Huntington Middle School Modernization
Kelso School District – No 458

Geotechnical Engineering Report

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Huntington Middle School Renovation and Gymnasium Addition
500 Redpath Street
Kelso, Washington

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

July 7, 2020
PBS Project 73400.004

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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 Huntington Middle School renovations and gymnasium addition located at 500 Redpath Street 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 renovations and gymnasium addition. 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

Seven borings were advanced to depths between 26.5 to 61.5 feet below the existing ground surface (bgs). The borings were logged and representative soil samples collected by a member of the PBS geotechnical engineering staff. In addition, two cone penetration tests (CPTs) were advanced to depths of approximately 29 and 59 feet bgs. The interpreted boring logs are presented as Figures A1 through A7 and 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 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, 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
- Liquefaction potential
- Seismic site hazard study that includes:
    - Discussion of geologic and seismic hazards impacting the site
    - Location of nearby faults
    - Evaluation of liquefaction potential
- Discussion of soil improvement options

- Discussion of foundation alternatives
- Shallow foundation design recommendations:
    - Minimum embedment
    - Allowable bearing pressure
    - Estimated settlement (total and differential)
    - Sliding coefficient
- Deep foundation options (if needed)
- Lateral earth pressures for embedded/retaining wall design, including:
    - Active, passive, and at-rest earth pressures
    - Seismic lateral force
    - Sliding coefficient
    - Groundwater and drainage considerations
- 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 2018 International Building Code (IBC) with State of Washington amendments
- Slab subgrade preparation recommendations

1.3 Project Understanding

PBS understands that the Kelso School District intends renovate the existing academic structures at Huntington Middle School and construct a new 5,500-square-foot gymnasium along the northwest side of the existing academic building, adjacent to the parking lot and bus lane.

2 SITE CONDITIONS

2.1 Surface Description

The site is located near the terminus of the Cowlitz River valley and is positioned east of the Cowlitz River, downslope and adjacent to Interstate 5. The school is bordered immediately to the west by North Kelso Avenue, to the south by densely vegetated slope and upslope residential properties, to the east by a densely vegetated slope and upslope Interstate 5, and to the north by North Kelso Avenue and an upslope residential property.

The school is composed of a primary academic building, oriented east to west, with two permanent structures located south of the primary academic structure, and a modular structure to the east. A track-and-field area is located south of the academic buildings and a large open grass field is located to the north. A staff parking lot is situated between the academic structures, and additional parking and bus drive lanes are located to the west at the front of the school.

Review of available LiDAR data indicates the site is surrounded by slopes to the north, east, and south that give rise to a higher terrace surface (WADNR, 2020). The academic structures are positioned on a higher surface that we interpret as a fluvial terrace associated with deposition by the Cowlitz River. The site slopes down from an elevation of approximately 37 feet above mean sea level (amsl) at the east end of the site where the modular structure is located, to elevations ranging from 17 to 21 feet along the west side of the academic buildings (NAVD88; WADNR, 2020). The contours on Figure 2 provide a coherent outline of this higher surface, with definitive slope break along the south, east, and north sides of the campus.

2.2 Geologic Setting

The site is located at the northern extent of the Portland Basin, a tectonic depression within the physiographic province of the Puget-Willamette Lowland (PWL). The PWL separates the Cascade Range from the Washington coastal range (Willapa Hills and Olympic Mountains) and extends from the Puget Sound to Eugene, Oregon (Yeats et al., 1996). At this location, the Portland Basin and PWL terminate against the geologic provinces of the Willapa Hills to the northwest and the South Cascades to the north and northeast.

The PWL 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 (Figure 3). Active northwest-trending faults accommodating clockwise rotation of the North American Plate are found throughout the Puget-Willamette lowland (Brocher et al., 2017; USGS, 2020). Older inactive faults and folds are found throughout the entire region, juxtaposing bedrock units, including the nearby Columbia Hills Anticline and Kelso fault (Figure 4).

2.2.1 Local Geology

The site is mapped as underlain by recent (Holocene) alluvium consisting of sand, gravel, silt, and peat. These sediments were deposited by the Cowlitz River and overlie older deformed sedimentary and volcanic rocks of Pliocene to Eocene age (Livingston, 1966; Figure 4). Pleistocene age river terraces positioned along the periphery of the Cowlitz River valley form flat surfaces higher in elevation than the Holocene alluvium. The rocks comprising the surrounding hillsides are deformed by northwest-trending anticline and syncline folds. Southeast of the site, the Cowlitz River valley is structurally bounded by the inactive north-south trending Kelso fault.

2.3 Subsurface Conditions

The site was explored by drilling seven borings, designated B-1 through B-7, to depths of 26.5 to 61.5 feet bgs. The drilling was performed by Holt Services, Inc., of Vancouver, Washington, using a track-mounted Mobile B-57 drill rig and mud rotary drilling techniques. Two additional cone penetration tests (CPTs) were completed to depths of approximately 29 and 59 feet bgs by Oregon Geotechnical Explorations using a track-mounted Geoprobe Model 6622 CPT rig.

PBS has summarized the subsurface units as follows:

SOFT SEDIMENTS (ML, CL, CH, SP-SM, SP, GP):	Interbedded fluvial sediments were encountered in borings B-1, B-2, B-4, B-5, and B-6 from the ground surface to the termination depth. In boring B-6, these soft sediments persisted to approximately 23 feet bgs before older terrace sediments were encountered. Fine-grained materials varied from low plasticity silts to high plasticity clays. Coarse-grained materials ranged in composition from poorly graded sand with silt to poorly graded gravel. Fine-grained materials were very soft to very stiff, with SPT N-values between 0 and 23 blows to advance the sampler 12 inches, were olive gray to brown in color, moist to wet, exhibited low to high plasticity, and contained fine-grained sand. Coarse-grained materials were very loose to dense, with SPT N-values between 0 and 37, primarily gray, moist to wet, with fine- to medium-grained sand, non-plastic fines, and included subrounded gravels at depth.
CONSOLIDATED SEDIMENTS (ML, CL, CH, SP-SM, SM):	Older terrace sediments were encountered in borings B-3 and B-7 from the ground surface to approximately 30 feet bgs. These materials were primarily coarse-grained sediments with lesser constituents of fine-grained materials. Materials varied from poorly graded sand with silt to silty sand. Materials were loose to medium dense with SPT N-values between 8 and 23, primarily brown in color, moist to wet, fine- to medium-grained sand, and contained low plasticity fines. With increasing depth, fine-grained materials were encountered at approximately 15 feet bgs. These materials are described as very stiff lean and fat clays with SPT N-values between 11 and 20, ranged in color from olive brown to greenish gray, were moist to wet, exhibited medium to high plasticity, and contained fine-grained sand and fine to coarse subrounded gravels.
WEATHERED BEDROCK (RX):	Weathered bedrock was encountered in boring B-3 at approximately 28 feet bgs. The material was weak, grayish green, friable, and platy. The material could be textured into sandy silt that was hard, with corresponding SPT N-values of greater than 50 blows required to advance the sampler 6 inches, exhibited low plasticity, and contained fine-grained sand.

The materials encountered within our borings were consistent with geologic mapping of the area. We note that B-3 and B-7, encountered more consolidated materials, beginning at the ground surface and throughout the entire borings, than the other borings. Softer materials were encountered from the ground surface to depths greater than 20 feet in other parts of the site.

2.4 Groundwater

Static groundwater was not directly measured in our borings due to the mud-rotary drilling techniques used. Pore pressure dissipation testing in CPT-1 indicates groundwater may be present at a depth of approximately 7 feet bgs at that location. Based on a review of regional groundwater logs available from the Washington State Department of Ecology, we anticipate that the static groundwater level is present at a depth of less than 10 feet bgs (WSDE, 2020). Please note that groundwater levels can fluctuate during the year depending on climate, irrigation season, extended periods of precipitation, drought, and other factors.

3 GEOLOGIC HAZARDS

Geologic and seismic hazards are defined as conditions associated with the geologic and seismic environment that could influence existing and/or proposed improvements. Geologic and seismic hazards that could affect the site's development are identified below and should be considered during the planning process.

3.1 Seismicity and Faulting

3.1.1 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.

3.1.1.1 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 500 to 530 years (Goldfinger et. al, 2012). 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 130 miles (210 km) from Kelso, Washington. The current US Geological Survey risk-based maximum credible earthquake for CSZ megathrust is M_W 9.0±0.2 (USGS, 2008).

3.1.1.2 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 40 km (Blair et al., 2011). Therefore, intraslab earthquakes are a seismic hazard that must be considered.

3.1.1.3 Crustal Earthquakes and Faults

Review of the US Geological Survey Quaternary Fault and Fold Database (USGS, 2006) indicate the site is not within close proximity (less than 25 km) to Quaternary faults (Figure 6). We note that the Kelso Fault is mapped as crossing the site (Figure 4); however, this fault is not considered active.

3.1.1.4 Historical Seismicity

Regional historical seismicity information was acquired from the Advanced National Seismic System (ANSS) Comprehensive Catalog, hosted by the Northern California Earthquake Data Center (NCEDC), and is presented on Figure 7. These data include earthquakes with magnitudes exceeding M 2.5, within a 150-km radius of the city of Kelso, Washington, and recorded between 1963 and 2017 (NCEDC, 2017). Magnitudes within the ANSS dataset are recorded as local magnitude, surface-wave magnitude, body-wave magnitude, moment magnitude, and magnitude of completeness.

3.2 Landslides

Landslides occur when masses of soil or rock lose stability due to over-saturation of the material, contributing to elevated pore water pressures; erosion of the terminal end of the slope, causing de-buttressing and generating additional instability of the overburden material; along geologic contacts; or as a combination of these processes in conjunction with one another. Seismically induced landslides may also occur during seismic events relating to the liquefaction of the soils in question or due to additional seismic loading. During such events, material may tumble, slide, or flow along the slide planes within the slope, along geologic contacts, or may protrude out of the exiting ground surface.

Based on a review of the WADNR Geologic Information Portals Landslide Catalog, the hills east of the site and upslope of Interstate 5 have numerous mapped landslides within the “Landslide Compilations” layer (WADNR, 2020b). In addition, a mapped landslide within the "Landslide Compilations" layer is located along the south side of the track at the toe of the terrace. This landslide deposit is distinguishable in WADNR LiDAR (WADNR, 2020a).

We note that the eastern upslope portion of the site has an appearance of a potential landslide deposit with several benches and several definitive slope breaks. This slope is not currently mapped as a landslide, and we cannot definitively say at this time if it is a landslide, or if it poses a hazard to downslope structures. Many unmapped, active, and inactive slides exist throughout the Pacific Northwest, and this may be one. Alternatively, this may be an erosional feature from incision of the prehistoric Cowlitz River. This slope has potential for instability due to seismic loading of a code-based seismic event.

3.2.1 Other Seismic Hazards

Other site-specific seismic hazards considered include fault rupture, seiche and tsunami inundation, liquefaction and lateral spreading, and earthquake shaking. Based on the location of the site's distance from any known Quaternary faults, the risk of fault rupture at the site is low. Due to the lack of free water bodies in the area and distance from the Pacific Ocean, the risk of seiche waves and tsunami inundation is absent. Based on the materials encountered during our explorations, and review of liquefaction susceptibility maps in the area, the risk of liquefaction at the site is moderate to high (WADNR, 2019b; Figure 5). Strong earthquake ground shaking will occur during a code-based seismic event on the CSZ as well as from local faults. Based on our current project understanding, our opinion is that effects of earthquake ground motions can be accounted for by using code-based design procedures and the code-based design response spectrum.

3.3 100-Year and 500-Year Floods

The site is located approximately 2,000 feet from the Cowlitz River, which is impounded by a levee system that provides flood protection (FEMA, 2015). Review of the FEMA Flood Insurance Map indicates the site is not expected to be impacted by a 100-year flood event (1% probability of flooding annually) or a 500-year flood event (0.2% probability of flooding annually) unless breaching or undermining of the levee system occurs.

3.4 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 moderate to high liquefaction hazard. The results of our analyses indicate approximately 5 inches of liquefaction settlement may occur following a code-based earthquake.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 Geotechnical Design Considerations

Soils encountered within the area of the proposed addition were generally very soft/loose. Borings B-3 and B-7 encountered stiffer/denser soils with higher relative SPT N-values. Our interpretation is that stiffer/denser, consolidated materials are more abundant along the east side of the campus between the modular building and staff parking lot. These materials are favorable for shallow foundations, without the need for soil improvement, as opposed to the materials encountered at the proposed gymnasium location, along the northwest side of the academic building, or the garden area to the southeast.

The proposed gymnasium location and garden area are underlain by zones of very loose to medium dense sand and silty sand that are susceptible to liquefaction resulting from a code-based earthquake. Conventional foundation support on shallow spread footings is not feasible at either location without some form of mitigation and consideration of earthquake risk.

4.2 Seismic Design Considerations

4.2.1 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. 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 2018 IBC, are summarized in Table 1.

Table 1. 2018 IBC Seismic Design Parameters

Parameter	Short Period	1 Second
Maximum Credible Earthquake Spectral Acceleration	S_S = 0.90	S_1 = 0.43
Site Class	D*	
Site Coefficient	Fa = 1.14	Fv = 1.87**
Adjusted Spectral Acceleration	S_{MS} = 1.02	S_{M1} = ***
Design Spectral Response Acceleration Parameters	S_{DS} = 0.68	S_{D1} = ***
MCE_G Peak Ground Acceleration	PGA = 0.41 g	
Site Amplification Factor at PGA	F_{PGA} = 1.19	
Site Modified Peak Ground Acceleration	PGA_M = 0.49 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. If the period of the structure is greater than 0.5 second, seismic site response analyses will be required.

** This value of Fv should only be used for the calculation of T_S

*** 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 $T > T_L$.

4.3 Foundation Alternatives

The soils at the proposed gymnasium location present a challenge for support of the proposed facility during a code-based earthquake. The site is underlain by very loose to medium dense, granular soils that are susceptible to liquefaction and compressible silt soils that are susceptible to consolidation settlement. The presence of soft, compressible and 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, below depths of 30 to 60 feet, would likely provide suitable support for deep foundations. We have developed two different foundation alternatives, which are discussed in the following paragraphs.

- Mitigate compressible and potentially liquefiable soils with soil improvement (stone columns/ modified aggregate piers 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 consolidation settlement and the associated differential settlement expected during a code-based earthquake. Foundations supported on piles or soils that have been improved can be used to support the proposed buildings; however, each has different levels of damage risk.

4.4 Soil Improvement

Due to the potential for liquefaction, soil improvement may be considered to adequately support structure foundations during a code-based earthquake. The detailed design for soil improvement, such as stone columns or DMS, are typically completed by a design-build contractor. Stone columns would provide suitable static support but would not provide adequate resistance to liquefaction in fine-grained silt soils. DSM can be used to provide both improved static support of new foundations and mitigate the effects of liquefaction.

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 30 feet thick. The current crust thickness is on the order of 6- to 8-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.

4.4.1 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 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

Due to the presence of fine-grained soils at the site, use of stone columns or vibro-compaction may be less effective than other techniques.

4.4.2 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.

4.5 Shallow Footings or Mats on Improved Soil

Shallow spread footings bearing on native soil 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 and types 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, soil improvement would need to extend to depths of 30 to 50 feet bgs.

4.5.1 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.

4.5.2 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.

4.5.3 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.

4.5.4 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).

4.5.5 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 2018 IBC.

4.6 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 approximately 30 to 60 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).

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

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.

4.7 Floor Slabs

If site soils are improved, 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 Imported Granular Materials 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.

4.8 Ground Moisture

4.8.1 General

The perimeter ground surface and hardscape 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.

4.8.2 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.

4.8.3 Vapor Flow Retarder

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

5 CONSTRUCTION RECOMMENDATIONS

5.1 Site Preparation

Construction of the proposed addition 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.

5.1.1 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.

5.1.2 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.

5.2 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.

5.3 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.

5.3.1 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).

5.3.2 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.

5.3.3 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.

5.3.4 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.

5.3.5 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.

5.3.6 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.

6 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.

7 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. 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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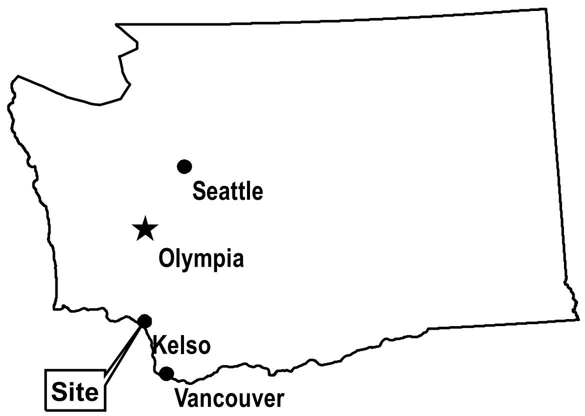
Figures

Source: ESRI Topographic



VICINITY MAP

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

DATE: JULY 2020 · PROJECT: 73400.004

PBS

FIGURE

1

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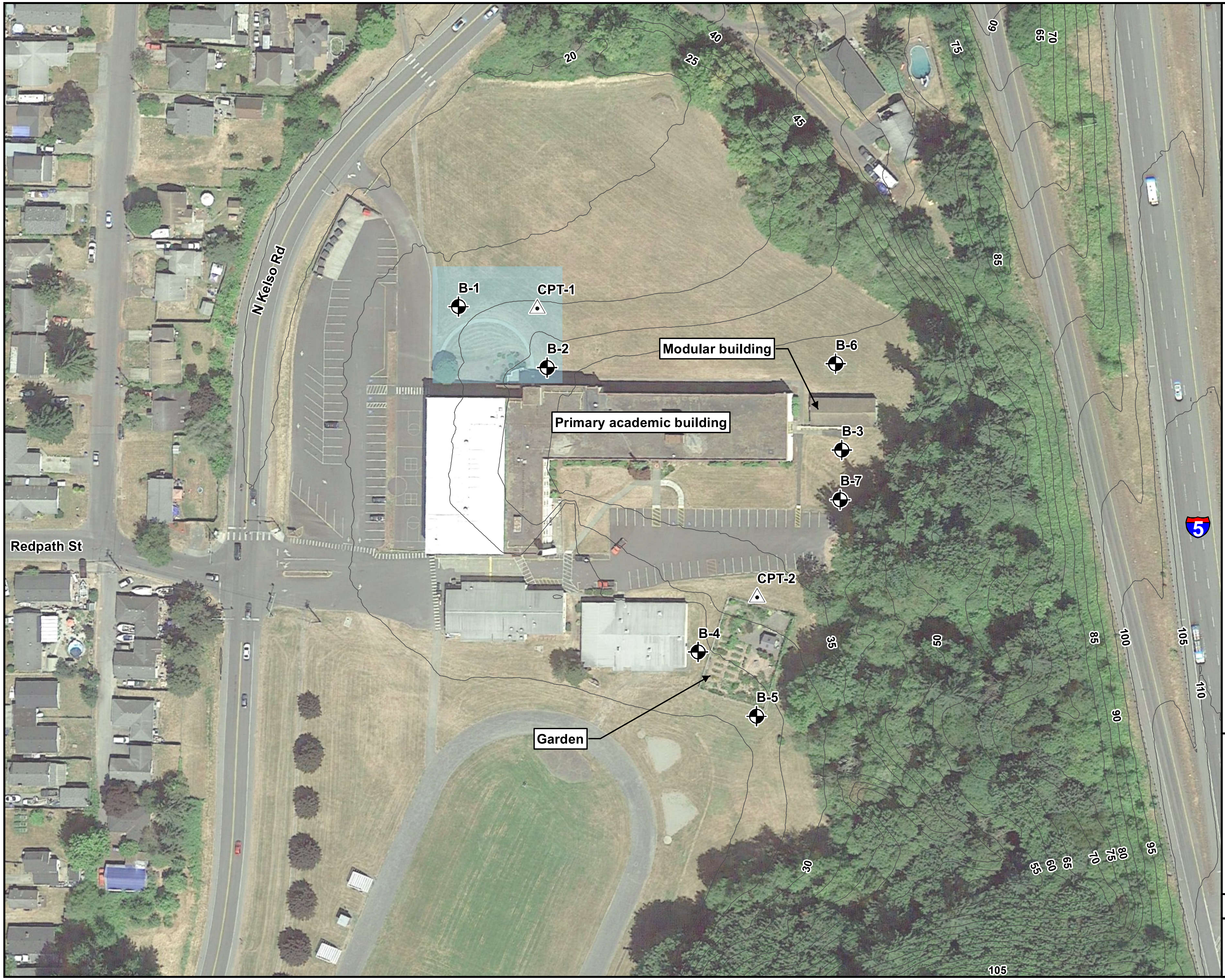

EXPLANATION

⊕ B-1 - Boring name and approximate location

△ CPT-1 - CPT name and approximate location

— 5-foot elevation contour

🟦 Proposed Gymnasium location

SOURCES: WADNR LiDAR (2020) and Google Earth (2018)

1 inch = 100 feet

0 50 100 200 Feet

SITE PLAN

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

DATE: JULY 2020 · PROJECT: 73400.004

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FIGURE
2

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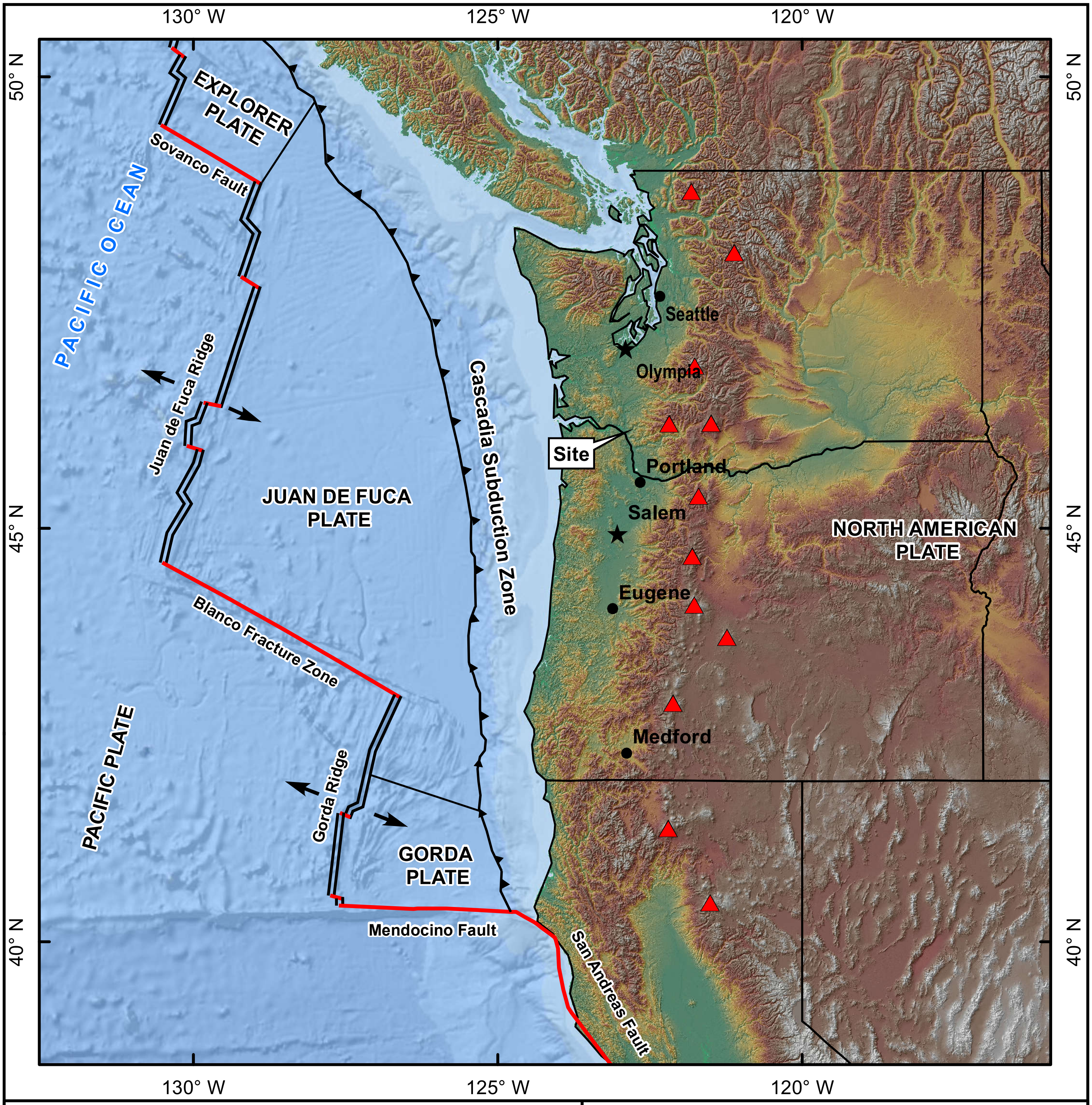

EXPLANATION

- ▲ Volcano
- 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

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

DATE: JULY 2020 · PROJECT: 73400.004

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FIGURE
3

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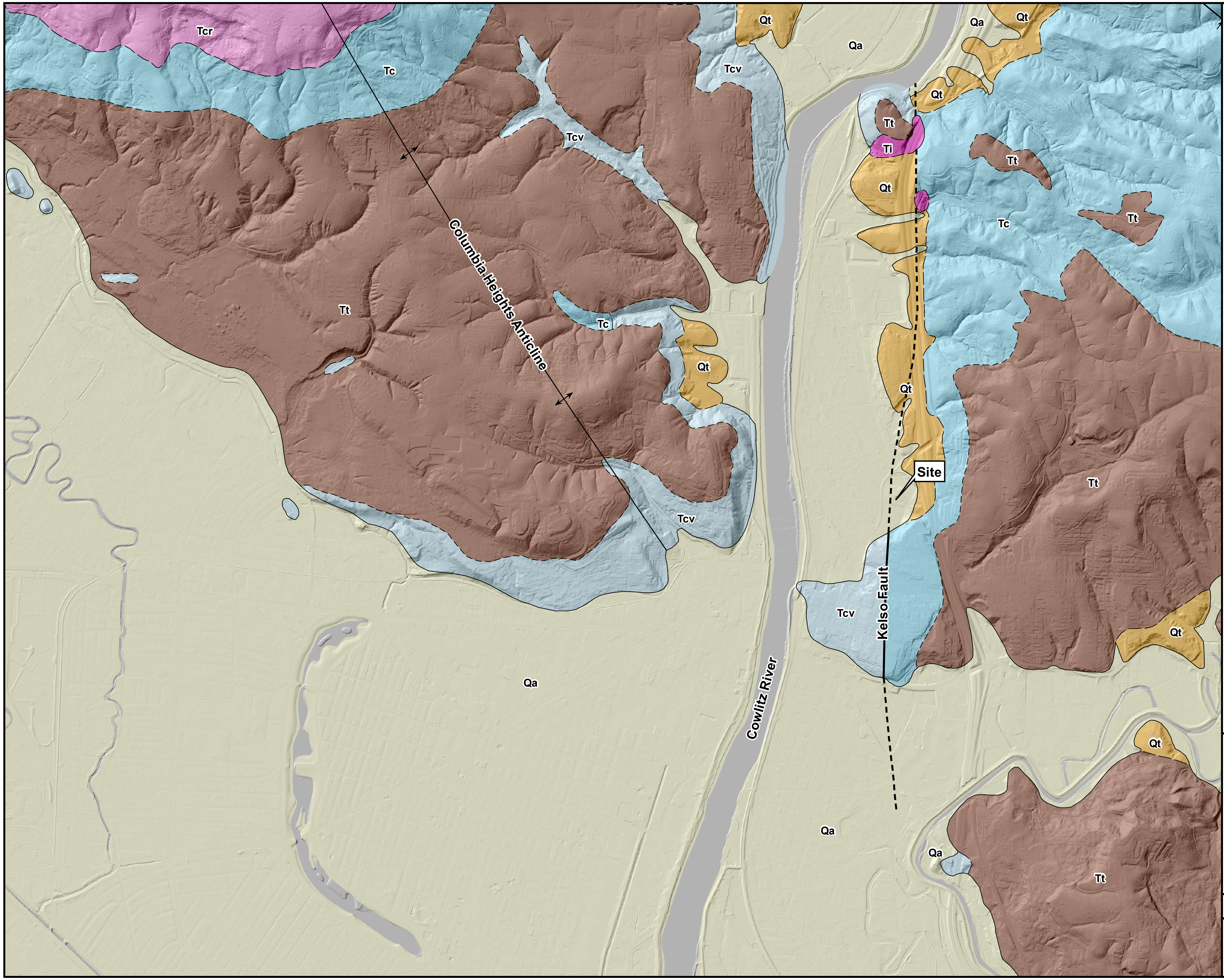

EXPLANATION

Qa	Alluvium - sand, gravel, silt, and peat deposits along stream courses
Qls	Landslide - areas of detached and slumped bedrock and overburden
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
┼	Syncline - solid where known; dashed where inferred

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

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

DATE: JULY 2020 · PROJECT: 73400.004

PBS

FIGURE
4

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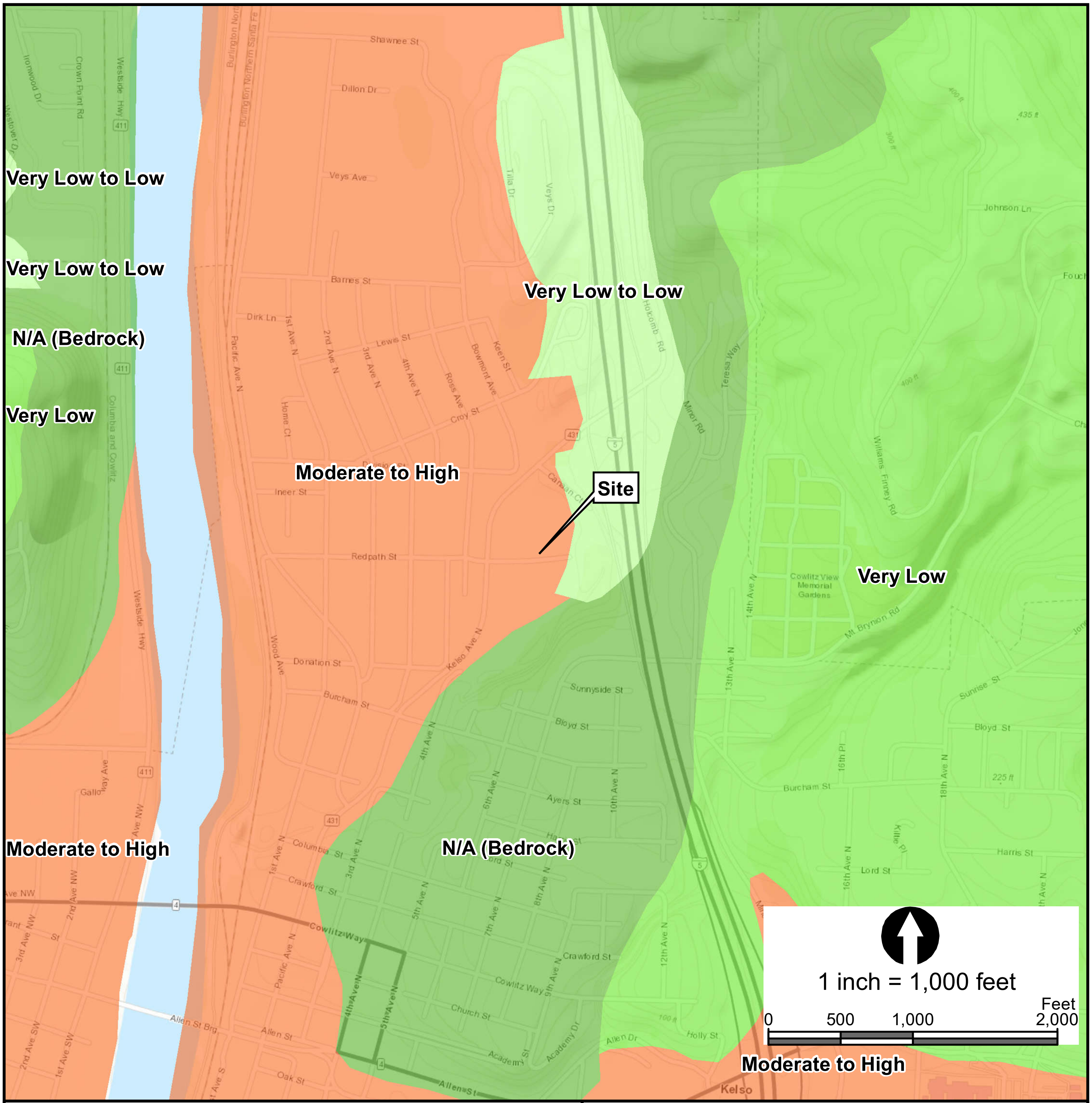

EXPLANATION

🟧 Liquefaction susceptibility: Moderate to high

🟩 Liquefaction susceptibility: Very low to low

🟩 Liquefaction susceptibility: Very low

🟩 Liquefaction susceptibility: Bedrock

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

LIQUEFACTION SUSCEPTIBILITY MAP

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

DATE: JULY 2020 · PROJECT: 0073400.004

PBS

FIGURE
5

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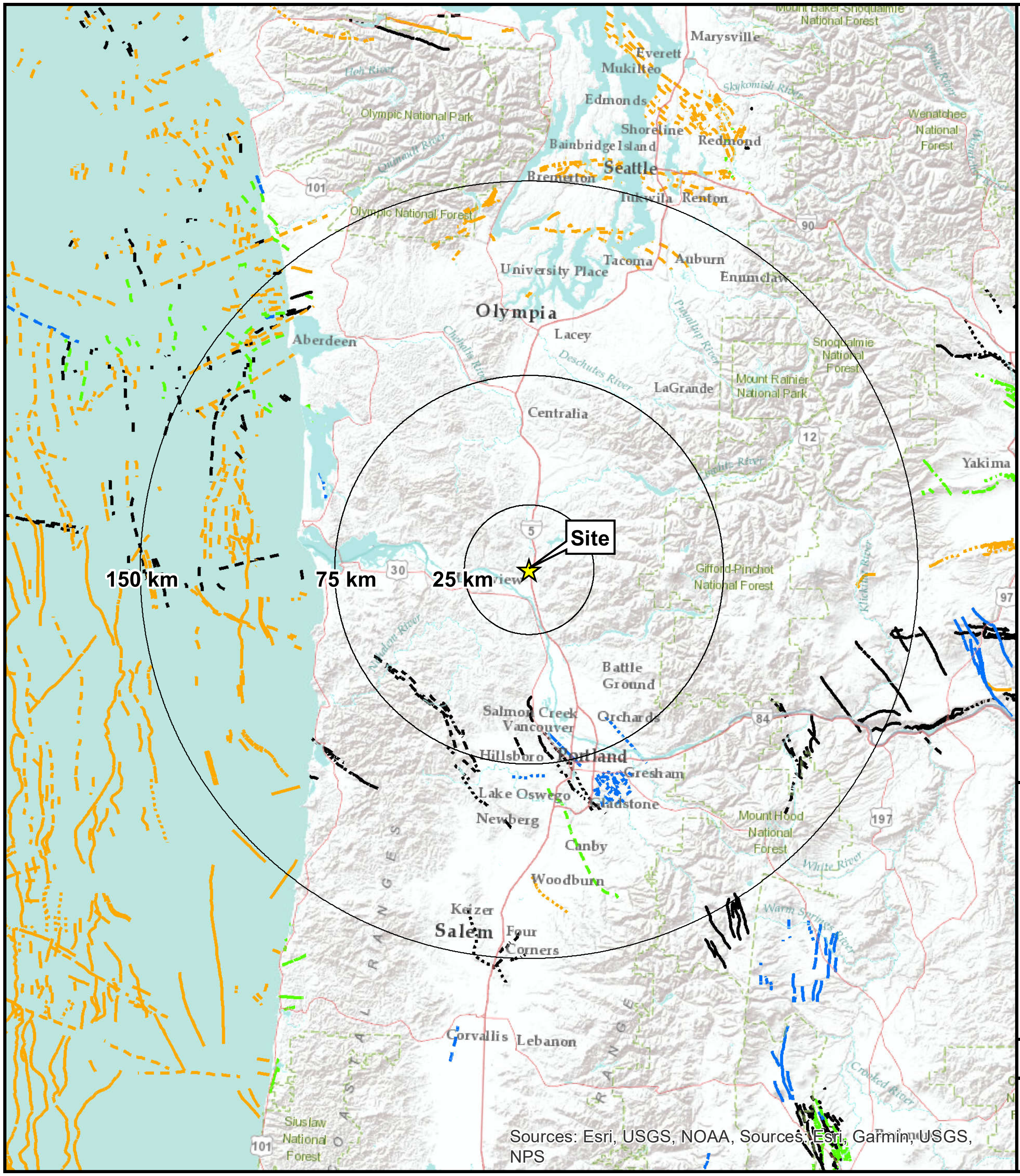

Sources: Esri, USGS, NOAA, Sources: Esri, Garmin, USGS, NPS

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

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

DATE: JULY 2020 · PROJECT: 73400.004

PBS

FIGURE

6

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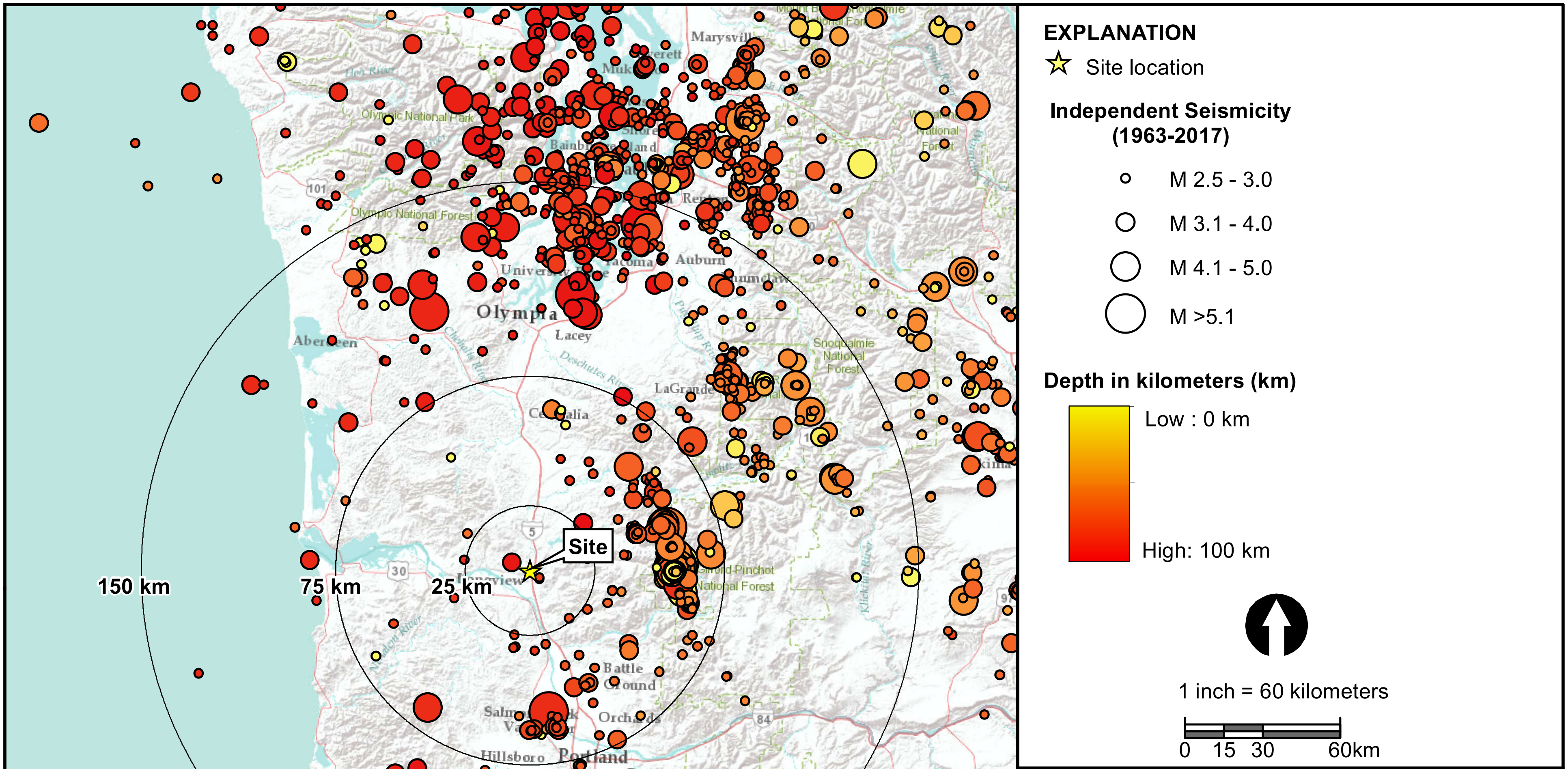

HISTORICAL SEISMICITY

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

DATE: JULY 2020 · PROJECT: 73400.004

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FIGURE

7

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Appendix A

Field Explorations

Appendix A: Field Explorations

A1 GENERAL

PBS explored subsurface conditions at the project site by advancing seven drilled borings and two cone penetration test (CPT) probes. The drilled borings were advanced to depths of 26.5 to 61.5 feet bgs on March 4, 5, and 24, 2020. The CPTs were completed to depths of approximately 29 to 59 feet bgs on February 28, 2020. 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 track-mounted Mobile B-57 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, 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 track-mounted Geoprobe Model 6622 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 A8 and A9. The results of shear wave velocity testing are included on Figure A10.

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.

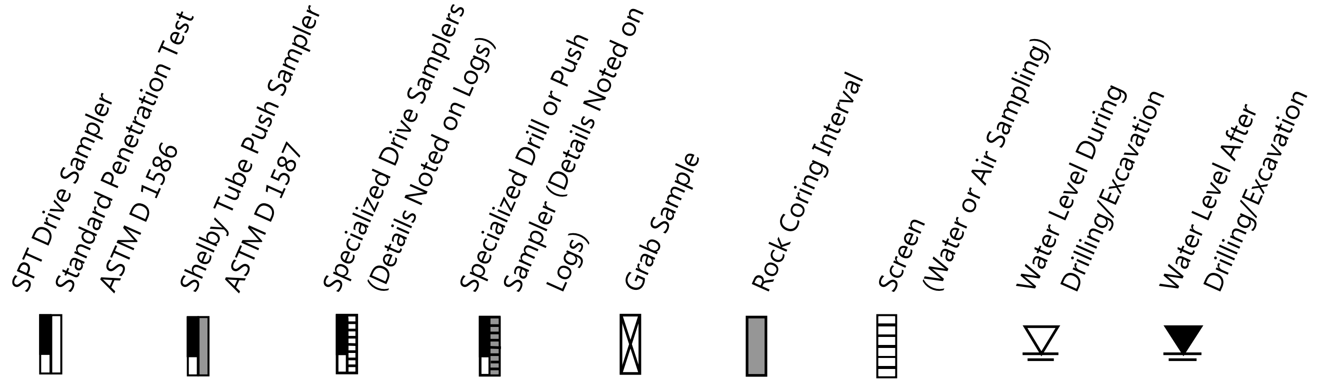
Cobble and boulder deposits may include a description of the matrix soils, as defined above.

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

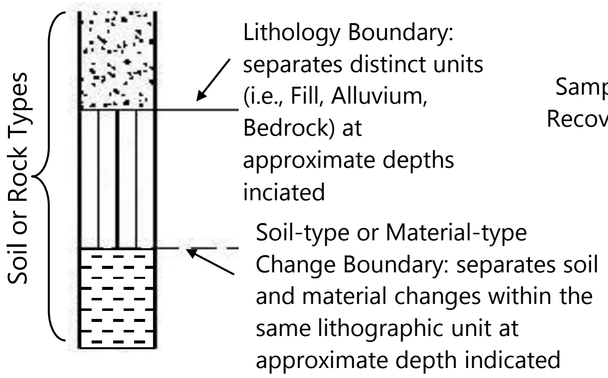
Table A-2
Key To Test Pit and Boring Log Symbols

SAMPLING DESCRIPTIONS

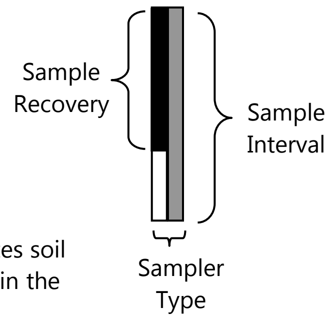


LOG GRAPHICS

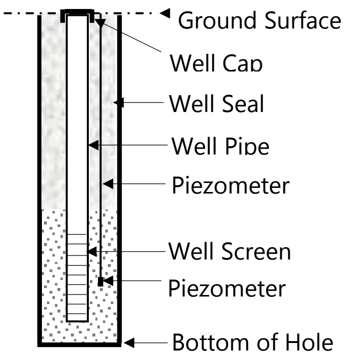
Soil and Rock



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.

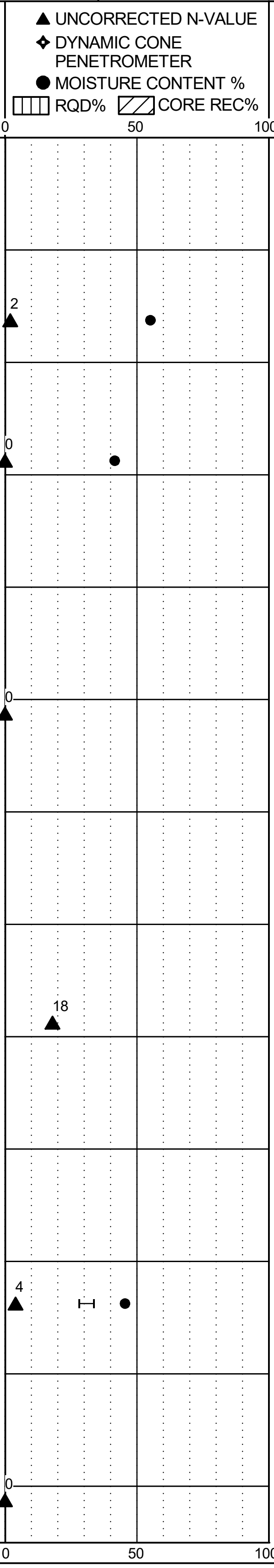
Rev. 02/2017

PBS	HUNTINGTON MIDDLE SCHOOL KELSO, WASHINGTON	**BORING B-1**
	PBS PROJECT NUMBER: 73400.004	APPROX. BORING B-1 LOCATION: (See Site Plan)

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	TESTING	SAMPLE TYPE SAMPLE ID	INSTALLATION AND COMMENTS
0.0	Very soft, olive brown, SILT (ML) with sand; low plasticity; fine sand; moist rootlets to 8 inches bgs			
2.5	Very soft, mottled olive gray to brown SILT (ML) with trace charcoal; low plasticity; moist		S-1	
4.0	Very loose/very soft, olive gray, sandy SILT (ML); non-plastic to low plasticity; fine sand; some iron oxide staining; wet	P200	S-2	P200 = 67%
8.0	Very loose, gray, poorly graded SAND (SP-SM) with silt to silty SAND (SM); non-plastic to low plasticity; fine sand; wet		S-3	Shelby pushed with mud pressure
			S-4	
	becomes medium dense, olive gray to light gray		S-5	Driller notes harder drilling
18.0	Soft, light gray, stratified (2 to 4 inch layers), SILT with sand to SILT (ML); low to medium plasticity; fine sand; wet	ATT	S-6	LL = 34 PL = 28 PI = 6
			S-7	
	becomes very soft, with trace woody debris		S-8	



DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: S. Cordes

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/04/2020

FIGURE A1

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

PBS PROJECT NUMBER:
73400.004

BORING B-1
(continued)

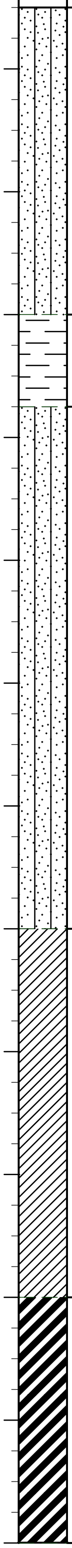
APPROX. BORING B-1 LOCATION:
(See Site Plan)

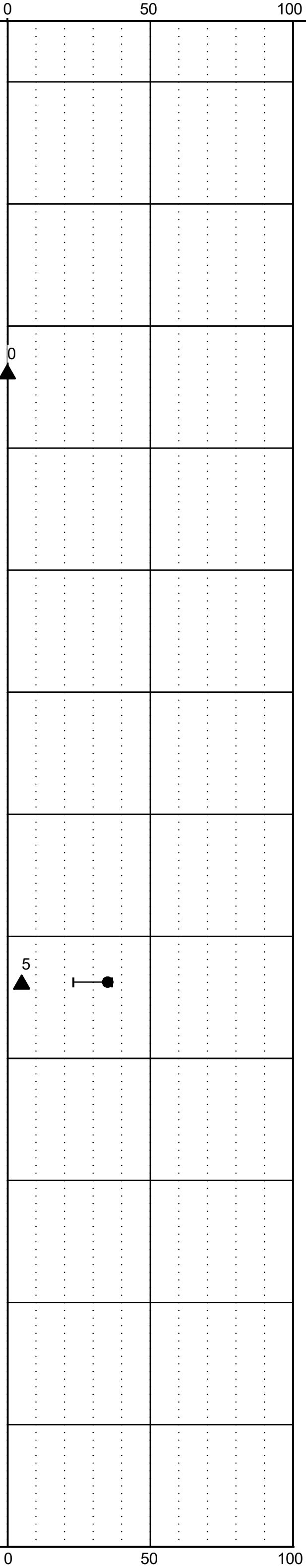
NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

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

INSTALLATION AND COMMENTS — Surface Conditions: Grass

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
25.0	Very loose, light gray, silty SAND (SM); low plasticity; fine to medium sand; trace, fine, subrounded gravel; wet	25.0				
30.0	Very soft, very dark brown to black, stratified (4 to 5 inch layers) ORGANIC SOIL (OL) to SILT (ML) with sand and woody debris; low to medium plasticity; fine sand; moist	30.0		S-9	0	
31.5	Very loose, gray, silty SAND (SM); low plasticity; fine sand; wet	31.5				
40.0	Medium stiff, gray, lean CLAY (CL); medium plasticity; moist	40.0	ATT	S-10	5	LL = 37 PL = 23 PI = 14
43.8						Rig chatter
46.0	Medium stiff, greenish gray, fat CLAY (CH); high plasticity; moist	46.0				Driller notes stiffer drilling
50.0						





DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: S. Cordes

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/04/2020

FIGURE A1

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

PBS PROJECT NUMBER:
73400.004

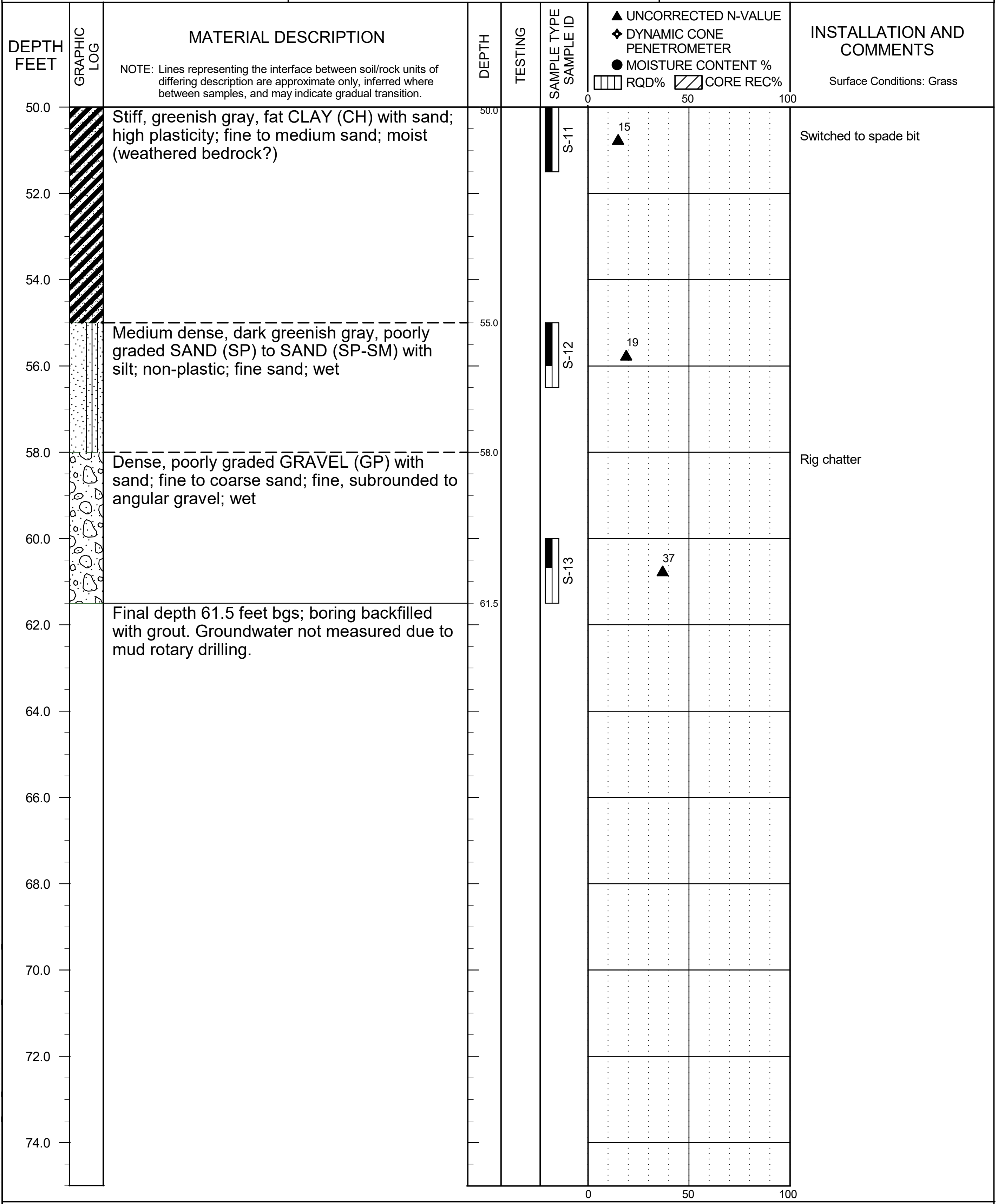
BORING B-1
(continued)

APPROX. BORING B-1 LOCATION:
(See Site Plan)

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	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
50.0	Stiff, greenish gray, fat CLAY (CH) with sand; high plasticity; fine to medium sand; moist (weathered bedrock?)	50.0		S-11	15	Switched to spade bit
55.0	Medium dense, dark greenish gray, poorly graded SAND (SP) to SAND (SP-SM) with silt; non-plastic; fine sand; wet	55.0		S-12	19	
58.0	Dense, poorly graded GRAVEL (GP) with sand; fine to coarse sand; fine, subrounded to angular gravel; wet	58.0		S-13	37	Rig chatter
61.5	Final depth 61.5 feet bgs; boring backfilled with grout. Groundwater not measured due to mud rotary drilling.	61.5				

DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: S. Cordes

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/04/2020

FIGURE A1

HUNTINGTON MIDDLE SCHOOL KELSO, WASHINGTON	**BORING B-2**	
PBS PROJECT NUMBER: 73400.004	APPROX. BORING B-2 LOCATION: (See Site Plan)	

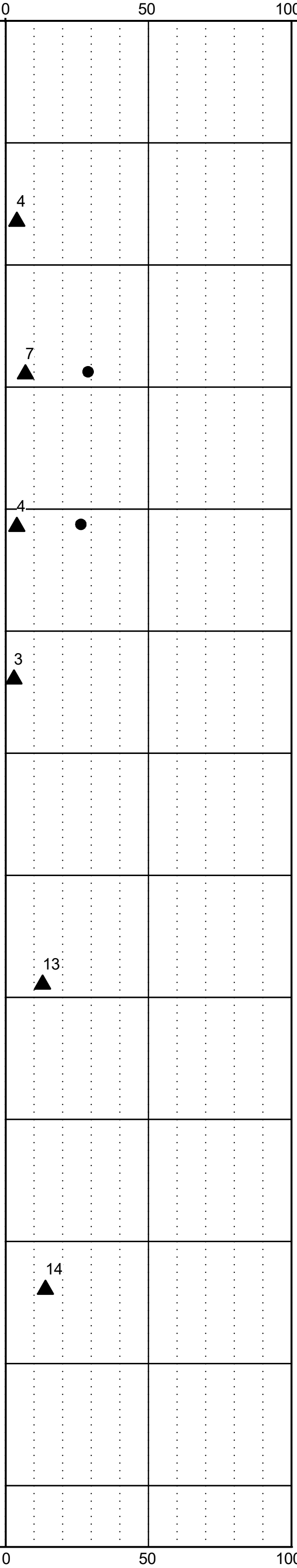
NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

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

INSTALLATION AND COMMENTS

Surface Conditions: Grass

DEPTH	MATERIAL DESCRIPTION	SAMPLE TYPE SAMPLE ID	UNCORRECTED N-VALUE
0.0	Brown SILT (ML); low plasticity; moist rootlets to 8 inches bgs		
2.5	Soft, brown, sandy SILT (ML); low plasticity; fine to medium sand; moist	S-1	4
5.5	Loose, brown, silty SAND (SM); low plasticity; fine to medium sand; moist	S-2	7
7.7	Very loose, gray, poorly graded SAND (SP); fine to medium sand; moist	S-3	4
10.0	Very soft, gray with olive brown mottling, stratified (2 to 3 inch layers) SILT to sandy SILT (ML); low plasticity; fine to medium sand; moist	S-4	3
15.0	Medium dense, brown, poorly graded SAND (SP); fine to medium sand; wet	S-5	13
15.6	Stiff, brown, sandy SILT (ML); low plasticity; fine to coarse sand; moist		
20.0	Medium dense, oxidized red and orange, brown to gray, poorly graded SAND (SP) to silty SAND (SM); non-plastic; fine to medium sand; wet	S-6	14



DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: S. Cordes

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/04/2020

FIGURE A2

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

PBS PROJECT NUMBER:
73400.004

BORING B-2
(continued)

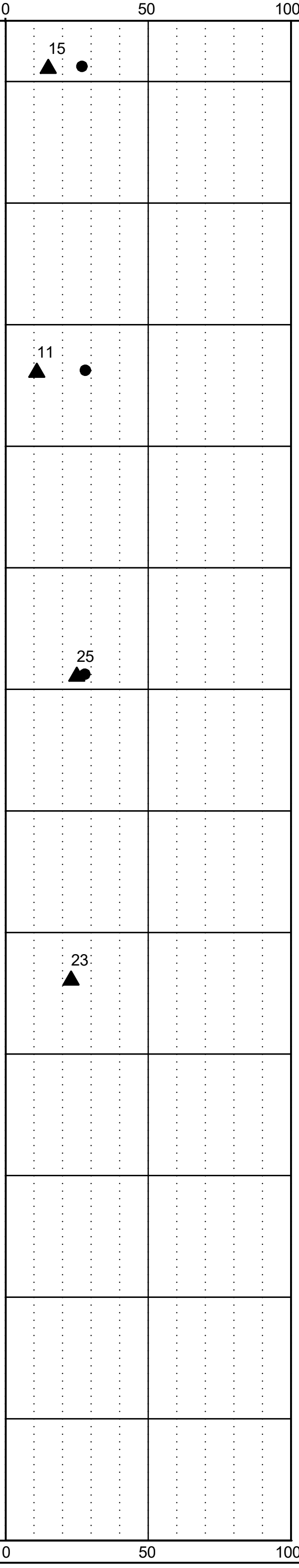
APPROX. BORING B-2 LOCATION:
(See Site Plan)

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	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	▲ UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
25.0	Medium dense, brown to gray, silty SAND (SM); non-plastic; fine to medium sand; wet	25.0	P200	S-7	15	P200 = 18%
28.0						Driller notes soft drilling
30.0	Stiff, mottled brown to gray SILT (ML); low plasticity; moist	30.0		S-8	11	
31.0	2-inch thick poorly graded sand lens Stiff, brown to gray SILT (ML) with sand; low plasticity; fine sand; wet	31.0				
35.0			P200	S-9	25	P200 = 83%
40.0	becomes dark olive brown			S-10	23	
40.5	Very stiff, dark greenish gray, lean CLAY (CL); medium plasticity; moist	40.5				
41.5	Final depth 41.5 feet bgs; boring backfilled with grout. Groundwater not measured due to mud rotary drilling.	41.5				



DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: S. Cordes

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/04/2020

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

BORING B-3

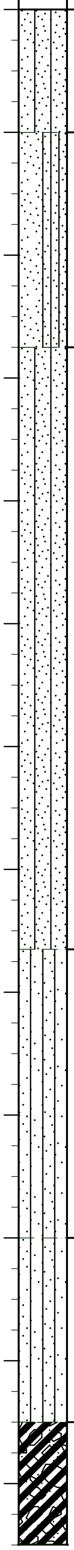
PBS PROJECT NUMBER:
73400.004

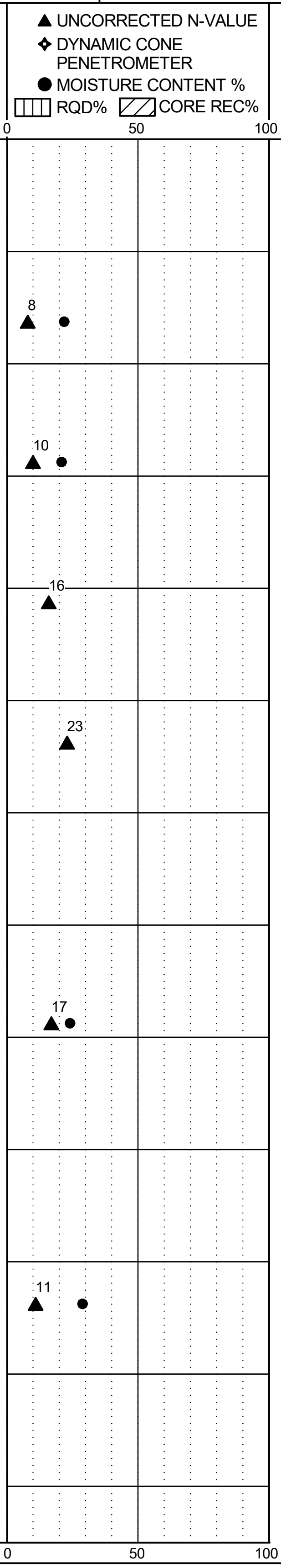
APPROX. BORING B-3 LOCATION:
(See Site Plan)

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

INSTALLATION AND COMMENTS — Surface Conditions: Grass

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
0.0	Brown silty SAND (SM); low plasticity; fine to medium sand; dry rootlets to 8 inches bgs	0.0				
2.0	Loose, brown, poorly graded SAND (SP-SM) with silt; non-plastic; fine to medium sand; moist increased fines becomes wet	2.0		S-1	8	
5.5	Loose, brown, silty SAND (SM); low plasticity; fine to medium sand; moist	5.5		S-2	10	
	becomes medium dense, mottled brown to red-brown; with 2-inch-thick, gray, SILT (ML) lens			S-3	16	
	4-inch-thick SILT (ML) lens; trace gravel			S-4	23	
						Rig chatter
	2-inch-thick, brown, poorly graded SAND (SP) lens					
15.3	Very stiff, olive brown, sandy SILT (ML); low to medium plasticity; fine to medium sand; moist	15.3		S-5	17	
20.0	Stiff, dark greenish gray, sandy SILT (ML); low plasticity; fine sand; wet	20.0	P200	S-6	11	P200 = 80%
23.0	Very stiff, dark greenish gray, gravelly fat CLAY (CH); high plasticity; fine to coarse, subrounded gravel; moist	23.0				Driller notes harder drilling





DEPTH FEET: 0.0, 2.0, 4.0, 6.0, 8.0, 10.0, 12.0, 14.0, 16.0, 18.0, 20.0, 22.0, 24.0

DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: S. Cordes

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/05/2020

FIGURE A3

HUNTINGTON MIDDLE SCHOOL KELSO, WASHINGTON	**BORING B-3** (continued)
PBS PROJECT NUMBER: 73400.004	APPROX. BORING B-3 LOCATION: (See Site Plan)

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

Legend: ▲ UNCORRECTED N-VALUE; ✧ DYNAMIC CONE PENETROMETER; ● MOISTURE CONTENT %; ▥ RQD%; ▨ CORE REC% (scale 0 – 50 – 100)

INSTALLATION AND COMMENTS

Surface Conditions: Grass

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
25.0–28.0	Very stiff, dark greenish gray, gravelly fat CLAY (CH); high plasticity; fine to coarse, subrounded gravel; moist	25.0		S-7	20	
28.0–31.5	Hard, greenish gray, sandy SILT (ML); low plasticity; fine sand; dry (severely weathered bedrock) friable, weak, platy	28.0		S-8	18-34-50/5"	
31.5	Final depth 31.5 feet bgs; boring backfilled with grout. Groundwater not measured due to mud rotary drilling.	31.5				

Depth scale: 26.0, 28.0, 30.0, 32.0, 34.0, 36.0, 38.0, 40.0, 42.0, 44.0, 46.0, 48.0, 50.0

DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: S. Cordes

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/05/2020

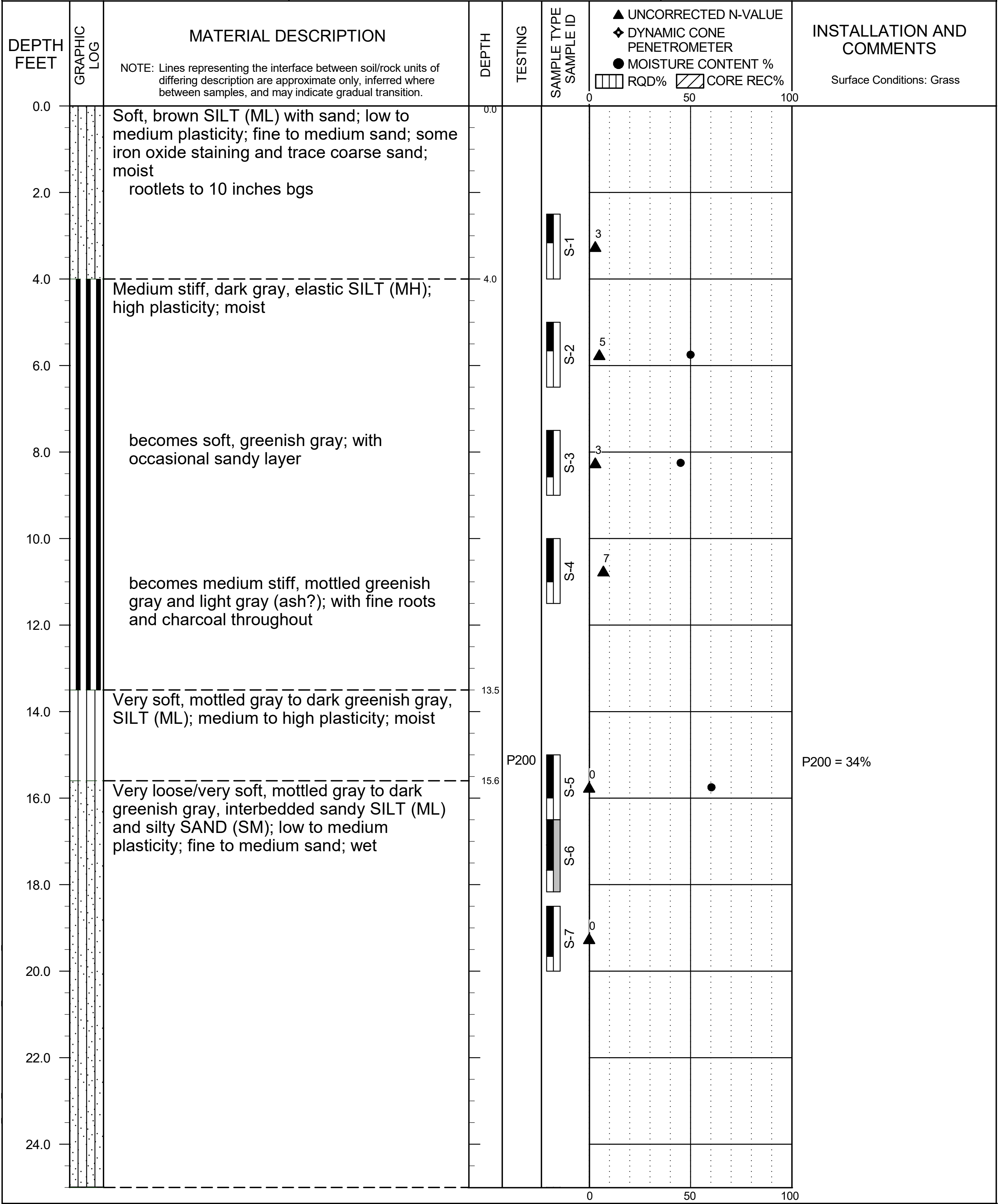
FIGURE A3

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

BORING B-4

PBS PROJECT NUMBER:
73400.004

APPROX. BORING B-4 LOCATION:
(See Site Plan)



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	MATERIAL DESCRIPTION
0.0	Soft, brown SILT (ML) with sand; low to medium plasticity; fine to medium sand; some iron oxide staining and trace coarse sand; moist rootlets to 10 inches bgs
4.0	Medium stiff, dark gray, elastic SILT (MH); high plasticity; moist becomes soft, greenish gray; with occasional sandy layer becomes medium stiff, mottled greenish gray and light gray (ash?); with fine roots and charcoal throughout
13.5	Very soft, mottled gray to dark greenish gray, SILT (ML); medium to high plasticity; moist
15.6	Very loose/very soft, mottled gray to dark greenish gray, interbedded sandy SILT (ML) and silty SAND (SM); low to medium plasticity; fine to medium sand; wet

SAMPLE ID	TESTING	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
S-1		3	
S-2		5	
S-3		3	
S-4		7	
S-5	P200	0	P200 = 34%
S-6			
S-7		0	

DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: S. Cordes

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/05/2020

FIGURE A4

BORING LOG 73400.004 B1-5 20200311.GPJ PBS DATATMPL GEO.GDT PRINT DATE: 4/10/20:RPG

BORING B-4
(continued)

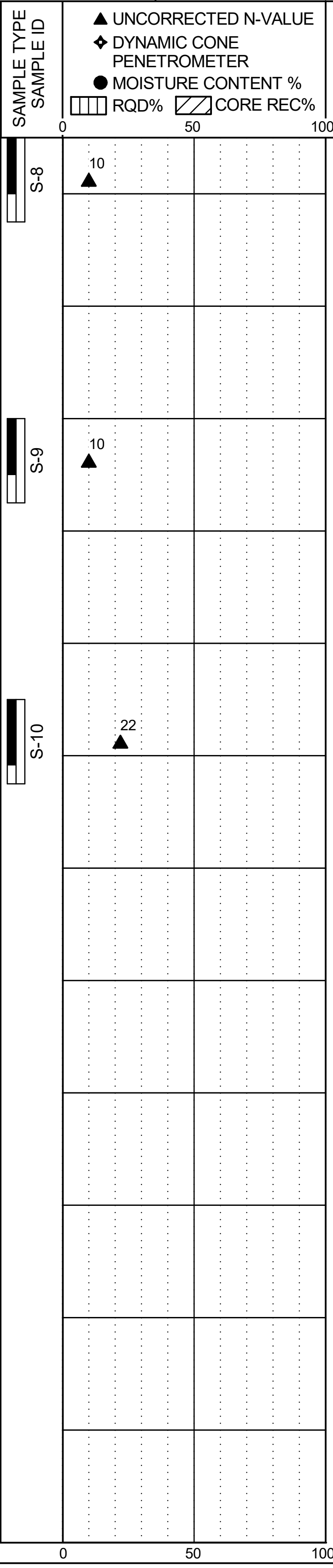
APPROX. BORING B-4 LOCATION:
(See Site Plan)

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	SAMPLE ID	UNCORRECTED N-VALUE
25.0	Loose, mottled orange-brown to gray, silty SAND (SM); low plasticity; fine sand; moist	25.0	S-8	10
30.0–31.5			S-9	10
35.0–36.5	becomes medium dense, gray		S-10	22
36.5	Final depth 36.5 feet bgs; boring backfilled with grout. Groundwater not measured due to mud rotary drilling.	36.5		

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

INSTALLATION AND COMMENTS

Surface Conditions: Grass



DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: S. Cordes

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/05/2020

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

BORING B-5

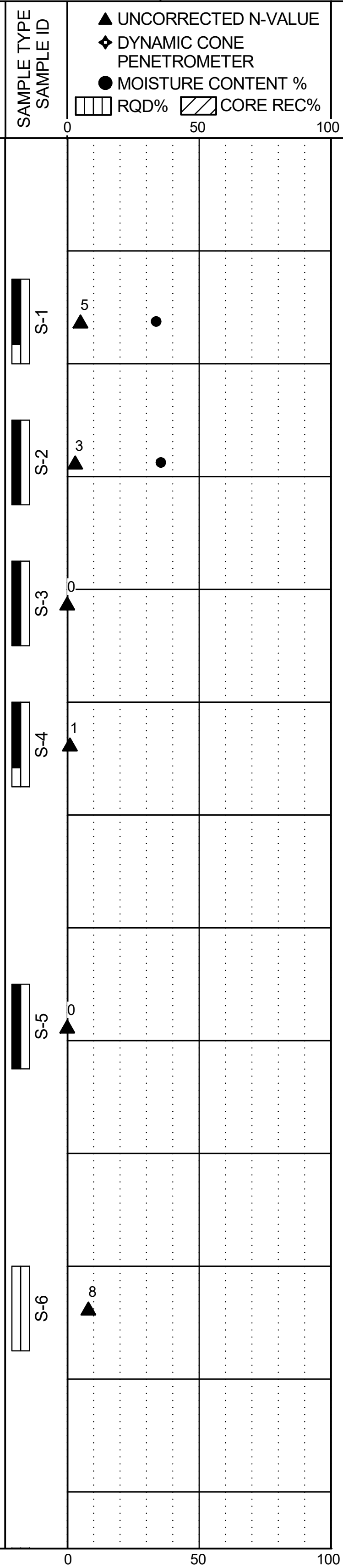
PBS PROJECT NUMBER:
73400.004

APPROX. BORING B-5 LOCATION:
(See Site Plan)

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

INSTALLATION AND COMMENTS

Surface Conditions: Grass



DEPTH FEET	MATERIAL DESCRIPTION	SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
0.0	Medium stiff/loose, mottled orange to gray, stratified (2 to 4 inch layers) sandy SILT (ML) to silty SAND (SM); low to medium plasticity; fine to medium sand; moist			
2.0		S-1	5	
4.0	becomes soft to very soft/very loose; dark greenish gray to dark gray; wet	S-2	3	
6.0				
8.0		S-3	0	
10.0	sand becomes fine to coarse	S-4	1	Driller notes abundant organics in mud return
12.0				
14.0	becomes mottled very dark gray to very dark greenish gray; trace organics	S-5	0	
16.0				
18.0				
20.0	becomes medium stiff/loose	S-6	8	No recovery
22.0				
24.0				Driller notes harder drilling

DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: S. Cordes

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/05/2020

FIGURE A5

BORING LOG 73400.004 B1-5 20200311.GPJ PBS DATATMPL GEO.GDT PRINT DATE: 4/10/20:RPG

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

PBS PROJECT NUMBER:
73400.004

BORING B-5
(continued)

APPROX. BORING B-5 LOCATION:
(See Site Plan)

DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION

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 | TESTING | SAMPLE TYPE SAMPLE ID

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

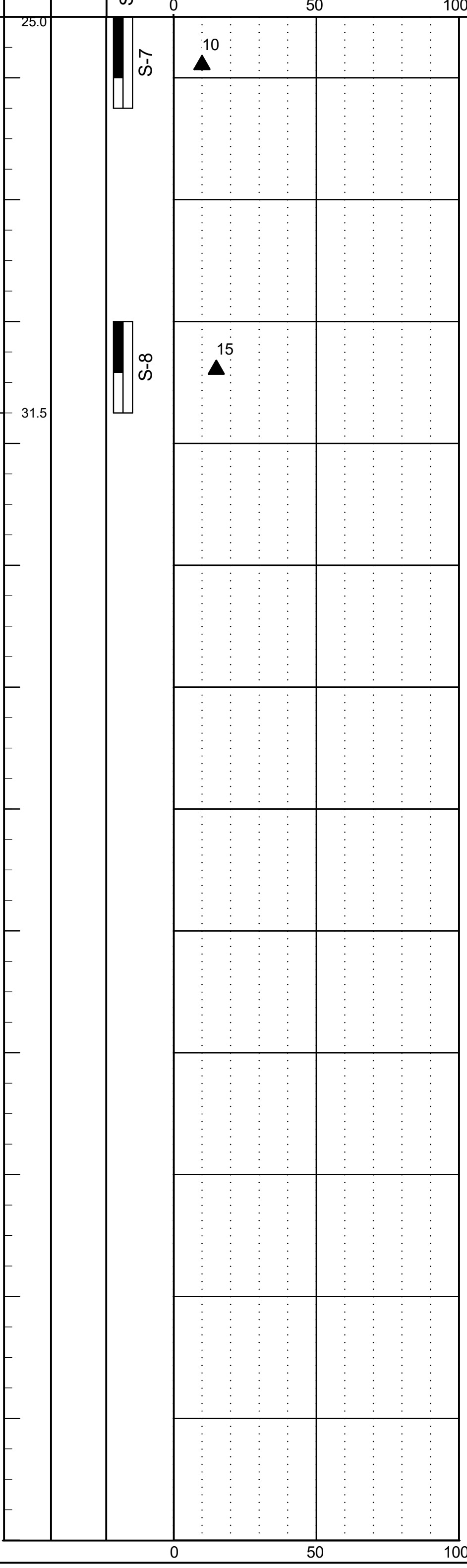
INSTALLATION AND COMMENTS

Surface Conditions: Grass

Loose, dark gray, silty SAND (SM); low plasticity; fine sand; moist

becomes medium dense

Final depth 31.5 feet bgs; boring backfilled with grout. Groundwater not measured due to mud rotary drilling.



26.0
28.0
30.0
32.0
34.0
36.0
38.0
40.0
42.0
44.0
46.0
48.0
50.0

DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: S. Cordes

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/05/2020

FIGURE A5

HUNTINGTON MIDDLE SCHOOL KELSO, WASHINGTON		**BORING B-6**
PBS PROJECT NUMBER: 73400.004		APPROX. BORING B-6 LOCATION: 46.154631, -122.903237

MATERIAL DESCRIPTION

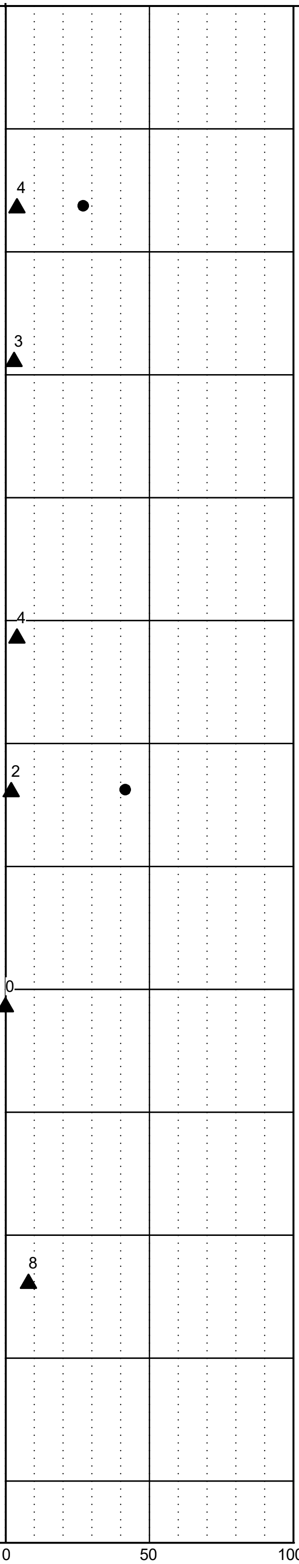
NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

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

INSTALLATION AND COMMENTS

Surface Conditions: Grass

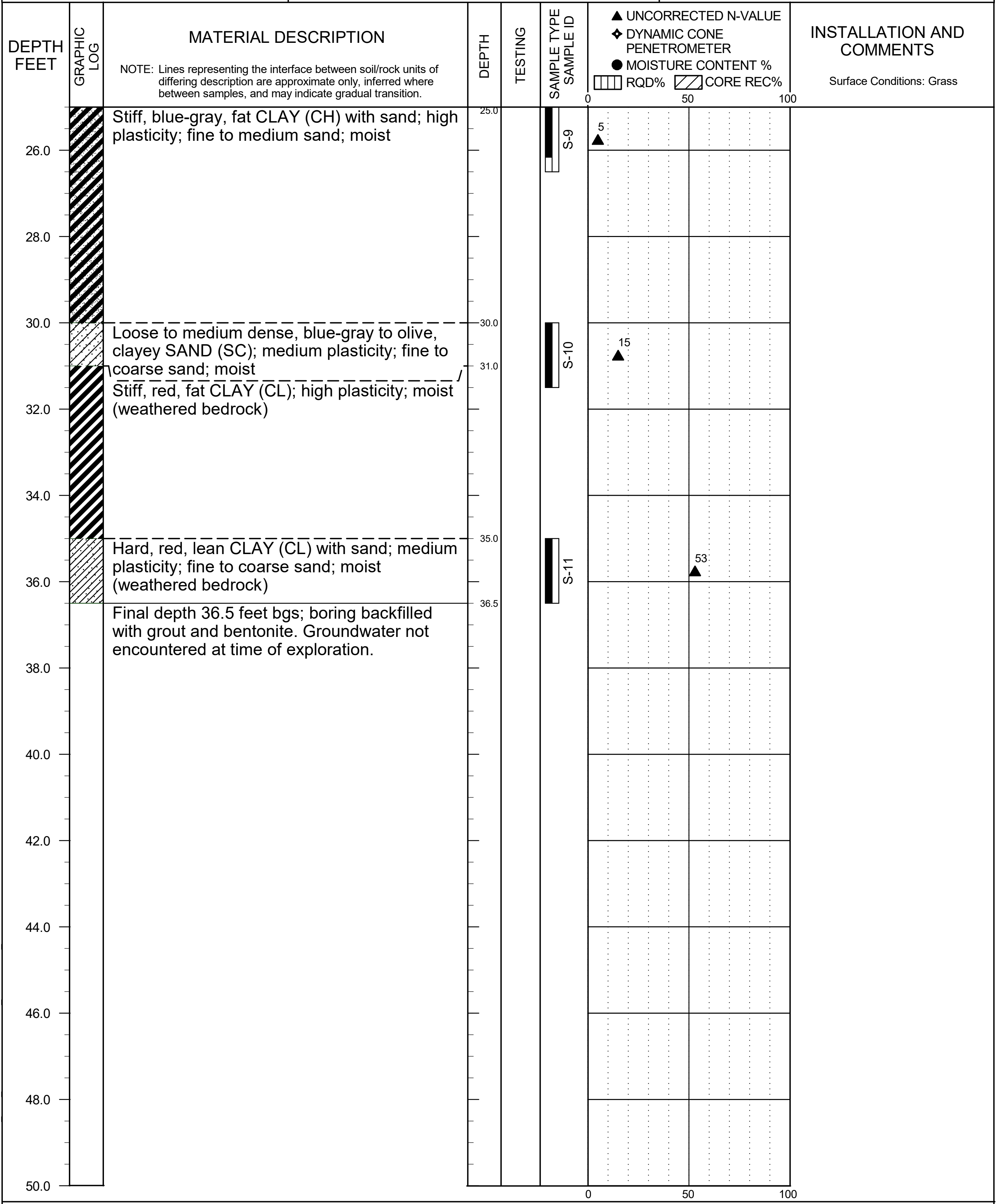
DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
0.0	Loose, brown, silty SAND (SM); non-plastic; fine to medium sand; moist	0.0				
2.0			P200	S-1	4	P200 = 38%
4.0				S-2	3	
	Soft, gray, lean CLAY (CL) with sand; medium plasticity; fine sand; moist organic odor	5.5				
6.0						
8.0				S-3		
	Soft, blue-gray, sandy lean CLAY (CL); medium plasticity; fine sand; moist	9.5				
10.0				S-4	4	
	Soft/very loose, blue-gray, sandy lean CLAY (CL) to clayey SAND (SC); medium plasticity; fine to coarse sand; moist	11.5				
12.0			P200	S-5	2	P200 = 55%
14.0				S-6		
	Very soft, dark gray, sandy lean CLAY (CL) with decomposed organics; medium plasticity; fine sand; moist to wet	15.5				
16.0				S-7	0	
18.0						
	Loose, gray to olive brown, poorly graded SAND (SP-SM) with silt; non-plastic; fine to coarse sand; moist to wet	18.5				
20.0				S-8	8	
22.0						
	Stiff, blue-gray, fat CLAY (CH) with sand; high plasticity; fine to medium sand; moist	23.5				
24.0						



DRILLING METHOD: Drag Bit
DRILLED BY: Holt Services, Inc.
LOGGED BY: D. Eibert

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/24/2020

FIGURE A6



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

25.0	Stiff, blue-gray, fat CLAY (CH) with sand; high plasticity; fine to medium sand; moist	S-9	5
30.0	Loose to medium dense, blue-gray to olive, clayey SAND (SC); medium plasticity; fine to coarse sand; moist	S-10	15
31.0	Stiff, red, fat CLAY (CL); high plasticity; moist (weathered bedrock)		
35.0	Hard, red, lean CLAY (CL) with sand; medium plasticity; fine to coarse sand; moist (weathered bedrock)	S-11	53
36.5	Final depth 36.5 feet bgs; boring backfilled with grout and bentonite. Groundwater not encountered at time of exploration.		

DRILLING METHOD: Drag Bit
DRILLED BY: Holt Services, Inc.
LOGGED BY: D. Eibert

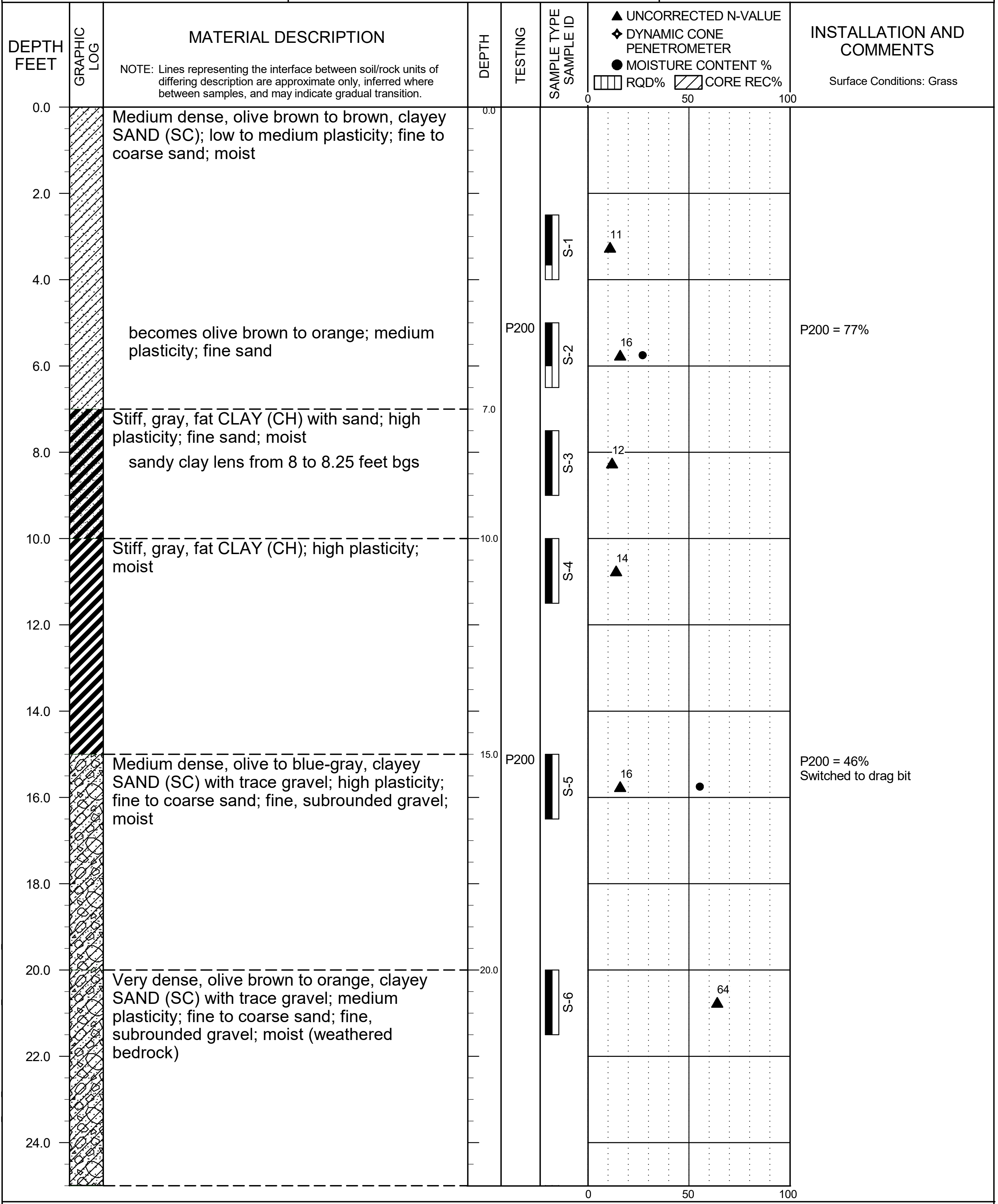
BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/24/2020

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

BORING B-7

PBS PROJECT NUMBER:
73400.004

APPROX. BORING B-7 LOCATION:
46.154277, -122.903220



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	MATERIAL DESCRIPTION	TESTING	SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
0.0	Medium dense, olive brown to brown, clayey SAND (SC); low to medium plasticity; fine to coarse sand; moist				
			S-1	11	
	becomes olive brown to orange; medium plasticity; fine sand	P200	S-2	16	P200 = 77%
7.0	Stiff, gray, fat CLAY (CH) with sand; high plasticity; fine sand; moist sandy clay lens from 8 to 8.25 feet bgs		S-3	12	
10.0	Stiff, gray, fat CLAY (CH); high plasticity; moist		S-4	14	
15.0	Medium dense, olive to blue-gray, clayey SAND (SC) with trace gravel; high plasticity; fine to coarse sand; fine, subrounded gravel; moist	P200	S-5	16	P200 = 46% Switched to drag bit
20.0	Very dense, olive brown to orange, clayey SAND (SC) with trace gravel; medium plasticity; fine to coarse sand; fine, subrounded gravel; moist (weathered bedrock)		S-6	64	

DRILLING METHOD: Mill Tooth/Drag Bit
DRILLED BY: Holt Services, Inc.
LOGGED BY: D. Eibert

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/24/2020

FIGURE A7

HUNTINGTON MIDDLE SCHOOL KELSO, WASHINGTON	**BORING B-7** (continued)
PBS PROJECT NUMBER: 73400.004	APPROX. BORING B-7 LOCATION: 46.154277, -122.903220

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

INSTALLATION AND COMMENTS

Surface Conditions: Grass

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	GRAPHIC LOG	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	N-VALUE (0 – 50 – 100)	INSTALLATION AND COMMENTS
26.0	▨	Very dense, blue-gray, olive, and orange, clayey SAND (SC); low plasticity; fine to medium sand; moist (weathered bedrock)	25.0		S-7	28-45-50/5 ▲	
		Final depth 26.5 feet bgs; boring backfilled with grout and bentonite. Groundwater not encountered at time of exploration.	26.5				
28.0							
30.0							
32.0							
34.0							
36.0							
38.0							
40.0							
42.0							
44.0							
46.0							
48.0							
50.0							

0 50 100

DRILLING METHOD: Mill Tooth/Drag Bit
DRILLED BY: Holt Services, Inc.
LOGGED BY: D. Eibert

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 89.9
LOGGING COMPLETED: 3/24/2020

FIGURE A7

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

CPT-1

Total depth: 58.89 ft, Date: 2/28/2020

Project: **73400.004 Huntington Middle School**
Location: **Kelso, Washington**

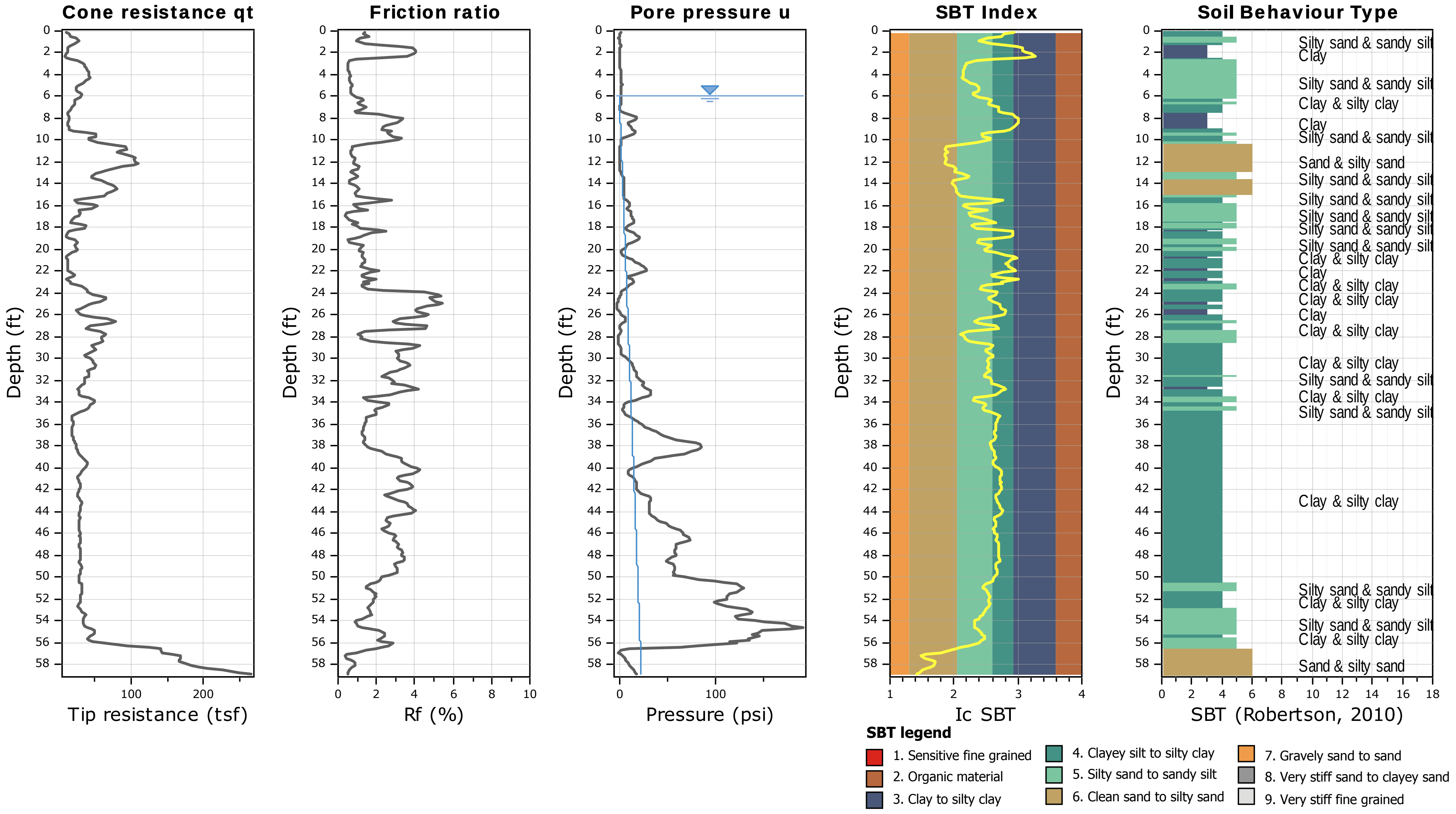


FIGURE A8

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

CPT-2
Total depth: 29.20 ft, Date: 2/28/2020

Project: **73400.004 Huntington Middle School**
Location: **Kelso, Washington**

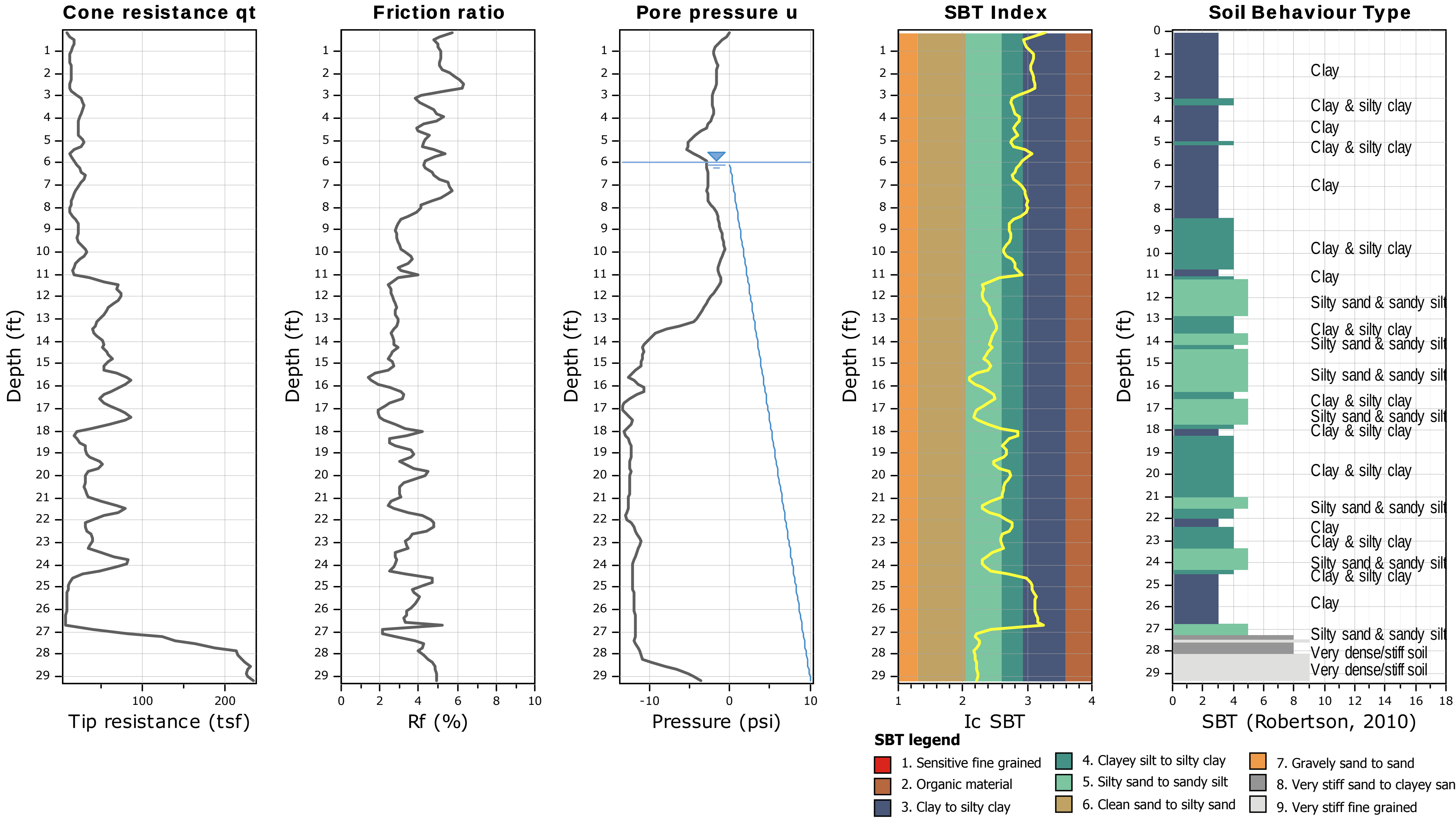
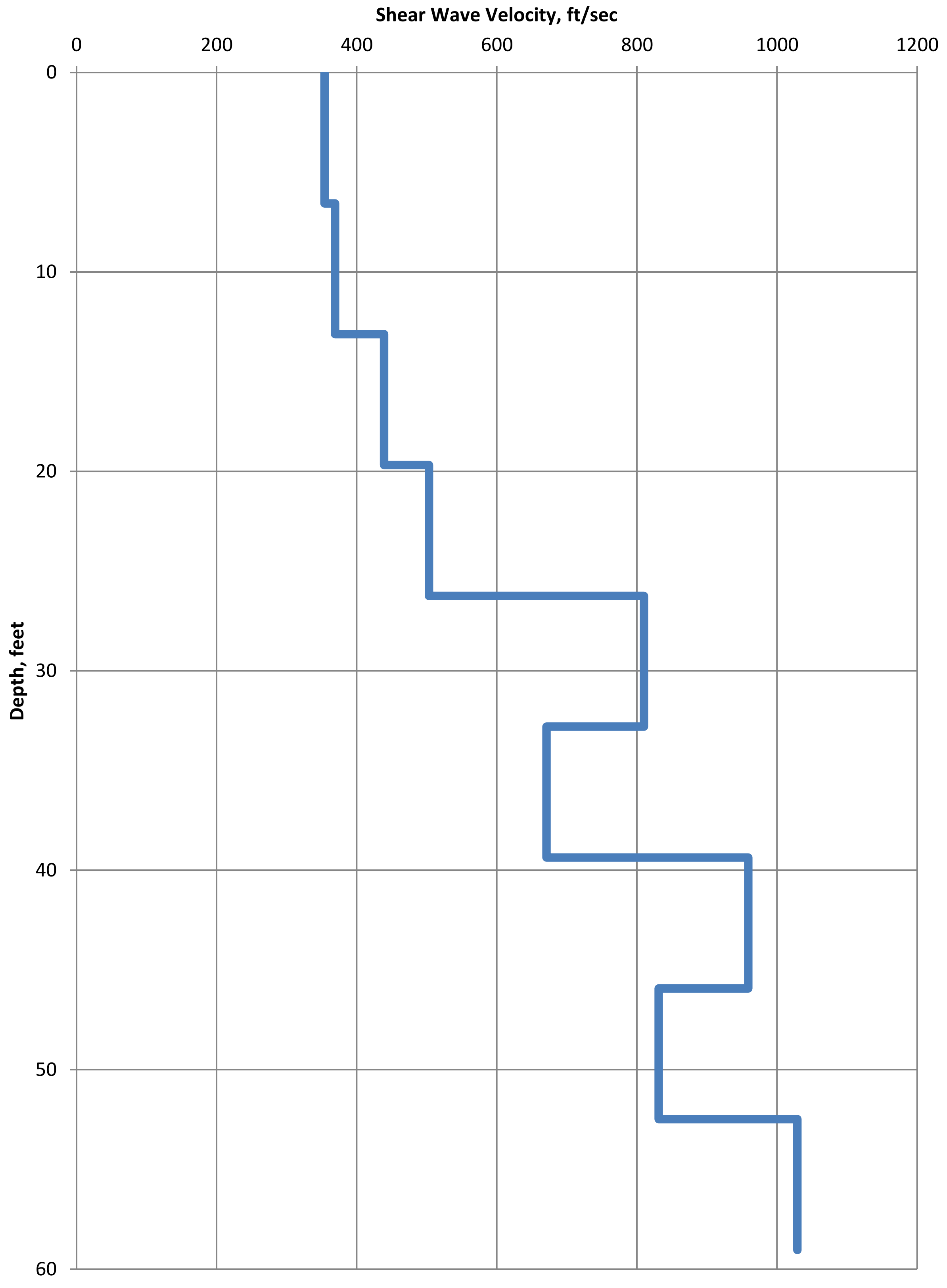


FIGURE A9



SHEAR WAVE VELOCITY PROFILE

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

APR 2020
73400.004

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 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.

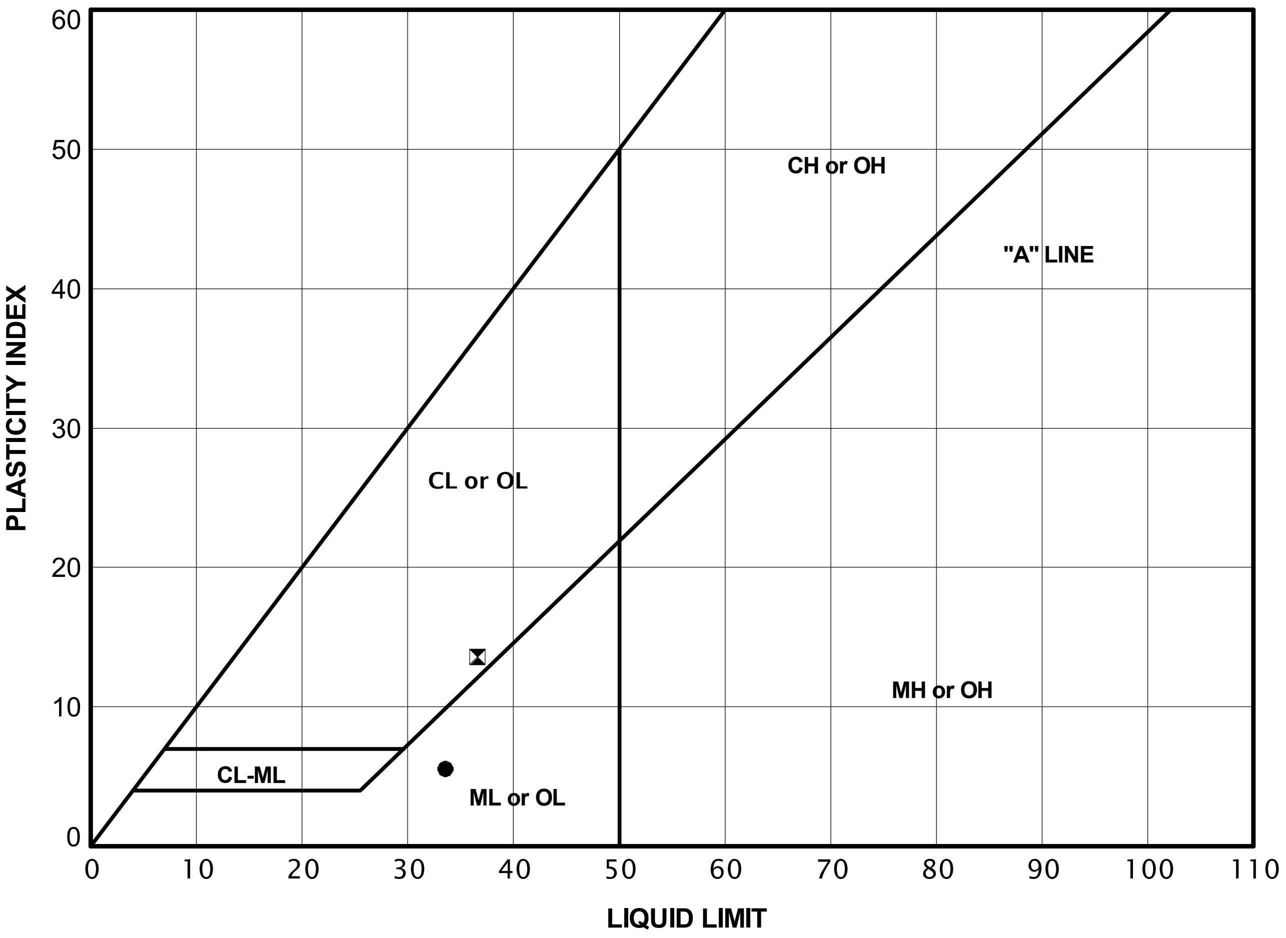
B2.4 Grain-Size Analyses (P200 Wash)

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.

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

PBS PROJECT NUMBER:
73400.004

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-1	S-6	20.0	45.4	NA	34	28	6
⧗	B-1	S-10	40.0	35.0	NA	37	23	14

_ATTERBERG LIMITS 73400.004_B1-5_20200311.GPJ PBS_DATATMPL_GEO.GDT PRINT DATE: 3/24/20:RPG

FIGURE B1

SUMMARY OF LABORATORY DATA

HUNTINGTON MIDDLE SCHOOL
KELSO, WASHINGTON

PBS PROJECT NUMBER:
73400.004

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-1	2.5		55.0							
B-1	S-2	5		41.5				67			
B-1	S-6	20		45.4					34	28	6
B-1	S-10	40		35.0					37	23	14
B-2	S-2	5		28.8							
B-2	S-3	7.5		26.3							
B-2	S-7	25		26.7				18			
B-2	S-8	30		27.9							
B-2	S-9	35		27.7				83			
B-3	S-1	2.5		21.8							
B-3	S-2	5		20.8							
B-3	S-5	15		24.0							
B-3	S-6	20		28.8				80			
B-4	S-2	5		49.9							
B-4	S-3	7.5		45.0							
B-4	S-5	15		60.2				34			
B-5	S-1	2.5		33.6							
B-5	S-2	5		35.4							
B-6	S-1	2.5		26.9				38			
B-6	S-5	12		41.5				55			
B-7	S-2	5		27.0				77			
B-7	S-5	15		55.3				46			

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.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

Huntington Middle School Modernization

Kelso School District – No 458

Ecological Letter of Findings

Ecological
Land Services

May 7, 2020

Kelso City Manager
203 South Pacific
P.O. Box 819
Kelso, WA 98626

Re: Letter of Findings - Huntington Middle School | Kelso, Washington

Dear Mr. Hamilton,

As requested, Ecological Land Services, Inc. (ELS) has assessed undeveloped area surrounding the Huntington Middle School Campus (Cowlitz County Tax Parcel 21582) on behalf of Kelso School District. The property is located at 500 Redpath Street in Kelso, Washington, within Section 26, Township 8 North, Range 2 West of the Willamette Meridian (Figure 1). ELS biologists conducted a reconnaissance of the study area on March 24, 2020 to determine the presence and extent of critical areas onsite. This memorandum provides a description of the study area's existing conditions and a summary of critical area findings in accordance with Kelso Municipal Code (KMC) *Title 17 Unified Development Code*, *Chapter 17.26 Environmentally Sensitive Areas*. The applicant is proposing a gym and parking lot expansion on the Huntington Middle School campus. The final location of the proposed development has not been determined; however, construction will be located within currently developed and/or regularly maintained areas onsite.

Site Description

The study area mainly consists of the forested area behind (east) of the Huntington Middle School campus (Figure 1). The study area slopes from east to west and contains small ravines. The study area is forested largely with native species, consisting mainly of include red alder (*Alnus rubra*), Douglas fir (*Pseudotsuga menziesii*), and big leaf maple (*Acer macrophyllum*). The understory contains native shrubs and herbaceous species including sword fern (*Polystichum munitum*), vine maple (*Acer circinatum*), salmonberry (*Rubus spectabilis*), and Indian plum (*Oemleria cerasiformis*); however, Himalayan blackberry (*Rubus armeniacus*) and English ivy (*Hedera helix*) dominate the understory. Interstate-5 lies directly east of the forested area with school buildings, athletic fields, and maintained school grounds to the west. Two streams (Stream A and Stream B) were delineated within the study area. The streams converge at a culvert located at the western edge of the forested area adjacent to the maintained grounds that conveys water to the west/northwest.

Methods

ELS researched various environmental informational websites including the Washington Department of Natural Resources (DNR) Forest Practices Application stream mapping website, the National Wetland Inventory (NWI), and the Natural Resources Conservation Service (NRCS) soil survey website, as well as conducted a site visit on March 24, 2020 to determine if critical areas were present onsite. During the site visit ELS mapped the centerline of two streams onsite using a GPS unit with sub-meter accuracy.

Findings

DNR stream mapping indicates two unknown streams and one Type-F (fish bearing) stream as occurring within the property. The NWI depicts two streams in the eastern portion of the property and two streams in the approximate DNR-mapped locations. ELS findings were not consistent with DNR or NWI maps, as there was no presence of a Type-F stream and no streams were found near the eastern property boundary; however, two unknown streams were delineated in the study area in the approximate DNR-mapped locations. The NRCS designates soils onsite as: Caples silty clay loam, somewhat poorly drained, 0-3 percent slopes; Kalama gravelly loam, moderately well drained, 30 to 60 percent slopes; and Kelso silt loam, moderately well drained, 15 to 30 percent slopes. Kalama gravelly loam and Kelso silt loam are mapped on the forested hillside and Caples is mapped across the developed school grounds. Caples silty clay loam is considered a hydric soil by the NRCS.

Streams

Stream A flows northwest from the southeast and is positioned in a ravine with sloped stream banks ranging from 20-45 percent. The wetted stream channel spanned approximately two to three feet at the time of visitation, and the channel substrate was composed of fine sediment and small to large cobble (1-8 inch) with low to moderate flow. Vegetation found below the ordinary high water mark (OHWM) of Stream A included skunk cabbage (*Lysichiton americanus*), pacific waterleaf (*Hydrophyllum tenuipes*), and lady fern (*Athyrium filix-femina*). Vegetation above the OHWM of Stream A included salmonberry but were largely dominated by Himalayan blackberry. Stream B was mapped north of Stream A and flows slightly southwest. Stream B’s channel is slightly steeper and spanned approximately two to five feet at the time of visitation, and the channel substrate was composed of fine sediment and small to large cobble (1-8 inch) with significant flow. The majority of Stream B was inaccessible due to overgrowth of Himalayan blackberry. Vegetation was not observed below the OHWM of Stream B and vegetation above the OHWM of Stream B was dominated by Himalayan blackberry. The streams converge to the west at the edge of the forested area and enter an underground stormwater system through a culvert that conveys water to the west/northwest beneath the school grounds. Riparian habitat function for the streams ends at this culvert. Flow within Streams A and B is seasonal and there is no fish habitat.

Conclusions

Streams A and B do not meet the technical criteria for Type Ns (nonfish bearing seasonal) streams according to KMC *17.26.060(A)(5)(d)*, which states that Type Ns waters includes “segments of natural waters within defined channels that are not Type S [shoreline], F, or Np [nonfish bearing perennial] waters. These are seasonal, nonfish habitat streams in which surface flow is not present for at least some portion of a year of normal rainfall and are not located downstream from any stream reach that is a Type Np water. Ns waters must be physically connected by an aboveground channel system to Type S, F, or N waters.” Because there is no aboveground connection to Type S, F, or N waters, Streams A and B are not regulated by the KMC and do not have designated buffers. Even without designated stream buffers, there will be no impact or functional loss of riparian habitat from project construction because future development will not extend into the forested area.

Limitations

ELS bases this report's determinations on standard scientific methodology and best professional judgment. In our opinion, local, state, and federal regulatory agencies should agree with our determinations. However, the information contained in this report should be considered preliminary and used at your own risk until it has been approved in writing by the appropriate regulatory agencies. ELS is not responsible for the impacts of any changes in environmental standards, practices, or regulations after the date of this report.

If you have additional questions, please feel free to contact me at steff@eco-land.com, or call (360) 578-1371.

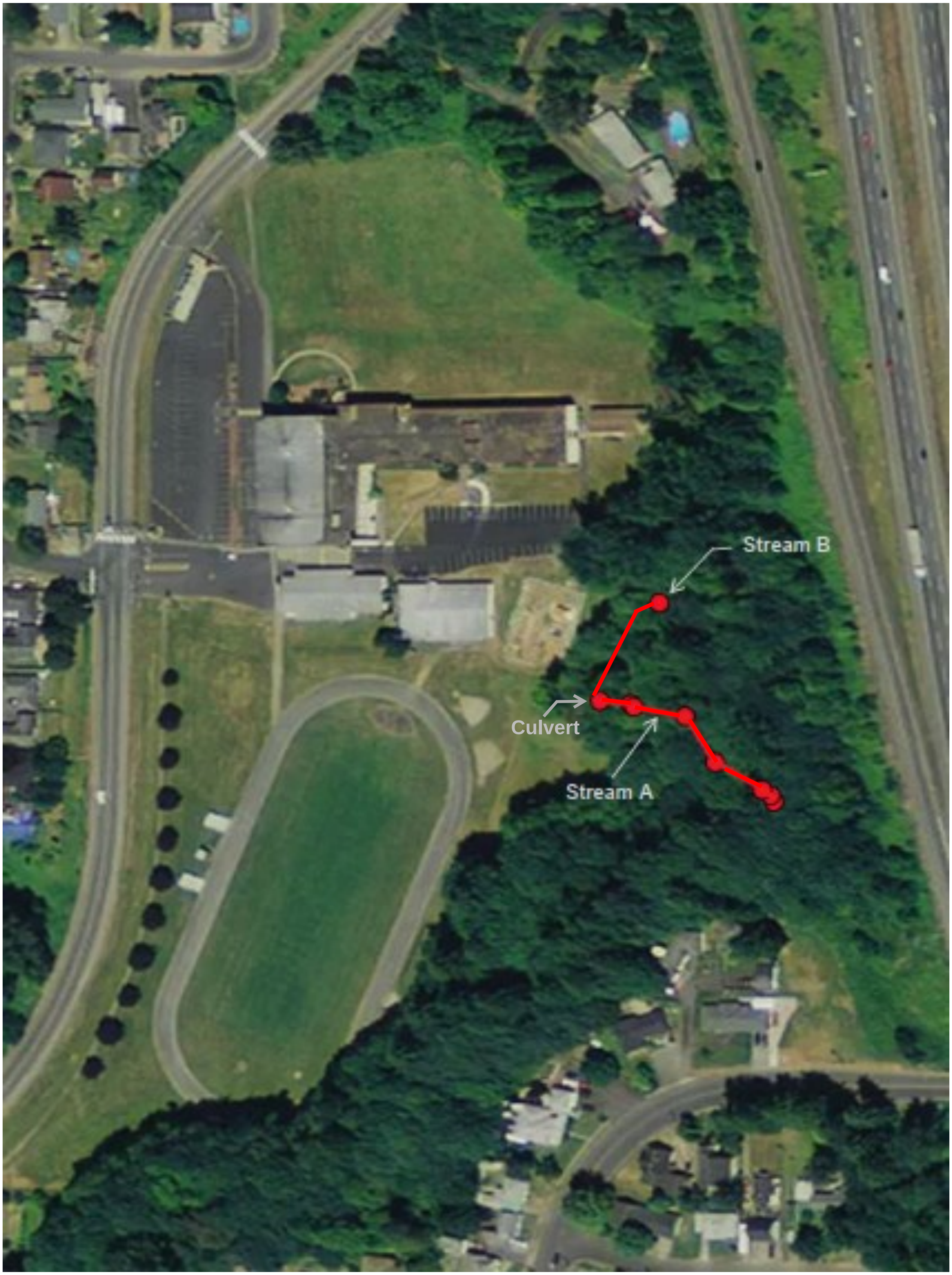
Sincerely,

Steffanie Taylor
Senior Biologist/Principal

Megan Mill
Biologist

Attachments:

Figure 1: Site Map



Ecological Land Services

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DATE: 5/1/2020
DWN: MM
PRJ. MGR: ST
PROJ.#:2742.04

Figure 1
Site Map
Huntington Middle School Memorandum
Kelso School District
Cowlitz County, Washington