Geotechnical Engineering Report

Larus Senior Apartments
7520 NE Bothell Way
Kenmore, Washington

Prepared for:
TWG Development LLC
1301 East Washington Street
Indianapolis, Indiana 46202

June 20, 2024
PBS Project 73658.000

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1 INTRODUCTION

1.1 General

This report presents results of PBS Engineering and Environmental LLC (PBS) geotechnical engineering services for the proposed development located at 7520 NE Bothell Way in Kenmore, Washington (site). The general site location is shown on the Vicinity Map, Figure 1. The locations of PBS' explorations in relation to existing and proposed site features are shown on the Site Plan, Figure 2.

1.2 Purpose and Scope

The purpose of PBS' services was to develop geotechnical design and construction recommendations in support of the planned new development. 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

Six borings were advanced to depths ranging from approximately 51.5 to 61.5 feet below the existing ground surface (bgs) within the development footprint. The borings were logged and representative soil samples collected by a member of the PBS geotechnical engineering staff. The approximate boring locations are shown on the Site Plan, Figure 2. The interpreted boring logs are presented as Figures A1 through A6 in Appendix A, Field Explorations.

1.2.3 Previous Subsurface Explorations

Previous nearby explorations reviewed for this report include three borings, designated B-1 through B-3 (NGA-B-1 through NGA-B-3 on Figure 2), completed by Nelson Geotechnical Associates, Inc. (NGA), in 2019.

The approximate locations of previous explorations are shown on Figure 2. Logs of the previous explorations are provided in Appendix C.

1.2.4 Soils Testing

Soil samples were returned to our laboratory and classified in general accordance with the Unified Soil Classification System (ASTM D2487) and/or the Visual-Manual Procedure (ASTM D2488). Laboratory tests included natural moisture contents, 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.

1.2.5 Geotechnical Engineering Analysis

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

1.2.6 Report Preparation

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

- Field exploration logs and site plan showing approximate exploration locations
- Laboratory test results
- Groundwater levels and considerations
- Liquefaction potential

- Building slab/mat recommendations, including:
    - Subgrade preparation
    - Moisture mitigation
    - Modulus of subgrade reaction
- Lateral earth pressures for retaining wall design including:
    - Active, passive, and at-rest earth pressures
    - Seismic lateral force
    - Allowable bearing pressure
    - 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
    - Subgrade preparation
    - Utility trench excavation and backfill requirements
    - Temporary and permanent slope inclinations
- Seismic design criteria in accordance with the 2021 International Building Code (IBC) with state of Washington amendments
- Recommended asphalt concrete (AC) pavement section

1.3 Project Understanding

The site is located approximately 0.2 miles north of the Sammamish River, on the north side of NE Bothell Way (SR 522). Based on review of the conceptual site plan (prepared by Wattenbarger Architects), PBS understands the proposed development will include construction of a six-story building at-grade, consisting of five floors of timber construction over one floor of concrete, with a building footprint of approximately 26,600 square feet and at-grade parking.

Based on our experience with similar projects, estimated maximum column and perimeter wall foundation loads will likely be less than 250 kips and 25 kips per linear foot, respectively. Slab loads will likely be less than 250 pounds per square foot (psf). PBS will coordinate with the structural engineer to determine estimated column loads and the building period.

2 SITE CONDITIONS

2.1 Surface Description

The site consists of two parcels totaling 1.18 acres. Parcel 0114100904 is currently occupied by a single-story commercial building with an asphalt parking lot. Parcel 0114100905 is currently occupied by a partially fenced single-story residence and grass field. The site is bordered by residential properties to the north, a driveway to the residential properties to the west, NE Bothell Wall to the south, and a commercial parking lot to the east. Review of available Washington Department of Natural Resources (WADNR) lidar and topographic data indicate the site generally slopes slightly to the south, with ground elevations ranging from a maximum of 38 feet (NAVD 88) in the northern portion of the site to 35 feet at the southern portion of the site.

2.2 Geologic Setting

The site is located within the southern Puget Lowland; a tectonic depression within the physiographic province that separates the Cascade Range from the Olympic Peninsula, and extends from the Puget Sound to Eugene, Oregon (Yeats et al., 1996). The Puget Lowland is situated along the Cascadia Subduction Zone (CSZ) where oceanic rocks of the Juan de Fuca Plate are subducting beneath the North American Plate, resulting in deformation and uplift of the Olympic Mountains and volcanism in the Cascade Range.

The site is located within the active Southern Whidbey Fault Zone, which has been interpreted as a complex zone of transpressional deformation with reverse and left-lateral senses of movement. This northwest-trending fault zone is up to 5 to 7 km wide, and extends more than 65 km across Possession Sound, southern Whidbey Island, and into the eastern Strait of Juan de Fuca. It has a slip rate of 0.2 to 1 mm/year and recurrence interval of 0.4 to 9.2 thousand years (Johnson et al., 2016).

The Puget Lowland has been repeatedly glaciated over the last 2 million years during the systematic advance and retreat of continental ice sheets moving southward from British Columbia (Booth et al., 2009). The modern topography reflects these cyclic modes of glacial scouring during advancement of the Puget Ice Lobe, with compacted glacial till forming undulating hills (drumlins) elongated in the direction of ice flow. These distinct features are apparent in the surrounding topographic high points within hillshade maps generated from lidar data (WADNR, 2024)

The site is mapped as underlain by Pleistocene-aged transitional beds, which are described as thinly bedded clay, silt, and fine sand, with layers of peat and gravel in the lower part of the unit. This unit can reach thicknesses of up to 90 feet bgs.

2.3 Subsurface Conditions

The site was explored by drilling six borings, designated B-1 through B-6, to depths of 51.5 to 61.5 feet bgs. The drilling was performed by Boretec 1, Inc., of Bellevue, Washington, using an EC-95 Track Drill rig and hollow-stem auger drilling techniques. The borings were logged and representative samples collected by a member of the PBS geotechnical staff.

PBS has summarized the subsurface units as follows:

TOPSOIL (SILTY SAND):	Dark brown to black, loose, silty sand with organic matter was encountered just below the ground surface in boring B-1.
ALLUVIUM (SAND with SILT and GRAVEL):	Sand, silt, and gravel of varying amounts was encountered just below the ground surface or topsoil in borings B-1 through B-6. This unit was interpreted as alluvium, and was generally brown to brown-gray, loose to dense, with fine to coarse sand and gravel.
PRE-FRASER DEPOSITS (SILT/SAND):	Silt with varying amounts of sand and gravel was encountered below the alluvium in borings B-1 through B-6. This unit was interpreted as Pre-Fraser deposits and was generally brown to gray, stiff to very stiff or medium dense to dense, with fine to coarse sand and fine gravel. The silt exhibited low plasticity.
PRE-OLYMPIA GLACIAL DEPOSITS (SANDY SILT/SILTY SAND):	Sand and silt of varying amounts was encountered below the Pre-Fraser deposits in borings B-1, B-2, B-4, and B-5. This unit was interpreted as Pre-Olympia glacial deposits and was generally gray, hard or dense to very dense, with fine to coarse sand. The silt exhibited low plasticity.

2.4 Groundwater

Static groundwater was encountered and measured at 22 feet bgs during our explorations. A piezometer was installed in boring B-2 to obtain depths to groundwater during different times of year. Please note that groundwater levels can fluctuate during the year depending on climate, irrigation season, extended periods of precipitation, drought, and other factors.

3 SEISMIC HAZARDS

Seismic hazards are geologic hazards resulting from seismicity (earthquakes). Earthquakes produce shaking and ground motions that can result in damage and destruction of buildings and infrastructure, fault rupture of the ground surface, liquefaction, lateral spreading, tsunamis, earthquake-induced landslides, and seiches. The site is located approximately 3 miles south from the approximately east-west trending Tacoma fault (fault no. 581; Brocher et al., 2017). Due to the location of the site away from slopes and water bodies, tsunamis, earthquake-induced landslides, seiches, and lateral spreading are not considered hazards for the site.

3.1 Liquefaction Potential

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

Based on a review of the Washington Division of Geology and Earth Resources and the King County Liquefaction Susceptibility Map (Palmer et al., 2004), the site is shown as having a very low liquefaction hazard; however, based on the soil types and relative density of site soils encountered in our explorations, our current opinion is that the risk of structurally damaging liquefaction settlement at the site is moderate to high. Due to the relatively flat topography of the site area, and the large distance (greater than 1,000 feet) between the site and the nearest free face at the Sammamish River, the risk of structurally damaging lateral spreading is low.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 Geotechnical Design Considerations

The project site is underlain by zones of loose, saturated, potentially liquefiable sand containing variable amounts of silt. Due to the potential for liquefaction to occur at the site as a result of a code-based earthquake, and based on our observations and analyses, support of the new structure on shallow foundations is not feasible without ground improvement or deep foundations.

Ground improvement, such as aggregate piers, or deep foundations, such as augercast piles, are necessary to mitigate the effects of liquefaction resulting from a code-based earthquake.

The grading and final development plans for the project had not been completed when this report was prepared. Once completed, PBS should be engaged to review the project plans and update our recommendations as necessary.

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 2021 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 seconds, a site-response analysis is not required to determine the spectral accelerations of liquefied soils and seismic design parameters can be determined using the pre-liquefaction site class, Site Class D. The seismic design criteria, in accordance with the 2021 International Building Code IBC with state of Washington amendments, are summarized in Table 1.

Table 1. 2021 IBC Seismic Design Parameters

Parameter	Short Period	1 Second
Maximum Credible Earthquake Spectral Acceleration	$S_s = 1.27$ g	$S_1 = 0.44$ g
Site Class	D[1]	
Site Coefficient	$F_a = 1.00$	$F_v = 1.86$[2,3]
Adjusted Spectral Acceleration	$S_{MS} = 1.27$ g	$S_{M1} = 0.82$ g[4]
Design Spectral Response Acceleration Parameters	$S_{DS} = 0.85$ g	$S_{D1} = 0.55$ g
MCE_G Peak Ground Acceleration	PGA = 0.54 g	
Site Amplification Factor at PGA	$F_{PGA} = 1.1$	
Site Modified Peak Ground Acceleration	$PGA_M = 0.594$ g	

g= Acceleration due to gravity

[1] Site Class D can be used if the fundamental period of the new structure is less than 0.5 seconds. If the fundamental period is larger than 0.5 seconds, site shall be classified as Site Class F and site-specific ground motion hazard analysis will be provided in an addendum to this report.

[2] A ground motion hazard analysis shall be performed in accordance with the American Society of Civil Engineers' Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7-16) Section 21.2, unless exempted in accordance with Exception 2 or 3 of Section 11.4.8.

[3] Use of site coefficient Fv = 1.856 requires adherence to Exception 2 criteria in Section 11.4.8 of ASCE 7-16.

[4] Site-specific site response analysis is not required for structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient Cs 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 $T > T_L$.

4.2.2 Liquefaction Evaluation

The susceptibility of site soils to liquefaction (i.e., sand-like or clay-like behavior) was evaluated using criteria established by Boulanger and Idriss (2005) and Bray and Sancio (2006). The results of our analyses indicate liquefaction at the site during a code-based earthquake will occur in soil layers between the depths of 22 and 55 feet bgs, except for B-2 where liquefaction extends to 60 feet bgs. Based on our analyses, the code-based earthquake would likely result in 6 to 13 inches of total liquefaction-induced settlement, with approximately 1 inch of differential liquefaction settlement over 20 feet.

The risk of surface manifestation of liquefaction could be reduced at the site by the presence of the existing non-liquefiable layer at the surface (i.e., "crust"). This crust is approximately 22 feet thick (represented by the unsaturated and non-liquefiable soil). 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 of approximately 29 feet, the crust would need to be on the order of 30 feet thick; therefore, liquefaction is expected to manifest at the surface.

Due to the relatively flat site topography, distance from the nearest free-face (e.g., river or slough bank), and relative discontinuity of liquefiable layers at the site, our current opinion is that the risk of structurally damaging lateral spreading associated with liquefaction is low.

Ground improvement such as aggregate piers could help reduce liquefaction settlement during a code-based earthquake. If deep foundations are implemented, the anticipated liquefaction settlement will impart downdrag loads on the piles from both the liquefied soil and the non-liquefiable crust.

4.3 Foundation Alternatives

The use of shallow foundations without ground improvement is not considered feasible due to the potential for liquefaction and the associated differential settlement expected during a code-based earthquake. PBS has developed recommendations for two foundation alternatives, which have different levels of damage risk:

- Mitigate potentially liquefiable soils with soil improvement (stone columns/ modified aggregate piers) in conjunction with a mat foundation.
- Support the structure on deep foundations. Despite the challenge of potentially high downdrag loads, the underlying glacial soils below depths of approximately 55 feet bgs would likely provide suitable support for deep foundations depending on estimated loads of the proposed structure and tolerable settlement.

4.3.1 Ground Improvement

Ground improvement could be considered for the project to support structure foundations by reducing potential settlement and mitigating liquefaction during a code-based earthquake. However, due to the density and thickness of the non-liquefiable crust, predrilling for ground improvement elements would be required, which would significantly increase construction costs. If ground improvement is used to decrease the liquefaction settlement potential below the crust, structures may be supported on shallow mat foundations.

The detailed design for ground improvement is typically completed by a design-build contractor. The type and extent of ground improvement should be determined by the specialty contractor based on the required project performance criteria.

4.3.2 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 glacially consolidated soils present at depths of approximately 55 feet bgs. 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 structural slabs are specifically designed, or slabs are supported on piles). Due to the presence of potentially liquefiable and loose soils, we recommend that the piles, if selected as the preferred foundation alternative, penetrate into glacial deposits at depths below 60 feet bgs, where they will derive their capacity from both shear resistance and end bearing.

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:

- Potential differential settlement between structures on piles and utilities or unsupported structures
- Requires specialty construction equipment and an experienced specialty contractor

Several deep foundation alternatives could be considered for the site, including driven piles, drilled shafts, augercast piles, and drilled displacement piles (DD). However, augercast piles embedded into glacially consolidated layers below a depth of 60 feet bgs would allow larger pile capacities and is typically a cost-effective alternative for these subsurface conditions.

DD piles can be used as an alternative to augercast piles to avoid construction issues that may arise during augercast pile installation. The use of DD piles can reduce the risks associated with augercast piles, such as soil flighting, due to the use of auger tools that can laterally displace and densify the soil around the pile during drilling. Consequently, soil is improved and would have higher values of side-shear resistance, which results in higher load carrying capacity at a shorter length compared to an augercast pile of similar diameter.

DD piles have the benefit of generating little to no spoils that would need to be removed from the site. However, penetrating DD piles into deep dense or hard soils with strong resistance will be more difficult. As a result, the depth to which they are effective will be limited, likely to depths of less than 80 feet.

Based on the benefits and challenges described above, PBS recommends supporting the building on DD piles or augercast piles. Augercast piles are typically installed with diameters ranging from 12 to 36 inches and lengths of up 100 feet. In practice, typical pile sizes range between 12 and 24 inches with minimum center-to-center spacing of 3 to 5 pile diameters. Typical DD pile diameters are 16 to 24 inches with lengths of 65 to 80 feet.

If augercast piles are selected as the preferred alternative, PBS can complete analyses to evaluate static and seismic geotechnical design capacities for the piles.

4.4 Floor Slabs

If the structure is supported on deep foundations, building floor slabs could be designed as structural slabs to fully span the distance between grade beams/pile caps, or they could follow conventional slab-on-grade design using the following recommendations if the owner accepts the risk of damage requiring slab repairs after the design earthquake.

Satisfactory subgrade support for building floor slabs can be obtained from the native silty sand to sandy silt or silt fill 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. A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade. Thicker aggregate sections may be necessary where undocumented fill is present, soft/loose soils are present at subgrade elevation, and/or during wet conditions. Imported granular material should be composed of crushed rock or crushed gravel that is relatively well graded between coarse and fine, contains no deleterious materials, has a maximum particle size of 1 inch, and has less than 5% by dry weight passing the US Standard No. 200 Sieve.

Floor slabs supported on a subgrade and base course prepared in accordance with the preceding recommendations may be designed using a modulus of subgrade reaction (k) of 80 pounds per cubic inch (pci).

4.5 Retaining Walls

The proposed new development may include retaining walls up to 5 feet tall for site grading. The following recommendations are based on the assumption of flat conditions in front of and behind the wall and fully drained backfill. For unrestrained walls allowed to rotate at least 0.005H about the base, where H is the height of the wall, we recommend using an active earth pressure calculated using an equivalent fluid weight (EFW) of 35 pounds per cubic foot (pcf). Where walls are constrained against rotation, we recommend using an at-rest earth pressure calculated using an EFW of 55 pcf. We recommend any retaining walls founded on native soil or compacted structural fill be provided with adequate drainage and backfilled with clean, angular, crushed rock fill, in accordance with the recommendations provided in section 5.3. For retained heights of less than 6 feet, seismic loading would not need to be considered. Recommended lateral earth pressure distributions are shown on Figure 3, Retaining Wall Earth Pressure Diagram. Additional lateral pressures due to surcharge loads can be estimated using the guidelines shown on Figure 4, Lateral Surcharge Detail.

Lateral loads can also be resisted by a passive resistance of 250 psf acting against retaining/embedded walls and foundations, and by friction acting on the base of concrete wall foundations using a friction coefficient of 0.35.

4.5.1 Drainage

Recommended lateral earth pressures assume that walls are fully drained and no hydrostatic pressures develop. For cantilevered concrete walls, a minimum 2-foot-wide zone of free-draining material should be installed immediately behind the wall. A 4-inch diameter perforated drainpipe should be installed at the base of the drain rock and routed to a suitable discharge point approved by the civil engineer.

4.6 Temporary and Permanent Slopes

Temporary excavation and slopes should not exceed the limits specified by local, state, and federal regulations. The stability of temporary excavations and slopes shall be the responsibility of the contractor. We recommend that temporary slopes of up to 10 feet tall, made in fill or thicknesses of native soils, not be steeper than 1.5H:1V (horizontal to vertical). The presence of seepage or groundwater may require that slopes be flattened further to remain stable.

We also make the following recommendations:

- Temporary cut slopes should be excavated with a smooth-bucket excavator, with the slope surface repaired if disturbed.
- Upslope surface runoff should be rerouted to not run down the face of the slopes.
- Slopes should be protected using plastic sheeting, flash coating, or tarps, as necessary, to reduce erosion.
- The duration that excavations or slopes are open should be limited to the shortest time possible (3 months or less).
- Equipment should not be allowed to induce vibration or infiltrate water above the slopes, and no surcharges are allowed within 10 feet of the slope crest.
- The conditions of the excavations and slopes should be monitored by a “competent person” as defined by Occupational Safety and Health Administration (OSHA) and the geotechnical engineer should be contacted if adverse conditions are observed.

PBS understands no permanent cut or fill slopes are anticipated for the project.

4.7 Ground Moisture

4.7.1 General

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

4.7.2 Perimeter Foundation Drains

Due to the moderate to high permeability of site soils and the depth of groundwater at the site, in our opinion, perimeter foundation drains would not be necessary for the proposed at-grade structure. If the building design changes, PBS should be consulted to revise this recommendation if needed.

4.7.3 Vapor Flow Retarder

A continuous, impervious vapor flow retarder must be installed over the ground surface under slabs of all structures. Vapor flow retarders are often required by flooring manufacturers to protect flooring and adhesives from moisture intrusion and mold. Many flooring manufacturers will warrant their product only if it is installed according to their recommendations. The PBS geotechnical team can provide additional information, as necessary, to assist with vapor flow retarder selection.

4.8 Pavement Design

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

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

Table 2. Minimum AC Pavement Sections

Traffic Loading	AC (inches)	Base Course (inches)	Subgrade
Pull-in Car Parking Only	2.5	9	Stiff subgrade as verified by PBS personnel*
Drive Lanes and Access Roads	3	9	

* Subgrade must pass proofroll

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

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

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

5 CONSTRUCTION RECOMMENDATIONS

5.1 Site Preparation

Construction of the proposed structure will involve clearing and grubbing of the existing vegetation or demolition of possible existing structures. In vegetated areas, site stripping should include removing topsoil, roots, and other deleterious materials to a minimum depth of 12 inches bgs. Demolition should include removing existing pavement, utilities, etc., throughout the proposed new development. Underground utility lines or other abandoned structural elements should also be removed. The voids resulting from removal of foundations or loose soil in utility lines should be backfilled with compacted structural fill. The base of these excavations should be excavated to stiff 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 Augercast Pile Installation

While advancing the auger to the required depth, balanced auger rotation and penetration rates are critical to ensuring the auger flights are filled with soil and the stability of the hole is maintained. This can be accomplished by balancing auger rotation and penetration rates. Controlling the rate of penetration during drilling and grout placement will help avoid lateral decompression of the ground inside the hole, the loosening of the in situ soil around the hole, and ground subsidence adjacent to the pile. The installation of augercast piles can be difficult while drilling into very dense glacially consolidated soils. The rate of penetration can be slowed and the overburden soils are then flighted by side loading of the auger. Reliably controlling the volume per unit length of the pile during withdrawal of the auger can also be difficult, which can lead to structural defects or necks in the pile, even with costly, less economical oversupply of concrete or grout. The drilling contractor must ensure that the pile has sound bearing and embedment into the bearing layer. One of the disadvantages of augercast piles is the generation of soil spoils that require collection and disposal. Handling spoils can be a significant issue when the soils are contaminated or if limited room is available on the site for handling the material.

5.1.2 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 stiff condition or be excavated and replaced with structural fill.

5.1.3 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 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

Minimal site grading is anticipated for the proposed development. Structural fill should be placed over subgrade that has been prepared in conformance with the Site Preparation and Wet/Freezing Weather and Wet Soil Conditions sections of this report. Structural fill material should consist of relatively well-graded soil, or an approved rock product that is free of organic material and debris, and contains particles not greater than 4 inches nominal dimension.

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

5.3.1 On-Site Soil

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

5.3.2 Imported Granular Materials

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

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

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% (by dry weight) passing the US Standard No. 200 Sieve. The imported granular material should be placed in one lift and compacted to at least 95% of the maximum dry density, as determined by ASTM D1557.

5.3.4 Foundation Base Aggregate

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

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% by dry weight passing the US Standard No. 200 Sieve, and should meet the standards prescribed by WSDOT SS 9-03.12(3) – Gravel Backfill for Pipe Zone Bedding. The pipe zone backfill should be compacted to at least 90% of the maximum dry density as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

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

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

5.3.6 Retaining Wall Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of 0.5H, where H is the height of the retaining wall, should consist of granular material meeting WSDOT SS 9-03.12(2) – Gravel Backfill for Walls. We recommend the granular wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the requirements provided in WSDOT SS 9-33.2 – Geosynthetic Properties, Table 3, for separation geotextile.

The wall backfill should be compacted to a minimum of 92% of the maximum dry density, as determined by ASTM D1557. However, backfill located within a horizontal distance of 3 feet from the retaining walls should only be compacted to approximately 90% of the maximum dry density, as determined by ASTM D1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as, jumping jack or vibratory plate compactor).

5.3.7 Stabilization Material

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

6 ADDITIONAL SERVICES AND CONSTRUCTION OBSERVATIONS

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

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

Satisfactory earthwork performance depends on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. We recommend that PBS be retained to observe general excavation, stripping, fill placement, foundation 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 or test pits. Such variations may result in changes to our recommendations and may require additional funds for expenses to attain a properly constructed project; therefore, we recommend a contingency fund to accommodate such potential extra costs.

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

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

8 REFERENCES

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Important Information about This

Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

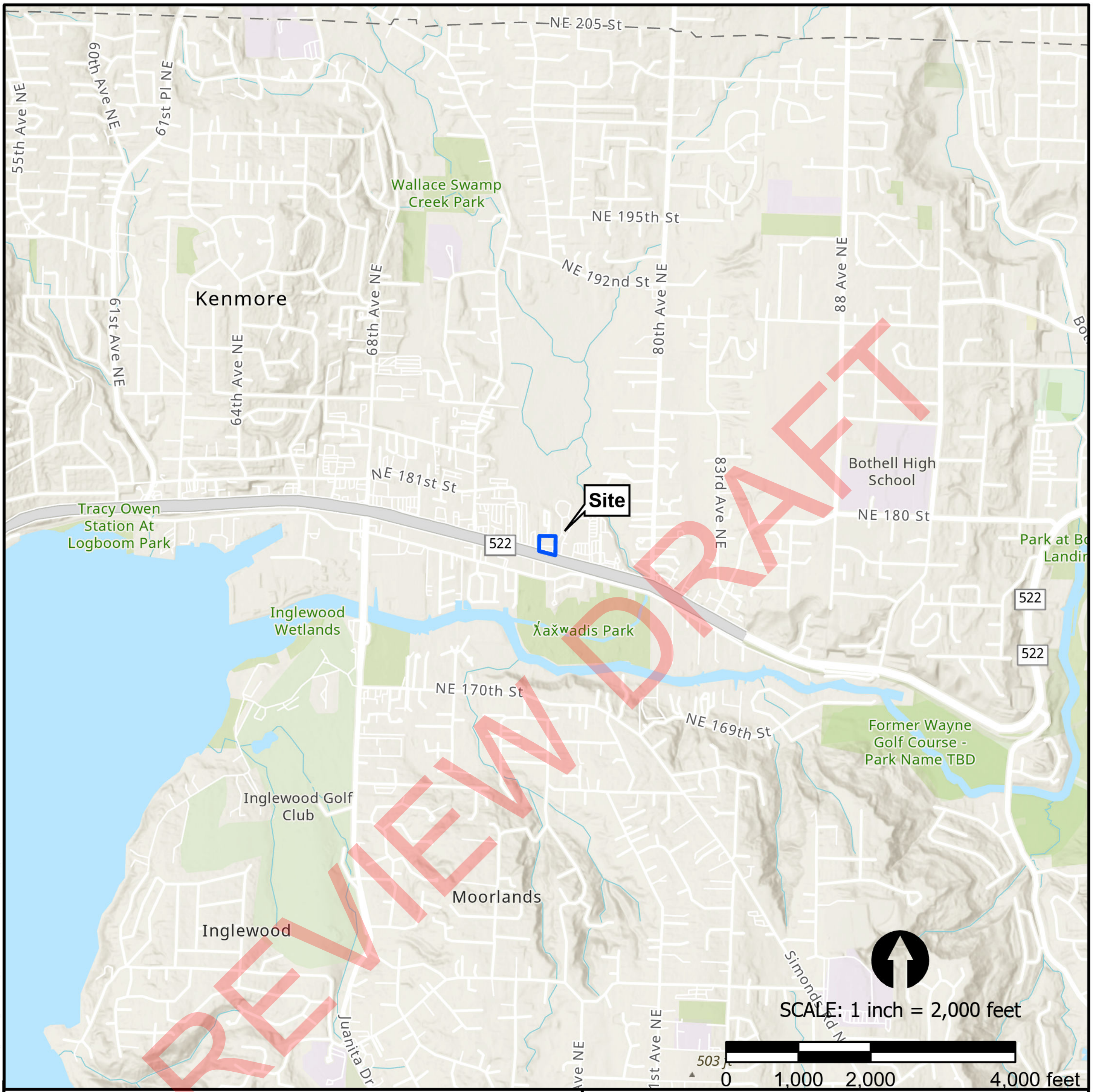
Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

GBA
GEOPROFESSIONAL
BUSINESS
ASSOCIATION

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

Figures



VICINITY MAP

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

DATE: JUN 2024 · PROJECT: 73658.000

PBS

FIGURE

1

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EXPLANATION

- ● B-1 - Boring name and approximate location
- ● B-2 - Boring with piezometer name and approximate location
- ● NGA-B-1 Nearby boring name and approximate location (Nelson Geotechnical Associates, Inc. 2019)

Notes: Google Earth 2022 imagery

Coordinate System: NAD 1983 2011 StatePlane Washington North FIPS 4601 Ft US

SCALE: 1 inch = 35 feet

0 17.5 35 70 feet

SITE PLAN

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

DATE: JUN 2024 · PROJECT: 73658.000

PBS

FIGURE
2

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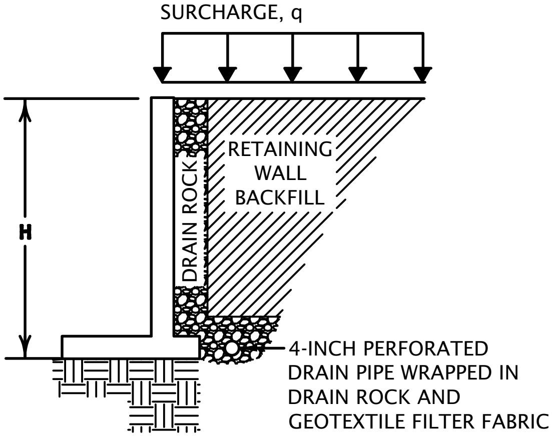
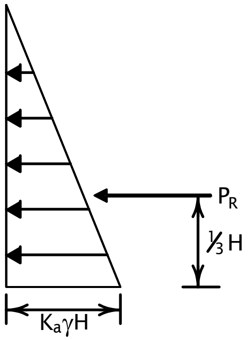
EARTH PRESSURES FOR RETAINING WALLS



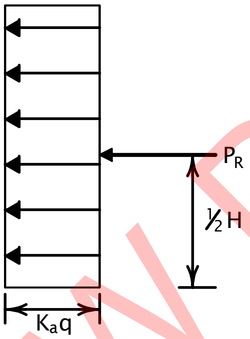
PARAMETER	VALUE
K_a	0.26
K_o	0.44
ΔK_{ae}	-
γ	120 pcf

ACTIVE EARTH PRESSURE



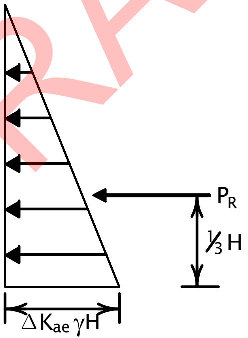
$P_R = K_a\,\gamma\,\frac{H}{2}^{2}$

SURCHARGE PRESSURE (ACTIVE)



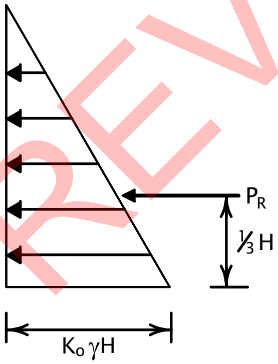
$P_R = K_a\,q\,H$

SEISMIC SURCHARGE PRESSURE



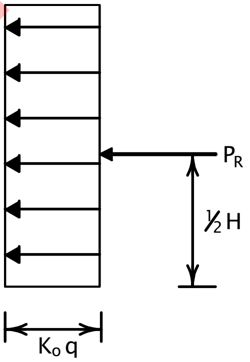
$P_R = \Delta K_{ae}\,\gamma\,\frac{H}{2}^{2}$

AT-REST EARTH PRESSURE



$P_R = K_o\,\gamma\,\frac{H}{2}^{2}$

SURCHARGE PRESSURE (AT-REST)



$P_R = K_o\,q\,H$

NOT TO SCALE

Legend

H = HEIGHT OF RETAINED SOIL (FEET)
Ka = ACTIVE EARTH PRESSURE COEFFICIENT
Ko = AT-REST EARTH PRESSURE COEFFICIENT
ΔKae = SEISMIC ACTIVE EARTH PRESSURE COEFFICIENT
P_R = RESULTANT LATERAL EARTH FORCE
γ = SOIL UNIT WEIGHT (PCF)

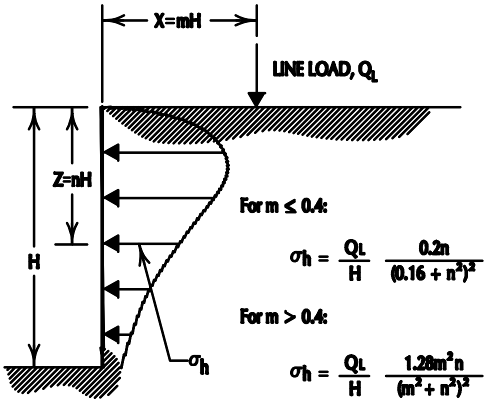
PBS

RETAINING WALL EARTH PRESSURE DIAGRAM

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

JUN 2024
73658.000
FIGURE
3

LATERAL SURCHARGE DETAIL



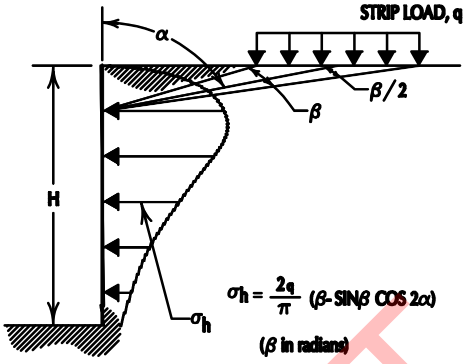
For m ≤ 0.4:

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

For m > 0.4:

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

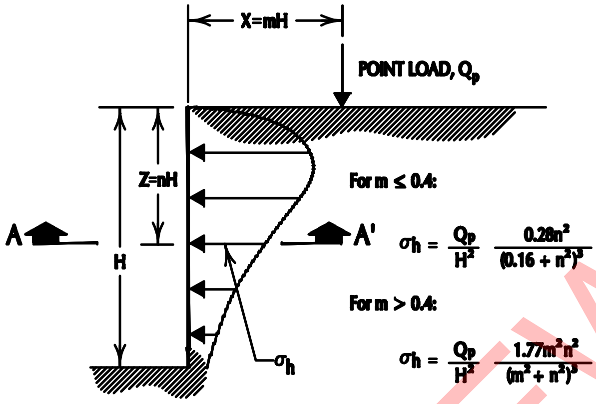
LINE LOAD PARALLEL TO WALL



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

(β in radians)

STRIP LOAD PARALLEL TO WALL

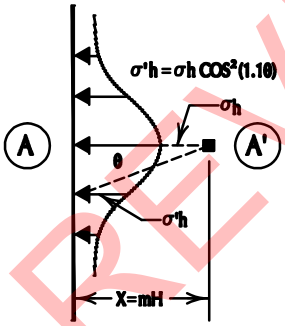


For m ≤ 0.4:

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

For m > 0.4:

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



$$\sigma'h = \sigma h \ \text{COS}^2 (1.1\theta)$$

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

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

NOT TO SCALE

Legend

H = HEIGHT OF RETAINED SOIL (FEET)
σ_h = LATERAL SURCHARGE PRESSURE
σ'_h = LATERAL SURCHARGE PRESSURE AT ANGLE θ

Filename: L:\Projects\73000\73600-73699\73658\GeoDWG\73658.000_EDP_20240620.dwg Layout Tab: 04_SURCHARGE User: Spencer Ambauen CAD Plot Date/Time: 6/20/2024 12:44:38 PM

LATERAL SURCHARGE DETAIL

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

JUN 2024
73658.000

FIGURE
4

Appendix A

Field Explorations

Appendix A: Field Explorations

A1 GENERAL

PBS explored subsurface conditions at the project site by advancing six borings to depths of up to approximately 51.5 feet to 61.5 feet bgs on May 15 and 16, 2024. 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 an EC-95 Track Drill rig provided and operated by Boretec 1, Inc., of Bellevue, Washington, using hollow-stem auger 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 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.

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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% 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 modifier words in general accordance with the ASTM D2488 Visual-Manual Procedure. "General Accordance" means that certain local and common descriptive practices may have been followed. In accordance with ASTM D2488, 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 explains the soil names and modifying terms used to describe fine- and coarse-grained soils.

Fine-Grained Soils (50% or greater fines passing 0.075mm, No. 200 sieve)
The primary soil type, i.e., SILT or CLAY, is designated through visual-manual procedures to evaluate soil toughness, dilatancy, dry strength, and plasticity. The following outlines the terminology used to describe fine-grained soils and may vary 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% 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, as follows. SILT soils with low to non-plastic behavior (i.e., PI < 7) may be classified using relative density.

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

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

Soil Descriptions

Coarse-Grained Soils (less than 50% fines)

Coarse-grained soil descriptions, i.e., SAND or GRAVEL, are based on the portion of materials passing a 3-inch (75mm) sieve. Coarse-grained soil group symbols are applied in accordance with ASTM D2488 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%. 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.

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

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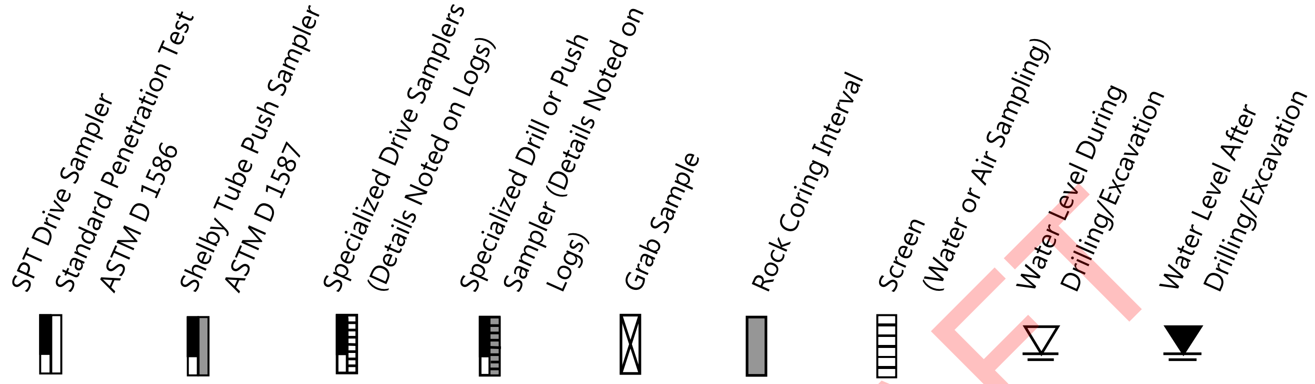
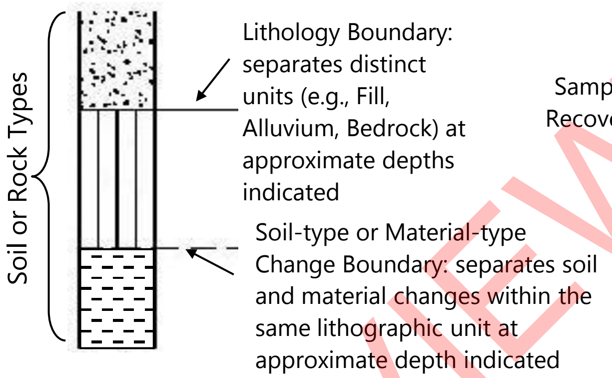
Table A-2
Key To Test Pit and Boring Log Symbols

SAMPLING DESCRIPTIONS

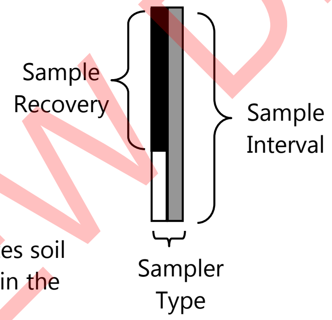


LOG GRAPHICS

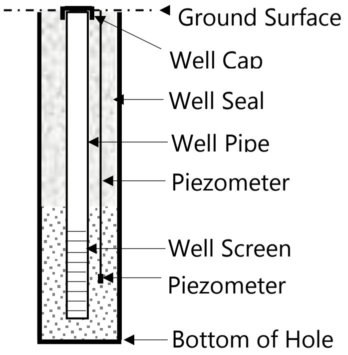
Soil and Rock



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	amsl	Above Mean Sea Level
CON	Consolidation	HCL	Hydrochloric Acid
UC	Unconfined Compressive Strength		

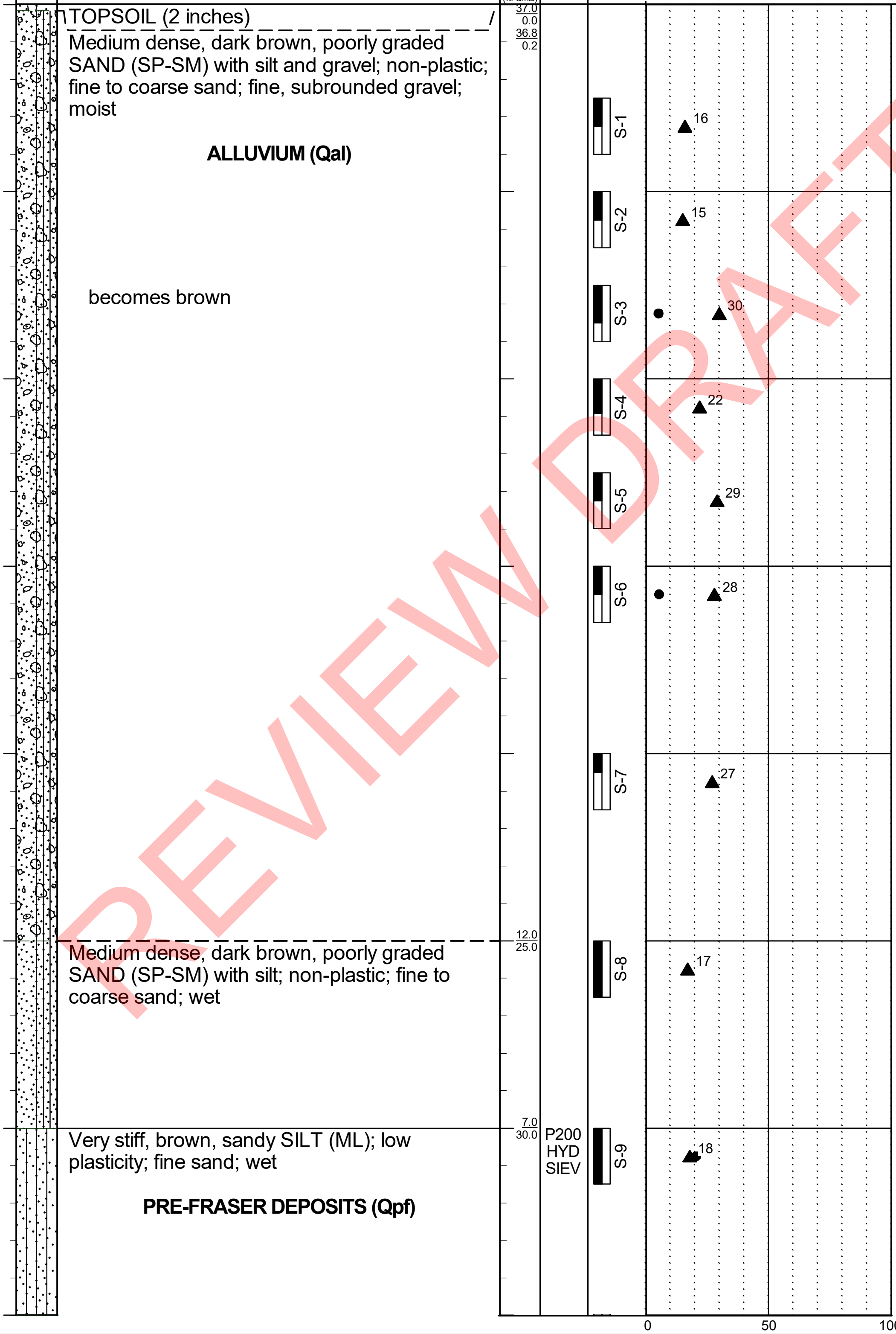
Details of soil and rock classification systems are available upon request.

PBS AN APEX COMPANY	LARUS SENIOR APARTMENTS KENMORE, WASHINGTON	**BORING B-1**
	PBS PROJECT NUMBER: 73658.000	APPROX. BORING B-1 LOCATION: (See Site Plan) Lat: 47.757376 Long: -122.239900

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	ELEV/DEPTH 37 (ft. amsl)	TESTING	SAMPLE TYPE SAMPLE ID	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	TOPSOIL (2 inches)	37.0/0.0				
	Medium dense, dark brown, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to coarse sand; fine, subrounded gravel; moist **ALLUVIUM (Qal)**	36.8/0.2		S-1	16	
5.0				S-2	15	
	becomes brown			S-3	30	
10.0				S-4	22	
				S-5	29	
15.0				S-6	28	
20.0				S-7	27	▼ 05/16/24
25.0	Medium dense, dark brown, poorly graded SAND (SP-SM) with silt; non-plastic; fine to coarse sand; wet	12.0/25.0		S-8	17	
30.0	Very stiff, brown, sandy SILT (ML); low plasticity; fine sand; wet **PRE-FRASER DEPOSITS (Qpf)**	7.0/30.0	P200 HYD SIEV	S-9	18	P200 = 51%
35.0						



DRILLING METHOD: Hollow-Stem Auger
DRILLED BY: Boretec 1, Inc.
LOGGED BY: V. Tavangar

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 60
LOGGING COMPLETED: 5/16/2024

FIGURE A1

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

PBS PROJECT NUMBER:
73658.000

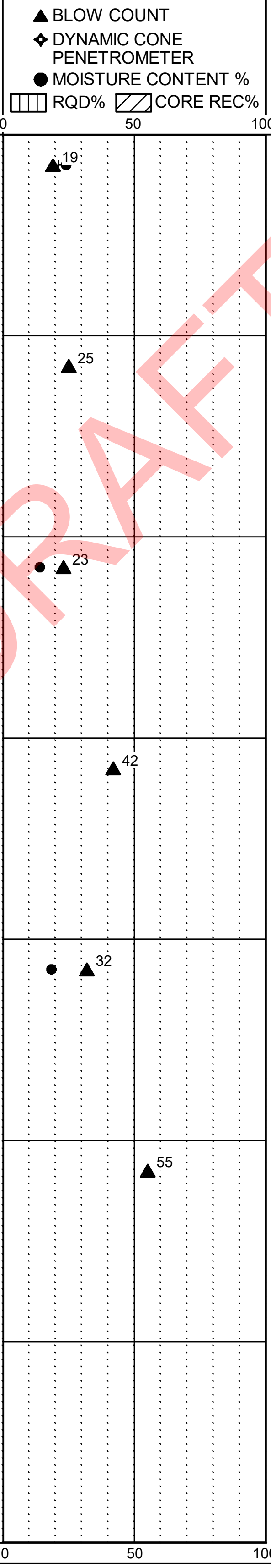
BORING B-1
(continued)

APPROX. BORING B-1 LOCATION:
(See Site Plan)
Lat: 47.757376 Long: -122.239900

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	ELEV / DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	BLOW COUNT	INSTALLATION AND COMMENTS
35.0	Very stiff, brown, sandy SILT (ML); low plasticity; fine sand; wet **PRE-FRASER DEPOSITS (Qpf)**	2.0 / 35.0	ATT	S-10	19	LL = 25 PL = 21 PI = 4
40.0	Medium dense, brown, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet	-3.0 / 40.0		S-11	25	
45.0	becomes gray			S-12	23	
50.0	Dense, gray, silty SAND (SM); non-plastic; fine to coarse sand; wet **PRE-OLYMPIA GLACIAL DEPOSITS (Qpog)**	-13.0 / 50.0		S-13	42	
55.0	Hard, gray, sandy SILT (ML); non-plastic to low plasticity; fine sand; wet	-18.0 / 55.0		S-14	32	
60.0	Very dense, gray, silty SAND (SM); non-plastic; fine sand; wet	-23.0 / 60.0		S-15	55	
	Final depth 61.5 feet bgs; boring backfilled with bentonite.	-24.5 / 61.5				
65.0						
70.0						



DRILLING METHOD: Hollow-Stem Auger
DRILLED BY: Boretec 1, Inc.
LOGGED BY: V. Tavangar

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 60
LOGGING COMPLETED: 5/16/2024

PBS
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LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

BORING B-2

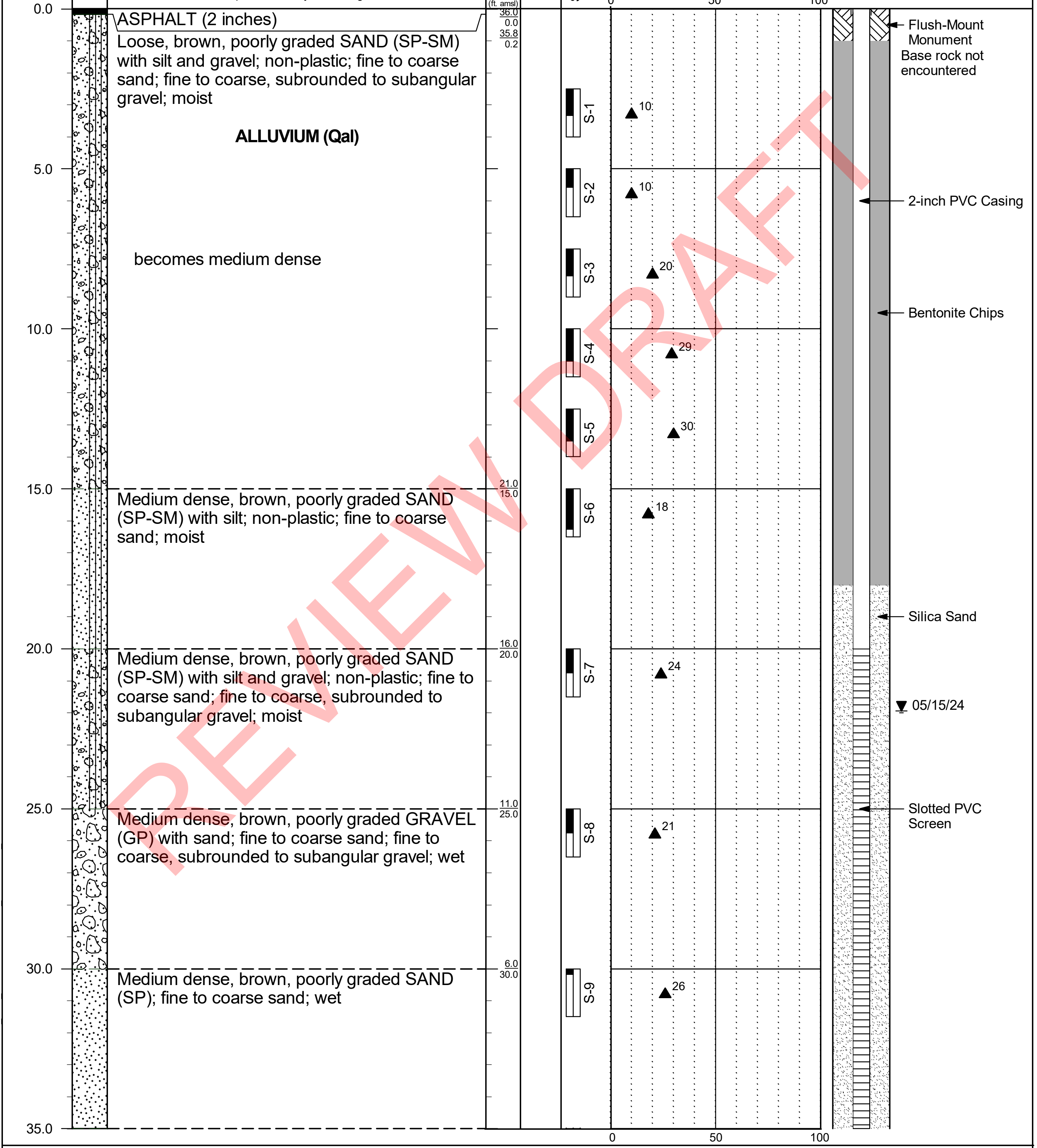
PBS PROJECT NUMBER:
73658.000

APPROX. BORING B-2 LOCATION:
(See Site Plan)
Lat: 47.757156 Long: -122.239723

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

▲ BLOW COUNT
✧ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

INSTALLATION AND COMMENTS
Surface Conditions: Asphalt



DEPTH FEET	ELEV/DEPTH 36 (ft. amsl)	MATERIAL DESCRIPTION	SAMPLE TYPE SAMPLE ID	BLOW COUNT
0.0	36.0/0.0	ASPHALT (2 inches)		
	35.8/0.2	Loose, brown, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to coarse sand; fine to coarse, subrounded to subangular gravel; moist **ALLUVIUM (Qal)**	S-1	10
5.0			S-2	10
		becomes medium dense	S-3	20
10.0			S-4	29
			S-5	30
15.0	21.0/15.0	Medium dense, brown, poorly graded SAND (SP-SM) with silt; non-plastic; fine to coarse sand; moist	S-6	18
20.0	16.0/20.0	Medium dense, brown, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to coarse sand; fine to coarse, subrounded to subangular gravel; moist	S-7	24
25.0	11.0/25.0	Medium dense, brown, poorly graded GRAVEL (GP) with sand; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet	S-8	21
30.0	6.0/30.0	Medium dense, brown, poorly graded SAND (SP); fine to coarse sand; wet	S-9	26
35.0				

Flush-Mount Monument
Base rock not encountered

2-inch PVC Casing

Bentonite Chips

Silica Sand

▼ 05/15/24

Slotted PVC Screen

BORING LOG W/ ELEV 73658.000 B1-6 20240521.GPJ PBS_DATATMPL_GEO.GDT PRINT DATE: 6/18/24:RPG

DRILLING METHOD: Hollow-Stem Auger
DRILLED BY: Boretec 1, Inc.
LOGGED BY: V. Tavangar

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 60
LOGGING COMPLETED: 5/15/2024

FIGURE A2

PBS
AN APEX COMPANY

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

PBS PROJECT NUMBER:
73658.000

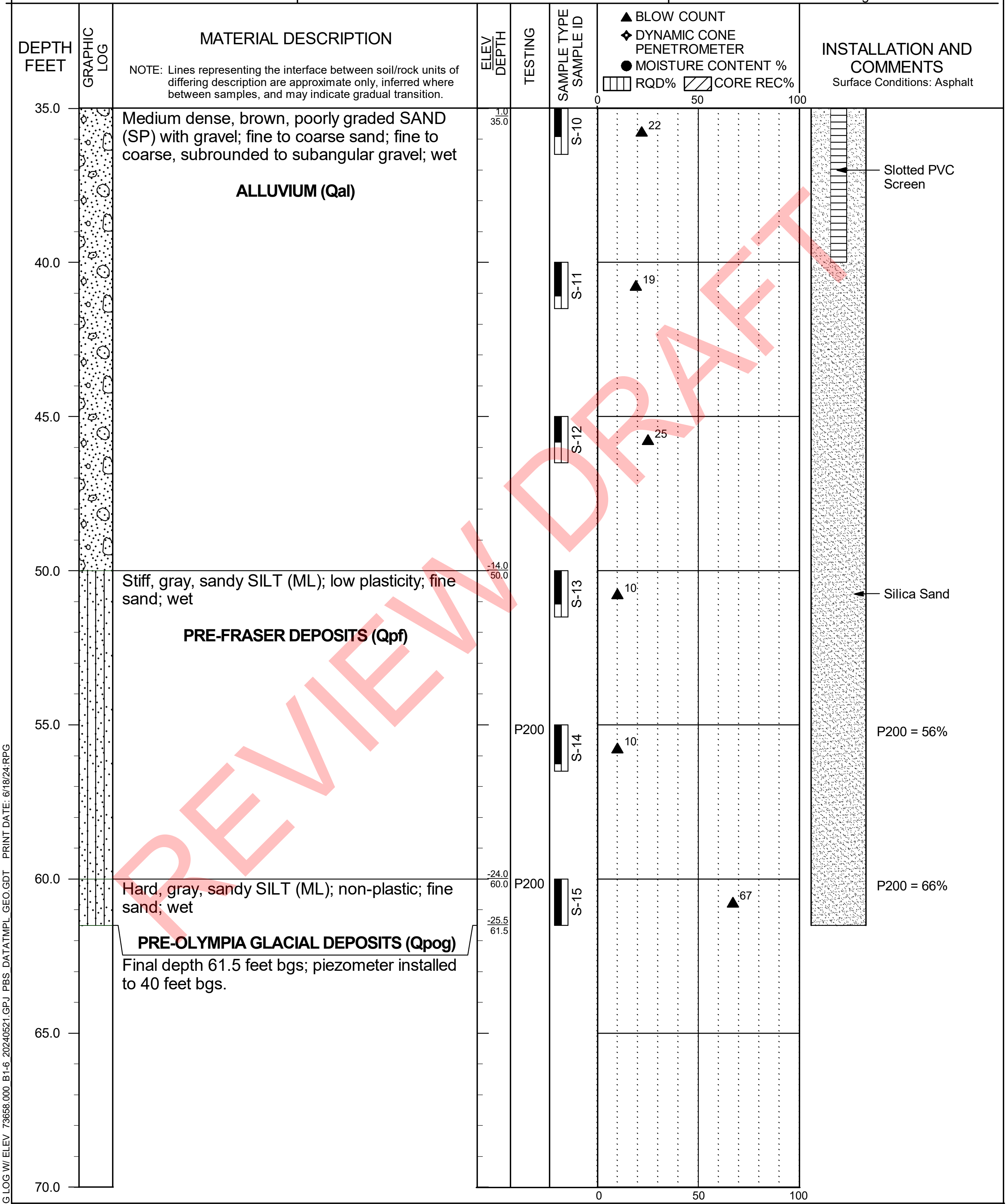
BORING B-2
(continued)

APPROX. BORING B-2 LOCATION:
(See Site Plan)
Lat: 47.757156 Long: -122.239723

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

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

Surface Conditions: Asphalt



DEPTH FEET	MATERIAL DESCRIPTION	ELEV DEPTH
35.0	Medium dense, brown, poorly graded SAND (SP) with gravel; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet **ALLUVIUM (Qal)**	1.0 35.0
50.0	Stiff, gray, sandy SILT (ML); low plasticity; fine sand; wet **PRE-FRASER DEPOSITS (Qpf)**	-14.0 50.0
60.0	Hard, gray, sandy SILT (ML); non-plastic; fine sand; wet **PRE-OLYMPIA GLACIAL DEPOSITS (Qpog)**	-24.0 60.0
61.5	Final depth 61.5 feet bgs; piezometer installed to 40 feet bgs.	-25.5 61.5

SAMPLE ID	BLOW COUNT	TESTING	INSTALLATION AND COMMENTS
S-10	22		
S-11	19		
S-12	25		
S-13	10		
S-14	10	P200	P200 = 56%
S-15	67	P200	P200 = 66%

DRILLING METHOD: Hollow-Stem Auger
DRILLED BY: Boretec 1, Inc.
LOGGED BY: V. Tavangar

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 60
LOGGING COMPLETED: 5/15/2024

FIGURE A2

PBS
AN APEX COMPANY

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

BORING B-3

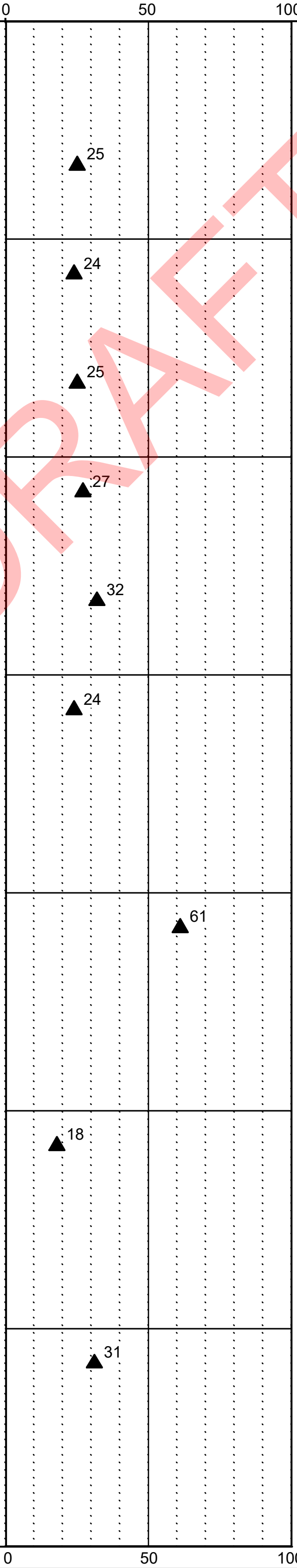
PBS PROJECT NUMBER:
73658.000

APPROX. BORING B-3 LOCATION:
(See Site Plan)
Lat: 47.757145 Long: -122.239240

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

Surface Conditions: Asphalt

▲ BLOW COUNT
◆ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%



DEPTH FEET	MATERIAL DESCRIPTION	ELEV DEPTH (35 ft. amsl)	TESTING	SAMPLE TYPE SAMPLE ID	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT (2 inches)	35.0 0.0				Base rock not encountered
	Medium dense, brown, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to coarse sand; fine to coarse, subrounded to subangular gravel; moist **ALLUVIUM (Qal)**	34.8 0.2		S-1	25	
5.0	Medium dense, brown, poorly graded SAND (SP-SM) with silt; non-plastic; fine to coarse sand; moist	30.0 5.0		S-2	24	
	Medium dense, brown, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to coarse sand; fine, subrounded to subangular gravel; moist	27.5 7.5		S-3	25	
10.0				S-4	27	
	becomes dense			S-5	32	
15.0	becomes medium dense			S-6	24	
20.0	becomes very dense			S-7	61	N-value likely overstated due to gravel ▼ 05/15/24
25.0	Very stiff, brown SILT (ML); low plasticity; wet **PRE-FRASER DEPOSITS (Qpf)**	10.0 25.0		S-8	18	
	Medium dense, poorly graded SAND (SP) with gravel; fine to coarse sand; fine, subrounded gravel; wet	9.0 26.0				
30.0	becomes dense, gray-brown; with fine to coarse, subrounded to subangular gravel			S-9	31	
35.0						

BORING LOG W/ ELEV 73658.000 B1-6 20240521.GPJ PBS_DATATMPL_GEO.GDT PRINT DATE: 6/18/24:RPG

DRILLING METHOD: Hollow-Stem Auger
DRILLED BY: Boretec 1, Inc.
LOGGED BY: V. Tavangar

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 60
LOGGING COMPLETED: 5/15/2024

FIGURE A3

PBS
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LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

BORING B-3
(continued)

PBS PROJECT NUMBER:
73658.000

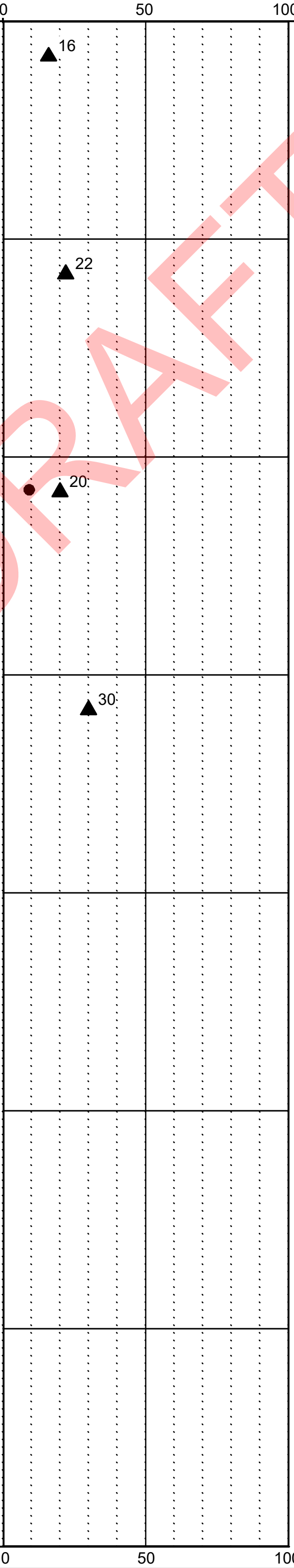
APPROX. BORING B-3 LOCATION:
(See Site Plan)
Lat: 47.757145 Long: -122.239240

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

▲ BLOW COUNT
✧ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD%
▨ CORE REC%

Surface Conditions: Asphalt

DEPTH FEET	MATERIAL DESCRIPTION	ELEV DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	BLOW COUNT	INSTALLATION AND COMMENTS
35.0	Medium dense, gray-brown, poorly graded SAND (SP) with gravel; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet **PRE-FRASER DEPOSITS (Qpf)**	0.0 35.0		S-10	16	
40.0				S-11	22	
45.0			P200 SIEV	S-12	20	P200 = 5%
50.0				S-13	30	
	Final depth 51.5 feet bgs; boring backfilled with bentonite.	-16.5 51.5				
55.0						
60.0						
65.0						
70.0						



DRILLING METHOD: Hollow-Stem Auger
DRILLED BY: Boretec 1, Inc.
LOGGED BY: V. Tavangar

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 60
LOGGING COMPLETED: 5/15/2024

PBS
AN APEX COMPANY

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

BORING B-4

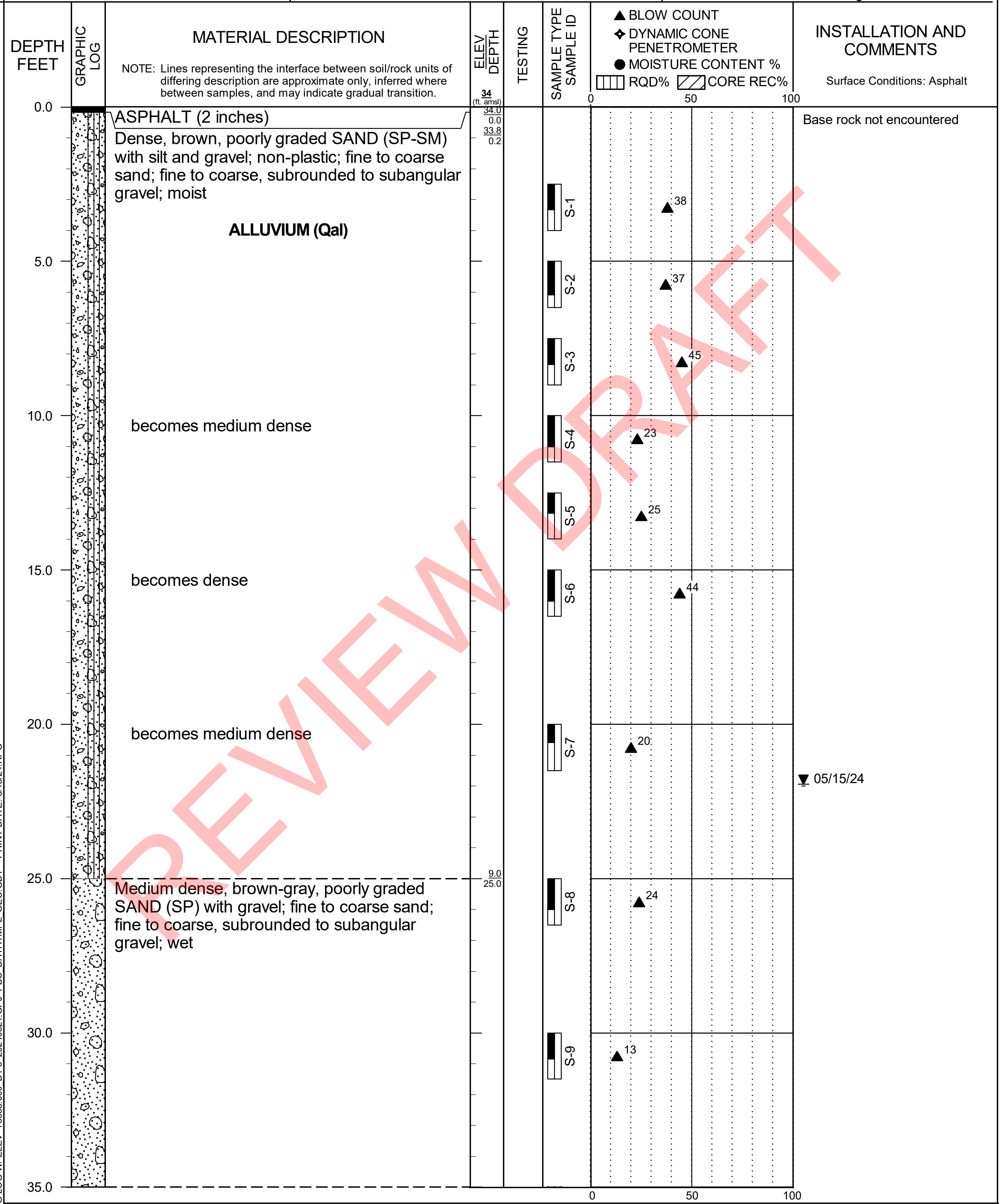
PBS PROJECT NUMBER:
73658.000

APPROX. BORING B-4 LOCATION:
(See Site Plan)
Lat: 47.757004 Long: -122.239894

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

▲ BLOW COUNT
✧ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

Surface Conditions: Asphalt



DEPTH FEET	MATERIAL DESCRIPTION	ELEV DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT (2 inches)	34 (ft amsl) 34.0 0.0				Base rock not encountered
	Dense, brown, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to coarse sand; fine to coarse, subrounded to subangular gravel; moist **ALLUVIUM (Qal)**	33.8 0.2		S-1	38	
5.0				S-2	37	
				S-3	45	
10.0	becomes medium dense			S-4	23	
				S-5	25	
15.0	becomes dense			S-6	44	
20.0	becomes medium dense			S-7	20	▼ 05/15/24
25.0	Medium dense, brown-gray, poorly graded SAND (SP) with gravel; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet	9.0 25.0		S-8	24	
30.0				S-9	13	
35.0						

DRILLING METHOD: Hollow-Stem Auger
DRILLED BY: Boretec 1, Inc.
LOGGED BY: V. Tavangar

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 60
LOGGING COMPLETED: 5/15/2024

FIGURE A4

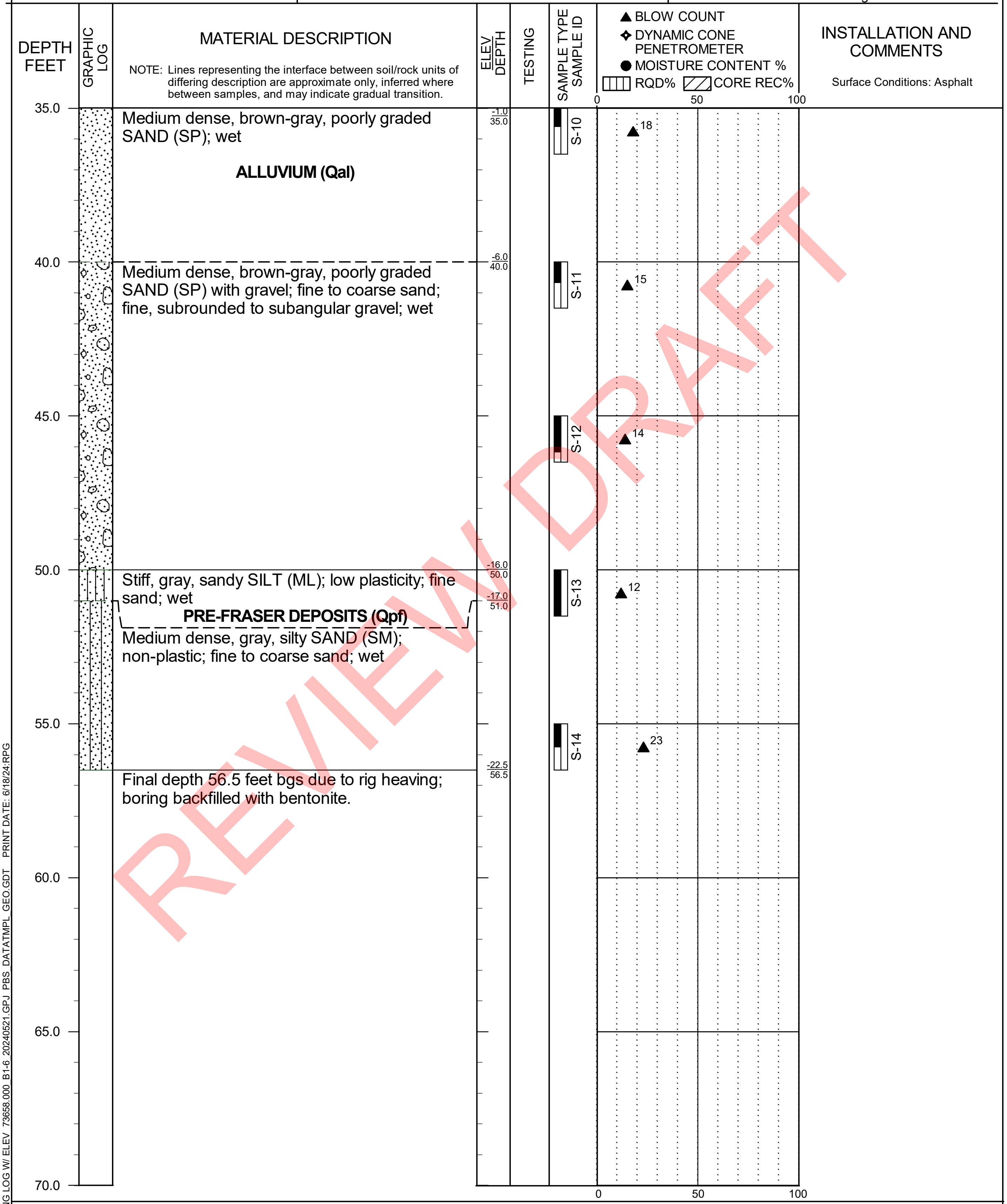
BORING LOG W/ ELEV 73658.000 B1-6 20240521.GPJ PBS DATATMPL GEO.GDT PRINT DATE: 6/18/24:RPG

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

PBS PROJECT NUMBER:
73658.000

BORING B-4
(continued)

APPROX. BORING B-4 LOCATION:
(See Site Plan)
Lat: 47.757004 Long: -122.239894



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

Surface Conditions: Asphalt

Depth (ft)	Material Description	Elev / Depth	Sample ID	Blow Count
35.0	Medium dense, brown-gray, poorly graded SAND (SP); wet **ALLUVIUM (Qal)**	-1.0 / 35.0	S-10	18
40.0	Medium dense, brown-gray, poorly graded SAND (SP) with gravel; fine to coarse sand; fine, subrounded to subangular gravel; wet	-6.0 / 40.0	S-11	15
45.0			S-12	14
50.0	Stiff, gray, sandy SILT (ML); low plasticity; fine sand; wet **PRE-FRASER DEPOSITS (Qpf)**	-16.0 / 50.0	S-13	12
51.0	Medium dense, gray, silty SAND (SM); non-plastic; fine to coarse sand; wet	-17.0 / 51.0		
55.0			S-14	23
56.5	Final depth 56.5 feet bgs due to rig heaving; boring backfilled with bentonite.	-22.5 / 56.5		

DRILLING METHOD: Hollow-Stem Auger
DRILLED BY: Boretec 1, Inc.
LOGGED BY: V. Tavangar

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 60
LOGGING COMPLETED: 5/15/2024

FIGURE A4

PBS
AN APEX COMPANY

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

BORING B-5

PBS PROJECT NUMBER:
73658.000

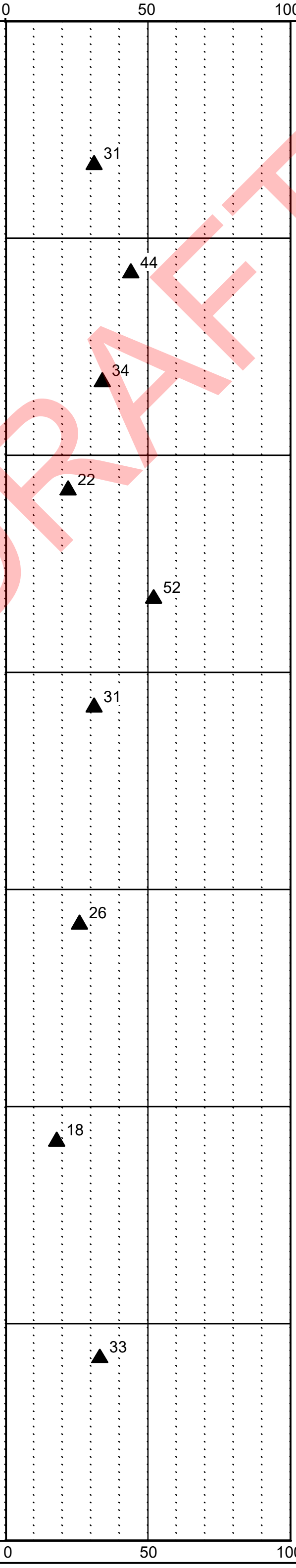
APPROX. BORING B-5 LOCATION:
(See Site Plan)
Lat: 47.756950 Long: -122.239238

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: ▲ BLOW COUNT; ✦ DYNAMIC CONE PENETROMETER; ● MOISTURE CONTENT %; ▥ RQD%; ▨ CORE REC%

Surface Conditions: Asphalt

DEPTH FEET	MATERIAL DESCRIPTION	ELEV / DEPTH (36 ft. amsl)	TESTING	SAMPLE TYPE / SAMPLE ID	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT (2 inches)	36.0 0.0				Base rock not encountered
	Dense, brown, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to coarse sand; fine to coarse, subrounded to subangular gravel; moist **ALLUVIUM (Qal)**	35.8 0.2		S-1	31	
5.0	Dense, gray-brown, poorly graded SAND (SP) with gravel; ; fine to coarse sand; fine to coarse, subrounded to subangular gravel; moist	31.0 5.0		S-2	44	
				S-3	34	
10.0	becomes medium dense			S-4	22	
	Very dense, gray-brown, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to coarse sand; fine to coarse, subrounded to subangular gravel; moist	23.5 12.5		S-5	52	
15.0	becomes dense			S-6	31	
20.0	Medium dense, brown, poorly graded GRAVEL (GP) with sand; fine to coarse sand; fine to coarse, subrounded to subangular gravel; moist	16.0 20.0		S-7	26	▼ 05/16/24
25.0	Medium dense, brown, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet	11.0 25.0		S-8	18	
30.0	becomes dense			S-9	33	
35.0						



DRILLING METHOD: Hollow-Stem Auger
DRILLED BY: Boretec 1, Inc.
LOGGED BY: V. Tavangar

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 60
LOGGING COMPLETED: 5/16/2024

FIGURE A5

PBS
AN APEX COMPANY

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

PBS PROJECT NUMBER:
73658.000

BORING B-5
(continued)

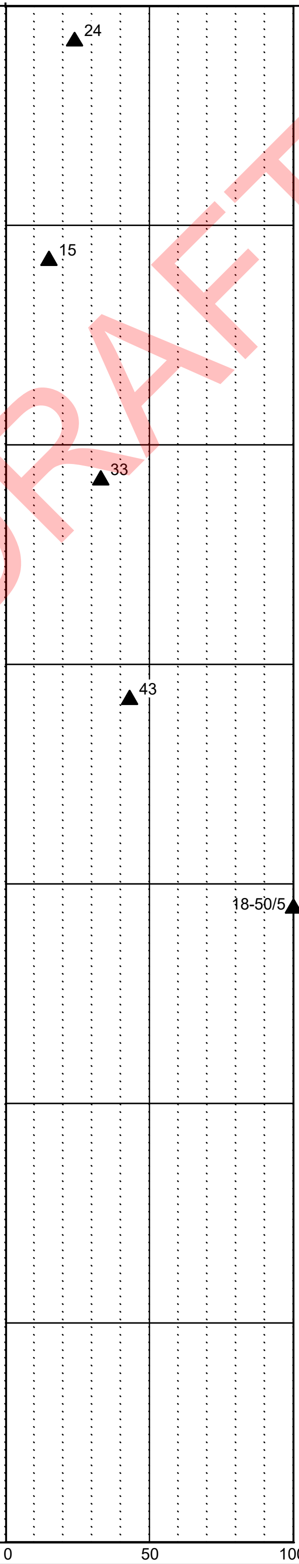
APPROX. BORING B-5 LOCATION:
(See Site Plan)
Lat: 47.756950 Long: -122.239238

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

▲ BLOW COUNT
✧ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

INSTALLATION AND COMMENTS
Surface Conditions: Asphalt

DEPTH FEET	MATERIAL DESCRIPTION	ELEV DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	BLOW COUNT	INSTALLATION AND COMMENTS
35.0	Medium dense, brown, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet **ALLUVIUM (Qal)**	1.0 35.0		S-10	24	
40.0				S-11	15	
45.0	Dense, gray, poorly graded SAND (SP) with gravel; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet **PRE-FRASER DEPOSITS (Qpf)**	-9.0 45.0		S-12	33	
50.0	Dense, gray, silty SAND (SM); non-plastic; fine to coarse sand; wet	-14.0 50.0		S-13	43	
	PRE-OLYMPIA GLACIAL DEPOSITS (Qpog) Dense, gray, silty SAND (SM) with gravel; non-plastic; fine to coarse sand; fine, subrounded gravel; wet	-15.0 51.0				
55.0	becomes very dense			S-14	18-50/5"	
	Final depth 56.0 feet bgs due to refusal in very dense sand and gravel; boring backfilled with bentonite.	-20.0 56.0				
60.0						
65.0						
70.0						



DRILLING METHOD: Hollow-Stem Auger
DRILLED BY: Boretec 1, Inc.
LOGGED BY: V. Tavangar

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 60
LOGGING COMPLETED: 5/16/2024

FIGURE A5

PBS
AN APEX COMPANY

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

BORING B-6

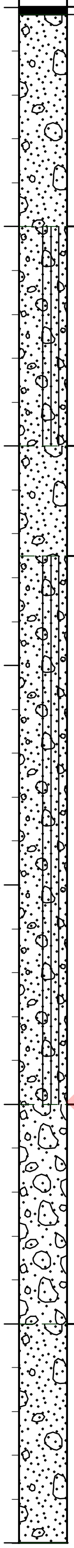
PBS PROJECT NUMBER:
73658.000

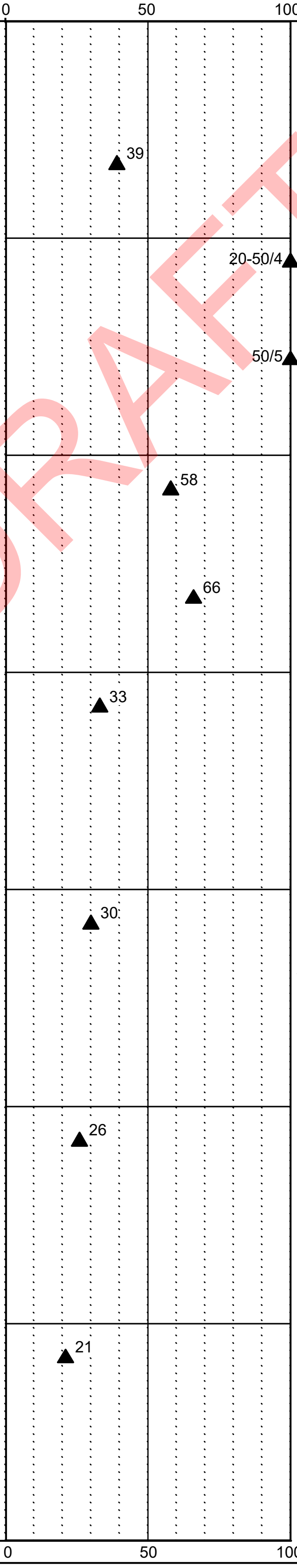
APPROX. BORING B-6 LOCATION:
(See Site Plan)
Lat: 47.756849 Long: -122.239173

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

▲ BLOW COUNT
✧ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

Surface Conditions: Asphalt





DEPTH FEET	MATERIAL DESCRIPTION	ELEV DEPTH 35 (ft. amsl)	TESTING	SAMPLE TYPE SAMPLE ID	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT (2 inches)	35.0 0.0				Base rock not encountered
	Dense, brown, poorly graded SAND (SP) with gravel; fine to medium sand; fine, subrounded to subangular gravel; moist **ALLUVIUM (Qal)**	34.8 0.2		S-1	39	
5.0	Very dense, light brown, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to medium sand; fine, subrounded to subangular gravel; moist	30.0 5.0		S-2 S-3	20-50/4 50/5	
10.0	Very dense, light brown, poorly graded SAND (SP) with gravel; fine to medium sand; fine, subrounded to subangular gravel; moist	25.0 10.0		S-4	58	
	Very dense, light brown, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to medium sand; fine, subrounded to subangular gravel; moist	22.5 12.5		S-5	66	
15.0	becomes dense, brown; with fine to coarse sand and increased gravel			S-6	33	
20.0	becomes medium dense			S-7	30	▼ 05/16/24
25.0	Medium dense, gray-brown, poorly graded GRAVEL (GP) with sand; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet	10.0 25.0		S-8	26	
30.0	Medium dense, gray, poorly graded SAND (SP) with gravel; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet	5.0 30.0		S-9	21	
35.0						

DRILLING METHOD: Hollow-Stem Auger
DRILLED BY: Boretec 1, Inc.
LOGGED BY: V. Tavangar

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 60
LOGGING COMPLETED: 5/16/2024

FIGURE A6

PBS AN APEX COMPANY

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

PBS PROJECT NUMBER:
73658.000

BORING B-6
(continued)

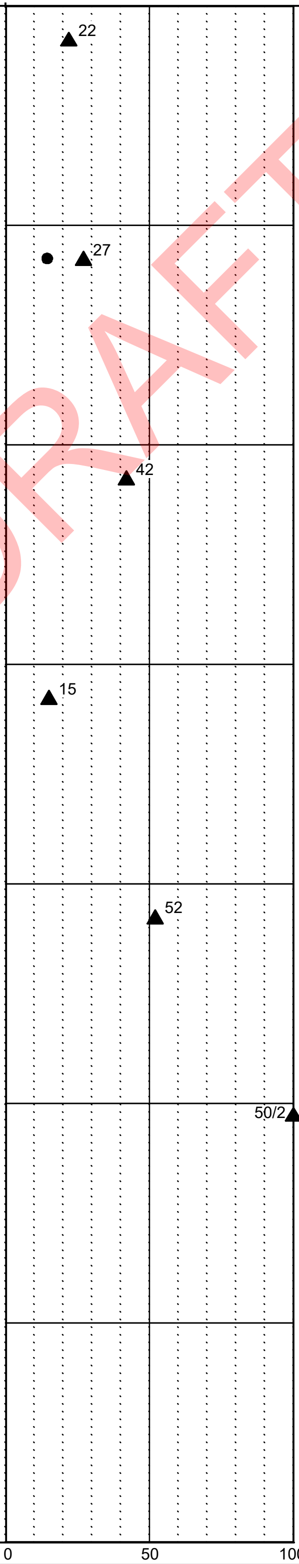
APPROX. BORING B-6 LOCATION:
(See Site Plan)
Lat: 47.756849 Long: -122.239173

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

▲ BLOW COUNT
◆ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

Surface Conditions: Asphalt

DEPTH FEET	MATERIAL DESCRIPTION	ELEV DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	BLOW COUNT	INSTALLATION AND COMMENTS
35.0	Medium dense, gray, poorly graded SAND (SP) with gravel; fine to coarse sand; fine to coarse, subrounded to subangular gravel; wet **ALLUVIUM (Qal)**	0.0 35.0		S-10	22	
40.0	Medium dense, gray, poorly graded SAND (SP-SM) with silt; non-plastic; fine to coarse sand; wet	-5.0 40.0	P200 SIEV	S-11	27	P200 = 8%
45.0	Dense, gray-brown, poorly graded SAND (SP); fine to coarse sand; wet **PRE-FRASER DEPOSITS (Qpf)**	-10.0 45.0		S-12	42	
50.0	Stiff, gray, sandy SILT (ML); low plasticity; fine sand; wet	-15.0 50.0		S-13	15	
51.0	Medium dense, gray, silty SAND (SM); non-plastic; fine sand; wet	-16.0 51.0				
55.0	Very dense, gray, silty SAND (SM); non-plastic; fine sand; wet **PRE-OLYMPIA GLACIAL DEPOSITS (Qpog)**	-20.0 55.0		S-14	52	
60.0				S-15	50/2"	
60.5	Final depth 60.5 feet bgs due to refusal in very dense sand; boring backfilled with bentonite.	-25.5 60.5				
65.0						
70.0						



DRILLING METHOD: Hollow-Stem Auger
DRILLED BY: Boretec 1, Inc.
LOGGED BY: V. Tavangar

BIT DIAMETER: 5 inches
HAMMER EFFICIENCY PERCENT: 60
LOGGING COMPLETED: 5/16/2024

Appendix B

Laboratory Testing

Appendix B: Laboratory Testing

B1 GENERAL

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

B2 CLASSIFICATION TESTS

B2.1 Visual Classification

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

B2.2 Moisture (Water) Contents

Natural moisture content determinations were made on samples of the fine-grained soils (that is, silts, clays, and silty sands). The natural moisture content is defined as the ratio of the weight of water to dry weight of soil, expressed as a percentage. The results of the moisture content determinations are presented on the exploration logs in Appendix A and on Figure B3, 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

Mechanical grain-size (sieve) analyses were performed on select samples to determine their particle size distribution. 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).

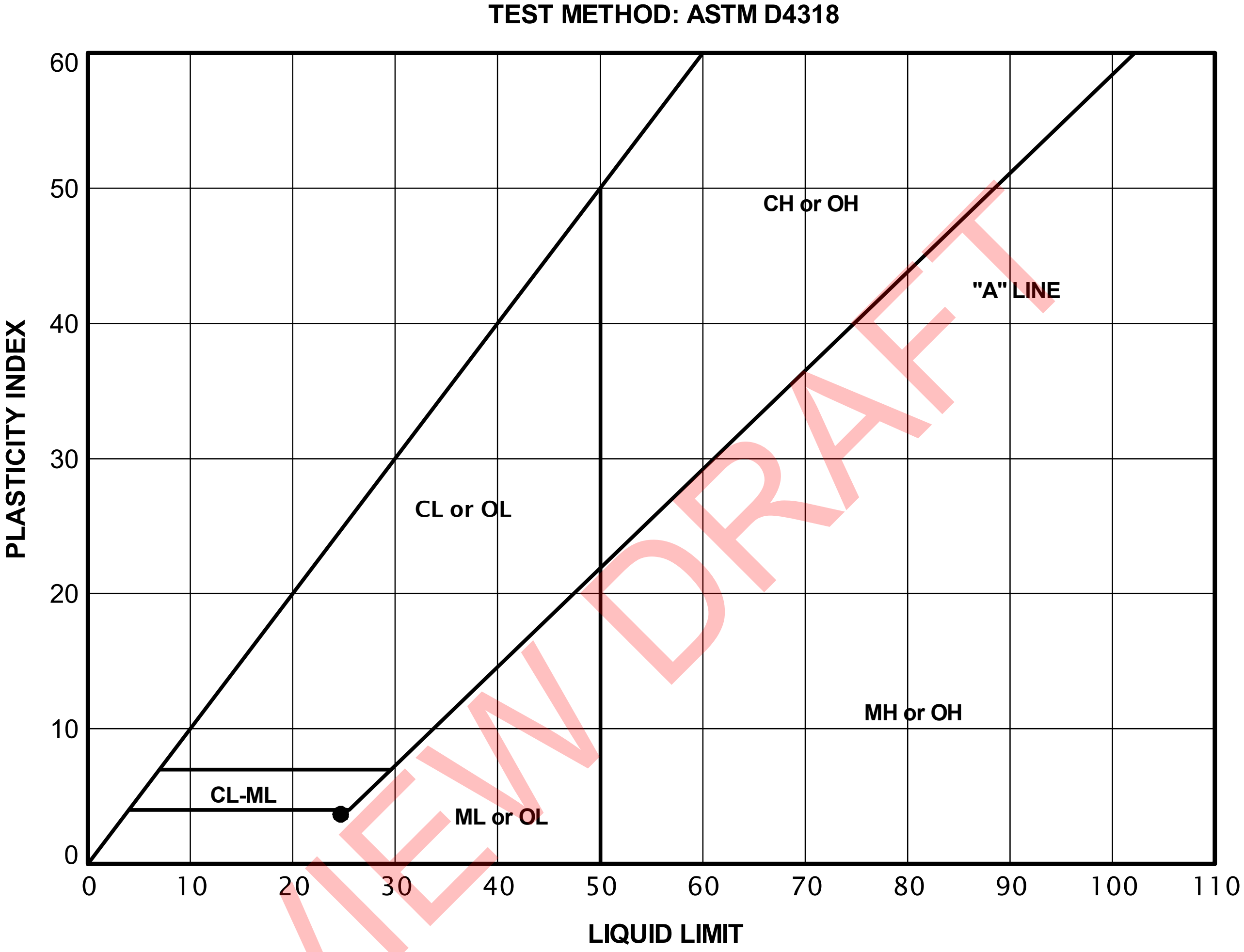
The results of the grain-size analyses are presented on exploration logs in Appendix A and on Figure B2, Particle-Size Analysis Test Results, and Figure B3, Summary of Laboratory Data, in Appendix B.

ATTERBERG LIMITS TEST RESULTS

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

PBS PROJECT NUMBER:
73658.000

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-10	35.0	24.0	NA	25	21	4

FIGURE B1

PBS
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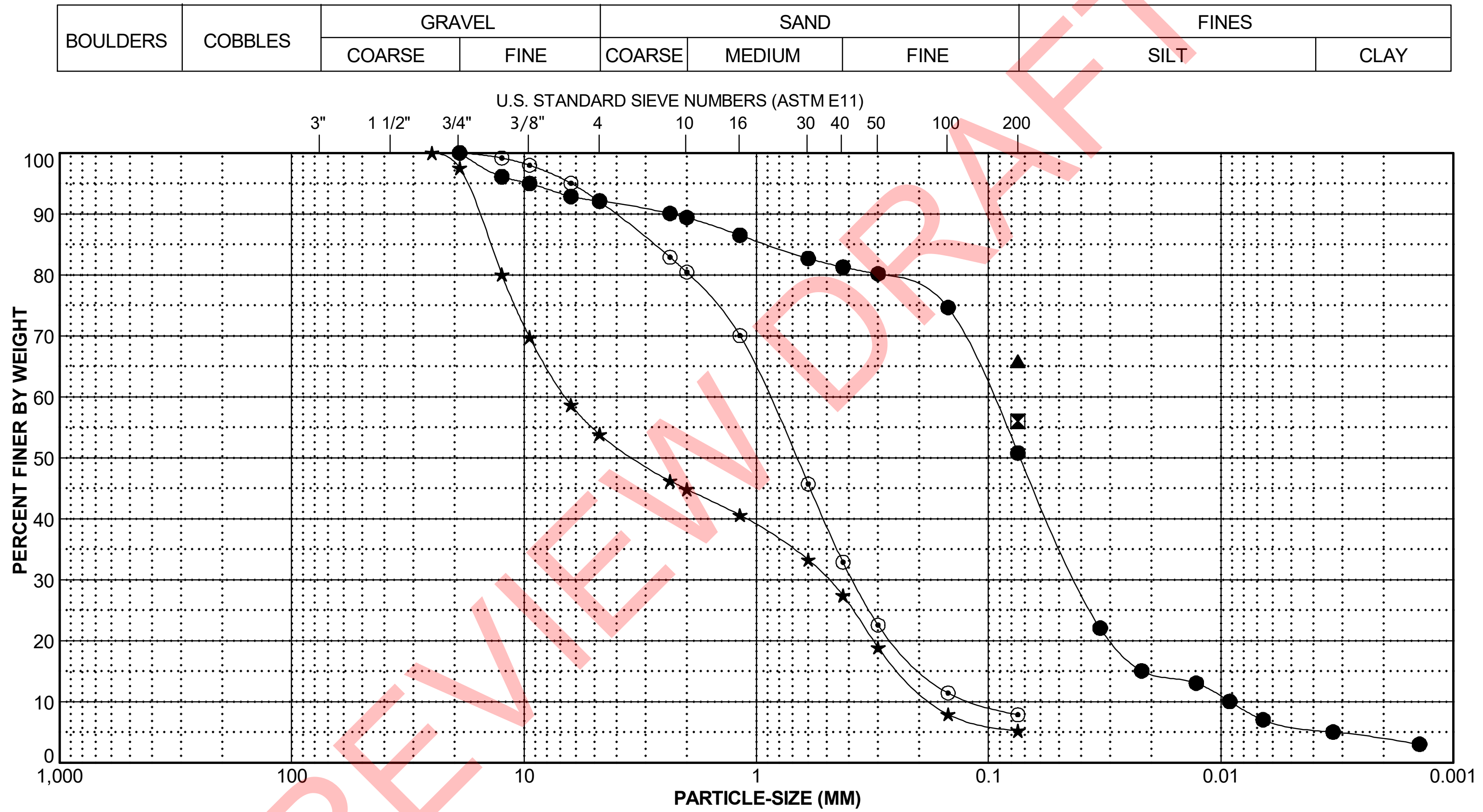
PARTICLE-SIZE ANALYSIS TEST RESULTS

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

PBS PROJECT NUMBER:
73658.000

TEST METHOD: ASTM C136/D422



KEY	EXPLORATION NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	D90 (MM)	D60 (MM)	D30 (MM)	D10 (MM)	D5 (MM)	GRAVEL (PERCENT)	SAND (PERCENT)	FINES (PERCENT)
●	B-1	S-9	30.0	20	2.3	0.10	0.0416	0.009	0.003	8	41	51
☒	B-2	S-14	55.0									56
▲	B-2	S-15	60.0									66
★	B-3	S-12	45.0	9	15.9	6.6	0.5	0.171		46	49	5
⊙	B-6	S-11	40.0	14	4.1	0.9	0.4	0.114		8	84	8

FIGURE B2

PBS
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SUMMARY OF LABORATORY DATA

LARUS SENIOR APARTMENTS
KENMORE, WASHINGTON

PBS PROJECT NUMBER:
73658.000

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-3	7.5	29.5	5.2							
B-1	S-6	15	22.0	5.4							
B-1	S-9	30	7.0	20.5		8	41	51			
B-1	S-10	35	2.0	24.0					25	21	4
B-1	S-12	45	-8.0	14.0							
B-1	S-14	55	-18.0	18.4							
B-2	S-14	55	-19.0	20.9				56			
B-2	S-15	60	-24.0	17.9				66			
B-3	S-12	45	-10.0	9.1		46	49	5			
B-6	S-11	40	-5.0	14.4		8	84	8			

FIGURE B3

Appendix C

Previous Subsurface Explorations by Others

VICINITY MAP

Not to Scale

N

Kenmore, WA

Project Number
1102819

Figure 1

Bozhko Residential
Development
Vicinity Map

NELSON GEOTECHNICAL
ASSOCIATES, INC.
NGA
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No.	Date	Revision	By	CK
1	5/16/19	Original	DPN	ABR

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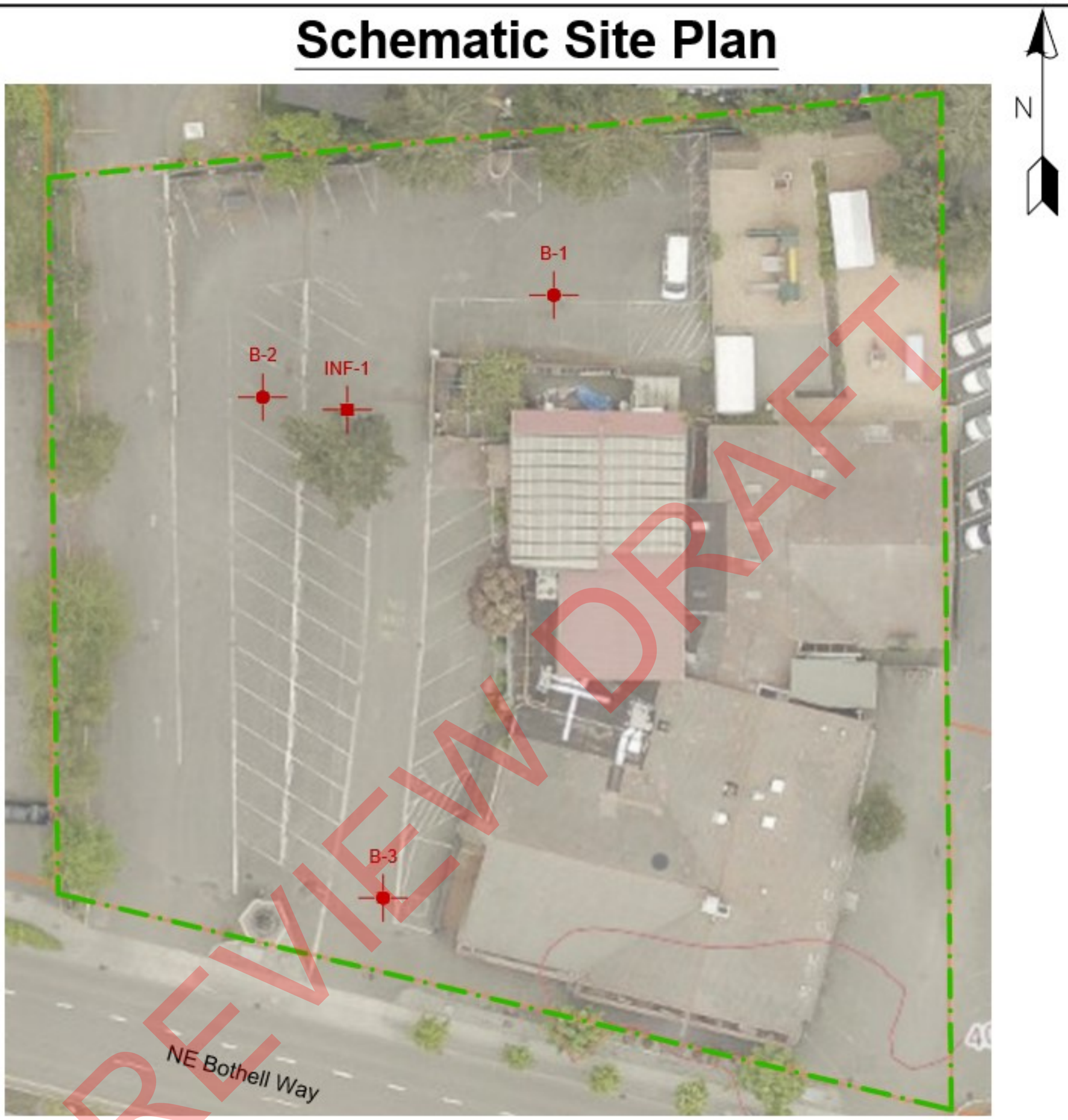
Schematic Site Plan



LEGEND

- - - Property line

B-1 Number and approximate location of boring

INF-1 Number and approximate location of infiltration test pit

0 40 80

Approximate Scale: 1 inch = 40 feet

Reference: Site Plan based on field measurements, observations, and aerial parcel map review.

Project Number
1102819

Figure 2

Bozhko Residential
Development
Schematic Site Plan

NELSON GEOTECHNICAL
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UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50 % RETAINED ON NO. 200 SIEVE	GRAVEL MORE THAN 50 % OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50 % OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50 % PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50 %	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50 % OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-93.

2) Soil classification using laboratory tests is based on ASTM D 2488-93.

3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS:

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water.

Wet - Visible free water or saturated, usually soil is obtained from below water table

Project Number
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Figure 3

Bozhko Residential
Development
Soil Classification Chart

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No.	Date	Revision	By	CK
1	5/16/19	Original	DPN	ABR

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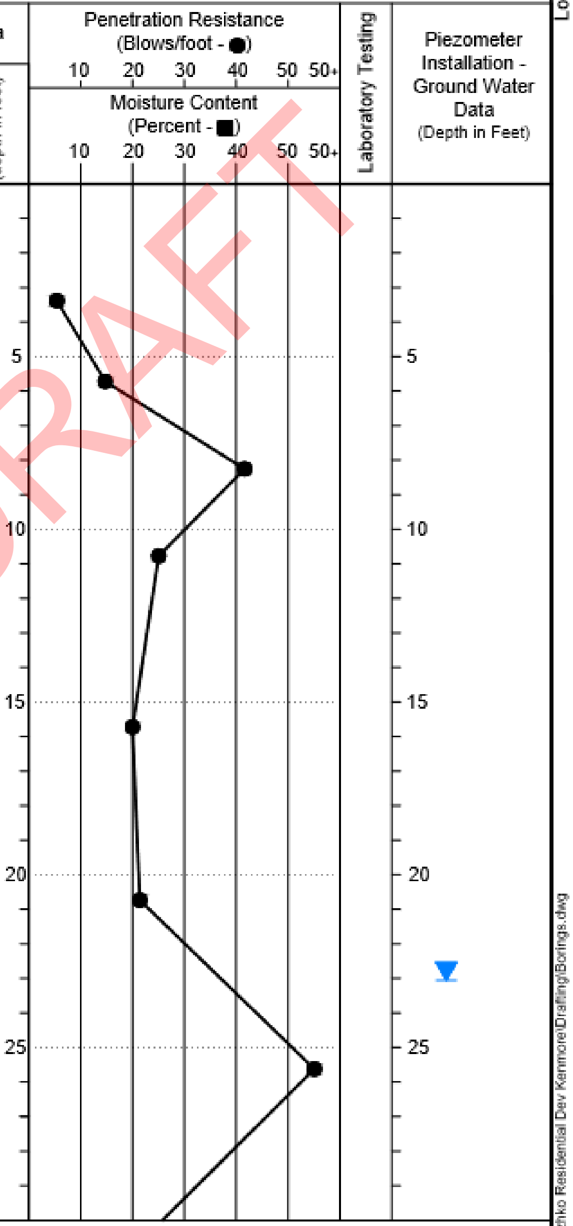
BORING LOG

B-1

Approximate Ground Surface Elevation: ??

Soil Profile			Sample Data	
Description	Graphic Log	Group Symbol	Blow Count	Sample Location (depth in feet)
2-inches asphalt Brown gravel with fine to coarse sand, silt, and trace organics (loose, moist) (**FILL**)			6	
-becomes light brown, medium dense			14	
Gray-brown, gravelly fine to coarse sand with silt (dense, moist)		SP-SM	41	
Light brown, silty fine to coarse sand with gravel and trace iron-oxide staining (medium dense, moist)		SM	25	
Gray-brown, fine to medium sand with silt and trace iron-oxide staining (medium dense, moist)		SP-SM	20	
Light brown, silty fine to coarse sand with gravel (medium dense, moist)		SM	21	
Light brown to gray-brown, fine to coarse sand with silt, gravel, and iron-oxide staining (very dense, moist)		SP-SM	50-6	



LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler

🟩	Solid PVC Pipe
🟩	Slotted PVC Pipe
🟥	Monument/ Cap to Piezometer
★	Liquid Limit
+	Plastic Limit
▨	Concrete
▨	Bentonite
▨	Native Soil
▨	Silica Sand
▼	Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

NOTE: Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Project Number
1102819
Figure 4
Page 1 of 2

Bozhko Residential
Development
Boring Log

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(509) 665-7696 / Fax: 665-7692

No.	Date	Revision	By	CK
1	5/16/19	Original	DPN	ABR

Logged by: ABR on 5/10/19

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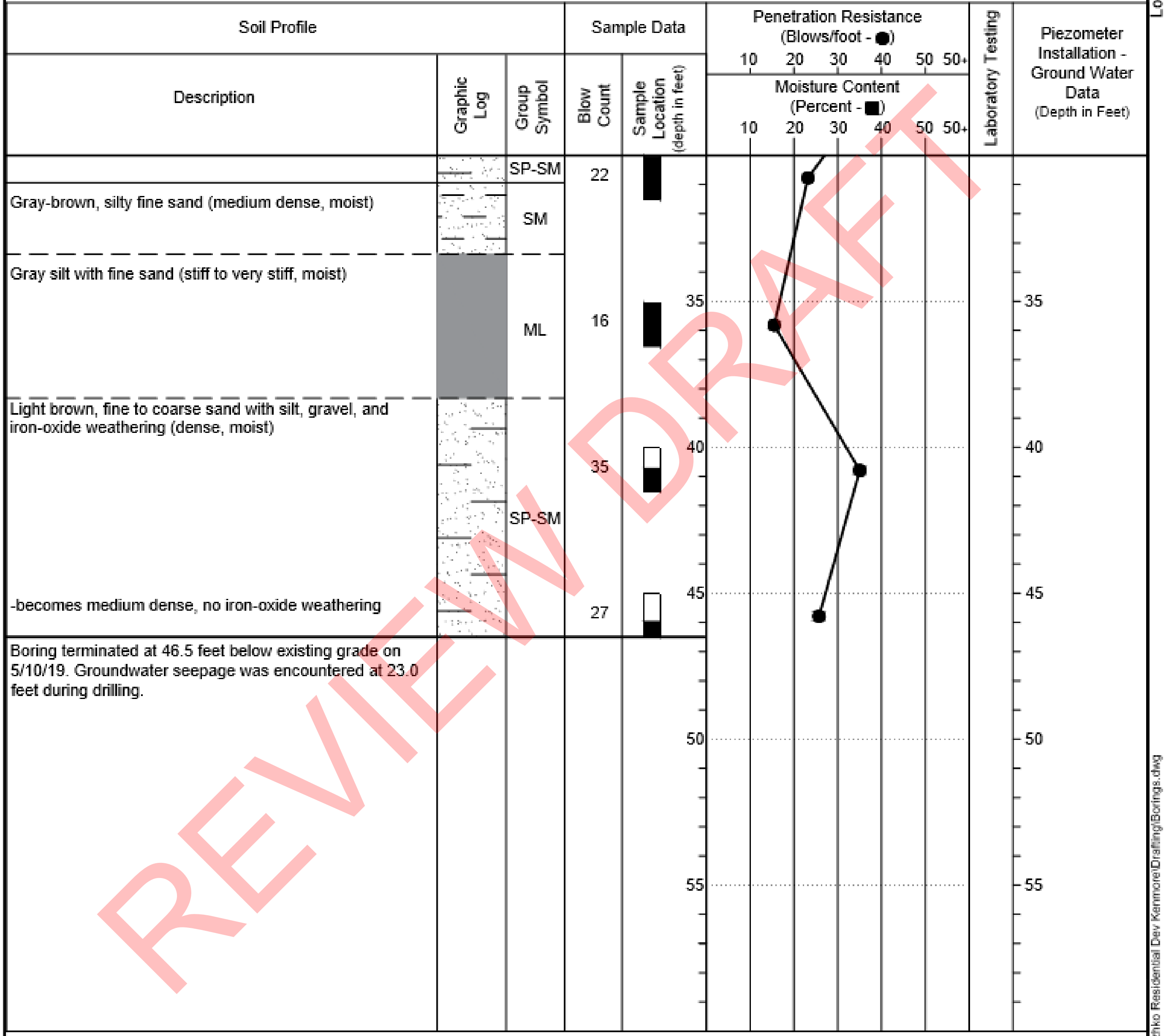
BORING LOG

B-1 (cont.)



Description	Group Symbol	Blow Count
	SP-SM	22
Gray-brown, silty fine sand (medium dense, moist)	SM	
Gray silt with fine sand (stiff to very stiff, moist)	ML	16
Light brown, fine to coarse sand with silt, gravel, and iron-oxide weathering (dense, moist)	SP-SM	35
-becomes medium dense, no iron-oxide weathering		27

Boring terminated at 46.5 feet below existing grade on 5/10/19. Groundwater seepage was encountered at 23.0 feet during drilling.

LEGEND

◧	Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
■	Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
☐	Solid PVC Pipe
⊟	Slotted PVC Pipe
🟥	Monument/ Cap to Piezometer
*	Liquid Limit
+	Plastic Limit
▨	Concrete
▨	Bentonite
▨	Native Soil
▨	Silica Sand
▼	Water Level
M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

NOTE: Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Project Number
1102819
Figure 4
Page 2 of 2

Bozhko Residential Development Boring Log

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East Wenatchee, WA 98802
(509) 665-7696 / Fax: 665-7692

No.	Date	Revision	By	CK
1	5/16/19	Original	DPN	ABR

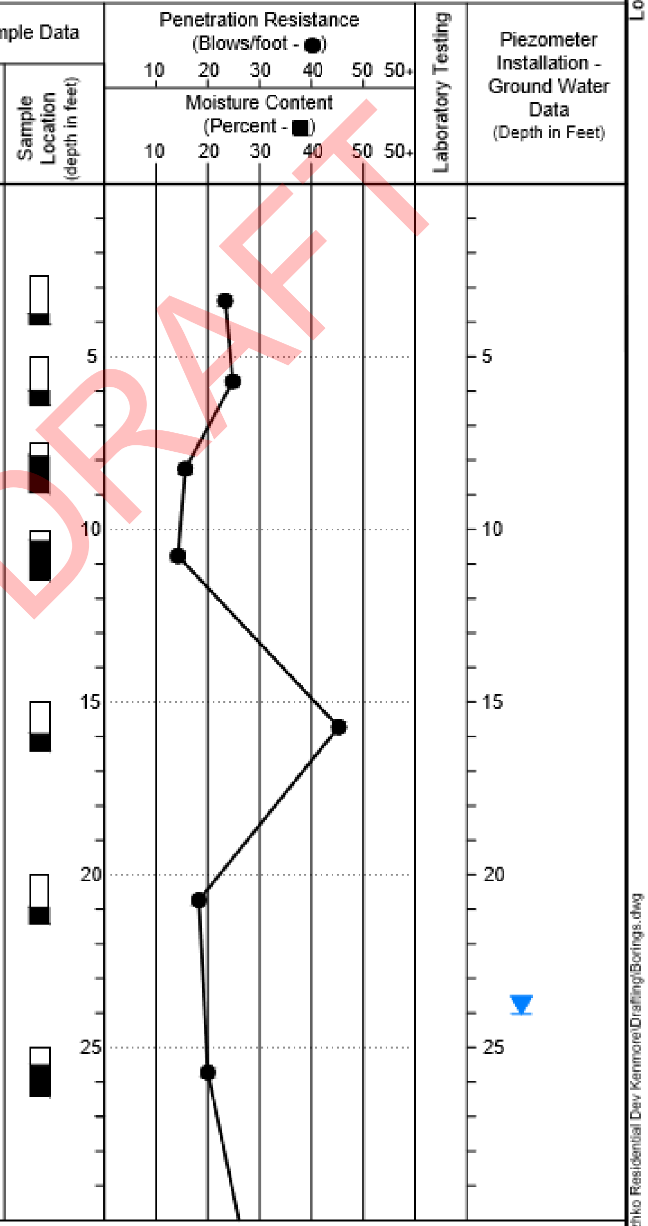
BORING LOG

B-2

Approximate Ground Surface Elevation: ??

Logged by: ABR on 5/10/19

Soil Profile: Description	Group Symbol	Blow Count
3-inches asphalt Brown gravel with fine to medium sand, silt, and trace organics (medium dense, moist) (<u>FILL</u>)		22 24
Light brown to gray, fine to medium sand with silt and trace iron-oxide staining (medium dense, moist)	SP-SM	17
Gray to gray-brown, silty fine sand with trace iron-oxide staining (medium dense, moist)	SM	13
Light brown, silty gravel with fine to coarse sand (dense, moist) -becomes brown, medium dense, with trace iron-oxide staining	GM	46 19
Light brown, fine to coarse sand with silt and gravel (medium dense, moist)	SP-SM	20
Gray-brown silt with fine sand and trace iron-oxide staining (very stiff, moist)	ML	
Gray-brown, fine to coarse sand with silt and gravel (medium dense, moist)	SP-SM	



LEGEND

- ▯ Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- ■ Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler

☐	Solid PVC Pipe
⊟	Slotted PVC Pipe
🟥	Monument/ Cap to Piezometer
*	Liquid Limit
+	Plastic Limit
▨	Concrete
▨	Bentonite
▨	Native Soil
▨	Silica Sand
▼	Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

NOTE: Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Project Number
1102819
Figure 5
Page 1 of 2

Bozhko Residential
Development
Boring Log

NGA
NELSON GEOTECHNICAL
ASSOCIATES, INC.
GEOTECHNICAL ENGINEERS & GEOLOGISTS

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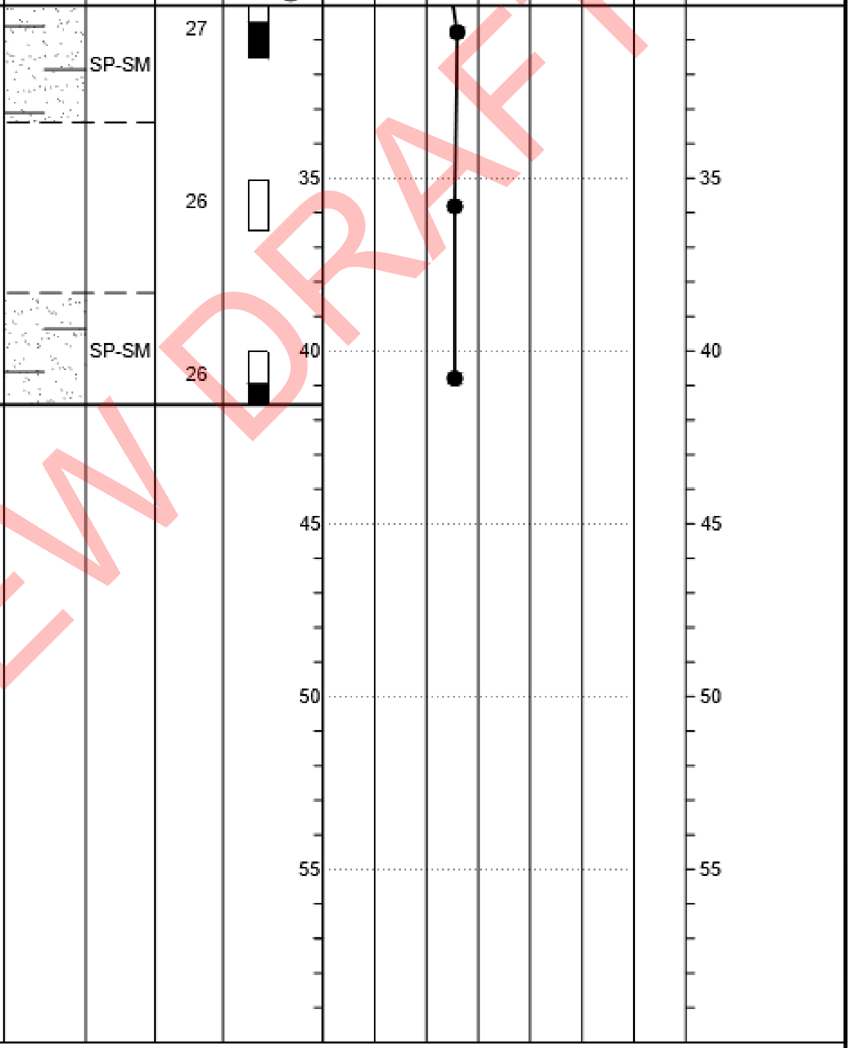
BORING LOG

B-2 (cont.)

Logged by: ABR on 5/10/19

Soil Profile: Description	Group Symbol	Blow Count
	SP-SM	27
-no recovery		26
Brown to gray-brown, gravelly fine to coarse sand with silt (medium dense, moist)	SP-SM	26
Boring terminated at 41.5 feet below existing grade on 5/10/19. Groundwater seepage was encountered at 24.0 feet during drilling.		



LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- 🟩 Solid PVC Pipe
- 🟩 Slotted PVC Pipe
- 🟥 Monument/ Cap to Piezometer
- ★ Liquid Limit
- \+ Plastic Limit
- ▨ Concrete
- ▨ Bentonite
- ▨ Native Soil
- ▨ Silica Sand
- ▼ Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
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Project Number
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Bozhko Residential Development Boring Log

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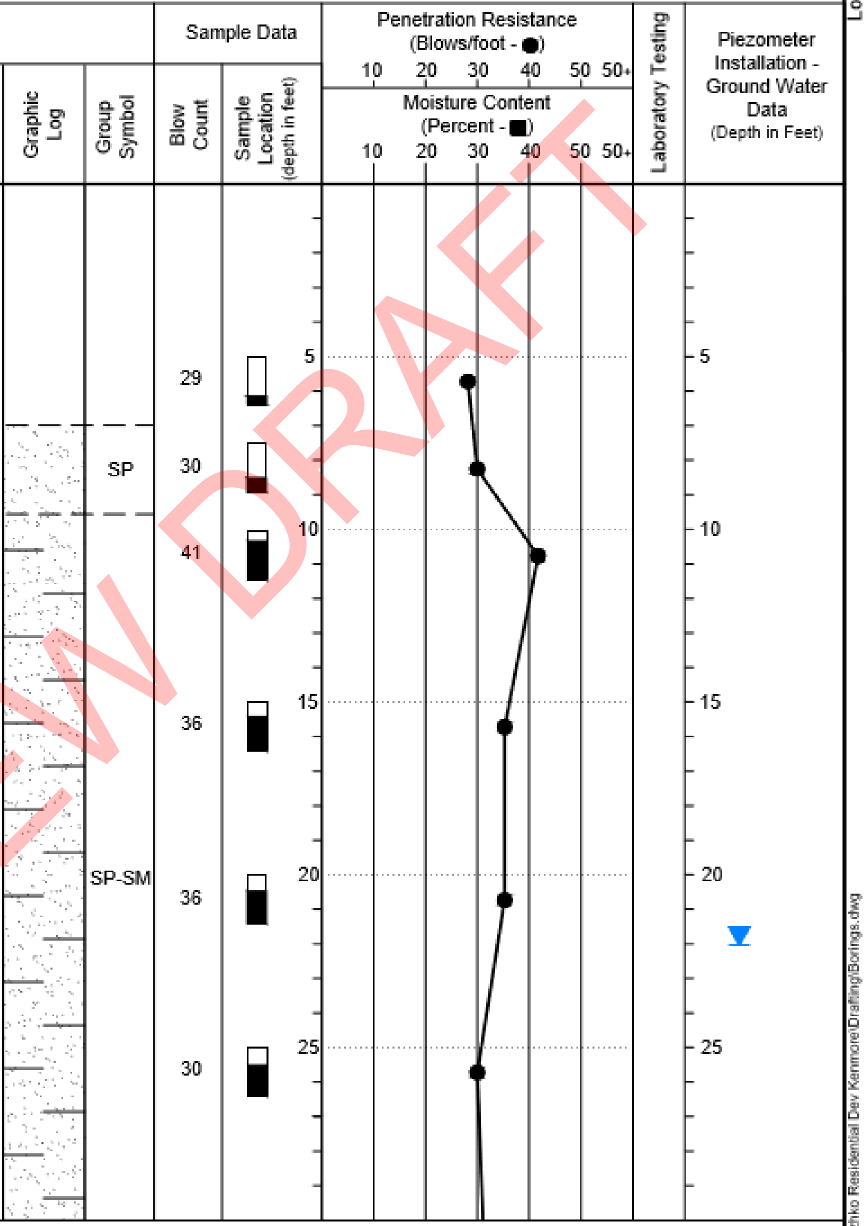
BORING LOG

B-3

Approximate Ground Surface Elevation: ??

Description	Group Symbol	Blow Count
2-inches asphalt Brown gravel with fine to coarse sand, silt, and trace organics (medium dense, moist) (FILL)		29
Light brown, gravelly fine to coarse sand with iron-oxide staining (medium dense to dense, moist)	SP	30
Brown to gray-brown, gravelly fine to coarse sand with silt and iron-oxide staining (dense, moist)		41
-becomes gray-brown, no iron oxide staining		36
	SP-SM	36
-becomes light brown, medium dense to dense		30



Logged by: ABR on 5/10/19

LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler

🟩	Solid PVC Pipe
🟩	Slotted PVC Pipe
🟥	Monument/ Cap to Piezometer
*	Liquid Limit
+	Plastic Limit
▨	Concrete
▨	Bentonite
▨	Native Soil
▨	Silica Sand
▼	Water Level
M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
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Project Number
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Bozhko Residential
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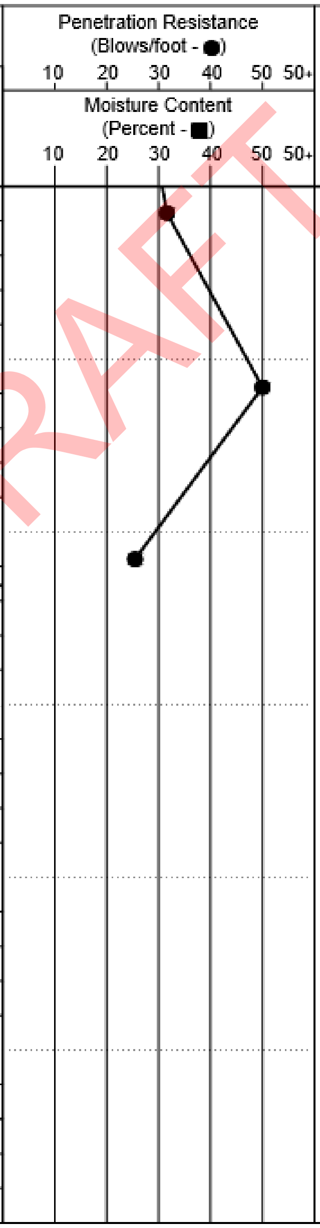
BORING LOG

B-3 (cont.)

Logged by: ABR on 5/10/19

Soil Profile: Description	Group Symbol	Blow Count	Sample Location (depth in feet)	Piezometer Installation - Ground Water Data (Depth in Feet)
-becomes gray-brown, dense	SP-SM	31	▯	
-becomes gray, fine to medium sand with silt and gravel, dense to very dense		50	■	35
Gray, silty fine to medium sand with gravel (medium dense, moist)	SM	26	■	40
Boring terminated at 41.5 feet below existing grade on 5/10/19. Groundwater seepage was encountered at 22.0 feet during drilling.				45 50 55



LEGEND

- ▯ Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- ■ Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- ☐ Solid PVC Pipe
- ⊟ Slotted PVC Pipe
- 🟥 Monument/ Cap to Piezometer
- * Liquid Limit
- \+ Plastic Limit
- ▨ Concrete
- ▨ Bentonite
- ▨ Native Soil
- ▨ Silica Sand
- ▼ Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
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Figure 6
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