Geotechnical Engineering Report

Wallace Elementary School Additions
410 Elm Street
Kelso, Washington

Prepared for:

Kelso School District
601 Crawford Street
Kelso, Washington 98626

August 30, 2018
PBS Project 73400.001

PBS
4412 SW CORBETT AVENUE
PORTLAND, OR 97239
503.248.1939 MAIN
866.727.0140 FAX
PBSUSA.COM

Geotechnical Engineering Report

Wallace Elementary School Additions
410 Elm Street
Kelso, Washington

Prepared for:
Kelso School District
601 Crawford Street
Kelso, Washington 98626

August 30, 2018
PBS Project 73400.001

Prepared by:

Shaun Cordes

Digitally signed by Shaun Cordes
Date: 2018.08.30 13:36:40 -07'00'

Shaun Cordes, LG, LEG
Engineering Geologist
PBS Engineering and Environmental Inc.

RYAN K. WHITE
39104

Digitally signed by Ryan White, PE, GE
Date: 2018.08.30 13:38:25 -07'00'

8/30/2018

Ryan White, PE, GE
Geotechnical Engineering Group Manager
PBS Engineering and Environmental Inc.

Reviewed by:

Saiid Behboodi

Saiid Behboodi, PE, GE
Principal Geotechnical Engineer
PBS Engineering and Environmental Inc

Table of Contents

SUPPORTING DATA

TABLES

FIGURES

APPENDICES

1 INTRODUCTION

1.1.1 General

This report presents results of PBS Engineering and Environmental Inc. (PBS) geotechnical engineering services for the proposed Wallace Elementary school additions in Kelso, Washington (site). The general site location is shown on the Vicinity Map, Figure 1. The locations of PBS' explorations in relation to existing 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 school additions. 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.

1.2.2 Subsurface Explorations

Subsurface conditions at the site were explored by advancing three borings and two cone penetration test (CPT) probes. The borings were advanced to a depth of 31.5 feet below the existing ground surface (bgs) in locations requested by the client. The borings were logged and representative soil samples collected by a member of the PBS geotechnical engineering staff. The CPTs were completed to a depth up to 81.5 feet bgs. The interpreted boring logs are presented as Figures A1 through A3 and the CPT logs are presented as Figures A4 and A5 in Appendix A, Field Explorations. The approximate boring and CPT locations are shown on the Site Plan, Figure 2.

1.2.3 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, grain-size analyses, one-dimensional consolidation, and Atterberg limits. Laboratory test results are included in the exploration logs in Appendix A, Field Explorations; and in Appendix B, Laboratory Testing.

1.2.4 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.

1.2.5 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
- Groundwater considerations
- Seismic site hazard study that includes:
    - Discussion of geologic and seismic hazards impacting the site
    - The location of nearby faults
    - Evaluation of liquefaction potential
- Shallow foundation design recommendations:

  - Minimum embedment
  - Allowable bearing pressure
  - Estimated settlement (total and differential)
  - Sliding coefficient
- Deep foundation recommendations:
  - Minimum depth of embedment
- Soil improvement recommendations
- Earthwork and grading, cut, and fill recommendations:
  - Structural fill materials and preparation, and reuse of on-site soils
  - Wet weather considerations
  - Utility trench excavation and backfill requirements
  - Temporary and permanent slope inclinations
- Seismic design criteria in accordance with the current International Building Code (IBC) with State of Washington amendments
- Slab and pavement subgrade preparation recommendations
- Recommended asphalt concrete (AC) pavement sections

1.3 Project Understanding

PBS understands that the Kelso School District intends to construct a new, two- to three-story building at the current Wallace Elementary School site. The new school will be constructed south of the existing school, in the area currently occupied by sports fields, allowing the school to remain open during construction. Development will also include new fire and drive lanes. At the time this report was prepared, the size, type, and location of structures and additions had not been finalized. Depending on the location of the structures, additional exploration and analyses may be required.

2 SITE CONDITIONS

2.1 Surface Description

Wallace Elementary School is located on the corner of Elm Street and S 5th Street in Kelso, Washington. The site and surrounding ground surface is relatively flat and located at an elevation of approximately 20 feet above mean sea level (amsl), as measured by available LiDAR data obtained from the Washington LiDAR Portal. Structures occupy approximately one-third of the site along the north end, play structures and an asphalt concrete surface occupy the middle of the site, and a grass field occupies the south.

2.2 Geologic Setting

The site is located north of the Portland-Vancouver basin within the physiographic province of the Puget-Willamette lowland that separates the Cascade Range from the Coast Range, and extends from the Puget Sound to Eugene, Oregon (Yeats et al., 1996). The Puget-Willamette lowlands are situated along the Cascadia Subduction Zone 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 (Figure 3).

The site is underlain by recent (Holocene) alluvium consisting of sand, gravel, silt, and peat deposits associated with the Cowlitz and Columbia River waterways (Livingston, 1966; Figure 4). These sediments were deposited over older deformed sedimentary rocks of Pliocene to Eocene age. These rocks comprise the surrounding hillsides and are deformed by northwest-trending anticline and syncline folds. The mouth of the Cowlitz River valley is bounded along the east side by the non-active Kelso Fault.

2.3 Subsurface Conditions

Subsurface conditions were explored by completing three borings, designated B-1 through B-3, to depths of up to 31.5 feet bgs, and advancing two CPTs to depths of 31 to 81.5 feet bgs. The drilling was performed by Holt Services, Inc., of Vancouver, Washington, using a truck-mounted Mobile B58 drill rig and mud-rotary drilling techniques. The CPTs were advanced by Oregon Geotechnical Explorations of Keizer, Oregon, using a truck-mounted Hogentogler 20-ton Electronic Dutch cone CPT rig. PBS has characterized the subsurface conditions to 31.5 feet using data collected from SPT samples and CPTs, and from 31.5 to 81.5 bgs using the CPT logs.

PBS has summarized the subsurface units as follows:

SAND and SILTY SAND:	Very loose to medium dense brown sand was encountered in all three borings and both CPTs. SPT N-values ranged from 4 to 17 and generally increased in density with depth. CPT-2 encountered sand and silty sand to 81.5 feet bgs with an increase in tip resistance and correlated N-values at a depth of approximately 18 feet bgs.
SILT, SANDY SILT, and CLAY:	A layer of very soft to medium stiff brown silt varying from 3- to 4-feet-thick was encountered in the upper 10 feet of the borings, with sand above and below it. The silt was non-plastic to medium plasticity with SPT N-values of 1 to 6. CPT-1 and CPT-2 encountered silt, sandy silt, and clay to approximately 10 to 12 feet bgs. CPT-2 identified interbeds of these materials within thicker sand and silty sand layers. The plasticity index (PI) of one sample was 15, medium plasticity.
GRAVEL:	A 5-foot-thick, medium dense, subrounded gravel with sand layer was encountered between 25 to 30 feet bgs in boring B-2, with an SPT N-Value of 16.

2.4 Groundwater

Static groundwater was encountered during our explorations at a depth of approximately 6 feet bgs. This is consistent with wet samples obtained by SPTs and CPT pore-pressure dissipation testing. Please note that groundwater levels can fluctuate during the year depending on climate, irrigation season, extended periods of precipitation, drought, and other factors.

2.4.1 100-Year and 500-Year Floods

The site is located adjacent to the Cowlitz River, which is impounded by a levee system that protects the city from flooding. Review of the FEMA Flood Insurance Map indicates the site is not expected to be impacted by a 100-year flood event (1 percent probability of flooding annually) or a 500-year flood event (0.2 percent probability of flooding annually) unless breaching or undermining of the levee system occurs.

2.5 Seismicity and Faulting

2.5.1 Seismic Sources

Several types of seismic sources exist in the Pacific Northwest, which are outlined below. Volcanic sources beneath the Cascade Range are not considered further in this study, as they rarely exceed about M=5.0 in size and are not considered to pose a significant ground-shaking hazard to the project site.

2.5.1.1 Cascadia Subduction Zone (CSZ) – Interface Earthquakes

The CSZ represents the boundary between the subducting Juan de Fuca tectonic plate and the overriding North American tectonic plate (Figure 3). Recurrence intervals for subduction zone earthquakes are based on studies of the geologic record, with studies estimating a recurrence interval between 300 to 600 years. Geologic evidence and written records from Japan suggest the most recent earthquake occurred in January 1700. The 1700 earthquake probably ruptured much of the approximate 620-mile (1,000 km) length of the CSZ and was estimated at moment magnitudes of M_W 9.0. The horizontal distance from the edge of the CSZ megathrust is located approximately 100 miles (150 km) from Portland. The current US Geological Survey risk-based maximum credible earthquake for CSZ megathrust is M_W 9.0±0.2 (USGS, 2008).

2.5.1.2 Intraslab Earthquakes

Intraslab earthquakes occur within the subducting slab. They are problematic in the sense that they do not have a surface expression or rupture the ground surface and their seismicity generates deformation along many faults within the slab (Kirby et al., 2002). The CSZ has generated significant intraslab destructive earthquakes including the 2001 M_W 6.8 Nisqually earthquake in the Puget lowland. The estimated depth to the subducting Juan de Fuca plate under Kelso is approximately 50 km (31 miles) (Blair et al., 2011). Therefore, intraslab earthquakes are a seismic hazard that must be considered.

2.5.1.3 Crustal Earthquakes and Faults

Review of the US Geological Survey Quaternary Fault and Fold Database (USGS, 2006) indicates the site is not within close proximity (less than 25 km, 15.5 miles) to Quaternary faults (Figure 6).

2.5.1.4 Historical Seismicity

Regional historical seismicity information was acquired from the Advanced National Seismic System (ANSS) Comprehensive Catalog, hosted by the Northern California Data Center (NCEDC), and is presented on Figure C2. These data include earthquakes with magnitudes exceeding M 2.5 within a 150-km (93 mile) radius of Portland, Oregon, recorded between 1963 and 2017 (NDEDC, 2017). Magnitudes within the ANSS dataset are recorded as local magnitude, surface-wave magnitude, body-wave magnitude, moment magnitude, and magnitude of completeness.

2.6 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 review of the liquefaction susceptibility map for Cowlitz County (Palmer et al., 2004; Figure 5), the site is shown as having a high liquefaction hazard. Based on the results of our analyses, we expect over 12 inches of liquefaction settlement and more than 2 feet of lateral spreading may occur following a code-based earthquake.

3 CONCLUSIONS AND RECOMMENDATIONS

3.1 Geotechnical Design Considerations

The project site is underlain by loose to medium dense and soft to medium stiff, saturated, potentially liquefiable sand with interbedded silt to depths greater than 80 feet bgs. Conventional foundation support on shallow spread footings is not feasible without some form of mitigation and consideration of earthquake risk. For our evaluation, we have considered two options for foundation support—each having different levels of risk associated with damage during an earthquake.

3.2 Seismic Design Considerations

3.2.1 Code-Based Seismic Design Parameters

The current seismic design criteria for this project are based on the 2015 IBC. Due to the potential for liquefaction of site soils, the site should be considered Site Class F. However, in accordance with ASCE 7-10, 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 E. If the period of the structure is greater than 0.5 second, seismic site response analyses will be required. The seismic design criteria, in accordance with the 2015 IBC, are summarized in Table 1.

Table 1. 2015 IBC Seismic Design Parameters

Parameter	Short Period	1 Second
Maximum Credible Earthquake Spectral Acceleration	S_S = 0.95	S_1 = 0.44
Site Class	E*	
Site Coefficient	Fa = 0.96	Fv = 2.40
Adjusted Spectral Acceleration	S_{MS} = 0.91	S_{M1}=1.05
Design Spectral Response Acceleration Parameters	S_{DS} = 0.61	S_{D1} = 0.70

g= Acceleration due to gravity
* Site Class E can be used if the fundamental period of the new structure is less than 0.5 second. If the period of the structure is greater than 0.5 second, seismic site response analyses will be required.

3.3 Foundation Alternatives

The soils at the site present a challenge for support of the proposed facility during a code-based earthquake. The site is underlain mostly by loose to medium dense, granular soils that are susceptible to liquefaction. The presence of liquefaction-susceptible soils and the associated potential of seismically-induced liquefaction settlement would affect footings, mats, and slabs.

Despite the challenges of supporting foundations on the shallow soils at the site, the underlying deeper soils would likely provide competent support for deep foundations, which could extend to depths of 50 feet bgs or greater. We have developed two different foundation alternatives, which are discussed in the following paragraphs.

- Mitigate potentially liquefiable soils with soil improvement (vibro-replacement stone columns or deep soil mixing [DSM]), used in conjunction with shallow spread footings with grade beams or a mat foundation.
- Use deep foundations.

The use of isolated shallow spread footings without soil improvement is not considered feasible due to potential for liquefaction and the associated differential settlement expected during a code-based earthquake. Footings supported on piles or soils that have been improved can be used to support the proposed building; however, each has different levels of damage risk.

3.4 Soil Improvement

The detailed design for soil improvement, such as stone columns, are typically completed by a design-build contractor. However, our evaluation suggests that stone columns and DSM are geotechnically feasible soil improvement alternatives.

Depending on the settlement limitations of the new structures, it may not be necessary to improve all the potentially liquefiable soils at the site. 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 25 to 30 feet thick. The current crust thickness is on the order of 6-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 (e.g., vibro-replacement) 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 3 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 40 feet bgs or deeper. The extent beyond the intended area of improvement should be approximately one-third the depth of improvement. This would correspond to approximately 25 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 percent or more.

3.5 Shallow Footings or Mats on Improved Soil

Shallow spread footings bearing on native silt and sand that has been improved with stone columns or DSM may be used to support loads associated with the proposed development. Stone columns can be used to reinforce soft non-plastic silt or loose granular soils to both mitigate liquefaction and provide improved bearing capacities for static (non-seismic) conditions. This technique involves advancing a vibratory probe to the target depth, then placing aggregate through the tip of the probe in lifts that are compacted by raising and lowering the probe. Depending on the spacing and diameter of the densified columns, soil types, and the depth of treatment, allowable bearing pressures of 2,500 to 5,000 pounds per square foot (psf) can be achieved beneath the spread footings. The actual diameter and spacing is typically determined by a specialty subcontractor, with the design reviewed by the project geotechnical engineer. Diameters typically range from about 24 to 36 inches, spaced about 6 to 8 feet on-center. Based on subsurface conditions encountered at the site, stone columns would need to extend to depths greater than 30 feet bgs.

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 250 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, or seismic ties, are not intended to vertically support column footings, but to help hold the building structure together during a code-based earthquake to provide for life safety. Grade beams between footings should be designed in accordance with the requirements of section 1810.3.12 of the 2015 IBC.

3.6 Deep Foundations

The impacts from post-earthquake settlement can be reduced by supporting the new building on piles. Piles would penetrate through the potentially liquefiable soils and derive their support from the underlying non-liquefiable soils present to depths of more than 80 feet bgs. We recommend that pile foundations for the proposed facilities, if used, consist of driven displacement piles such as closed-end steel pipe piles. Supporting the building on piles will provide support for the structure during an earthquake but will not provide vertical support to at-grade slabs (unless specifically designed and supported on piles). However, resisting the loads associated with lateral spreading using piles may not be feasible. Doing so may require a significant number of additional piles and using higher strength, thicker-walled piles.

Advantages of pile foundations include:

- No significant static or seismically-induced foundation settlement
- Uses locally available equipment and experienced local contractors

Disadvantages of pile foundations include:

- Differential settlement between pile-supported facilities and utilities or non-pile supported structures
- Requires specialty construction equipment and an experienced specialty contractor
- Piles would likely be more than 100 feet long

If pile foundations are used, additional specific design recommendations for pile foundations will be necessary, depending the type and size of piles selected. This could include additional exploration to estimate the required length of piles and consideration of lateral capacities that might control pile design.

Based on our discussions with the design team, we understand that shallow foundations supported on soil improvement (stone columns) is the most feasible option.

3.7 Floor Slabs

If site soils are improved using stone columns, satisfactory subgrade support for building floor slabs can be obtained from the silt and sand subgrade prepared in accordance with our recommendations presented in the Site Preparation, Wet/Freezing Weather and Wet Soil Conditions, and Select Granular Fill sections of this report. If the new structure is supported on piles and the slab is not designed to be pile supported, settlement, damage, and repair or replacement of the slab should be anticipated following a code-based earthquake.

A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade. Depending on the design of the stone columns, it may be necessary to provide a 12- to 24-inch-thick working surface, which would help distribute foundation and slab loads. 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 1½ inch, and has less than 5 percent by dry weight passing the US Standard No. 200 Sieve.

Floor slabs supported on an improved subgrade and base course prepared in accordance with the preceding recommendations may be designed using a modulus of subgrade reaction (k) of 150 pounds per cubic inch (pci). Alternatively, if the slab is designed to be supported on unimproved soil, it should be designed using a modulus of subgrade reaction (k) of 100 pci.

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 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 two trucks and five school buses per day for a 20-year design life.

The minimum recommended pavement section thicknesses are provided in Table 2. Depending on weather conditions at the time of construction, a thicker aggregate base course section could be required to support construction traffic during preparation and placement of the pavement section.

Table 2. Minimum AC Pavement Sections

Traffic Loading	AC (inches)	Base Course (inches)	Subgrade
Pull-in Car Parking Only	3	9	Firm subgrade as verified by PBS personnel*
Drive Lanes and Access Roads	4	12	

* 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 percent 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 new structure will involve clearing and grubbing of the existing vegetation or demolition of possible existing structures. Demolition should include removal of 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.

4.2 Excavation

The near-surface soils at the site can be excavated with conventional earthwork equipment. Sloughing and caving should be anticipated. 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. Trench cuts should stand relatively vertical to a depth of approximately 4 feet bgs, provided no groundwater seepage is present in the trench walls. 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 some sloughing may 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 vertical walls are desired for cuts deeper than 4 feet bgs. 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 Structural Fill

The extent of site grading is currently unknown; however, PBS estimates that cuts and fills will be on the order of up to 2 feet to raise the grades within the proposed site. 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 percent 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.

4.3.1 On-Site Soil

On-site soils encountered in our explorations are generally suitable for placement as structural fill during dry weather when moisture content 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 percent of the maximum dry density, as determined by ASTM D1557 (modified proctor).

4.3.2 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 percent 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 percent of the maximum dry density, as determined by ASTM D1557.

4.3.3 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 percent (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 percent of the maximum dry density, as determined by ASTM D1557.

4.3.4 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 percent of the maximum dry density, as determined by ASTM D1557.

4.3.5 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 percent 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 percent 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 percent 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 percent 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 percent 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 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

4.3.6 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 percent 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

At the time of this report was prepared, the size, type, and location of structures and additions had not been finalized. Depending on the location of the structures, additional exploration and analyses may be required. 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, 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 CPTs. 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.

7 REFERENCES

ASCE. (2010). Minimum Design Loads for Buildings and Other Structures (ASCE 7-10).

Blair, J. L., McCrory, P. A., Oppenheimer, D. H., and Waldhauser, F. (2011, revised 2013). A Geo-referenced 3D model of the Juan de Fuca slab and associated seismicity: US Geological Survey Data Series 633, v.1.2, https://pubs.usgs.gov/ds/633/.

Federal Emergency Management Agency (FEMA). 2015. [MAP] Flood Insurance Rate Map Cowlitz County, Washington and Incorporated Areas. National Flood Insurance Program, Map Number 53015C0518G, Panel; 518 of 1020.

IBC. (2015). International Building Code. Country Club Hills, IL: International Code Council, Inc. Washington State Amendments to the International Building Code 2009 Edition, Effective July 1, 2010.

Kirby, Stephen H., Wang, Kelin, Dunlop, Susan (2002). The Cascadia Subduction Zone and related subduction systems—seismic structure, intraslab earthquakes and processes, and earthquake hazards: US Geological Survey Open-File Report 02-328.

Livingston, V. E. (1966). [Map] Geology and mineral resources of the Kelso-Cathlamet area, Cowlitz and Wahkiakim Counties, Washington. Washington Division of Mines and Geology, Bulletin 54, scale 1:24,000.

NCEDC (2017), Northern California Earthquake Data Center. UC Berkeley Seismological Laboratory, dataset doi:10.7932/NCEDC.

Palmer, S. P., Magsino, S. L., Bilderback, E. L., Poelstra, J. L., Folger, D. S., and Niggemann, R. A. (2004). [Map] Liquefaction Susceptibility Map of Cowlitz County, Washington. Washington Division of Geology and Earth Resources, Open File Report 2004-20, scale 1:100,000.

US Geological Survey (USGS). (2006). Quaternary fault and fold database for the United States, accessed March 13, 2018, from USGS web site: http//earthquake.usgs.gov/hazards/qfaults/

US Geological Survey (USGS). (2008). Earthquake hazards program: Conterminous states probabilistic maps and data.

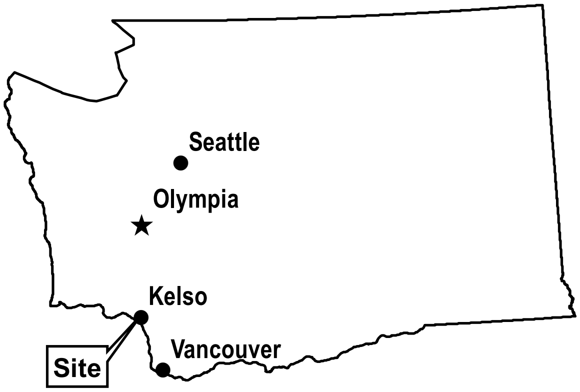
Washington State Department of Transportation (WSDOT SS). (2018). Standard Specifications for Road, Bridge, and Municipal Construction, M 41-10, Olympia, Washington.

Yeats, R. S., Graven, E. P., Werner, K. S., Goldfinger, Chris, and Popowski, T. A. (1996). Tectonics of the Willamette Valley, Oregon, in Rogers, A. M., Walsh, T. J., Kockelman, W. J., and Priest, G. R., eds., Assessing earthquake hazards and reducing risk in the Pacific Northwest: US Geological Survey Professional Paper 1650, v. 1, p. 183–222.

Yelin. S., and Patton, H. J. (1991). Seismotectonics of the Portland, Oregon, Region. Bulletin of the Seismological Society of America, Vol. 81, No. 1, pp/ 109-130.

Figures

Source: ESRI Topographic



VICINITY MAP

WALLACE ELEMENTARY SCHOOL ADDITIONS
KELSO, WASHINGTON

DATE: AUG 2018 · PROJECT: 73400.001

PBS

FIGURE

1

L:\Projects\73000\73400-73499\73400\73400.001\gis\fig_01_WA.mxd

EXPLANATION

⊕ B-1 - Boring name and approximate location

▲ CPT-1 - CPT name and approximate location

SOURCES: Google Earth 2017

1 inch = 50 feet

0 25 50 100'

SITE PLAN

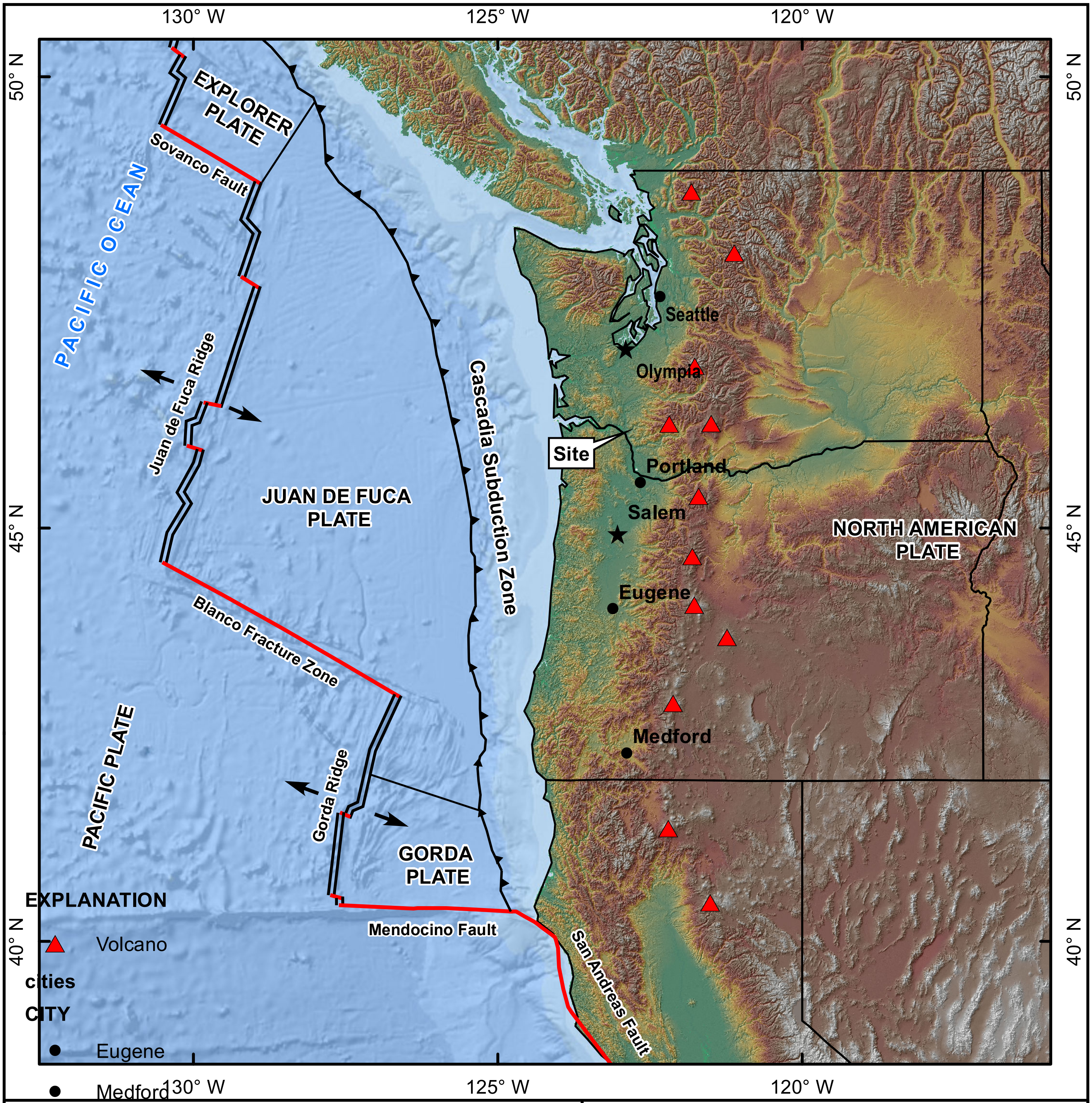
WALLACE ELEMENTARY
SCHOOL ADDITIONS
KELSO, WASHINGTON

DATE: AUG 2018 · PROJECT: 73400.001

PBS

FIGURE
2

L:\Projects\73000\73400-73499\73400\73400.001\gis\fig_02.mxd



EXPLANATION

- 🔺 Volcano

cities

CITY

- ● Eugene
- ● Medford
- ● Portland
- ★ Salem
- Transform boundary
- Spreading ridge
- Thrust fault

Sources:
1) SRTM 30-meter DEM
2) ESRI World Oceans Basemap
3) USGS Tectonic Plate Boundaries

TECTONIC SETTING OF THE PACIFIC NORTHWEST

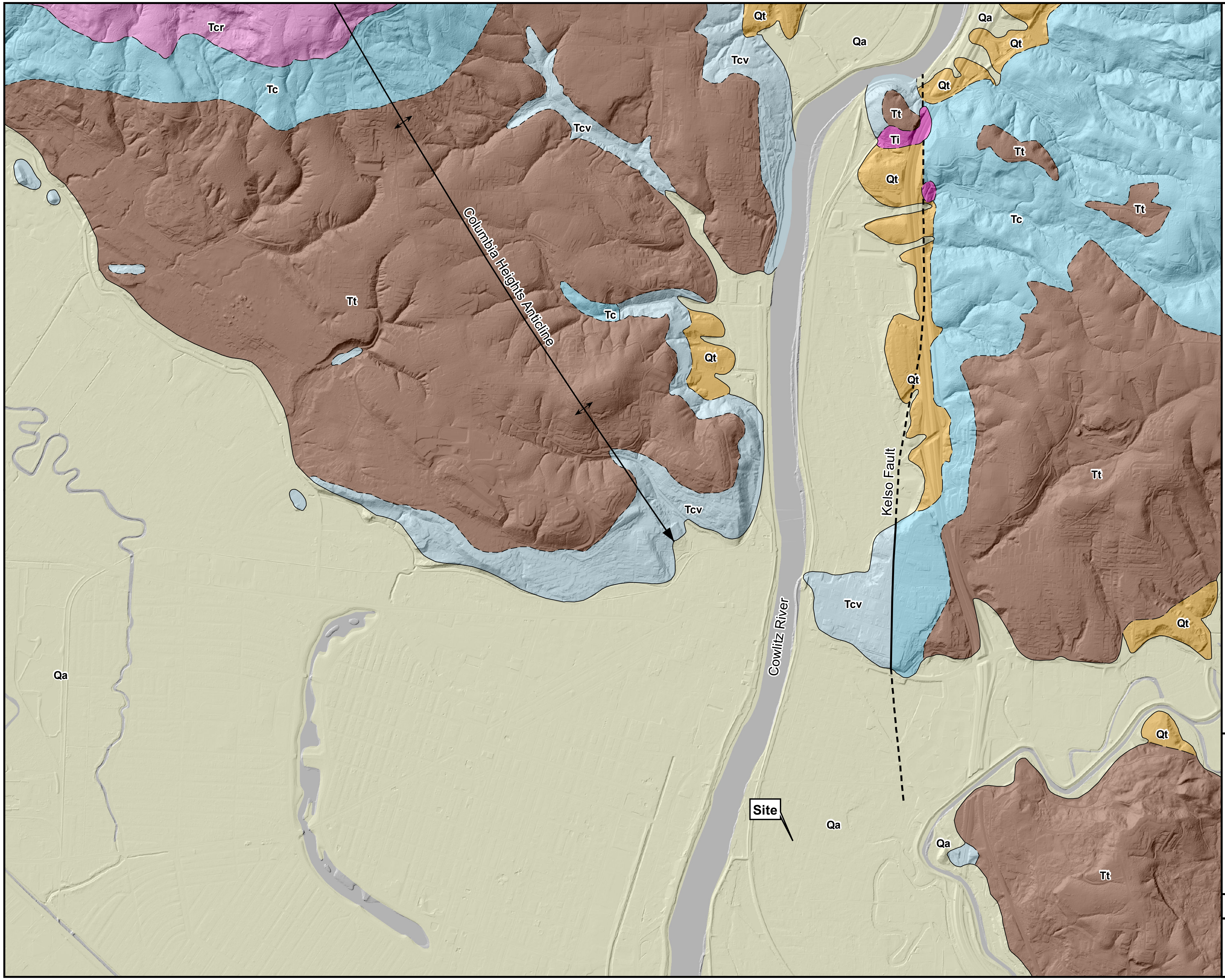
WALLACE ELEMENTARY SCHOOL ADDITIONS
KELSO, WASHINGTON

DATE: AUG 2018 · PROJECT: 73400.001

PBS

FIGURE
3

L:\Projects\73000\73400-73499\73400\73400.001\gis\fig_03_tectonics.mxd



EXPLANATION

Qa	Alluvium - sand, gravel, silt, and peat deposits along stream courses
Qt	Terrace deposits - silt and fine sand along valley walls
Tt	Troutdale Formation - sedimentary rocks
Tcr	Columbia River Basalt Group - volcanic rocks
Ti	Cowlitz Formation - intrusive rocks
Tcv	Cowlitz Formation - volcanic rocks
Tc	Cowlitz Formation - sedimentary rocks
―	Contact - solid where known; dashed where inferred
━	Fault - solid where known; dashed where inferred
↕►	Anticline with arrow indicating direction of plunge

SOURCES: Washington Department of Natural Resources LiDAR 2017,2010,2005 and Livingston, V.E., Jr., 1966, Geology and mineral resources of the Kelso-Cathlamet area, Cowlitz and Wahkiakum Counties, Washington: Washington Division of Mines and Geology, Bulletin 54, scale 1:24,000

1 inch = 2,000 feet

0 1,000 2,000 4,000ft

GEOLOGIC MAP

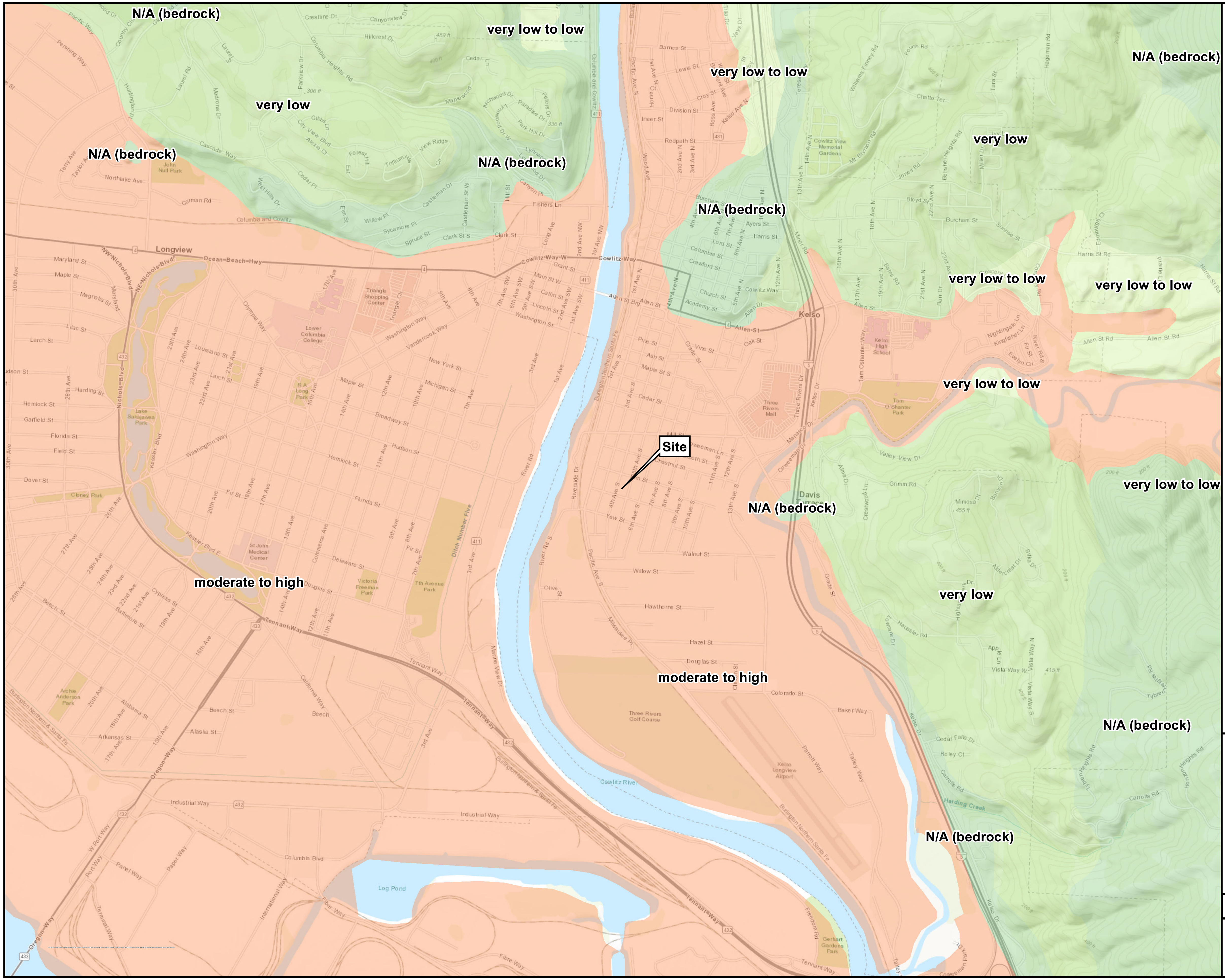
WALLACE ELEMENTARY
SCHOOL ADDITIONS
KELSO, WASHINGTON

DATE: AUG 2018 · PROJECT: 73400.001

PBS

FIGURE
4

L:\Projects\73000\73400-73499\73400\73400.001\gis\fig_04.mxd



EXPLANATION

- 🟧 Liquefaction susceptibility: Moderate to high
- 🟨 Liquefaction susceptibility: Very low to low
- 🟩 Liquefaction susceptibility: Very low
- 🟩 Liquefaction susceptibility: Bedrock

NOTE: Boundaries are approximate

SOURCES: Liquefaction Susceptibility Map of Cowlitz County, Washington by Palmer et al. (2004), ESRI Topographic Basemap

1 inch = 2,000 feet

0 1,000 2,000 4,000ft

LIQUEFACTION SUSCEPTIBILITY MAP

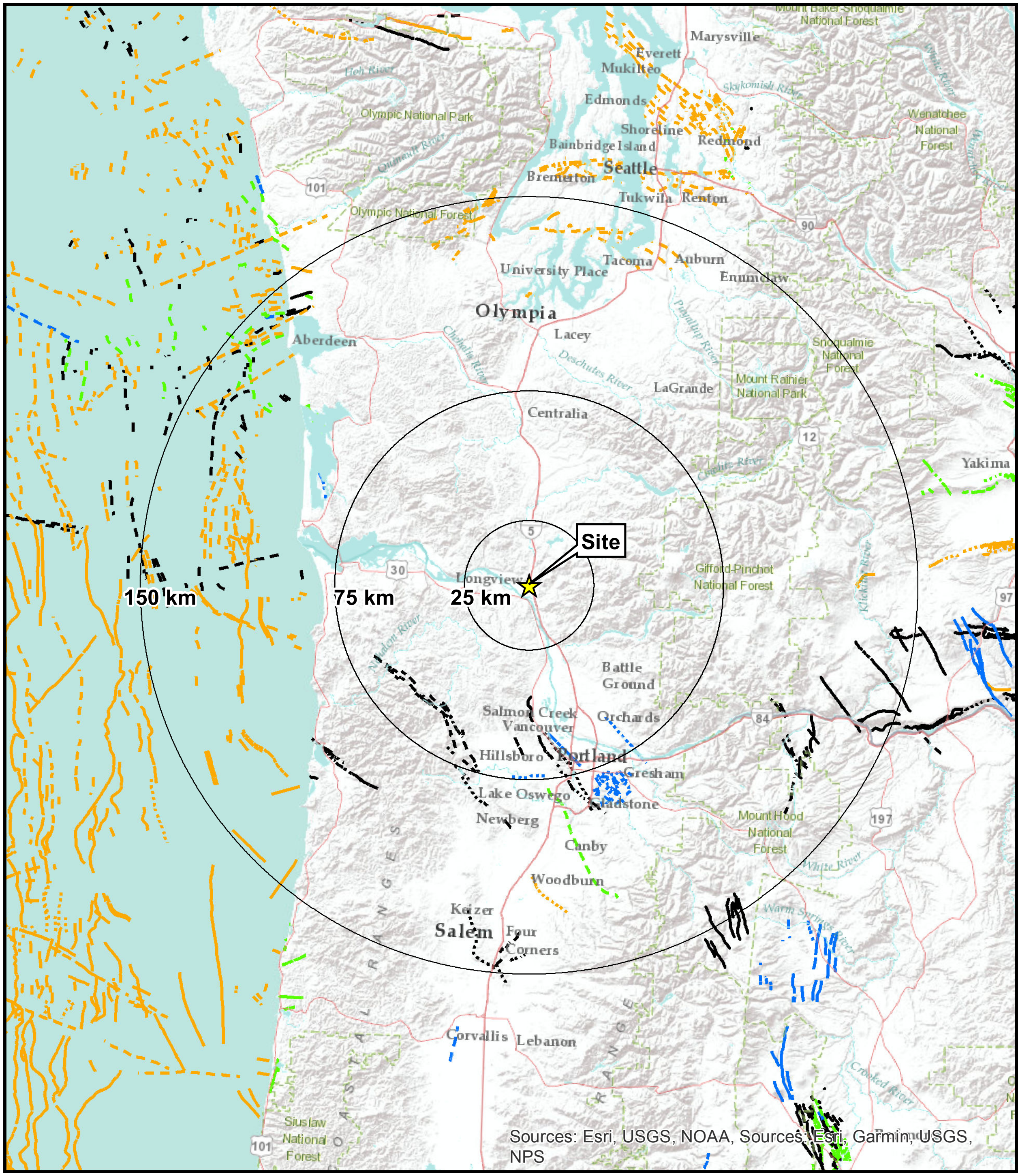
WALLACE ELEMENTARY
SCHOOL ADDITIONS
KELSO, WASHINGTON

DATE: AUG 2018 · PROJECT: 73400.001

PBS

FIGURE
5

\\pbsenv.lan\\Projects\\73000\\73400-73499\\73400\\73400.001\\gis\\fig_05_liquefaction.mxd



Sources: Esri, USGS, NOAA, Sources: Esri, Garmin, USGS, NPS

\\pbsenv.lan\I\Projects\73000\73400-73499\73400\73400.001\gis\fig_06_Qfaults_8.5x11_portrait.mxd

EXPLANATION

USGS (2006) Quaternary fault traces; solid where well constrained, dashed where moderately constrained, and dotted where inferred

🟧 < 15,000 years - latest Quaternary

🟩 < 130,000 years - late Quaternary

🟦 < 750,000 years - middle and late Quaternary

⬛ < 1.6 million years - undifferentiated Quaternary

1 inch = 60 kilometers

0 15 30 60km

REGIONAL FAULT MAP

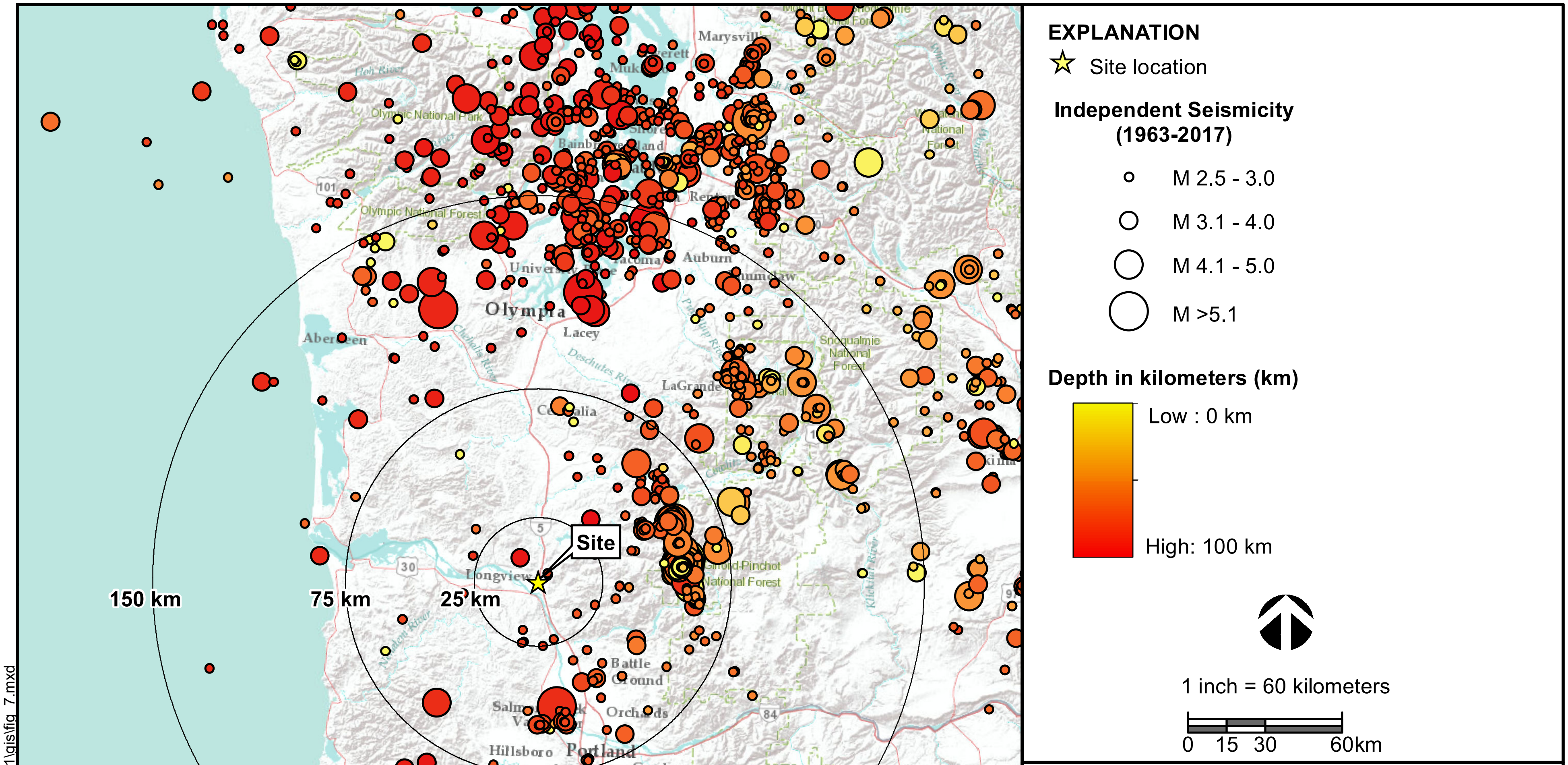
WALLACE ELEMENTARY SCHOOL ADDITIONS KELSO, WASHINGTON

DATE: AUG 2018 · PROJECT: 73400.001

PBS

FIGURE

6



HISTORICAL SEISMICITY

WALLACE ELEMENTARY SCHOOL ADDITIONS KELSO, WASHINGTON

DATE: AUG 2018 · PROJECT: 73400.001

PBS

FIGURE

7

\\pbsenv.lan\I\Projects\73000\73400-73499\73400\73400.001\gis\fig_7.mxd

Appendix A

Field Explorations

Appendix A: Field Explorations

A1 GENERAL

PBS explored subsurface conditions at the project site by advancing three drilled borings and 2 cone penetration test (CPT) probes. The drilled borings were advanced to 31.5 feet bgs on July 12, 2018. The two CPTs were completed to depths of up to 81.5 feet bgs on June 28, 2018. The approximate locations of the explorations are shown on Figure 2, Site Plan. The procedures used to advance the borings, 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 Mobile B58 drill rig provided and operated by Holt Services, Inc., of Vancouver, Washington, 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 (OD), 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 CONE PENETRATION TESTS (CPTs)

A3.1 Field Procedures

Explorations CPT-1 and CPT-2 were advanced using a truck mounted Hogentogler 20-ton Electronic Dutch cone CPT rig. CPTs were performed by Oregon Geotechnical Explorations and results were reviewed and used for site specific seismic design calculations.

Before the start of testing, the truck is jacked up and leveled on four pads to provide a stable reaction for the cone thrust. 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.

A3.2 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 completed 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 A4 and A5. The results of shear wave velocity testing are included on Figure A6.

A4 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

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.

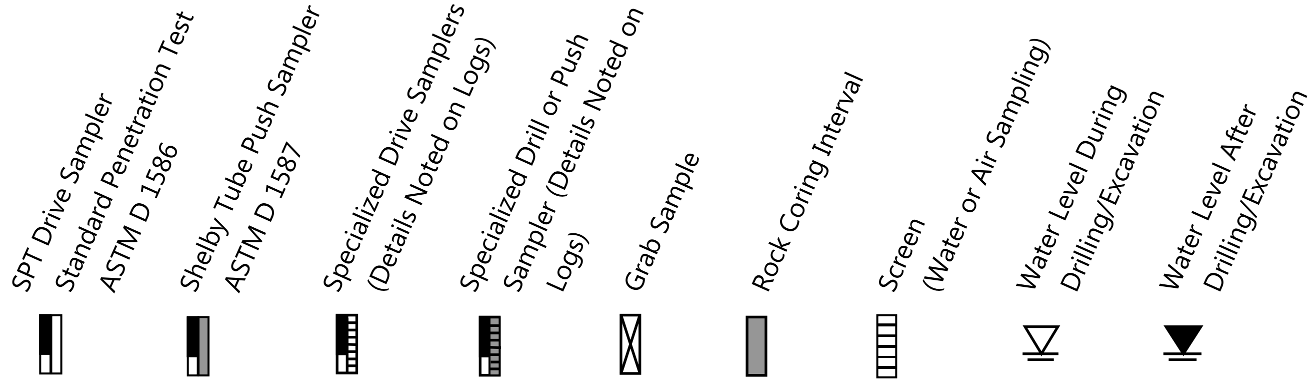
Cobble and boulder deposits may include a description of the matrix soils, as defined above.

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

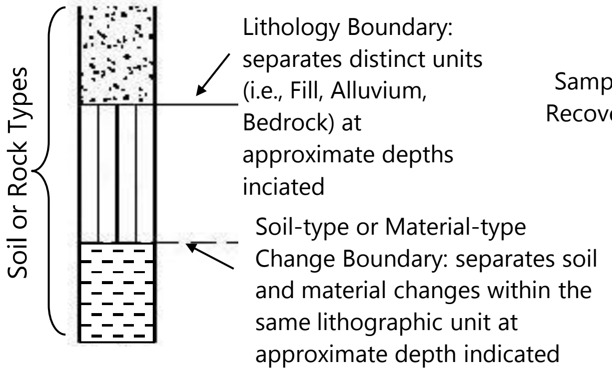
Table A-2
Key To Test Pit and Boring Log Symbols

SAMPLING DESCRIPTIONS

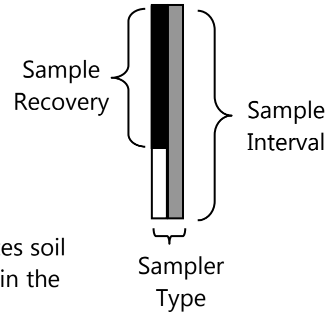


LOG GRAPHICS

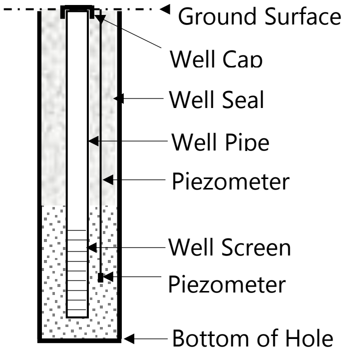
Soil and Rock



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

WALLACE ELEMENTARY SCHOOL
KELSO, WASHINGTON

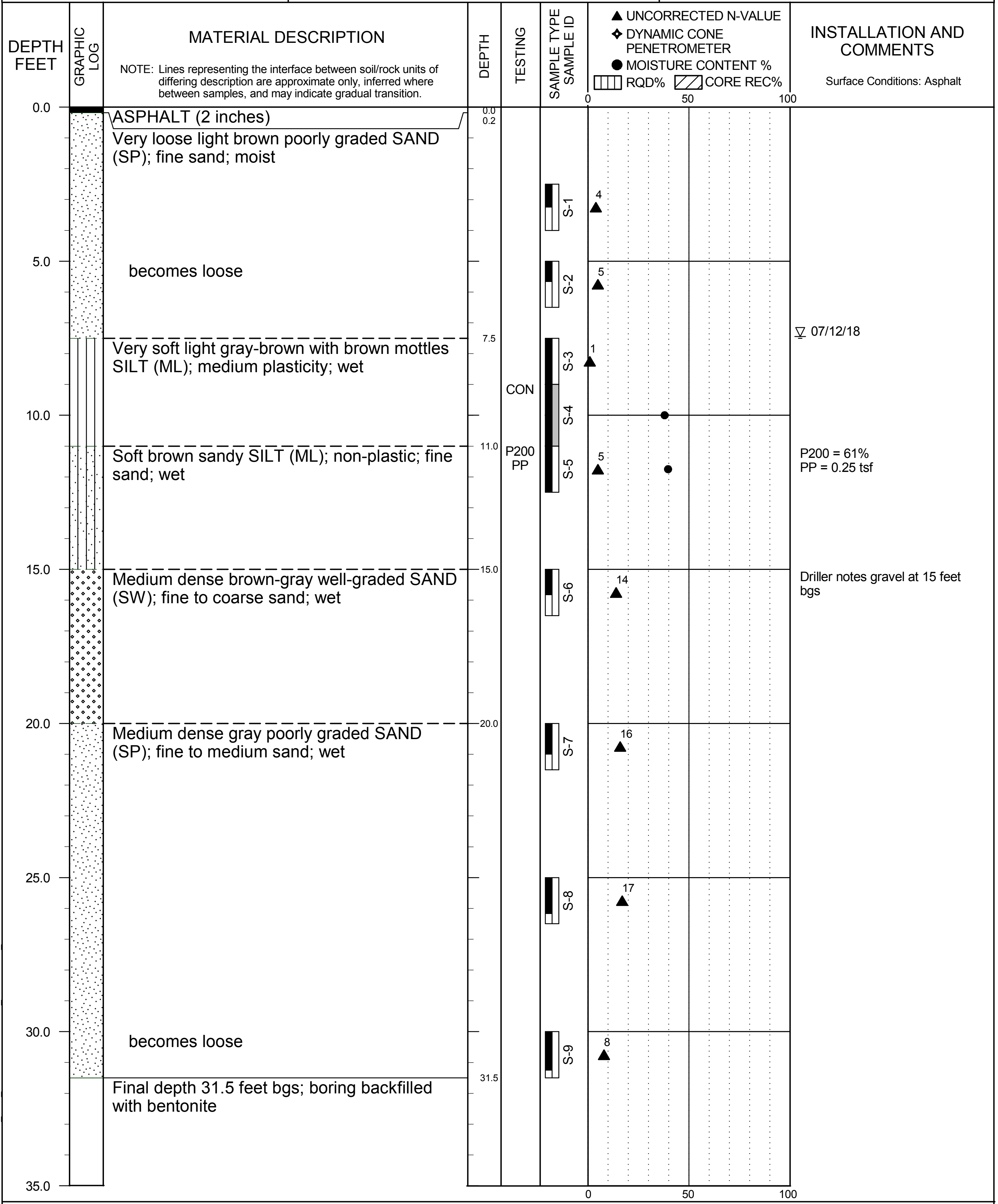
BORING B-1

PBS PROJECT NUMBER:
73400.001

APPROX. BORING B-1 LOCATION:
46.134334,-122.912667

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: Asphalt



DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
0.0	ASPHALT (2 inches)	0.0				
	Very loose light brown poorly graded SAND (SP); fine sand; moist	0.2		S-1	4	
5.0	becomes loose			S-2	5	
	Very soft light gray-brown with brown mottles SILT (ML); medium plasticity; wet	7.5		S-3	1	∇ 07/12/18
10.0			CON	S-4		
	Soft brown sandy SILT (ML); non-plastic; fine sand; wet	11.0	P200 PP	S-5	5	P200 = 61% PP = 0.25 tsf
15.0	Medium dense brown-gray well-graded SAND (SW); fine to coarse sand; wet	15.0		S-6	14	Driller notes gravel at 15 feet bgs
20.0	Medium dense gray poorly graded SAND (SP); fine to medium sand; wet	20.0		S-7	16	
25.0				S-8	17	
30.0	becomes loose			S-9	8	
	Final depth 31.5 feet bgs; boring backfilled with bentonite	31.5				
35.0						

DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: B. Portwood

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 83
LOGGING COMPLETED: 7/12/18

FIGURE A1

WALLACE ELEMENTARY SCHOOL
KELSO, WASHINGTON

BORING B-2

PBS PROJECT NUMBER:
73400.001

APPROX. BORING B-2 LOCATION:
46.134487,-122.912117

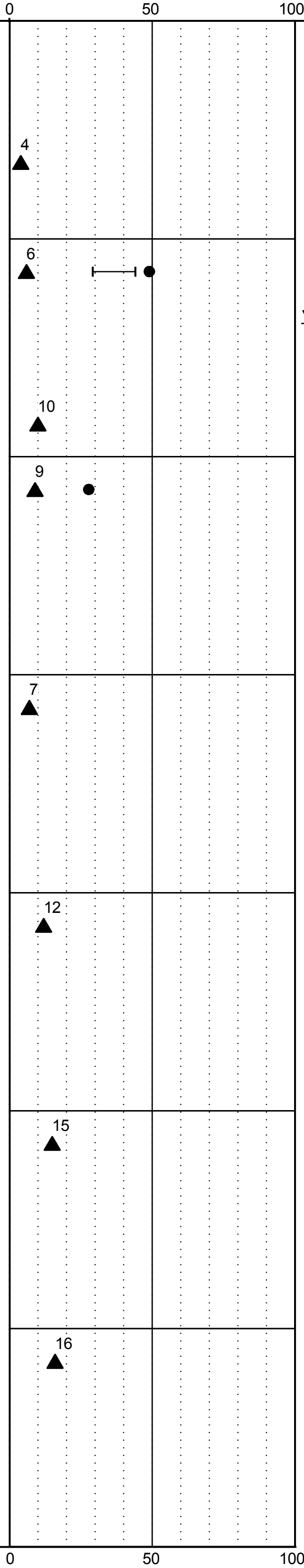
NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

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

INSTALLATION AND COMMENTS

Surface Conditions: Asphalt

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
0.0	ASPHALT (2 inches)	0.0				
	Very loose brown poorly graded SAND (SP-SM) with silt; non-plastic; fine sand; moist	0.2		S-1	4	
5.0	Medium stiff brown with gray mottles SILT (ML); medium plasticity; moist to wet	5.0	ATT	S-2	6	LL = 44 PL = 29 PI = 15 ∇ 07/12/18 Groundwater at 6 feet bgs
				S-3		
	Loose brown poorly graded SAND (SP-SM) with silt; non-plastic; fine sand; wet	8.0		S-4	10	
10.0			P200	S-5	9	P200 = 6%
15.0	becomes gray-brown; fine to medium sand			S-6	7	
						Driller notes gravel at 18 feet bgs
20.0	becomes medium dense			S-7	12	
25.0	Medium dense brown-gray with red-brown clasts well-graded GRAVEL (GW) with sand; fine to coarse sand; fine to coarse, subrounded gravel; wet	25.0		S-8	15	
30.0	Medium dense brown-gray poorly graded SAND (SP); fine sand; wet	30.0		S-9	16	
	Final depth 31.5 feet bgs; boring backfilled with bentonite	31.5				
35.0						



DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: B. Portwood

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 83
LOGGING COMPLETED: 7/12/18

FIGURE A2

WALLACE ELEMENTARY SCHOOL
KELSO, WASHINGTON

BORING B-3

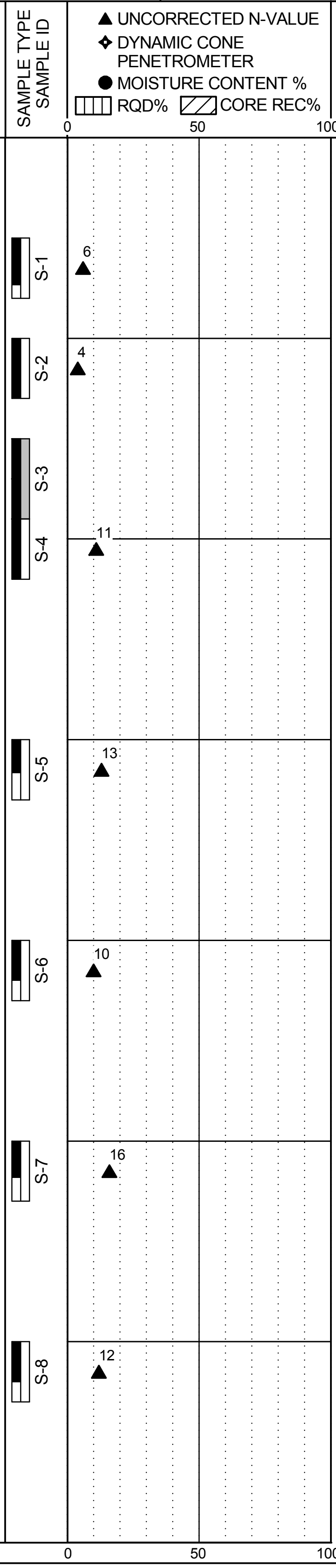
PBS PROJECT NUMBER:
73400.001

APPROX. BORING B-3 LOCATION:
46.133648,-122.912425

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

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	INSTALLATION AND COMMENTS
0.0	ROOT ZONE (2 inches)	0.0 0.2	
	Loose brown poorly graded SAND (SP); fine sand; moist		
5.0	Soft brown to brown with gray mottles SILT (ML); low to medium plasticity; moist	5.0	▽ 07/12/18
	Medium dense brown poorly graded SAND (SP); fine sand; wet	9.0	
10.0	Medium dense gray well-graded SAND (SW); fine to coarse sand; wet 3-inch layer of brown silt	11.0	
15.0			Driller notes gravel at 16.5 feet bgs
20.0	becomes loose, brown-gray		
25.0	becomes medium dense		
30.0	Medium dense gray poorly graded SAND (SP); fine sand; wet	30.0	
	Final depth 31.5 feet bgs; boring backfilled with bentonite	31.5	
35.0			



DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: B. Portwood

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 83
LOGGING COMPLETED: 7/12/18

FIGURE A3

PBS Engineering and Environmental Inc.
4412 SW Corbett Avenue
Portland, Oregon 97239
www.pbsusa.com

CPT: CPT-1
Total depth: 32.32 ft, Date: 6/28/2018

Project: **73400.001 Wallace Elementary School**
Location: **Kelso, Washington**

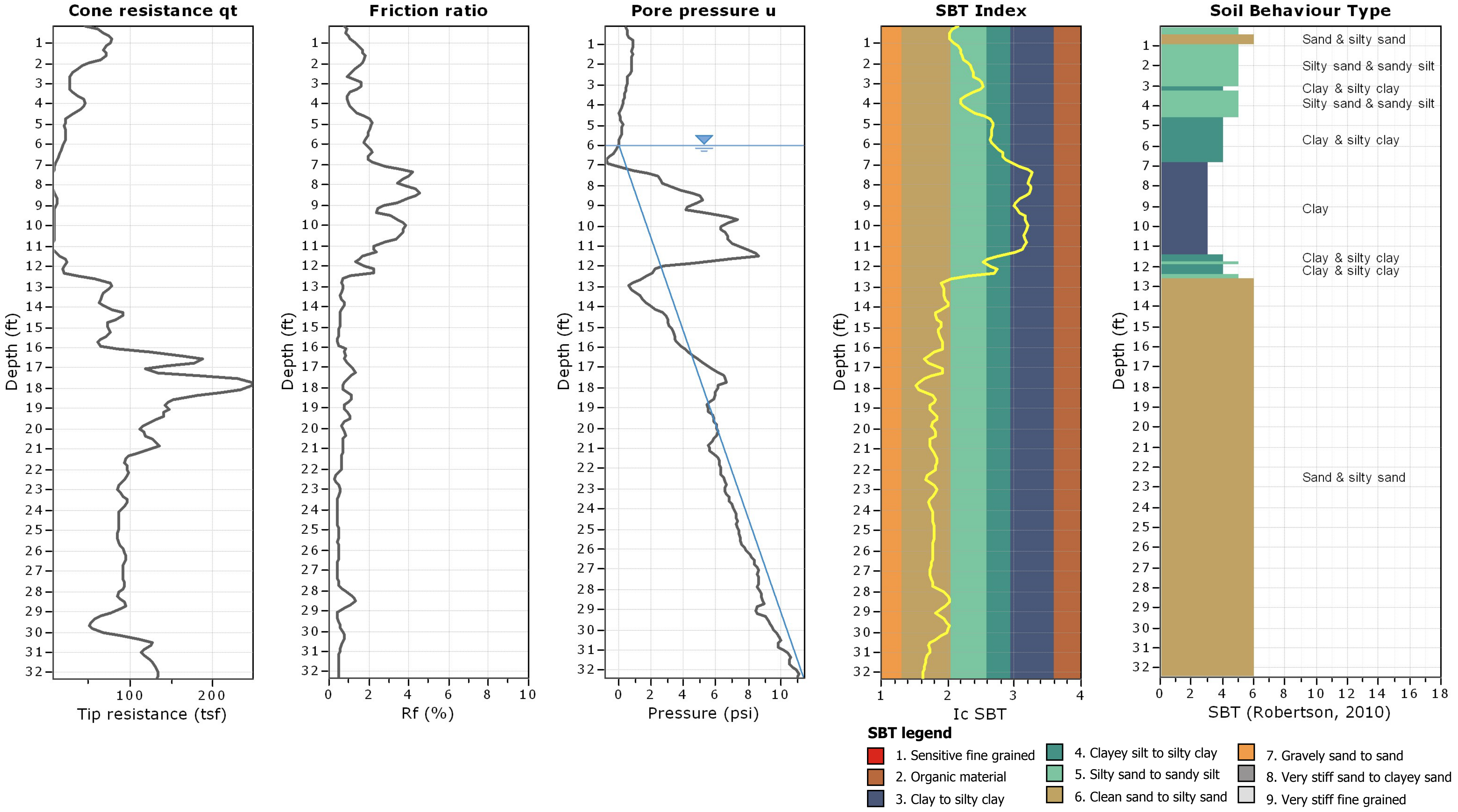


FIGURE A4

PBS Engineering and Environmental Inc.
4412 SW Corbett Avenue
Portland, Oregon 97239
www.pbsusa.com

CPT: CPT-2
Total depth: 81.53 ft, Date: 6/28/2018

Project: **73400.001 Wallace Elementary School**
Location: **Kelso, Washington**

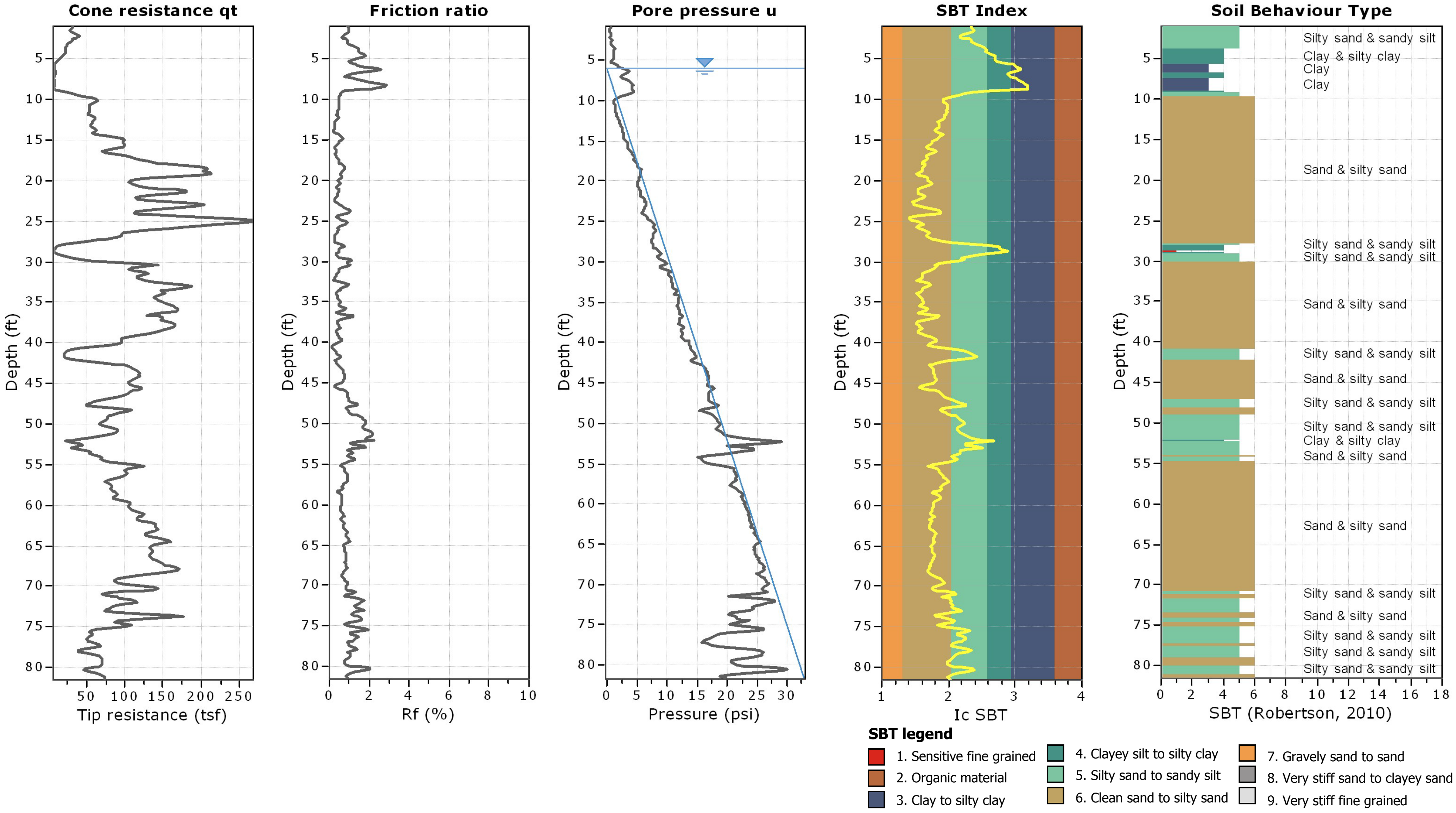
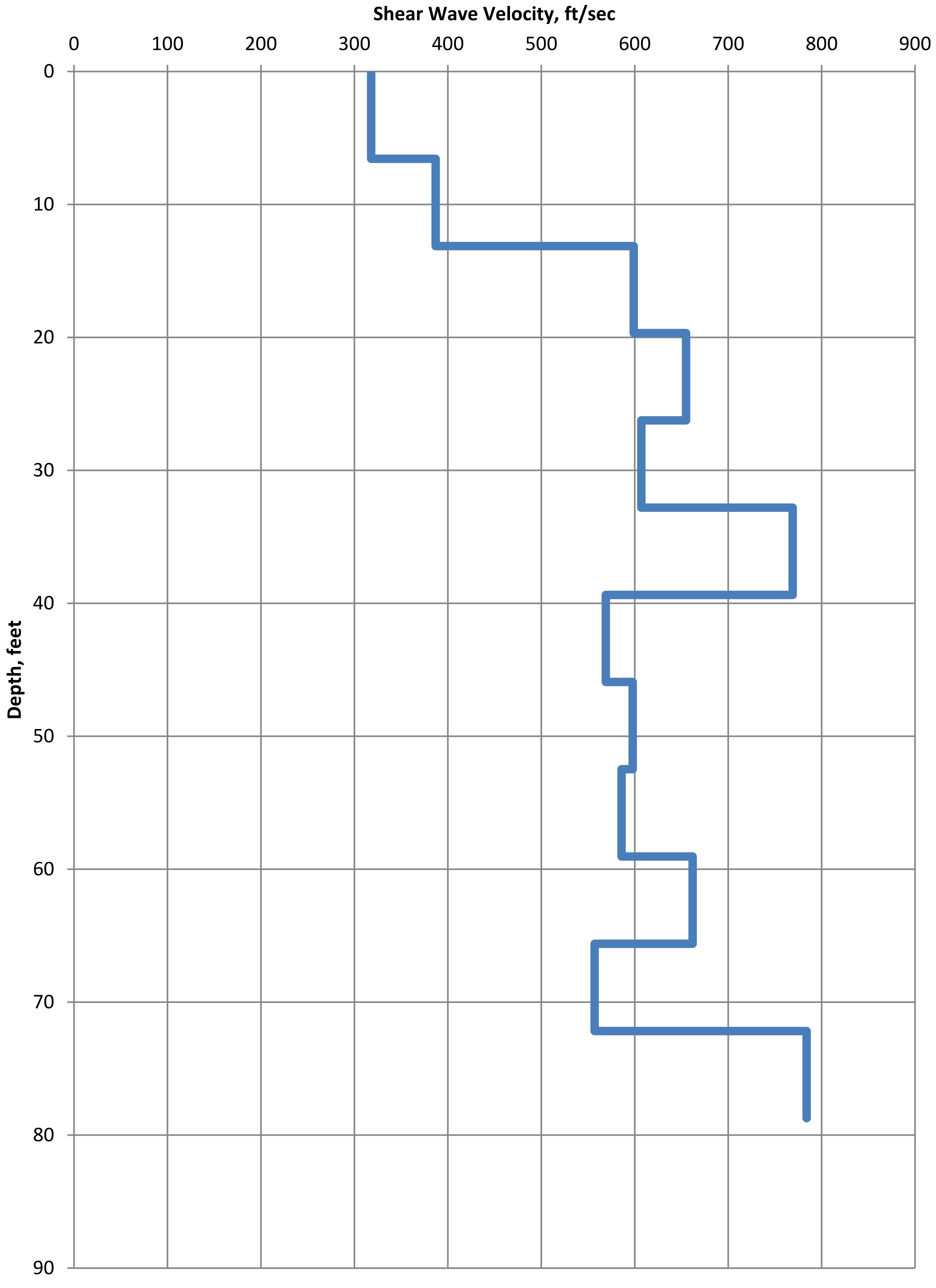


FIGURE A5



PBS

SHEAR WAVE VELOCITY PROFILE

WALLACE ELEMENTARY SCHOOL
KELSO, WASHINGTON

AUG 2018
73400.001

FIGURE
A6

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 logs of the borings in Appendix A and on Figure B2, Summary of Laboratory Data, in Appendix B.

B2.3 Atterberg Limits

Atterberg limits were determined on select samples for the purpose of classifying soils into various groups for correlation. The results of the Atterberg limits test, which included liquid and plastic limits, are plotted on Figure B1, Atterberg Limits Test Results, and on the explorations logs in Appendix A where applicable.

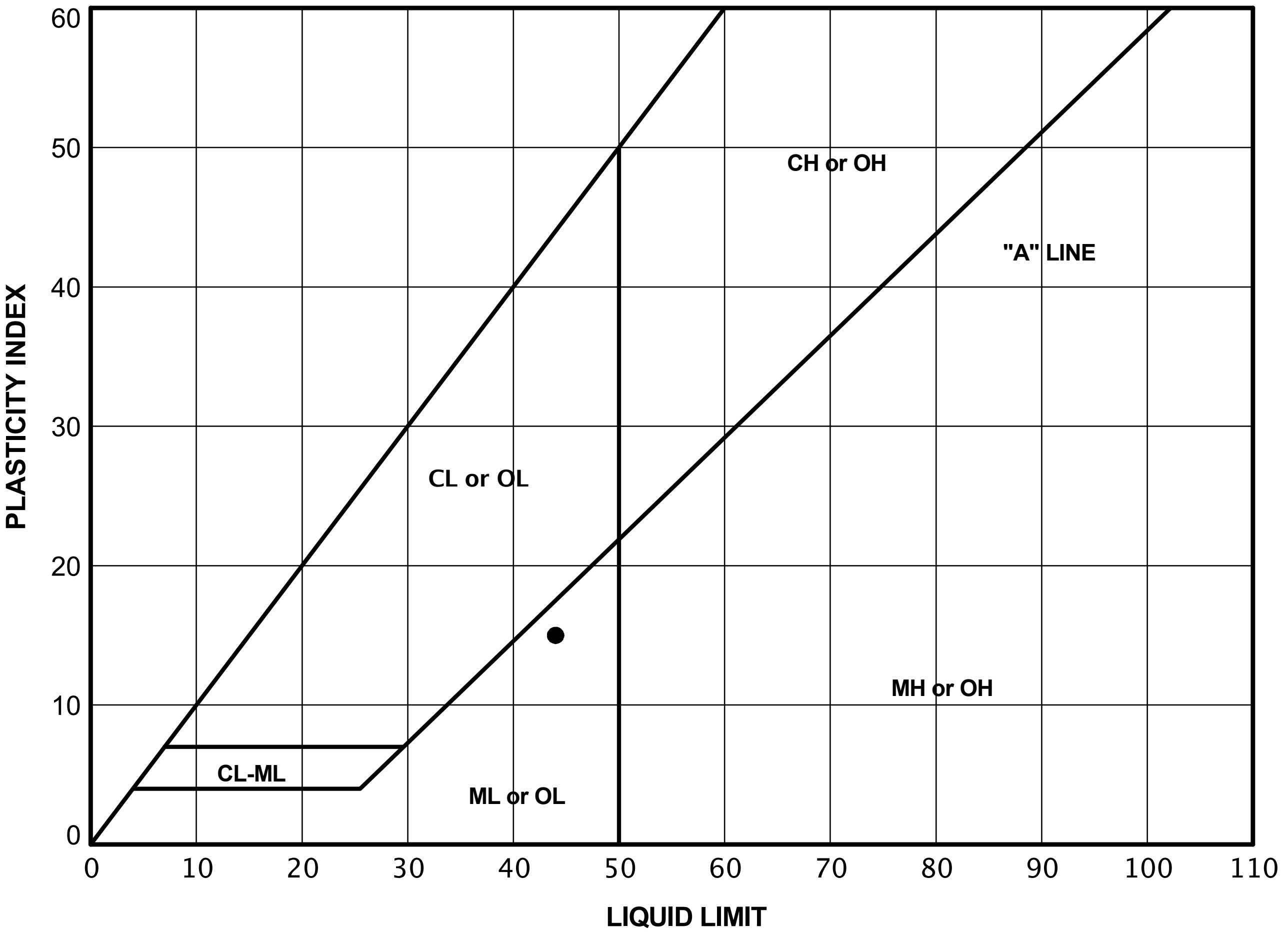
B2.4 Grain-Size Analyses (P200 Wash)

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.

B2.5 One-Dimensional Consolidation

One-dimensional consolidation testing was conducted on a relatively undisturbed sample collected from B-1 at a depth of 9 feet to obtain quantitative data for use in evaluating potential settlement resulting from loads imposed from proposed fill, pavement, and structures at the site. The test specimen was placed in a one-dimensional, fixed-ring consolidometer and loads were applied to the specimen. The resulting change in thickness of the soil sample was monitored with time. Upon completion of primary consolidation, the next load increment was applied. The specimen was kept moist until the first load increment was applied, at which point the specimen was inundated with water. The results of the consolidation tests are presented on Figure B2. The curve of the plot shows the percent strain that occurred in the test specimen under various magnitudes of applied constant load.

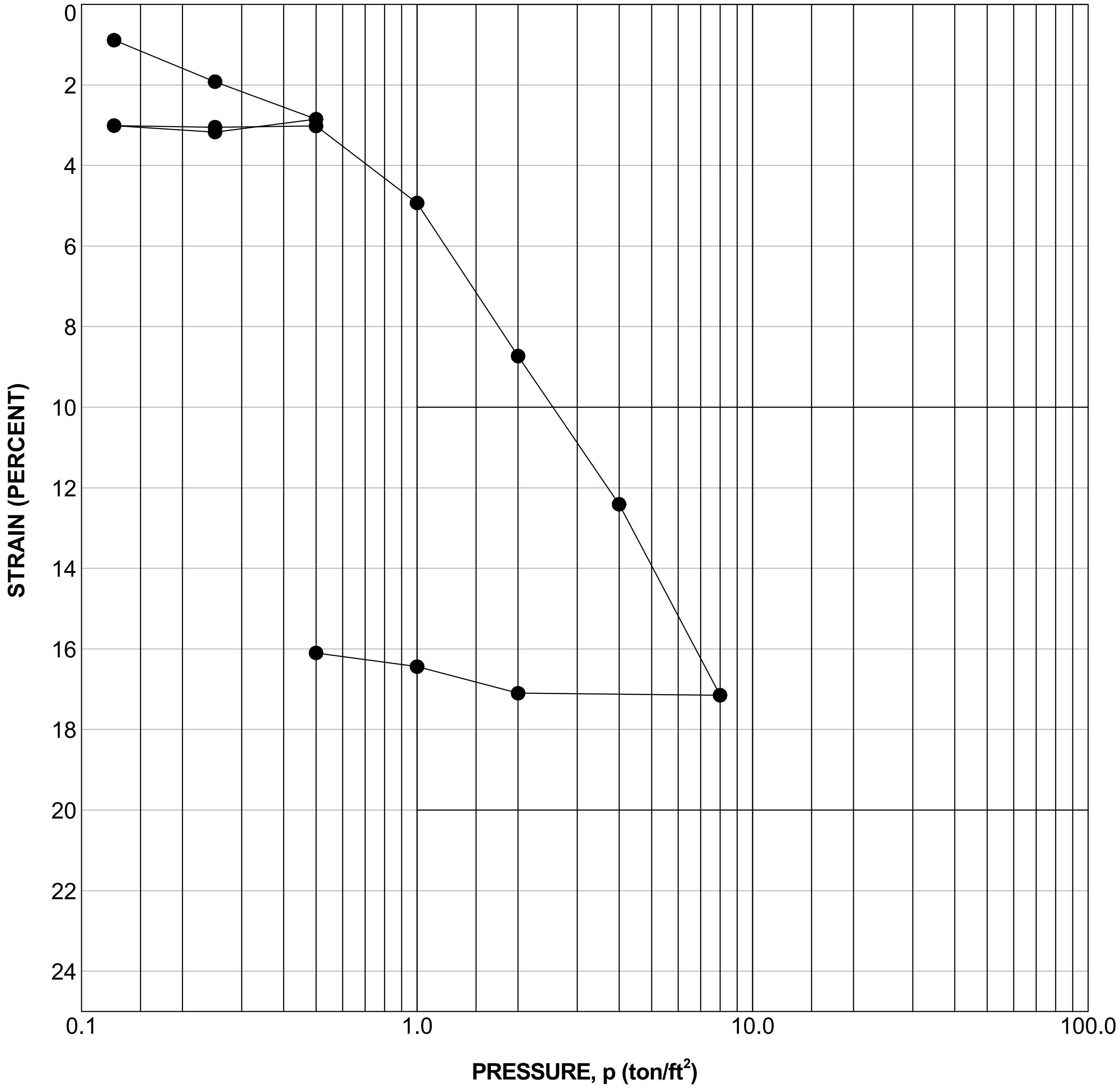
TEST METHOD: ASTM D4318



KEY	EXPLORATION NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	NATURAL MOISTURE CONTENT (PERCENT)	PERCENT PASSING NO. 40 SIEVE (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-2	S-2	5.0	48.9	NA	44	29	15

CONSOLIDATION TEST RESULTS

WALLACE ELEMENTARY SCHOOL KELSO, WASHINGTON	PBS PROJECT NUMBER: 73400.001



KEY	EXPLORATION NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	INITIAL MOISTURE CONTENT (PERCENT)	FINAL MOISTURE CONTENT (PERCENT)	INITIAL DRY DENSITY (PCF)
●	B-1	S-4	9.0	37.9	34.4	76.1

FIGURE B2

SUMMARY OF LABORATORY DATA

WALLACE ELEMENTARY SCHOOL
KELSO, WASHINGTON

PBS PROJECT NUMBER:
73400.001

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-4	9		37.9							
B-1	S-5	11		39.6				61			
B-2	S-2	5		48.9					44	29	15
B-2	S-5	10		27.7				6			

FIGURE B3