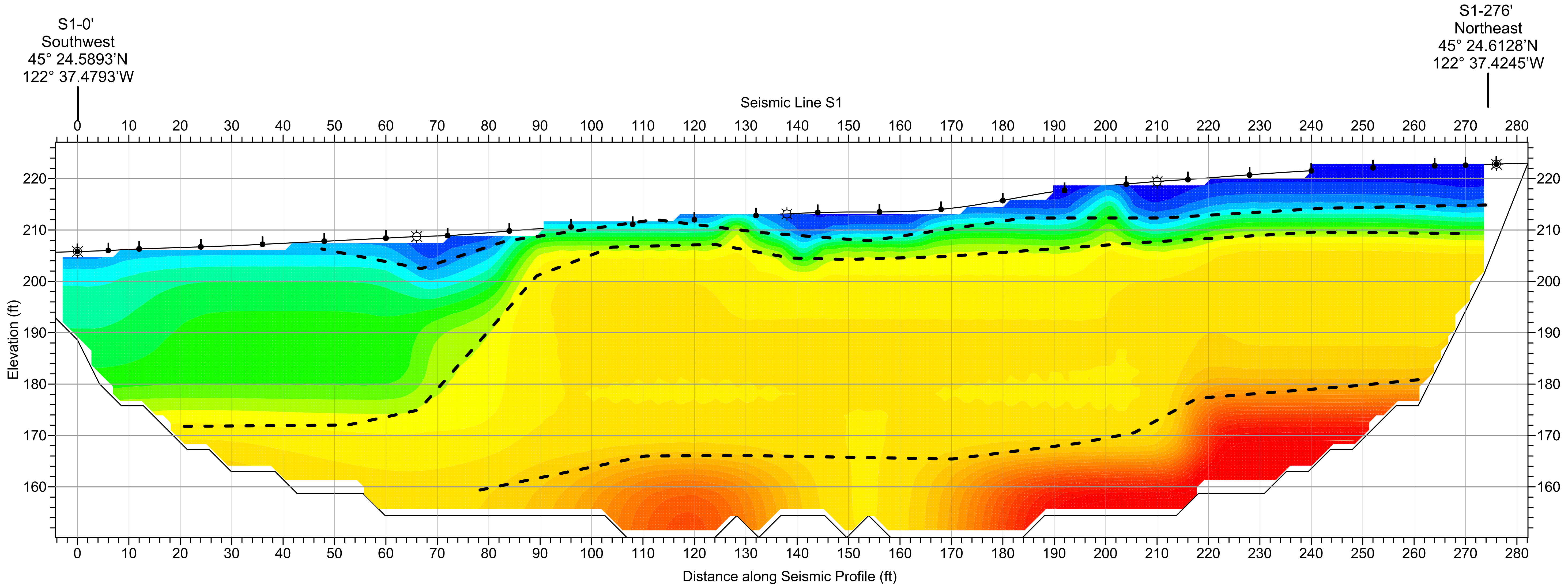
The inversion of seismic refraction data does not produce a unique model. Theoretically, there are an infinite number of models that will fit the data as well as the models presented in this report. Further, many geologic materials have similar seismic velocity. We have presented models and interpretations 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 as a result of decisions made or actions taken based upon this report.

RESPECTFULLY SUBMITTED;
EARTH DYNAMICS LLC

Daniel Lauer
Partner - Senior Geophysicist

APPENDIX A

Seismic Refraction Profile Models



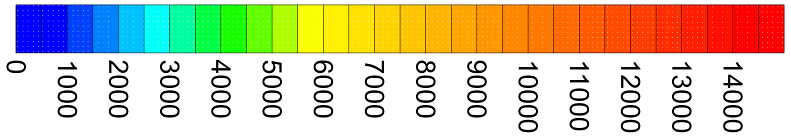
Dashed lines are interpreted seismic velocity interfaces

☼ Shot point

• Geophone

Horizontal Scale: 1" = 20'
Vertical Scale: 1" =20'

Horizontal Latitude and Longitude (WGS 1984) surveyed with
Trimble GeoXH 6000 GPS Receiver (Post-processed estimated horzontal accuracy < 1 foot)
Geophone elevations surveyed with level and rod tied to reported GPS elevation.



Compressional wave Velocity (ft/s)

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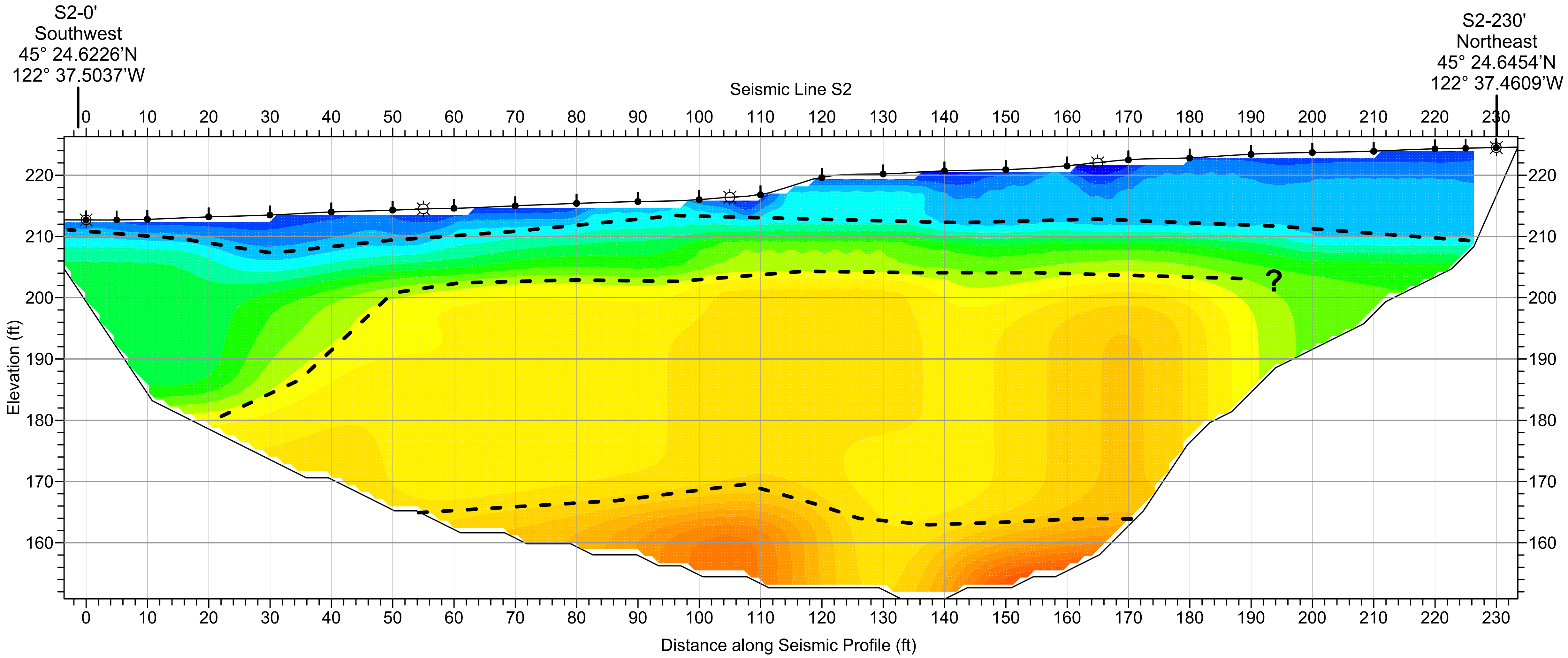
Seismic Profile S1 - SeisOpt2D Model

Job #: 20212	Date: 8/17/2020	Figure: A-1



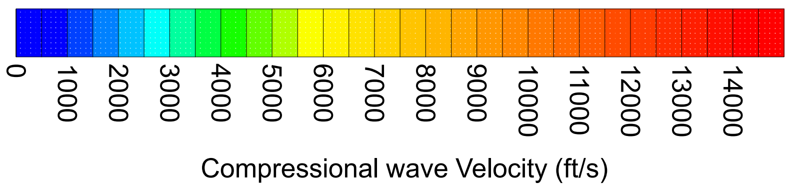
Dashed lines are interpreted seismic velocity interfaces

☼ Shot point

• Geophone

Horizontal Scale: 1" = 20'
Vertical Scale: 1" =20'

Horizontal Latitude and Longitude (WGS 1984) surveyed with
Trimble GeoXH 6000 GPS Receiver (Post-processed estimated horzontal accuracy < 1 foot)
Geophone elevations surveyed with level and rod tied to reported GPS elevation.



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Seismic Profile S2 - SeisOpt2D Model

Job #: 20212
Date: 8/17/2020
Figure: A-2