Geotechnical Engineering Report

Woodland Library
828 Goerig Street
Woodland, Washington

Prepared for:
Fort Vancouver Regional Libraries
1007 East Mill Plain Boulevard
Vancouver, Washington 98663

September 22, 2022
PBS Project 71959.000

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1 INTRODUCTION

1.1 General

This report presents results of PBS Engineering and Environmental Inc. (PBS) geotechnical engineering services for the proposed library located at 828 Goerig Street in Woodland, Washington (site). The general site location is shown on the Vicinity Map, Figure 1. The locations of PBS' explorations in relation to existing and proposed site features are shown on the Site Plan, Figure 2.

1.2 Purpose and Scope

The purpose of PBS' services was to develop geotechnical design and construction recommendations in support of the planned new library. This was accomplished by performing the following scope of services.

1.2.1 Literature and Records Review

PBS reviewed various published geologic maps of the area for information regarding geologic conditions and hazards at or near the site. PBS also reviewed previously completed reports for the project site and vicinity.

1.2.2 Subsurface Explorations

Five borings were advanced to depths ranging from approximately 11.5 to 36.5 feet below the existing ground surface (bgs) within the development footprint. The borings were logged and representative soil samples collected by a member of the PBS geotechnical engineering staff. The interpreted boring logs are presented as Figures A1 through A5 in Appendix A, Field Explorations.

PBS excavated two test pits within the proposed development footprint to depths of up to 9 feet bgs. The test pits were logged and representative soil samples collected by a member of the PBS geotechnical engineering staff. Interpreted test pit logs are included as Figures A6 and A7 in Appendix A, Field Explorations.

Two cone penetration tests (CPT) probes were advanced to depths of approximately 60 and 82 feet bgs. The CPT logs are presented as Figures A8 and A9 in Appendix A, Field Explorations. Shear wave velocities collected in CPT-1 are presented as Figure A10. The approximate boring, test pit, and CPT locations are shown on the Site Plan, Figure 2.

1.2.3 Field Infiltration Testing

Two cased-hole, falling-head field infiltration tests were completed in test pits TP-1 and TP-2 within the proposed development at a depth of 5 feet bgs. Infiltration testing was monitored by PBS geotechnical engineering staff.

1.2.4 Soils Testing

Soil samples were returned to our laboratory and classified in general accordance with the Unified Soil Classification System (ASTM D2487) and/or the Visual-Manual Procedure (ASTM D2488). Laboratory tests included natural moisture contents and grain-size analyses. Laboratory test results are included in the exploration logs in Appendix A, Field Explorations; and in Appendix B, Laboratory Testing.

1.2.5 Geotechnical Engineering Analysis

Data collected during the subsurface exploration, literature research, and testing were used to develop site-specific geotechnical design parameters and construction recommendations.

1.2.6 Report Preparation

This Geotechnical Engineering Report summarizes the results of our explorations, testing, and analyses, including information relating to the following:

- Field exploration logs and site plan showing approximate exploration locations
- Laboratory test results
- Infiltration test results
- Groundwater considerations
- Liquefaction potential
- Foundation alternatives
- Soil improvement alternatives
- Updated shallow foundation design recommendations:
    - Minimum embedment
    - Allowable bearing pressure
    - Estimated settlement (total and differential)
    - Sliding coefficient
- Earthwork and grading, cut, and fill recommendations:
    - Structural fill materials and preparation, and reuse of on-site soils
    - Utility trench excavation and backfill requirements
    - Temporary and permanent slope inclinations
    - Wet weather considerations
- Updated seismic design criteria in accordance with the 2018 International Building Code (IBC) with state of Washington amendments
- Pavement subgrade preparation recommendations
- Recommended asphalt concrete (AC) pavement sections

1.3 Project Understanding

PBS previously performed subsurface explorations at the site and presented the results and recommendations in a geotechnical engineering report dated April 11, 2017. PBS understands previous preliminary plans included development and construction of a one-story, approximately 10,000 square-foot, wood-frame building with slab-on-grade floors, as well as a new parking lot and book-drop access way.

When our previous geotechnical engineering report was completed in 2017, the proposed location of the new library was within the footprint of the existing funeral home that was to be demolished. Subsequent to the preparation of our report, the location of the new library building was moved east approximately 200 feet, to an area where no geotechnical explorations had been completed. Five geotechnical borings were previously completed at the site to depths of 11.5 to 36.5 feet bgs and presented in our 2017 report.

The extent of site grading is currently unknown; however, based on provided information, excavation to depths of up to 5 feet may be required. Based on our experience, we assume the proposed building loads will be less than 100 kips for columns, up to 3 kips per linear foot for walls, and less than about 250 pounds per square foot (psf) for slab-on-grade floors.

2 SITE CONDITIONS

2.1 Surface Description

The proposed project development will occupy the approximately 2.4-acre parcel in Woodland, Washington, just west of Interstate-5. The site is bounded on the east and north by Lakeshore Drive, to the west by an existing residence and Goerig Street, and to the south by commercial buildings and grassland. The northwestern portion of the parcel (southeast of the intersection of Lakeshore Drive and Goerig Street) was previously occupied by a two-story, wood-framed funeral home with a concrete walkway and asphalt concrete parking areas and drive aisles. The funeral home has been demolished. The majority of the site, east of the previous development, is undeveloped and vegetated with grass. The site is generally flat with elevations ranging from approximately 25 to 32 feet (NAVD88).

2.2 Geologic Setting

The project site is located within the northern portion of the Portland Basin. The Portland Basin and Willamette Valley form a tectonic depression within the physiographic province of the Puget-Willamette Lowland that separates the Cascade Range from the Coast Range and extends from the Puget Sound in Washington to Eugene, Oregon (Yeats et al., 1996). The Puget-Willamette Lowland is situated along the Cascadia Subduction Zone (CSZ) where oceanic rocks of the Juan de Fuca Plate are subducting beneath the North American Plate, resulting in deformation and uplift of the Coast Range and volcanism in the Cascade Range. Northwest-trending faults accommodating clockwise rotation of the North American Plate are found throughout the Puget-Willamette lowland (Brocher et al., 2017; USGS, 2022).

The greater Portland Basin is underlain by Columbia River Basalt Group (CRBG) flows consisting of numerous fine-grained volcanic eruptions between approximately 17 million years ago (Ma) and 6 Ma from fissures located in eastern Oregon, eastern Washington, and western Idaho (Beeson et al., 1991). These fissures released thousands of square kilometers, inundating areas east of the Cascade Range and entering western Oregon through a Miocene gap in the Cascade Range (present day Columbia Gorge) before reaching the ocean. Magmatic compositions of the CRBG allow the flows to be subdivided into distinct formations that can be further divided into members-based geochemical, paleomagnetic, and lithological properties.

Numerous northwest-trending faults govern the topography within the basin. Uplift and down dropping of crustal blocks have created topographic high points by offsetting regional-scale flood basalts and down dropping basement rocks, creating infilled depressions and sediment basins. Of these deposits, the Pliocene Troutdale Formation is the most widespread unit within the basin overlying CRBG volcanic flows. These friable to moderately strong conglomerates, with minor interbeds of sandstone and claystone, consist of well-rounded CRBG clasts and other exotic metamorphic and plutonic clasts. Younger quaternary deposits have accumulated above these conglomerates.

Cyclical Pleistocene cataclysmic floods deposited sediments and recarved the landscape within the Portland Basin more than 40 times over a 3,000-year timespan (Burns and Coe, 2012). As floodwaters entered the basin from the Columbia River Gorge, they slowed, depositing suspended sediments and bed loads. Topographic highpoints within the basin deflected floodwaters and generated areas that were scoured and eroded into older sediments and bedrock. These geomorphic features dominate the modern-day landscape and are indistinguishable within the Portland Basin LiDAR data (WADNR 2022; DOGAMI, 2022).

2.3 Local Geology

The site is mapped as underlain by unconsolidated silts, sands, and gravels (Qa) of the Holocene and Pleistocene, originating from the existing Lewis River (Evarts, 2004). These deposits are estimated to be around 250 feet thick and were placed later than the Lake Missoula flood deposits. They range from poorly graded to well-graded material and are commonly cross bedded. The alluvium also reportedly contains much reworked material from the eruptive products of Mount St. Helens.

2.4 Subsurface Conditions

The site was explored by drilling five borings, designated B-1 through B-5, to depths of 36.5 feet bgs, excavating two test pits, designated TP-1 and TP-2, to depths of 8 to 9 feet bgs, and advancing two cone penetrometer test (CPT) probes, designated CPT-1 and CPT-2, to depths of up 60 to 80 feet bgs. The drilling was performed by Western States Soil Conservation, Inc., of Hubbard, Oregon, using a truck-mounted drill rig and mud rotary drilling techniques. The test pit excavation was performed by Dan J. Fischer Excavation, Inc., of Forest Grove, Oregon, using a Case 580 Super N equipped with a 24-inch toothed bucket. The CPT probes were advanced using a 20-ton truck, mounted with a Vertek CPT 10 cm^2 electric seismic piezo cone, owned and operated by Geotechnical Explorations, Inc., of Keizer, Oregon.

PBS has summarized the subsurface units as follows:

SURFACE MATERIALS:	Approximately 2 to 6 inches of topsoil consisting of gray, poorly graded sand with silt and trace organics, such as roots, was encountered within both test pits and within borings B-3, B-4, and B-5. The silt exhibited low plasticity, and the sand was generally fine grained. Approximately 3 inches of concrete and 1.5 inches of AC over 10.5 inches of crushed rock aggregate base was encountered within borings B-1 and B-2.
FILL (GRAVEL):	Gravel fill was observed beneath the concrete in B-1. The gravel was medium dense, rounded to subangular, and encountered to a depth of 4 feet bgs. This material had an SPT N-value of 13, classifying it as medium dense.
UPPER SILT and SILTY SAND:	Brown silt with sand and silty was encountered beneath the pavement section and topsoil to depths of approximately 5 feet bgs in B-2 and B-4, TP-1 and TP-2. Moisture contents were between 35 and 45%, with over 20% fine-grained sand in the silt samples. SPT N-values ranged from 1 to 2, classifying it as very soft to soft. Silty sand, not silt, was observed to a depth of about 5 feet in both test pit TP-1 and TP-2.
SAND:	Loose to medium dense sand containing variable amounts of silt and gravel was encountered in all borings, test pits, and CPTs completed at the site. The sand was present to the depths explored in the borings, test pits, and CPT-2. The sand terminated at a depth of approximate 60 feet in CPT-1. The relative density generally increased to medium dense below about 25 feet bgs. SPT N-values ranged from 4 to 18. Laboratory testing indicated the sand generally contained less than 10% silt.
LOWER SILT:	An approximate 5-foot-thick zone of soft, fine-grained clay to silt was noted in the CPT logs at a depth of approximately 30 feet bgs. Medium stiff to stiff clay and silt was encountered below a depth of approximately 60 feet in CPT-1 to the depth explored.

2.5 Groundwater

Static groundwater was measured at a depth of approximately 7 feet in B-2 at the time of our exploration. Due to the use of mud-rotary drilling techniques groundwater was not measured in the other borings. Groundwater was not encountered to the 8-to-9-foot depth explored in the test pits. Pore pressure dissipation testing indicates static groundwater could be present at near 6 feet bgs, though it was not observed in the deeper, test pit excavations. Please note that groundwater levels can fluctuate during the year depending on climate, irrigation season, extended periods of precipitation, drought, and other factors.

2.6 Infiltration Testing

PBS completed two cased-hole, falling-head infiltration tests in test pits TP-1 and TP-2 at a depth of 5 feet bgs. The infiltration tests were conducted in general accordance with the Stormwater Management Manual for Western Washington (SWMMEW) procedures. The infiltration tests were conducted within 6-inch inside diameter casing. The casing was filled with water to achieve a minimum 1-foot-high column of water. After a period of saturation, the height of the water column in the pipe was then measured initially and at regular, timed intervals. Results of our field infiltration testing are presented in Table 1.

Table 1. Infiltration Test Results

Test Location	Depth (feet bgs)	Field Measured Infiltration Rate (in/hr)	Soil Classification
TP-1	5	28	SP
TP-2	5	28	SP

The infiltration rates listed in Table 1 are not permeability/hydraulic conductivities, but field-measured rates, and do not include correction factors related to long-term infiltration rates. The design engineer should determine the appropriate correction factors to account for the planned level of pre-treatment, maintenance, vegetation, siltation, etc. Field-measured infiltration rates are typically reduced by a minimum factor of 2 to 4 for use in design.

Soil types can vary significantly over relatively short distances. The infiltration rates noted above are representative of one discrete location and depth. Installation of infiltration systems within the layer the field rate was measured is considered critical to proper performance of the systems.

3 CONCLUSIONS AND RECOMMENDATIONS

3.1 Geotechnical Design Considerations

The project site is underlain by very soft, highly compressible silt and loose, saturated, potentially liquefiable silt and sand soils. Conventional foundation support on shallow spread footings is not feasible without some form of mitigation and consideration of risk. We have considered two options for foundation support, each having different levels of risk associated with damage during an earthquake.

The following sections provide a more detailed discussion of our analysis and recommendations.

3.2 Seismic Design Criteria

3.2.1 Liquefaction and Lateral Spreading

Liquefaction is defined as a decrease in the shear resistance of loose, saturated, cohesionless soil (e.g., sand) or low plasticity silt soils, due to the buildup of excess pore pressures generated during an earthquake. This results in a temporary transformation of the soil deposit into a viscous fluid. Liquefaction can result in ground settlement, foundation bearing capacity failure, and lateral spreading of ground.

Based on a review of the Washington Division of Geology and Earth Resources, the site is shown as having a moderate to high liquefaction hazard. Based on the soil types and relative density of site soils encountered in our explorations, our current opinion is that the risk of structurally damaging liquefaction settlement at the site is high.

Section 11.8.3 of ASCE 7-16 requires liquefaction evaluation for site-peak ground accelerations, earthquake magnitudes, and source characteristics consistent with the MCE-level peak ground acceleration (MCE_G). The liquefaction analyses were conducted using magnitude-acceleration-distance pairs consistent with the 2014 USGS deaggregation, which forms the basis for the 2018 IBC. A mean moment 9.0 was used for the Cascadia Subduction Zone earthquake. A peak ground surface acceleration (PGA_M) value of 0.46 g was used for the subduction zone earthquake. For the purpose of our liquefaction studies, we have assumed groundwater is present at a depth of approximately 9 feet bgs.

The potential for liquefaction at the site was evaluated using the simplified procedure as described by Idriss and Boulanger (2014). The simplified procedure compares the cyclic shear stresses (referred to as the CSR) induced within a soil profile during an earthquake with the ability of the soil to resist these stresses (referred to as the CRR). The stresses induced within the profile are estimated on the basis of earthquake magnitude and the accelerations within the profile. The ability of the soil to resist these stresses is based on its strength, as characterized by SPT N-values or CPT tip resistance normalized for overburden pressures and corrected for other factors such as fines content, i.e., silt and clay materials passing the US No. 200 sieve. The factor of safety against liquefaction can then be calculated as the CRR/CSR. As the factor of safety against liquefaction approaches 1.0, an increased risk of cyclic strength loss and liquefaction-induced settlement exists.

Our analysis indicates zones of silt and sand present below groundwater have factors of safety of less than 1.0 and are potentially liquefiable as a result of a code-based earthquake. We estimated the liquefaction-induced, free-field, liquefaction-induced settlement as a result of the code-based earthquake will be on the order of 4 to 5 inches. If liquefaction occurs at the site, lateral spreading of 12 to 18 inches could also occur.

3.2.2 Code-Based Seismic Design Parameters

The current seismic design criteria for this project are based on the 2018 IBC. Due to the potential for liquefaction of site soils, the site should be considered Site Class F. However, in accordance with ASCE 7-16, for structures having a fundamental period of less than 0.5 second, a site-response analysis is not required to determine the spectral accelerations of liquefied soils and seismic design parameters can be determined using the pre-liquefaction site class, Site Class D. The seismic design criteria, in accordance with the 2018 IBC, are summarized in Table 2.

Table 2. 2018 IBC Seismic Design Parameters

Parameter	Short Period	1 Second
Maximum Credible Earthquake Spectral Acceleration	S_s = 0.82 g	S_1 = 0.39 g
Site Class	D*	
Site Coefficient	F_a = 1.17	F_v = 1.91**
Adjusted Spectral Acceleration	S_{MS} = 0.96 g	S_{M1} = ***
Design Spectral Response Acceleration Parameters	S_{DS} = 0.64 g	S_{D1} = ***
MCE_G Peak Ground Acceleration	PGA = 0.37 g	
Site Amplification Factor at PGA	F_{PGA} = 1.23	
Site Modified Peak Ground Acceleration	PGA_M = 0.46 g	

g= Acceleration due to gravity

* Site Class D can be used if the fundamental period of the new structure is less than 0.5 second.

** This value of F_v shall only be used to calculate T_s

*** Site-specific site response analysis is not required for structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$.

3.3 Foundation Alternatives

The soils at the site present several challenges for support of the proposed new library. Potential seismically-induced settlement would affect both footings and slabs. Soft silt and clay soils create challenges for both bearing capacity and static settlement. We have developed two different foundation alternatives, which are discussed in the paragraphs that follow. These include:

- Creating a surficial, non-liquefiable "crust" with soil improvement and using shallow spread footings, or
- Using deep foundations.

Due to low soil bearing capacity and large estimated consolidation settlement under the estimated static foundation loads, shallow spread footings should only be used in conjunction with soil improvement. Shallow spread footings could be used in conjunction with preloading/surcharging of the building pad, but this would not reduce the risk of liquefaction settlement and resulting damage. Without soil improvement or first preloading the building pad area, shallow footings are not considered feasible.

3.4 Soil Improvement

The detailed design for soil improvement, such as stone columns or deep soil mixing (DSM), are typically completed by a design-build contractor. Depending on the settlement limitations of the new structures, improving all the potentially liquefiable soils at the site may not be necessary. The risk of surface manifestation of liquefaction can be reduced by a non-liquefiable layer at the surface (i.e., "crust"). Using the estimated ground surface acceleration associated with a design-level earthquake, methods developed by Ishihara (1985), and the liquefiable layer thickness at the site, the crust would need to be on the order of 30 feet thick. The current crust thickness is on the order of 9 feet thick. Using soil improvement techniques to increase the thickness of the crust would allow for the use of shallow spread footings. Because improving the crust does not improve the potentially liquefiable layers at greater depths, liquefaction settlement below the improved soil would probably still occur.

3.4.1 Stone Columns

Installation of stone columns is a common method to mitigate liquefaction. Stone columns incorporate a vibratory probe that is advanced to the target depth, with the void created filled with compacted crushed rock as the probe is extracted, creating a series of stone columns. Advancing the probe as it vibrates can densify loose cohesionless sand, while the replacement with crushed rock acts to improve soft, fine-grained soils that cannot be densified due to their fine-grained nature by reinforcing them with better materials. Stone columns also provide a path for faster dissipation of excess pore water pressures during earthquake events, further reducing liquefaction potential.

Depending on the application, stone columns can be 2 to 4 feet in diameter and installed in a grid at about 6 to 10 feet on-center. The actual diameter and spacing is typically determined by a specialty subcontractor, with the design reviewed by the project geotechnical engineer. We recommend stone columns extend to depths of at least 30 feet bgs or deeper. The extent beyond the intended area of improvement should be approximately one-half the depth of improvement. This would correspond to approximately 15 feet beyond the edge of footings. Stone columns can be used in conjunction with appropriately designed building foundation systems, including spread footings and mats.

3.4.2 Deep Soil Mixing

As an alternative to the stone columns, a method of mixing cement into the subsurface soils may be used to form columns or walls of cement-amended soils. Using this methodology, either dry or wet cement is injected into the ground with a series of paddles/blades. The paddles rotate during installation creating a generally uniform column of cement-amended soil, which provides greatly increased allowable bearing pressures. The building loads are then supported on shallow foundations resting on the amended soil. In addition, if the columns are installed in an overlapping or touching linear array, the line of columns provides significant shear resistance to lateral soil loads. Often, the linear arrays are arranged in a box pattern forming a series of boxes, or cells, across the site. Experience has shown that the native soil retained in the box pattern has a reduced risk of liquefaction.

Soil mixing would incorporate 2- to 3-foot diameter columns installed in an overlapping pattern having a compressive strength of about 200 pounds per square inch (psi). Treatment area ratios can range from 10 to 30% or more.

3.5 Shallow Footings on a Non-Liquefiable Crust Created with Soil Improvement

The risk of surface manifestation of liquefaction can be reduced by a non-liquefiable layer at the surface (i.e., "crust"). Using the estimated ground surface acceleration associated with a design-level earthquake, methods developed by Ishihara (1985), and the liquefiable layer thickness at the site, the crust would need to be on the order of 30 feet thick. The current crust thickness is on the order of 9 feet thick. Using soil improvement techniques to increase the thickness of the crust to a depth of approximately 30 feet would allow for the use of shallow spread footings. Because improving the crust does not improve the potentially liquefiable layers at greater depths, liquefaction settlement on the order of 2 to 3 inches would probably still occur.

Additionally, we recommend all footings be connected with grade beams. Specific recommendations for design and construction of both footings and grade beams are included in the following sections.

3.5.1 Minimum Footing Widths/Design Bearing Pressure

Continuous wall and spread footings should be at least 18 and 24 inches wide, respectively. The design allowable bearing pressure will be determined based on the size and spacing of stone columns, but will not likely be less than 2,500 psf. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads. For footings supported on soil improved with stone columns, allowable bearing pressures may be increased by one-third for seismic and wind.

Footings will settle in response to column and wall loads. Based on our evaluation of the subsurface conditions and our analysis, we estimate post-construction settlement will be less than 1 inch for the column and perimeter foundation loads. Differential settlement will be on the order of one-half of the total settlement. The magnitude of seismic settlement will be a function of the soil improvement design and method.

3.5.2 Footing Embedment Depths

PBS recommends that all footings be founded a minimum of 18 inches below the lowest adjacent grade. The footings should be founded below an imaginary line projecting upward at a 1H:1V (horizontal to vertical) slope from the base of any adjacent, parallel utility trenches or deeper excavations.

3.5.3 Footing Preparation

Excavations for footings should be carefully prepared to a neat and undisturbed state. A representative from PBS should confirm suitable bearing conditions and evaluate all exposed footing subgrades. Observations should also confirm that loose or soft materials have been removed from new footing excavations and concrete slab-on-grade areas. Localized deepening of footing excavations may be required to penetrate loose, wet, or deleterious materials.

PBS recommends a layer of compacted, crushed rock be placed over the footing subgrades to help protect them from disturbance due to foot traffic and the elements. The footing subgrade should be in a dense or stiff condition prior to pouring concrete. Based on our experience, approximately 4 inches of compacted crushed rock will be suitable beneath the footings.

3.5.4 Lateral Resistance

Lateral loads can be resisted by passive earth pressure on the sides of footings and grade beams, and by friction at the base of the footings. A passive earth pressure of 150 pounds per cubic foot (pcf) may be used for footings confined by native soils and new structural fills. The allowable passive pressure has been reduced by a factor of two to account for the large amount of deformation required to mobilize full passive resistance. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent unpaved areas should not be considered when calculating passive resistance. For footings supported on native soils or new structural fills, use a coefficient of friction equal to 0.35 when calculating resistance to sliding. These values do not include a factor of safety (FS).

3.5.5 Grade Beams

Grade beams are not intended to vertically support column footings, but to help hold the facility structure together during a design-level earthquake and reduce the impacts of lateral spreading. Grade beams between footings should be designed in accordance with the requirements of section 1810.3.12 of the 2018 IBC.

For lateral spreading, grade beams should be designed to resist the force on the perpendicular grade beam and/or perimeter foundation. The force acting on the perpendicular foundation will include a passive pressure (triangular distribution) from an equivalent fluid unit weight of 500 pcf and friction on the base of the footing/grade beam using a friction coefficient of 0.35.

3.6 Deep Foundations

The impacts from post-earthquake settlement and static settlement can be reduced by supporting the structure on piles. Piles will penetrate through the potentially liquefiable soils and derive their support from dense sand and/or gravel. We recommend that pile foundations for the proposed building, if used, consist of driven displacement piles such as closed-end steel pipe or driven grout piles. Supporting building columns on piles will provide support for the structure during an earthquake, but will not provide support to the slab-on-grade floors.

Frequently, structures founded on piles do not have structurally supported floor slabs. Therefore, the floor slabs and under-slab utilities will likely crack and settle significantly during the design-level earthquake. Consequently, major floor slab repair, as well as repair of major under-slab utilities, will likely be necessary after a design-level earthquake. If structural fill is planned in the building pad area, will additional slab settlement will occur due to increased pressure. Floor slabs can be structurally supported, but this can be costly, particularly for structures with large footprints.

Advantages of pile foundations include:

- Ability to support the structure when penetrating soft soils
- No significant static and seismically induced foundation settlement

Disadvantages of pile foundations include:

- Likely major repair of the floor slab and under-slab utilities after a design-level earthquake
- Some risk of slab settlement
- Requires specialty construction equipment and experienced specialty contractor
- High cost

Boring explorations for our preliminary evaluation and subsequent CPT explorations did not encounter a suitable bearing stratum. Although non-liquefiable soils were generally encountered below a depth of 60 feet bgs, piles would likely need to penetrate 30 feet or more into those soils and still may not provide sufficient resistance to lateral spreading. Our current understanding is that soil improvement is being considered for support of the new structure at the site.

3.7 Floor Slabs

Without soil improvement that extends to the full depth of potentially liquefiable soils, or structural support of the building slab on piles that derive their capacity below potentially liquefiable soils, some damage and associated repair of the building slab should be anticipated following a code-based earthquake.

For static conditions, satisfactory subgrade support for building floor slabs can be obtained from the near-surface silt and sand subgrades prepared in accordance with our recommendations presented in the Site Preparation, Wet/Freezing Weather and Wet Soil Conditions, and Imported Granular Materials sections of this report. A minimum 12-inch-thick layer of imported granular material should be placed and compacted over the prepared (compacted) subgrade. Imported granular material should be composed of crushed rock or crushed gravel that is relatively well graded between coarse and fine, contains no deleterious materials, has a maximum particle size of 1 inch, and has less than 5% by dry weight passing the US Standard No. 200 Sieve.

Floor slabs supported on a compacted subgrade and base course prepared in accordance with the preceding recommendations, may be designed using a modulus of subgrade reaction (k) of 100 pounds per cubic inch (pci) for unimproved soils or 175 pci for improved soils with a 2-foot-thick load transfer platform of compacted crushed rock.

3.8 Ground Moisture

3.8.1 General

The perimeter ground surface and hard-scape should be sloped to drain away from all structures and away from adjacent slopes. Gutters should be tight-lined to a suitable discharge and maintained as free-flowing. All crawl spaces should be adequately ventilated and sloped to drain to a suitable, exterior discharge.

3.8.2 Perimeter Footing Drains

Due to the relatively low permeability of site soils and the potential for perched groundwater at the site, we recommend perimeter foundation drains be installed around all proposed structures.

The foundation subdrainage system should include a minimum 4-inch diameter perforated pipe in a drain rock envelope. A non-woven geotextile filter fabric, such as Mirafi 140N or equivalent, should be used to completely wrap the drain rock envelope, separating it from the native soil and footing backfill materials. The invert of the perimeter drain lines should be placed approximately at the bottom of footing elevation. Also, the subdrainage system should be sealed at the ground surface. The perforated subdrainage pipe should be laid to drain by gravity into a non-perforated solid pipe and finally connected to the site drainage stem at a suitable location. Water from downspouts and surface water should be independently collected and routed to a storm sewer or other positive outlet. This water must not be allowed to enter the bearing soils.

3.8.3 Vapor Flow Retarder

A continuous, impervious barrier must be installed over the ground surface in the crawl space and under slabs of all structures. Barriers should be installed per the manufacturer’s recommendations.

3.9 Temporary and Permanent Slopes

All temporary cut slopes should be excavated with a smooth-bucket excavator, with the slope surface repaired if disturbed. In addition, upslope surface runoff should be rerouted to not run down the face of the slopes. Equipment should not be allowed to induce vibration or infiltrate water above the slopes, and no surcharges are allowed within 25 feet of the slope crest.

Permanent cut and fill slopes up to 10 feet high can be inclined at 2H:1V in medium dense or better silty sand and sand or compacted structural fill. If slow seepage is present, use of a rock blanket or a suitably revegetated, reinforced erosion control blanket may be required. PBS should be consulted if seepage is present; additional erosion control measures, such as additional drainage elements, and/or flatter slopes, may also be required. Exposed soils that are soft or loose may also require these measures. Fill slopes should be over-built and cut back into compacted structural fill at the design inclination using a smooth-bucket excavator. Erosion control is critical to maintaining slopes.

3.10 Pavement Design

The provided pavement recommendations were developed using the American Association of State Highway and Transportation Officials (AASHTO) design methods and references the associated Washington Department of Transportation (WSDOT) specifications for construction. Our evaluation considered a maximum of one truck per day for a 20-year design life.

The minimum recommended pavement section thicknesses are provided in Table 3. Depending on weather conditions at the time of construction, and due to the presence of soft silt at the surface in some areas of the site, a thicker aggregate base course section could be required to support construction traffic during preparation and placement of the pavement section.

Table 3. Minimum AC Pavement Sections

Traffic Loading	AC (inches)	Base Course (inches)	Subgrade
Pull-in Car Parking Only	3	9	Dense/stiff subgrade as verified by PBS personnel*
Drive Lanes and Access Roads	3.5	9	

* Subgrade must pass proofroll

The asphalt cement binder should be selected following WSDOT SS 9-02.1(4) – Performance Graded Asphalt Binder. The AC should consist of ½-inch hot mix asphalt (HMA) with a maximum lift thickness of 3 inches. The AC should conform to WSDOT SS 5-04.3(7)A – Mix Design, WSDOT SS 9-03.8(2) – HMA Test Requirements, and WSDOT SS 9-03.8(6) – HMA Proportions of Materials. The AC should be compacted to 91% of the maximum theoretical density (Rice value) of the mix, as determined in accordance with ASTM D2041, following the guidelines set in WSDOT SS 5-04.3(10) – Compaction.

Heavy construction traffic on new pavements or partial pavement sections (such as base course over the prepared subgrade) will likely exceed the design loads and could potentially damage or shorten the pavement life; therefore, we recommend construction traffic not be allowed on new pavements, or that the contractor take appropriate precautions to protect the subgrade and pavement during construction.

If construction traffic is to be allowed on newly constructed road sections, an allowance for this additional traffic will need to be made in the design pavement section.

4 CONSTRUCTION RECOMMENDATIONS

4.1 Site Preparation

Construction of the proposed structure will involve clearing and grubbing of the existing vegetation or demolition of possible existing structures. In vegetated areas, site stripping should include removing topsoil, roots, and other deleterious materials to a minimum depth of 12 inches bgs. Demolition should include removing existing pavement, utilities, etc., throughout the proposed new development. Underground utility lines or other abandoned structural elements should also be removed. The voids resulting from removal of foundations or loose soil in utility lines should be backfilled with compacted structural fill. The base of these excavations should be excavated to firm native subgrade before filling, with sides sloped at a minimum of 1H:1V to allow for uniform compaction. Materials generated during demolition should be transported off site or stockpiled in areas designated by the owner's representative.

4.1.1 Proofrolling/Subgrade Verification

Following site preparation and prior to placing aggregate base over shallow foundation, floor slab, and pavement subgrades, the exposed subgrade should be evaluated either by proofrolling or another method of subgrade verification. The subgrade should be proofrolled with a fully loaded dump truck or similar heavy, rubber-tire construction equipment to identify unsuitable areas. If evaluation of the subgrades occurs during wet conditions, or if proofrolling the subgrades will result in disturbance, they should be evaluated by PBS using a steel foundation probe. We recommend that PBS be retained to observe the proofrolling and perform the subgrade verifications. Unsuitable areas identified during the field evaluation should be compacted to a firm condition or be excavated and replaced with structural fill.

4.1.2 Wet/Freezing Weather and Wet Soil Conditions

Due to the presence of fine-grained silt and sands in the near-surface materials at the site, construction equipment may have difficulty operating on the near-surface soils when the moisture content of the surface soil is more than a few percentage points above the optimum moisture required for compaction. Soils disturbed during site preparation activities, or unsuitable areas identified during proofrolling or probing, should be removed and replaced with compacted structural fill.

Site earthwork and subgrade preparation should not be completed during freezing conditions, except for mass excavation to the subgrade design elevations. We recommend the earthwork construction at the site be performed during the dry season.

Protection of the subgrade is the responsibility of the contractor. Construction of granular haul roads to the project site entrance may help reduce further damage to the pavement and disturbance of site soils. The actual thickness of haul roads and staging areas should be based on the contractors' approach to site development, and the amount and type of construction traffic. The imported granular material should be placed in one lift over the prepared undisturbed subgrade and compacted using a smooth-drum, non-vibratory roller. A geotextile fabric should be used to separate the subgrade from the imported granular material in areas of repeated construction traffic. Depending on site conditions, the geotextile should meet Washington State Department of Transportation (WSDOT) SS 9-33.2 – Geosynthetic Properties for soil separation or stabilization. The geotextile should be installed in conformance with WSDOT SS 2-12.3 – Construction Geosynthetic (Construction Requirements) and, as applicable, WSDOT SS 2-12.3(2) – Separation or WSDOT SS 2-12.3(3) – Stabilization.

4.1.3 Compacting Test Pit Locations

The test pit excavations were backfilled using the excavator bucket and relatively minimal compactive effort; therefore, soft spots can be expected at these locations. We recommend that the relatively uncompacted soil be removed from the test pits to a depth of at least 3 feet below finished subgrade elevation in pavement areas and to full depth in building areas. The resulting excavation should be backfilled with structural fill.

4.2 Excavation

The near-surface soils at the site can be excavated with conventional earthwork equipment. Sloughing and caving should be anticipated. Severe caving was observed in the test pit excavations and limited the depth of excavation at test pit locations. All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. The contractor is solely responsible for adherence to the OSHA requirements. Open excavation techniques may be used provided the excavation is configured in accordance with the OSHA requirements, groundwater seepage is not present, and with the understanding that sloughing and caving will occur. Trenches/excavations should be flattened if sloughing occurs or seepage is present. Use of a trench shield or other approved temporary shoring is recommended. If dewatering is used, we recommend that the type and design of the dewatering system be the responsibility of the contractor, who is in the best position to choose systems that fit the overall plan of operation.

4.3 Slopes

If the project will include slopes or open excavation, temporary and permanent cut slopes up to 10 feet high may be inclined at 1.5H:1V and 2H:1V, respectively. Access roads and pavements should be located at least 5 feet from the top of temporary slopes. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face.

4.4 Structural Fill

The extent of site grading is currently unknown; however, PBS estimates that cuts and fills will be less than 5 feet. Structural fill should be placed over subgrade that has been prepared in conformance with the Site Preparation and Wet/Freezing Weather and Wet Soil Conditions sections of this report. Structural fill material should consist of relatively well-graded soil, or an approved rock product that is free of organic material and debris, and contains particles not greater than 4 inches nominal dimension.

The suitability of soil for use as compacted structural fill will depend on the gradation and moisture content of the soil when it is placed. As the amount of fines (material finer than the US Standard No. 200 Sieve) increases, soil becomes increasingly sensitive to small changes in moisture content and compaction becomes more difficult to achieve. Soils containing more than about 5% fines cannot consistently be compacted to a dense, non-yielding condition when the water content is significantly greater (or significantly less) than optimum.

If fill and excavated material will be placed on slopes steeper than 5H:1V, these must be keyed/benched into the existing slopes and installed in horizontal lifts. Vertical steps between benches should be approximately 2 feet.

4.4.1 On-Site Soil

On-site soils encountered in our explorations are generally suitable for placement as structural fill for mass grading to raise the site during moderate, dry weather when moisture contents can be maintained by air drying and/or addition of water. The fine-grained fraction of the site soils are moisture sensitive, and during wet weather, may become unworkable because of excess moisture content. In order to reduce moisture content, some aerating and drying of fine-grained soils may be required. The material should be placed in lifts with a maximum uncompacted thickness of approximately 8 inches and compacted to at least 92% of the maximum dry density, as determined by ASTM D1557 (modified proctor).

4.4.2 Imported Granular Materials

Imported granular material used during periods of wet weather or for haul roads, building pad subgrades, staging areas, etc., should be pit or quarry run rock, crushed rock, or crushed gravel and sand, and should meet the specifications provided in WSDOT SS 9-03.14(2) – Select Borrow. In addition, the imported granular material should be fairly well graded between coarse and fine, and of the fraction passing the US Standard No. 4 Sieve, less than 5% by dry weight should pass the US Standard No. 200 Sieve.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 9 inches and be compacted to not less than 95% of the maximum dry density, as determined by ASTM D1557.

4.4.3 Base Aggregate

Base aggregate for floor slabs and beneath pavements should be clean crushed rock or crushed gravel. The base aggregate should contain no deleterious materials, meet specifications provided in WSDOT SS 9-03.9(3) – Crushed Surfacing Base Course, and have less than 5% (by dry weight) passing the US Standard No. 200 Sieve. The imported granular material should be placed in one lift and compacted to at least 95% of the maximum dry density, as determined by ASTM D1557.

4.4.4 Foundation Base Aggregate

Imported granular material placed at the base of excavations for spread footings, slabs-on-grade, and other below-grade structures should be clean, crushed rock or crushed gravel and sand that is fairly well graded between coarse and fine. The granular materials should contain no deleterious materials, have a maximum particle size of 1½ inch, and meet WSDOT SS 9-03.12(1)A – Gravel Backfill for Foundations (Class A). The imported granular material should be placed in one lift and compacted to not less than 95% of the maximum dry density, as determined by ASTM D1557.

4.4.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 2 feet above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of 1 inch and less than 10% by dry weight passing the US Standard No. 200 Sieve, and should meet the standards prescribed by WSDOT SS 9-03.12(3) – Gravel Backfill for Pipe Zone Bedding. The pipe zone backfill should be compacted to at least 90% of the maximum dry density as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

Within pavement areas or beneath building pads, the remainder of the trench backfill should consist of well-graded granular material with a maximum particle size of 1½ inches, less than 10% by dry weight passing the US Standard No. 200 Sieve, and should meet standards prescribed by WSDOT SS 9-03.19 – Bank Run Gravel for Trench Backfill. This material should be compacted to at least 92% of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department. The upper 2 feet of the trench backfill should be compacted to at least 95% of the maximum dry density, as determined by ASTM D1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads), trench backfill placed above the pipe zone should consist of excavated material free of wood waste, debris, clods, or rocks greater than 6 inches in diameter and meet WSDOT SS 9-03.14 – Borrow and WSDOT SS 9-03.15 – Native Material for Trench Backfill. This general trench backfill should be compacted to at least 90% of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

4.4.6 Stabilization Material

Stabilization rock should consist of pit or quarry run rock that is well-graded, angular, crushed rock consisting of 4- or 6-inch-minus material with less than 5% passing the US Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material. WSDOT SS 9-13.1(5) – Quarry Spalls can be used as a general specification for this material with the stipulation of limiting the maximum size to 6 inches.

5 ADDITIONAL SERVICES AND CONSTRUCTION OBSERVATIONS

In most cases, other services beyond completion of a final geotechnical engineering report are necessary or desirable to complete the project. Occasionally, conditions or circumstances arise that require additional work that was not anticipated when the geotechnical report was written. PBS offers a range of environmental, geological, geotechnical, and construction services to suit the varying needs of our clients.

PBS should be retained to review the plans and specifications for this project before they are finalized. Such a review allows us to verify that our recommendations and concerns have been adequately addressed in the design.

Satisfactory earthwork performance depends on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. We recommend that PBS be retained to observe general excavation, stripping, fill placement, footing subgrades, soil improvement, and/or pile installation. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

6 LIMITATIONS

This report has been prepared for the exclusive use of the addressee, and their architects and engineers, for aiding in the design and construction of the proposed development and is not to be relied upon by other parties. It is not to be photographed, photocopied, or similarly reproduced, in total or in part, without express written consent of the client and PBS. It is the addressee's responsibility to provide this report to the appropriate design professionals, building officials, and contractors to ensure correct implementation of the recommendations.

The opinions, comments, and conclusions presented in this report are based upon information derived from our literature review, field explorations, laboratory testing, and engineering analyses. It is possible that soil, rock, or groundwater conditions could vary between or beyond the points explored. If soil, rock, or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that PBS is notified immediately so that we may reevaluate the recommendations of this report.

Unanticipated fill, soil and rock conditions, and seasonal soil moisture and groundwater variations are commonly encountered and cannot be fully determined by merely taking soil samples or completing explorations such as soil borings or test pits. Such variations may result in changes to our recommendations and may require additional funds for expenses to attain a properly constructed project; therefore, we recommend a contingency fund to accommodate such potential extra costs.

The scope of work for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

If there is a substantial lapse of time between the submission of this report and the start of work at the site, if conditions have changed due to natural causes or construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, this report should be reviewed to determine the applicability of the conclusions and recommendations presented herein. Land use, site conditions (both on and off site), or other factors may change over time and could materially affect our findings; therefore, this report should not be relied upon after three years from its issue, or in the event that the site conditions change.

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Important Information about This

Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:
- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

GBA
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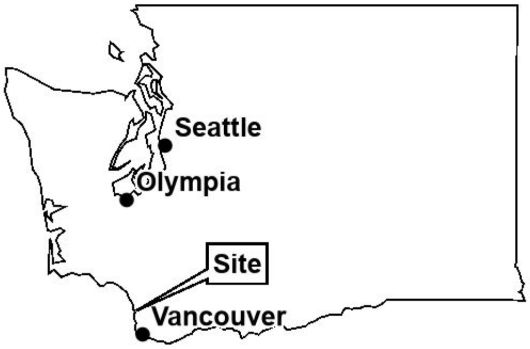
Figures



VICINITY MAP

WOODLAND LIBRARY
WOODLAND, WASHINGTON

DATE: SEP 2022 · PROJECT: 71959.000

PBS

FIGURE

1

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EXPLANATION

▦ TP-1 - Test pit name and approximate location with infiltration test

△ CPT-1 - CPT name and approximate location

◐ B-1 - Boring name and approximate location (PBS, 2017)

⌐ ¬ Approximate building footprint

Notes: Google Earth 2021

Coordinate System: NAD 1983 2011 StatePlane Washington South FIPS 4602 Ft US

SCALE: 1 inch = 50 feet

0 25 50 100 feet

SITE PLAN

WOODLAND LIBRARY
WOODLAND, WASHINGTON

DATE: SEP 2022 · PROJECT: 71959.000

PBS

FIGURE

2

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Appendix A

Field Explorations

Appendix A: Field Explorations

A1 GENERAL

PBS explored subsurface conditions at the project site by drilling five borings to depths of up to 36.5 feet bgs on February 28, 2017, excavating test pits to depths of up to 9 feet bgs on August 31, 2022, and advancing two cone penetration tests (CPTs) to depths of up to 82 feet bgs on August 12, 2022. The approximate locations of the explorations are shown on Figure 2, Site Plan. The procedures used to advance the borings, test pits, and CPTs, collect samples, and other field techniques are described in detail in the following paragraphs. Unless otherwise noted, all soil sampling and classification procedures followed engineering practices in general accordance with relevant ASTM procedures. "General accordance" means that certain local drilling/excavation and descriptive practices and methodologies have been followed.

A2 BORINGS

A2.1 Drilling

Borings were advanced using a truck-mounted drill rig provided and operated by Western States Soil Conservation, Inc., of Hubbard, Oregon, using mud rotary drilling techniques. The borings were observed by a member of the PBS geotechnical staff, who maintained a detailed log of the subsurface conditions and materials encountered during the course of the work.

A2.2 Sampling

Disturbed soil samples were taken in the borings at selected depth intervals. The samples were obtained using a standard 2-inch outside diameter, split-spoon sampler following procedures prescribed for the standard penetration test (SPT). Using the SPT, the sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance (N-value). The N-value provides a measure of the relative density of granular soils such as sands and gravels, and the consistency of cohesive soils such as clays and plastic silts. The disturbed soil samples were examined by a member of the PBS geotechnical staff and then sealed in plastic bags for further examination and physical testing in our laboratory.

A2.3 Boring Logs

The boring logs show the various types of materials that were encountered in the borings and the depths where the materials and/or characteristics of these materials changed, although the changes may be gradual. Where material types and descriptions changed between samples, the contacts were interpreted. The types of samples taken during drilling, along with their sample identification number, are shown to the right of the classification of materials. The N-values and natural water (moisture) contents are shown farther to the right.

A3 TEST PITS

A3.1 Excavation

Test pits were excavated using a Case 580 Super N excavator equipped with a 24-inch-wide, toothed bucket provided and operated by Dan J. Fisher Excavating, Inc., of Forest Grove, Oregon. The test pits were observed by a member of the PBS geotechnical staff, who maintained a detailed log of the subsurface conditions and materials encountered during the course of the work.

A3.2 Sampling

Representative disturbed samples were taken at selected depths in the test pits. The disturbed soil samples were examined by a member of the PBS geotechnical staff and sealed in plastic bags for further examination.

A3.3 Test Pit Logs

The test pit logs show the various types of materials that were encountered in the excavations and the depths where the materials and/or characteristics of these materials changed, although the changes may be gradual. Where material types and descriptions changed between samples, the contacts were interpreted. The types of samples taken during excavation, along with their sample identification number, are shown to the right of the classification of materials. The natural water (moisture) contents are shown farther to the right. Measured seepage levels, if observed, are noted in the column to the right.

A4 CONE PENETRATION TESTS (CPT)

A4.1 Field Procedures

Two CPT probes were advanced using a 20-ton truck mounted with a Vertek CPT 10 cm^2 electric seismic piezo cone owned and operated by Geotechnical Explorations, Inc., of Keizer, Oregon. During the test, the instrumented cone is hydraulically pushed into the ground at the rate of about 2 centimeters per second (cm/s), and readings of cone tip resistance, sleeve friction, and pore pressure are digitally recorded every second. As the cone advances, additional cone rods are added such that a “string” of rods continuously advances through the soil. As the test progresses, the CPT operator monitors the cone resistance and its deviation from vertical alignment.

For CPT soundings in which seismic data were collected, conventional CPT testing is temporarily halted at 2-meter intervals to collect seismic data. A seismograph integrated with the CPT is used to record the arrival time of seismic waves generated by striking a steel beam positioned at least 10 feet from the cone rods and coupled to the ground surface by the weight of the beam and operator to prevent the beam from moving when struck.

Each side of the beam is struck several times, and each signal produced by a blow is closely examined for signal and noise content, after which the waveform is selected and the arrival time of the shear wave is determined and recorded. After a complete set of seismic data are recorded, the cone is advanced to the next depth, and the procedure is repeated until the hole is complete.

A4.2 CPT Logs

In accordance with the applicable ASTM standard, the vertical axis is designated for the depth, while the horizontal axis displays the magnitude of the test values recorded. Recorded values include tip and shaft resistance and pore pressure. Final plotting scales are determined after all the tests are complete and take into consideration maximum test values and depths recorded for the project. This information is used to calculate the friction ratio and is correlated to material types, which are presented graphically in a column to the right. The CPT logs are included as Figures A8 and A9. The results of shear wave velocity testing are included on Figure A10.

A5 MATERIAL DESCRIPTION

Initially, samples were classified visually in the field. Consistency, color, relative moisture, degree of plasticity, and other distinguishing characteristics of the soil samples were noted. Afterward, the samples were reexamined in the PBS laboratory, various standard classification tests were conducted, and the field classifications were modified where necessary. The terminology used in the soil classifications and other modifiers are defined in Table A-1, Terminology Used to Describe Soil.

Table A-1
Terminology Used to Describe Soil
1 of 2

Soil Descriptions

Soils exist in mixtures with varying proportions of components. The predominant soil, i.e., greater than 50 percent based on total dry weight, is the primary soil type and is capitalized in our log descriptions (SAND, GRAVEL, SILT, or CLAY). Smaller percentages of other constituents in the soil mixture are indicated by use of modifier words in general accordance with the ASTM D2488-06 Visual-Manual Procedure. "General Accordance" means that certain local and common descriptive practices may have been followed. In accordance with ASTM D2488-06, group symbols (such as GP or CH) are applied on the portion of soil passing the 3-inch (75mm) sieve based on visual examination. The following describes the use of soil names and modifying terms used to describe fine- and coarse-grained soils.

Fine-Grained Soils (50% or greater fines passing 0.075 mm, No. 200 sieve)
The primary soil type, i.e., SILT or CLAY is designated through visual-manual procedures to evaluate soil toughness, dilatency, dry strength, and plasticity. The following outlines the terminology used to describe fine-grained soils, and varies from ASTM D2488 terminology in the use of some common terms.

Primary soil NAME, Symbols, and Adjectives			Plasticity Description	Plasticity Index (PI)
SILT (ML & MH)	**CLAY (CL & CH)**	**ORGANIC SOIL (OL & OH)**		
SILT		Organic SILT	Non-plastic	0 – 3
SILT		Organic SILT	Low plasticity	4 – 10
SILT/Elastic SILT	Lean CLAY	Organic SILT/ Organic CLAY	Medium Plasticity	10 – 20
Elastic SILT	Lean/Fat CLAY	Organic CLAY	High Plasticity	20 – 40
Elastic SILT	Fat CLAY	Organic CLAY	Very Plastic	>40

Modifying terms describing secondary constituents, estimated to 5 percent increments, are applied as follows:

Description	% Composition	
With Sand	% Sand ≥ % Gravel	15% to 25% plus No. 200
With Gravel	% Sand < % Gravel	
Sandy	% Sand ≥ % Gravel	≤30% to 50% plus No. 200
Gravelly	% Sand < % Gravel	

Borderline Symbols, for example CH/MH, are used when soils are not distinctly in one category or when variable soil units contain more than one soil type. **Dual Symbols,** for example CL-ML, are used when two symbols are required in accordance with ASTM D2488.

Soil Consistency terms are applied to fine-grained, plastic soils (i.e., PI ≥ 7). Descriptive terms are based on direct measure or correlation to the Standard Penetration Test N-value as determined by ASTM D1586-84, as follows. SILT soils with low to non-plastic behavior (i.e., PI < 7) may be classified using relative density.

Consistency Term	SPT N-value	Unconfined Compressive Strength	
		tsf	kPa
Very soft	Less than 2	Less than 0.25	Less than 24
Soft	2 – 4	0.25 – 0.5	24 – 48
Medium stiff	5 – 8	0.5 – 1.0	48 – 96
Stiff	9 – 15	1.0 – 2.0	96 – 192
Very stiff	16 – 30	2.0 – 4.0	192 – 383
Hard	Over 30	Over 4.0	Over 383

Table A-1
Terminology Used to Describe Soil
2 of 2

Soil Descriptions

Coarse - Grained Soils (less than 50% fines)

Coarse-grained soil descriptions, i.e., SAND or GRAVEL, are based on the portion of materials passing a 3-inch (75mm) sieve. Coarse-grained soil group symbols are applied in accordance with ASTM D2488-06 based on the degree of grading, or distribution of grain sizes of the soil. For example, well-graded sand containing a wide range of grain sizes is designated SW; poorly graded gravel, GP, contains high percentages of only certain grain sizes. Terms applied to grain sizes follow.

Material NAME	Particle Diameter	
	Inches	**Millimeters**
SAND (SW or SP)	0.003 – 0.19	0.075 – 4.8
GRAVEL (GW or GP)	0.19 – 3	4.8 – 75
Additional Constituents:		
Cobble	3 – 12	75 – 300
Boulder	12 – 120	300 – 3050

The primary soil type is capitalized, and the fines content in the soil are described as indicated by the following examples. Percentages are based on estimating amounts of fines, sand, and gravel to the nearest 5 percent. Other soil mixtures will have similar descriptive names.

Example: Coarse-Grained Soil Descriptions with Fines

>5% to < 15% fines (Dual Symbols)	≥15% to < 50% fines
Well graded GRAVEL with silt: GW-GM	Silty GRAVEL: GM
Poorly graded SAND with clay: SP-SC	Silty SAND: SM

Additional descriptive terminology applied to coarse-grained soils follow.

Example: Coarse-Grained Soil Descriptions with Other Coarse-Grained Constituents

Coarse-Grained Soil Containing Secondary Constituents	
With sand or with gravel	≥ 15% sand or gravel
With cobbles; with boulders	Any amount of cobbles or boulders.

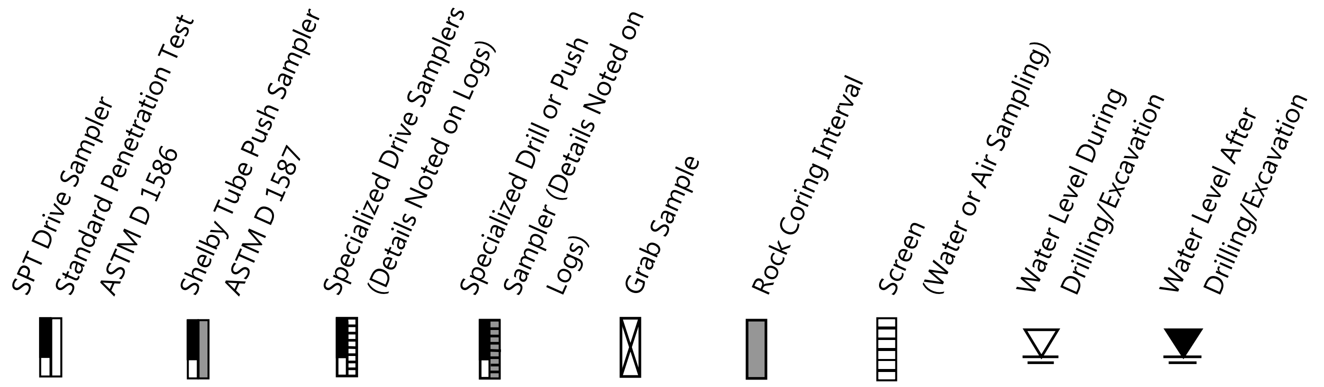
Cobble and boulder deposits may include a description of the matrix soils, as defined above.

Relative Density terms are applied to granular, non-plastic soils based on direct measure or correlation to the Standard Penetration Test N-value as determined by ASTM D1586-84.

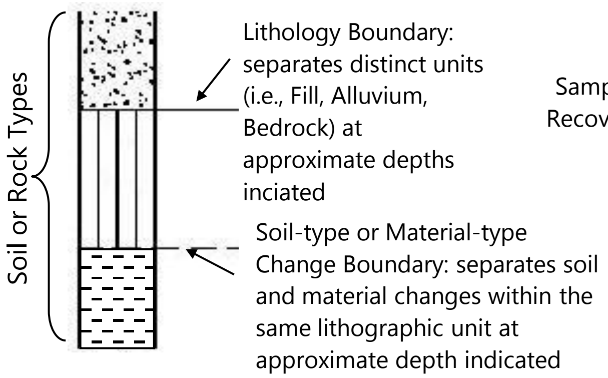
Relative Density Term	SPT N-value
Very loose	0 – 4
Loose	5 – 10
Medium dense	11 – 30
Dense	31 – 50
Very dense	> 50

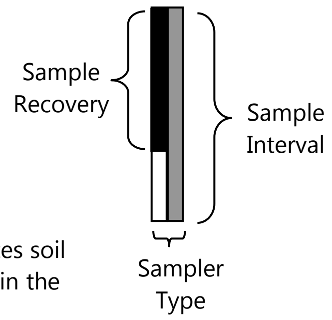
Table A-2
Key To Test Pit and Boring Log Symbols

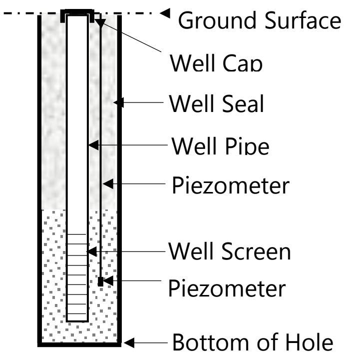
SAMPLING DESCRIPTIONS



LOG GRAPHICS


Soil and Rock


Sampling Symbols


Instrumentation Detail

Geotechnical Testing Acronym Explanations

PP	Pocket Penetrometer	HYD	Hydrometer Gradation
TOR	Torvane	SIEV	Sieve Gradation
DCP	Dynamic Cone Penetrometer	DS	Direct Shear
ATT	Atterberg Limits	DD	Dry Density
PL	Plasticity Limit	CBR	California Bearing Ratio
LL	Liquid Limit	RES	Resilient Modulus
PI	Plasticity Index	VS	Vane Shear
P200	Percent Passing US Standard No. 200 Sieve	bgs	Below ground surface
OC	Organic Content	MSL	Mean Sea Level
CON	Consolidation	HCL	Hydrochloric Acid
UC	Unconfined Compressive Strength		

Details of soil and rock classification systems are available on request.

Rev. 02/2017

PBS

WOODLAND LIBRARY
WOODLAND, WASHINGTON

BORING B-1

PBS PROJECT NUMBER:
71959.000

APPROX. BORING B-1 LOCATION:
(See Site Plan)
Lat: 45.90412 Long: -122.74405

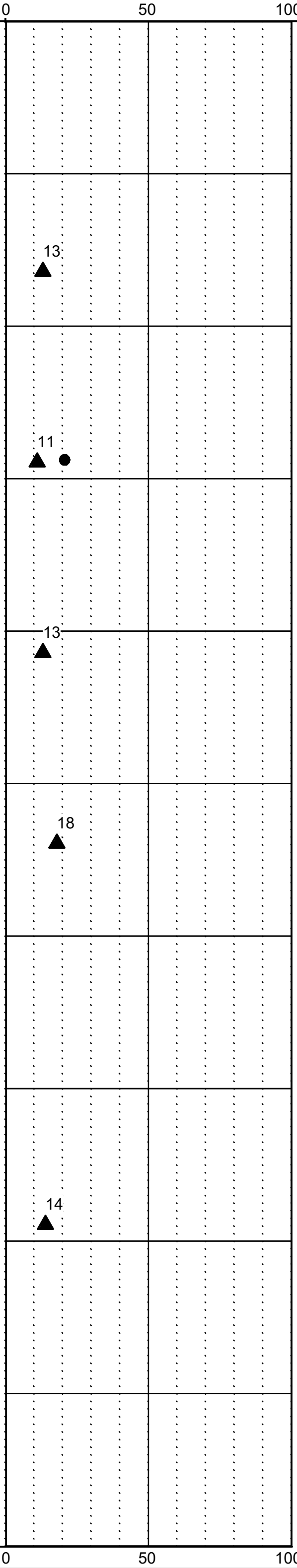
NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

▲ UNCORRECTED N-VALUE
✧ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

INSTALLATION AND COMMENTS

Surface Conditions: Concrete

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	INSTALLATION AND COMMENTS
0.0	CONCRETE (3 inches)	0.0 0.3			
	Medium dense poorly graded GRAVEL (GP); fine to coarse, rounded to subangular gravel; moist **FILL**			S-1	
4.0	Medium dense gray poorly graded SAND (SP-SM) with silt; non-plastic; fine sand; moist	4.0	P200	S-2 S-3 S-4 S-5	P200 = 8%
16.0	Medium dense poorly graded SAND (SP) with gravel; fine to coarse sand; fine, rounded to subangular gravel; moist	16.0			
20.0					



DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: T. Rikli

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 92.8
LOGGING COMPLETED: 2/28/2017

FIGURE A1

BORING B-1
(continued)

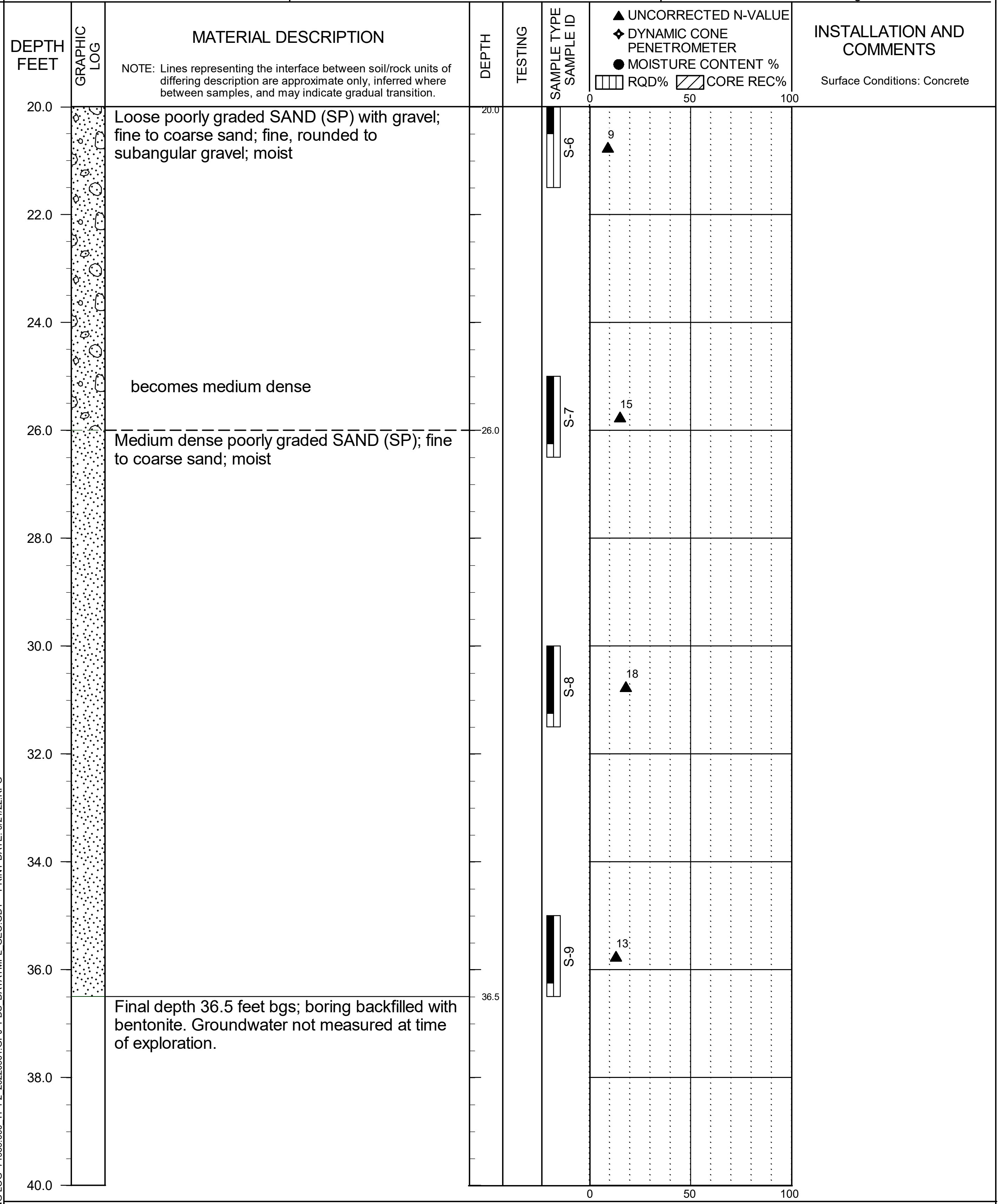
APPROX. BORING B-1 LOCATION:
(See Site Plan)
Lat: 45.90412 Long: -122.74405

DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | DEPTH | TESTING | SAMPLE TYPE SAMPLE ID | INSTALLATION AND COMMENTS

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

▲ UNCORRECTED N-VALUE
◆ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

Surface Conditions: Concrete



Loose poorly graded SAND (SP) with gravel; fine to coarse sand; fine, rounded to subangular gravel; moist

becomes medium dense

Medium dense poorly graded SAND (SP); fine to coarse sand; moist

Final depth 36.5 feet bgs; boring backfilled with bentonite. Groundwater not measured at time of exploration.

DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: T. Rikli

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 92.8
LOGGING COMPLETED: 2/28/2017

FIGURE A1

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BORING B-2

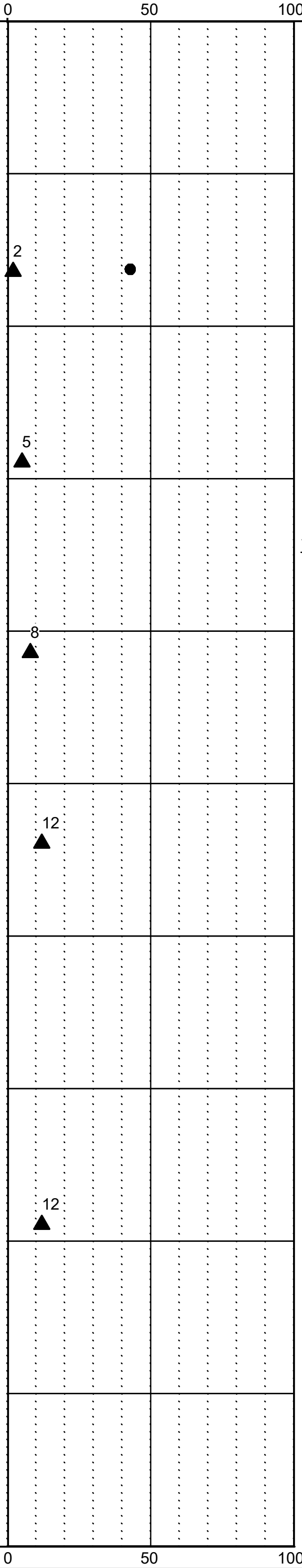
PBS PROJECT NUMBER:
71959.000

APPROX. BORING B-2 LOCATION:
(See Site Plan)
Lat: 45.90422 Long: -122.74357

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

▲ UNCORRECTED N-VALUE
✧ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

Surface Conditions: Asphalt



DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
0.0	ASPHALT (1.5 inches)	0.0				
	BASE ROCK (10.5 inches)	0.1				
	Soft brown SILT (ML) with sand; low plasticity; fine sand; moist	1.0				
2.0			P200	S-1	2	P200 = 78%
4.0						
	Loose gray poorly graded SAND (SP); fine to medium sand; moist	5.0		S-2	5	
6.0						⊻ 02/28/17
8.0				S-3	8	
10.0	becomes medium dense			S-4	12	
12.0						
14.0				S-5	12	
16.0	Loose gray poorly graded SAND (SP) with gravel; fine to coarse sand; fine, rounded to subangular gravel; moist	16.0				
18.0						
20.0						

DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: T. Rikli

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 92.8
LOGGING COMPLETED: 2/28/2017

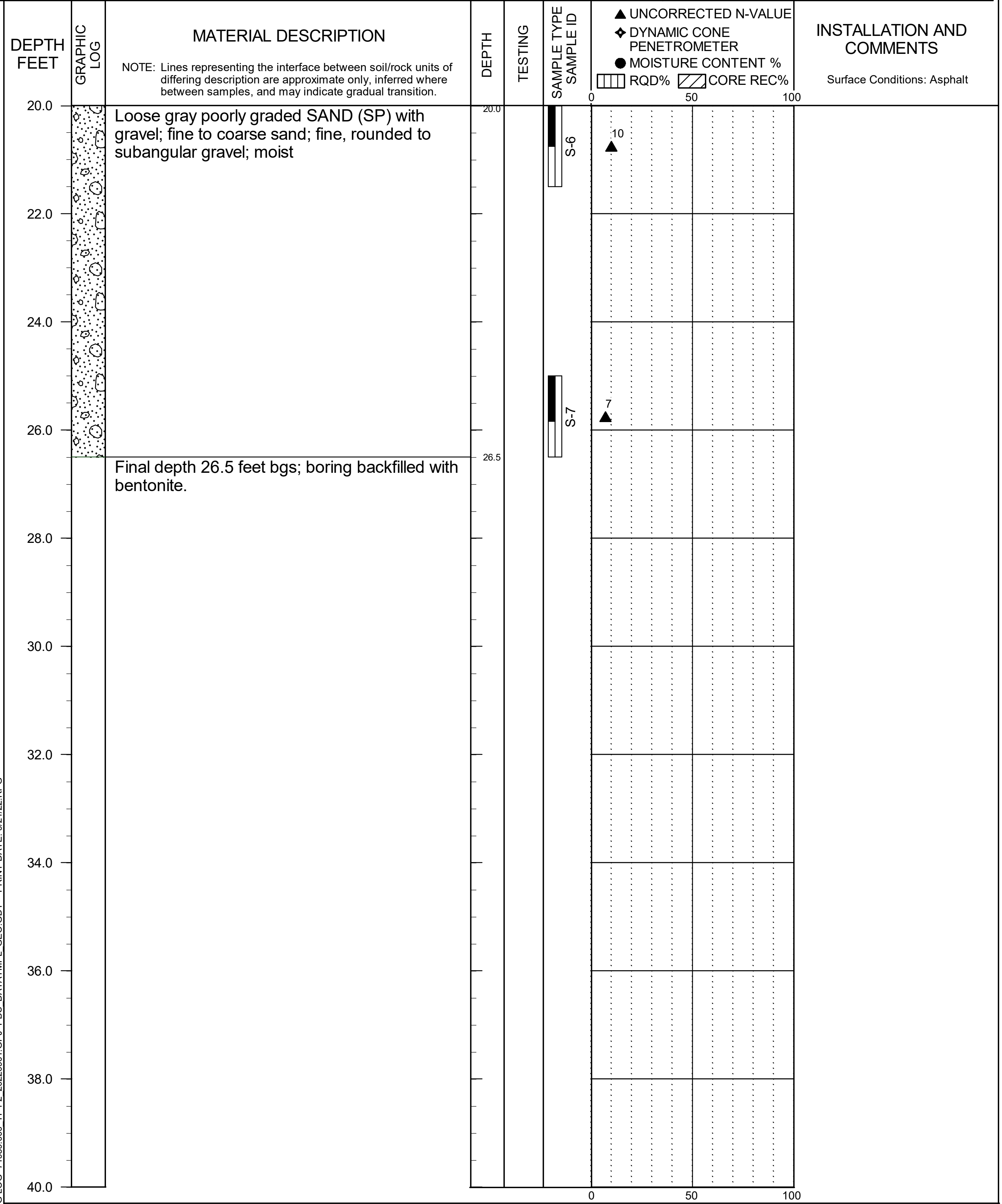
FIGURE A2

	WOODLAND LIBRARY WOODLAND, WASHINGTON	**BORING B-2** (continued)
	PBS PROJECT NUMBER: 71959.000	APPROX. BORING B-2 LOCATION: (See Site Plan) Lat: 45.90422 Long: -122.74357

INSTALLATION AND COMMENTS

Surface Conditions: Asphalt

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.



DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	SAMPLE ID	UNCORRECTED N-VALUE
20.0	Loose gray poorly graded SAND (SP) with gravel; fine to coarse sand; fine, rounded to subangular gravel; moist	20.0	S-6	10
			S-7	7
26.5	Final depth 26.5 feet bgs; boring backfilled with bentonite.	26.5		

DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: T. Rikli

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 92.8
LOGGING COMPLETED: 2/28/2017

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WOODLAND, WASHINGTON

BORING B-3

PBS PROJECT NUMBER:
71959.000

APPROX. BORING B-3 LOCATION:
(See Site Plan)
Lat: 45.90403 Long: -122.74348

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

▲ UNCORRECTED N-VALUE
✧ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

Surface Conditions: Grass

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	UNCORRECTED N-VALUE (0 – 50 – 100)	INSTALLATION AND COMMENTS
0.0		TOPSOIL (2 inches)	0.0				
		Loose gray poorly graded SAND (SP); fine to coarse sand; moist	0.2				
					S-1	4	
					S-2	5	
					S-3	5	
					S-4	7	
		Final depth 11.5 feet bgs; boring backfilled with bentonite. Groundwater not measured at time of exploration.	11.5				
20.0							

BORING LOG 71959.000 TP1-2 20220901.GPJ PBS DATATMPL GEO.GDT PRINT DATE: 9/21/22:RPG

DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: T. Rikli

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 92.8
LOGGING COMPLETED: 2/28/2017

FIGURE A3

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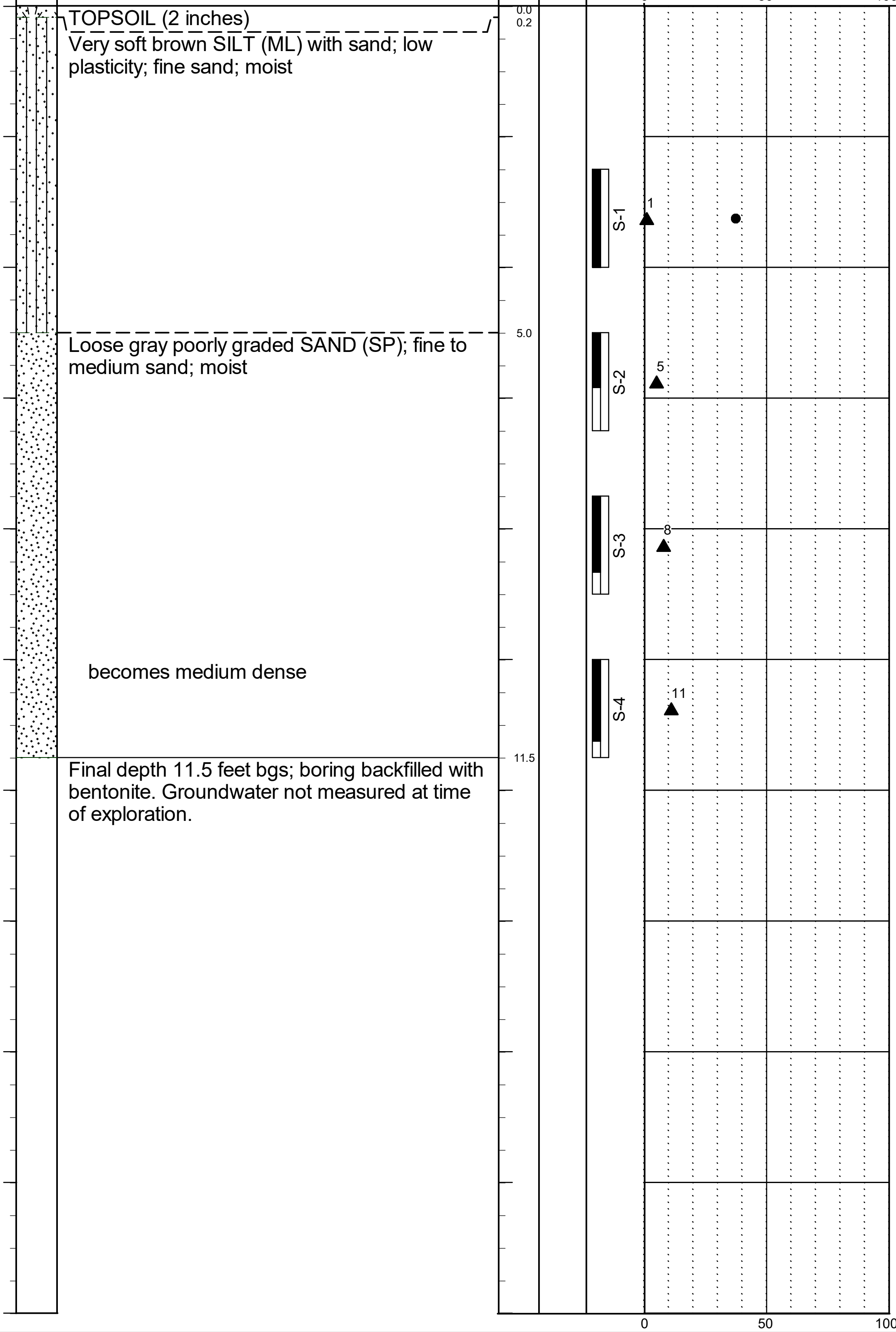
BORING B-4

PBS PROJECT NUMBER:
71959.000

APPROX. BORING B-4 LOCATION:
(See Site Plan)
Lat: 45.90423 Long: -122.74341

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	▲ UNCORRECTED N-VALUE ✦ DYNAMIC CONE PENETROMETER ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
		NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.				0 50 100	Surface Conditions: Grass
0.0		TOPSOIL (2 inches)	0.0 0.2				
		Very soft brown SILT (ML) with sand; low plasticity; fine sand; moist					
2.0							
					S-1	1	
4.0							
		Loose gray poorly graded SAND (SP); fine to medium sand; moist	5.0				
					S-2	5	
6.0							
8.0					S-3	8	
10.0		becomes medium dense					
					S-4	11	
		Final depth 11.5 feet bgs; boring backfilled with bentonite. Groundwater not measured at time of exploration.	11.5				
12.0							
14.0							
16.0							
18.0							
20.0						0 50 100	



DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: T. Rikli

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 92.8
LOGGING COMPLETED: 2/28/2017

FIGURE A4
Page 1 of 1

BORING LOG 71959.000 TP1-2 20220901.GPJ PBS DATATMPL GEO.GDT PRINT DATE: 9/21/22:RPG

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WOODLAND, WASHINGTON

BORING B-5

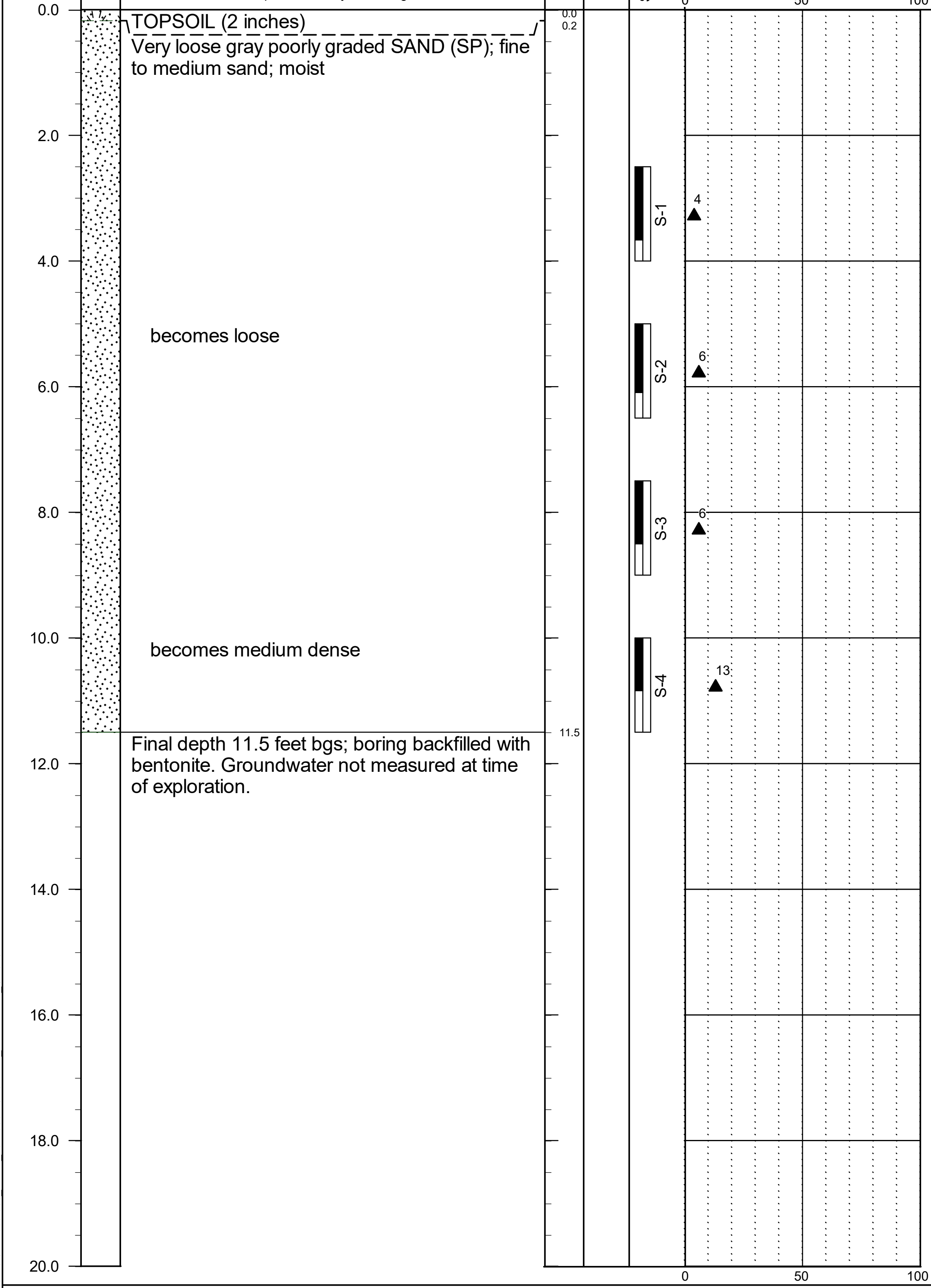
PBS PROJECT NUMBER:
71959.000

APPROX. BORING B-5 LOCATION:
(See Site Plan)
Lat: 45.90380 Long: -122.74346

DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | DEPTH | TESTING | SAMPLE TYPE SAMPLE ID | ▲ UNCORRECTED N-VALUE ◆ DYNAMIC CONE PENETROMETER ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% | INSTALLATION AND COMMENTS

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

Surface Conditions: Grass



TOPSOIL (2 inches)

Very loose gray poorly graded SAND (SP); fine to medium sand; moist

becomes loose

becomes medium dense

Final depth 11.5 feet bgs; boring backfilled with bentonite. Groundwater not measured at time of exploration.

DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: T. Rikli

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 92.8
LOGGING COMPLETED: 2/28/2017

FIGURE A5

BORING LOG 71959.000 TP1-2 20220901.GPJ PBS DATATMPL GEO.GDT PRINT DATE: 9/21/22:RPG

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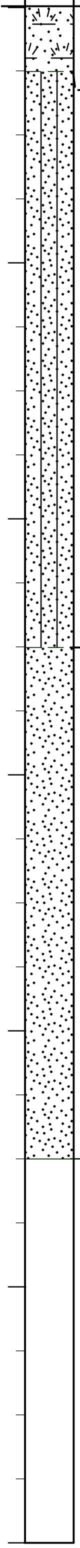
TEST PIT TP-1

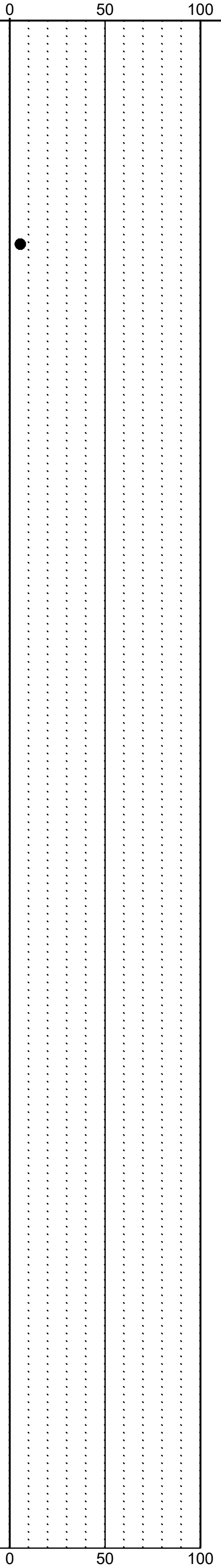
PBS PROJECT NUMBER:
71959.000

APPROX. TEST PIT TP-1 LOCATION:
(See Site Plan)
Lat: 45.9240536 Long: -122.7429857

Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

Surface Conditions: Grass





DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	◆ DYNAMIC CONE PENETROMETER ⊠ STATIC PENETROMETER ● MOISTURE CONTENT %	COMMENTS
0.0	TOPSOIL: Gray, poorly graded SAND (SP-SM) with silt and roots; low plasticity; fine sand; dry	0.0				
	Gray, silty SAND (SM); non-plastic; fine sand; dry	0.5				
			P200	☒ S-1	●	P200 = 12%
2.0						
4.0						
						Infiltration testing completed at 5 feet bgs
	Gray, poorly graded SAND (SP); fine sand; dry	5.0		☒ S-2		
6.0						Severe caving from 6 to 9 feet bgs
8.0						
	Final depth 9.0 feet bgs due to severe caving; test pit backfilled with excavated material to existing ground surface. Groundwater not encountered at time of exploration.	9.0				
10.0						
12.0						

LOGGED BY: F. Jarman
COMPLETED: 8/31/2022

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: CASE 580N with 24" Bucket

FIGURE A6
Page 1 of 1

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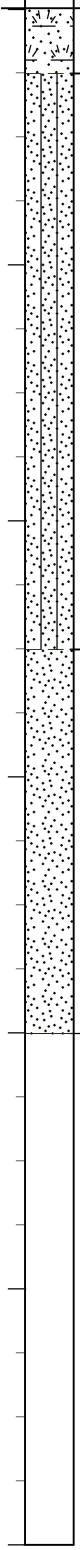
WOODLAND, WASHINGTON

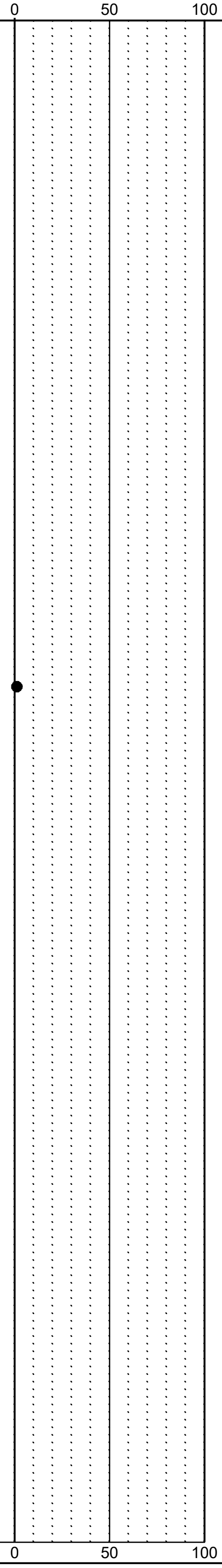
PBS PROJECT NUMBER:
71959.000

TEST PIT TP-2

APPROX. TEST PIT TP-2 LOCATION:
(See Site Plan)
Lat: 45.9038587 Long: -122.7432164

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	❖ DYNAMIC CONE PENETROMETER ☒ STATIC PENETROMETER ● MOISTURE CONTENT % 0 50 100	COMMENTS Surface Conditions: Grass
0.0		TOPSOIL: Gray, sandy SILT (ML) with roots; low plasticity; fine sand; dry	0.0				
		Gray, silty SAND (SM); non-plastic; fine sand; dry	0.5		S-1		
2.0							
4.0							Infiltration testing completed at 5 feet bgs
		Gray, poorly graded SAND (SP); fine sand; dry	5.0	P200 SIEV	S-2	●	P200 = 5%
6.0							Severe caving from 6 to 8 feet bgs
8.0		Final depth 8.0 feet bgs due to severe caving; test pit backfilled with excavated material to existing ground surface. Groundwater not encountered at time of exploration.	8.0				
10.0							
12.0							





LOGGED BY: F. Jarman
COMPLETED: 8/31/2022

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: CASE 580N with 24" Bucket

FIGURE A7
Page 1 of 1

PBS Engineering and Environmental Inc.
4412 S Corbett Avenue
Portland, Oregon 97239
http://pbsusa.com

CPT-1

Total depth: 82.02 ft
Date: 8/12/2022

Project: **71959.000**
Location: **Woodland, Washington**

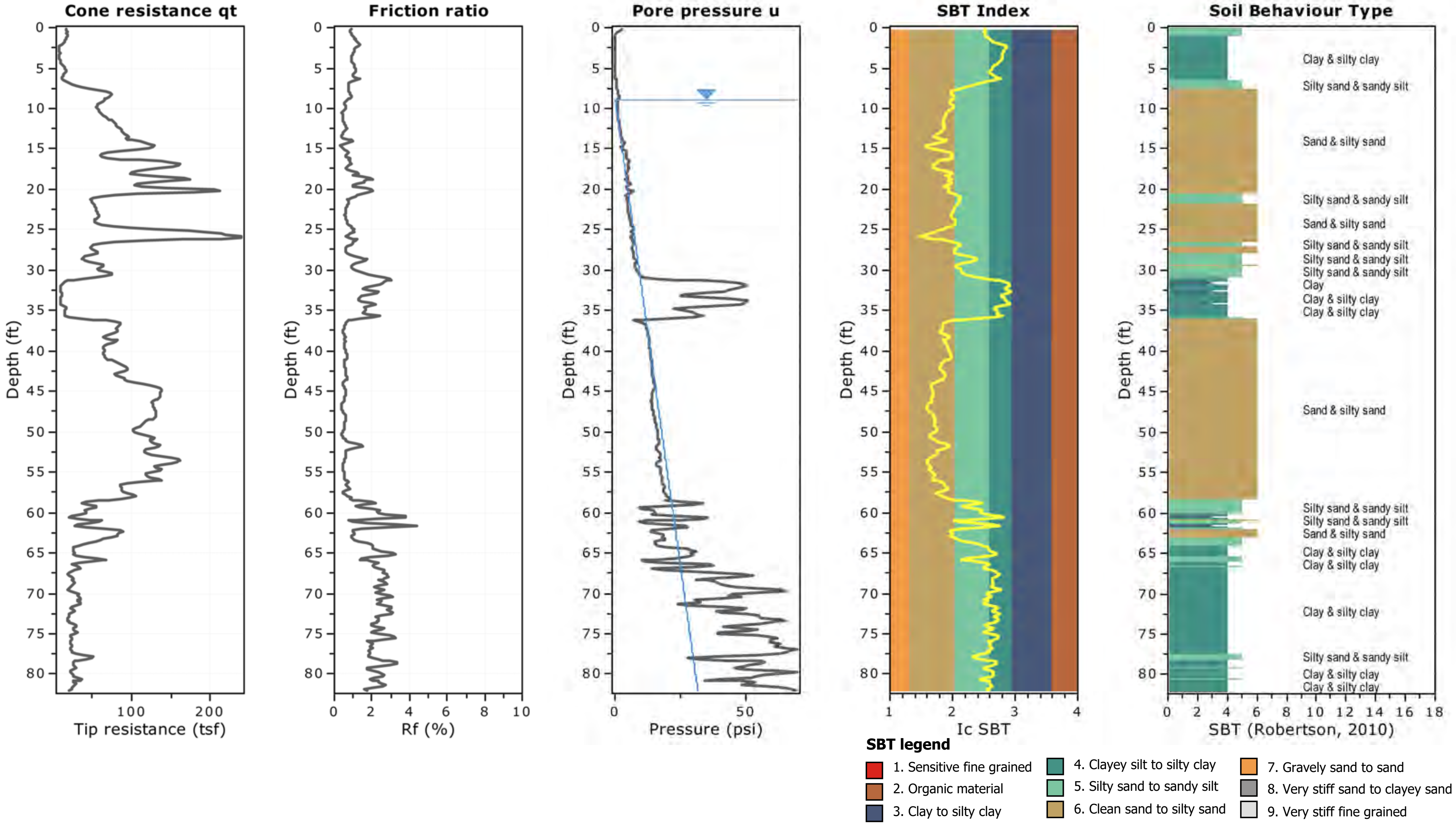


FIGURE A8

PBS Engineering and Environmental Inc.
4412 S Corbett Avenue
Portland, Oregon 97239
http://pbsusa.com

CPT-2

Total depth: 60.04 ft
Date: 8/12/2022

Project: **71959.000**
Location: **Woodland, Washington**

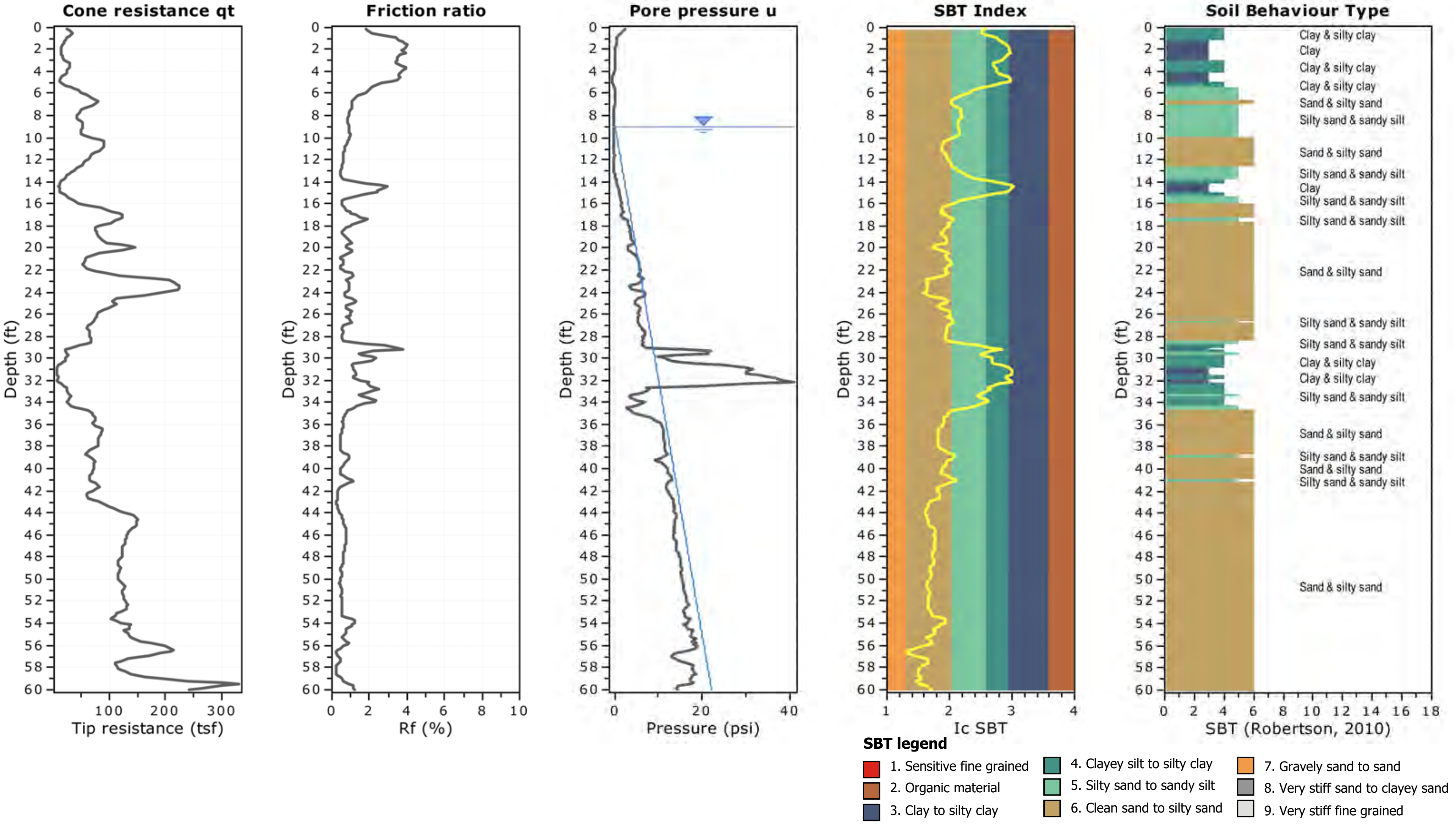
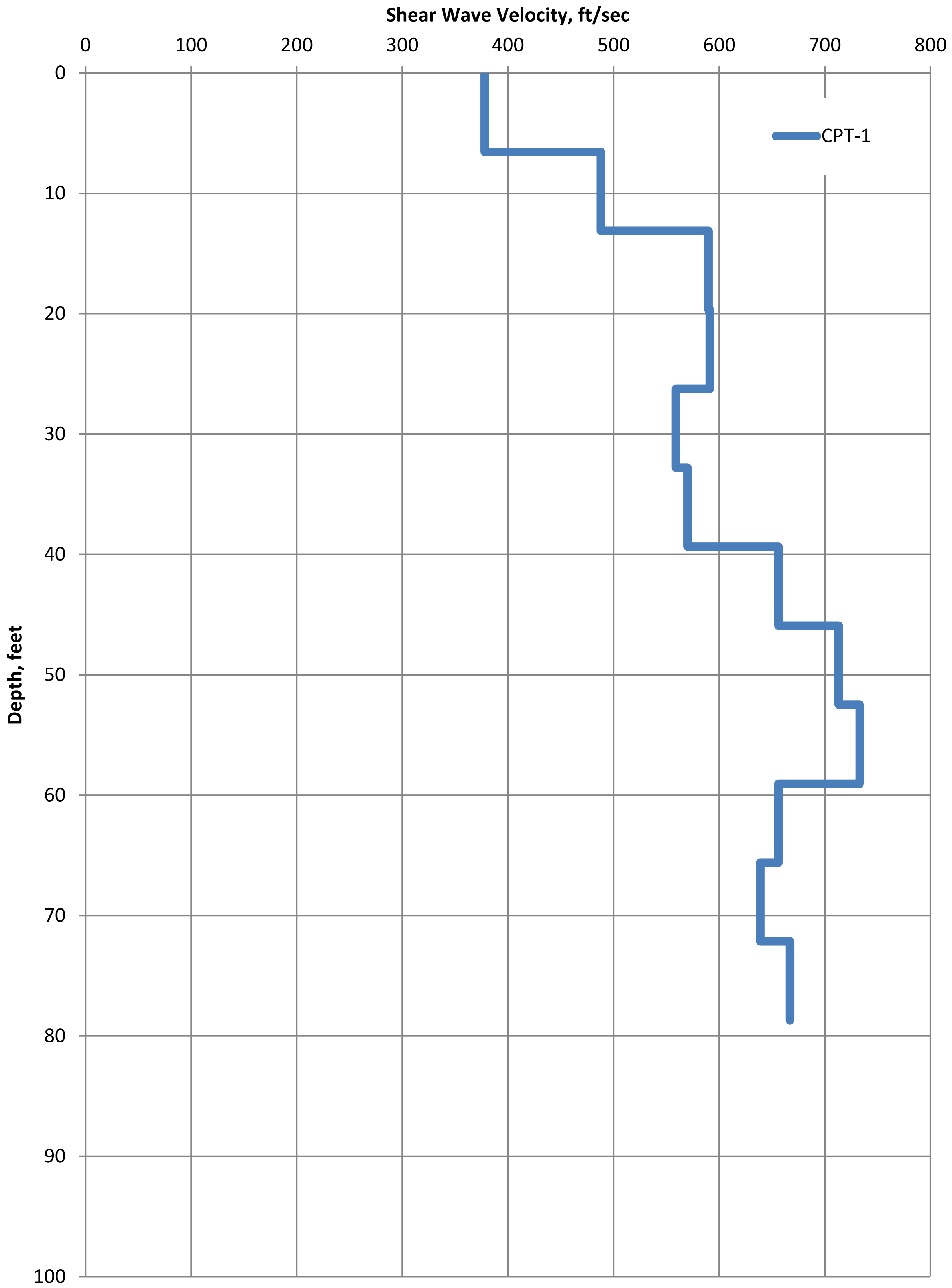


FIGURE A9



SHEAR WAVE VELOCITY PROFILE

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FIGURE
A10

Appendix B

Laboratory Testing

Appendix B: Laboratory Testing

B1 GENERAL

Samples obtained during the field explorations were examined in the PBS laboratory. The physical characteristics of the samples were noted and field classifications were modified where necessary. During the course of examination, representative samples were selected for further testing. The testing program for the soil samples included standard classification tests, which yield certain index properties of the soils important to an evaluation of soil behavior. The testing procedures are described in the following paragraphs. Unless noted otherwise, all test procedures are in general accordance with applicable ASTM standards. "General accordance" means that certain local and common descriptive practices and methodologies have been followed.

B2 CLASSIFICATION TESTS

B2.1 Visual Classification

The soils were classified in accordance with the Unified Soil Classification System with certain other terminology, such as the relative density or consistency of the soil deposits, in general accordance with engineering practice. In determining the soil type (that is, gravel, sand, silt, or clay) the term that best described the major portion of the sample is used. Modifying terminology to further describe the samples is defined in Table A-1, Terminology Used to Describe Soil, in Appendix A.

B2.2 Moisture (Water) Contents

Natural moisture content determinations were made on samples of the fine-grained soils (that is, silts, clays, and silty sands). The natural moisture content is defined as the ratio of the weight of water to dry weight of soil, expressed as a percentage. The results of the moisture content determinations are presented on the exploration logs in Appendix A and on Figure B2, Summary of Laboratory Data, in Appendix B.

B2.3 Grain-Size Analyses

Mechanical grain-size (sieve) analyses were performed on select samples to determine their particle size distribution. The results of the sieve analyses are presented on Figure B1, Particle-Size Analysis Test Results, in Appendix B.

Washed sieve analyses (P200) were completed on samples to determine the portion of soil samples passing the No. 200 Sieve (i.e., silt and clay). The results of the P200 test results are presented on the exploration logs in Appendix A and on Figure B2, Summary of Laboratory Data, in Appendix B.

SUMMARY OF LABORATORY DATA

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SAMPLE INFORMATION				MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT (PERCENT)	PLASTIC LIMIT (PERCENT)	PLASTICITY INDEX (PERCENT)
B-1	S-2	5		20.6				8			
B-2	S-1	2.5		42.8				78			
B-4	S-1	2.5		37.3							
TP-1	S-1	1.5		5.5				12			
TP-2	S-2	5		1.1		0	95	5			

PBS

PARTICLE-SIZE ANALYSIS TEST RESULTS

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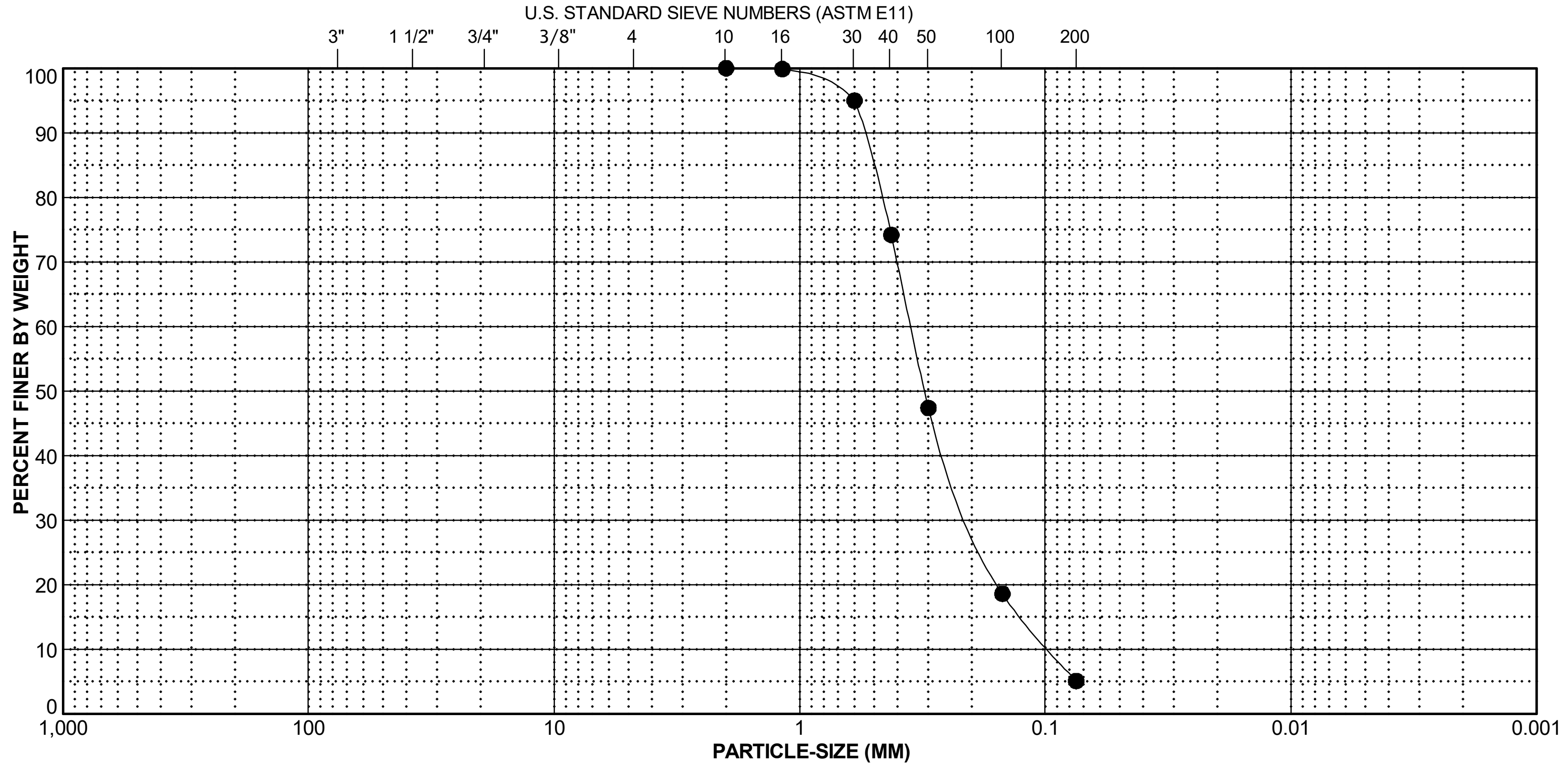
TEST METHOD: ASTM C136/D422

BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY



KEY	EXPLORATION NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	D90 (MM)	D60 (MM)	D30 (MM)	D10 (MM)	D5 (MM)	GRAVEL (PERCENT)	SAND (PERCENT)	FINES (PERCENT)
●	TP-2	S-2	5.0	1	0.6	0.4	0.2	0.097			95	5

FIGURE B1