Geotechnical Engineering Report

Harmony at Arlington
7417 204th Street Northeast
Arlington, Washington

Prepared for:
Harmony at Arlington, LLC
2033 E Lake Sammamish Place SE
Sammamish, Washington 98075

May 21, 2024
PBS Project 73672.000

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Executive Summary

PBS Engineering and Environmental LLC (PBS) has prepared a Geotechnical Engineering Report (GER) for the proposed Harmony at Arlington project located at 7417 204th Street Northeast in Arlington, Washington. This Executive Summary is provided for convenience only and should only be used in conjunction with the full GER (text and attachments) for the project in design. PBS is not responsible for utilization of less than the complete report.

Topic	Summary Statement	Report Section
Project Understanding	The two parcels will be developed with approximately 60 three-story, three- to four-bedroom, wood-frame townhomes placed on fill a few feet above existing grades. The following loads were provided by the structural engineer: • Max wall loads 2 to 4 kips per linear foot • Max slab loads 250 psf	1.3
Subsurface Conditions	Soil profile includes: • Existing gravel fill to depths of 0.5 to 5 feet bgs • Medium dense to dense sand with silt and cobbles and silty gravel to depths of 7.5 to 25 feet bgs • Dense, well-graded sand with silt/cobbles to depths of 25 feet bgs • Liquefiable, medium dense, poorly graded sand with silt below 25 feet bgs and extending to the depths explored (31.5 to 71.5 feet bgs) Groundwater is present at depths of 9 to 17 feet bgs.	2.4
Foundation Alternatives	Spread footing foundations over ground improvement: • Ground improvement (such as stone columns) is required • Could be designed for an allowable bearing pressure of about 3,000 psf Mat slab foundations on non-liquefiable crust: • Average allowable bearing pressure 1,000 psf • Modulus of subgrade reaction (k) 100 pounds per cubic inch (pci) • 1.5-foot-thick crush rock pads are required Stiffened conventional foundations over reinforced fill: • Min 2.5-foot-thick reinforced structural fill section with geogrid • Grade beams or additional reinforcement to stiffen the footings/slabs • Allowable bearing pressure 2,000 psf • Modulus of subgrade reaction (k) 150 pounds per cubic inch (pci) Perimeter foundation drains are recommended.	4.3
Soil Improvement	Due to the potential for liquefaction at the site, soil improvement is recommended as one alternative for the project.	4.3.1

Topic	Summary Statement	Report Section
Site Preparation	• On-site soils are generally suitable for placement as structural fill. • Existing fill may be reused if it meets the requirements for structural fill.	5.1
Limitations	This section contains important information about the limitations of this geotechnical engineering report.	7

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 7417 204th Street Northeast in Arlington, Washington (site). The general site location is shown on the Vicinity Map, Figure 1. The locations of PBS' explorations in relation to existing site features are shown on the Site Plan, Figure 2.

PBS previously completed geotechnical engineering services for a different project (Reserve at Arlington) at the project site for Reserve at Arlington Partners LLLP, owned by AVS Community Development, LLC (AVS), and presented the results in a geotechnical engineering report (GER) dated July 8, 2019 (PBS, 2019).

1.2 Purpose and Scope

The purpose of PBS' services for the Harmony at Arlington project 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 geologic and hazard mapping of the site area and previous nearby explorations 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

For the previous Reserve at Arlington project, subsurface conditions were explored by completing four borings and two test pits with pilot infiltration testing. The borings were advanced to depths of 31.5 to 71.5 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 A4 in Appendix A, Field Explorations.

The two test pits were excavated within the proposed development footprint to depths of 10 feet bgs. The test pits were logged and representative soil samples collected by a member of the PBS geotechnical engineering staff. Interpreted test pit logs are included as Figure A5 in Appendix A, Field Explorations.

Two piezometers were constructed within borings B-2 and B-4 to monitor the depth of groundwater and seasonal fluctuations; however, only B-4 appears to still exist on the site.

No additional subsurface explorations were completed specifically for the Harmony at Arlington project.

1.2.3 Field Infiltration Testing

Two small-scale pilot infiltration tests (PIT) were completed in test pits TP-1 and TP-2 within the property development at a depth of 5 feet bgs. Infiltration testing was monitored by PBS geotechnical engineering staff.

1.2.4 Soils Testing

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

1.2.5 Geotechnical Engineering Analysis

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

1.2.6 Report Preparation

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

- Field exploration logs and site plan showing approximate exploration locations
- Laboratory test results
- Infiltration test results
- Groundwater levels and considerations
- Liquefaction potential
- Shallow foundation (conventional and mat slabs) recommendations:
    - Ground improvement
    - Minimum embedment
    - Allowable bearing pressure
    - Estimated settlement
    - Sliding coefficient
- 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
    - Utility trench excavation and backfill requirements
    - Slab and pavement subgrade preparation
    - Wet weather considerations
    - Temporary and permanent slope inclinations
- Seismic design criteria in accordance with the 2018 International Building Code (IBC) with state of Washington amendments
- Slab-on-grade design recommendations
- Asphalt concrete (AC) pavement section recommendations

1.3 Project Understanding

PBS understands the client intends to develop two parcels (31051100304000 and 31051100400700) on the northwest corner of Highway 9 and 204th Street NE. Development plans are currently in the conceptual stages; however, the proposed development will include approximately 60 three-story, three- to four-bedroom, wood-frame townhomes with paved parking and drive lanes, utilities, and stormwater facilities.

Based on conversations with the structural engineer, we understand loads for the three-story buildings will be about 2 to 4 kips per linear foot for walls and less than 250 pounds per square foot (psf) for floors. The extent of site grading is currently unknown; however, based on preliminary conversations, fills on the order of 5 feet may be required.

2 SITE CONDITIONS

2.1 Surface Description

The site is currently an undeveloped rectangular parcel of land bordered to the east by Washington Highway 9, to the south by a commercial structure and paved parking area, to the west by open parcels with several outbuilding structures or sheds, and to the north by a commercial property and Portage Creek. Based on available topographic data, the site is generally flat, with the majority of ground surface elevations ranging between 120 to 122 feet above mean sea level (amsl) (NAVD88; WADNR, 2019a). The eastern portion of the property gently rises to 124 feet amsl as the site slopes to meet Highway 9. Outside of the site, the ground surface is relatively flat in all directions except for the shallow drainage of Portage Creek immediately north of the site.

2.2 Regional Geologic Setting

The site is located within the northern portion of the 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. Numerous faults are located throughout the Puget Lowland, including prominent east- and west-northwest-trending faults of the Darrington-Devils Mountain fault zone approximately 17 km north of the site (USGS fault no. 574) and the Utsalady Point Island fault zone approximately 24 km to the northwest of the site (USGS fault no. 573; USGS, 2019).

The Arlington and greater Seattle area 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.

2.3 Local Geology

Pleistocene glacial scouring, deposition, and fluvial reworking of material dominate the topography around the site. Review of lidar topographic data indicate the hills east and west of the site have expansive northwest-trending drumlin complexes parallel to Pleistocene glacial advancement (WADNR, 2019a). North of the site, numerous river terraces, meander belts, and oxbows can be identified as the modern-day Stillaguamish River incises into and reworks glacial outwash sediments.

Review of published geologic mapping of the Arlington West 7.5-minute quadrangle indicates the site is mapped within Quaternary glacial outwash of the Vashon Drift Marysville Sand Member (Minard, 1985). This unconsolidated, stratified, massive outwash sand was deposited during the early Quaternary by meltwaters flowing southward from the retreat of the Vashon Glacier near the end of the Frasier Glaciation. Thicknesses of this unit are believed to be between 60 to 120 feet within the mapped area.

2.4 Subsurface Conditions

The site was previously explored by drilling four borings, designated B-1 through B-4, to depths of 31.5 to 71.5 feet bgs. The drilling was performed by Holt Services, Inc., of Seattle, Washington, using a truck-mounted CME-85 drill rig and mud rotary drilling techniques. Two test pits, designated TP-1 and TP-2, were also excavated to a depth of 10 feet bgs for the purpose of infiltration testing. Test pit excavation was performed by Taylor's Excavations, Inc., of Seattle, Washington, using a track-mounted Komatsu 170LC excavator and a 36-inch toothed bucket.

PBS has summarized the subsurface units as follows:

GRAVEL FILL TO CLAYEY SAND FILL (GW FILL/SC FILL):	Variable fill consisting of well-graded gravel fill to clayey sand with gravel was encountered at the ground surface to 5 feet bgs in boring B-2 and to 6 inches bgs in boring B-3. The material in B-2 was dense, gray to brown, moist, fine- to coarse-grained sand, with medium plasticity fines and subrounded gravels.
SAND WITH SILT TO SILTY GRAVEL (SP-SM/GM):	Sand with silt and cobbles to silty gravel was encountered in all borings and test pits: from the surface to 7.5 feet bgs in boring B-1, from 5 to 15 feet bgs in boring B-2, between the surface and 25 feet bgs in boring B-3, from the surface to 10 feet bgs in boring B-4, and from the surface to approximately 4 feet bgs in test pits TP-1 and TP-2. The material was medium dense to dense, brown to gray, moist to wet, contained fine- to medium-grained sand, and non-plastic fines.
WELL-GRADED SAND WITH SILT AND COBBLES (SW):	Well-graded sand with cobbles was encountered in TP-1 and TP-2 from approximately 4 feet bgs to a termination depth of 10 feet bgs and well-graded sand with silt was encountered in boring B-1 from 15 to 25 feet bgs. The material was dense, light brown to brown, moist, with fine- to coarse-grained sand, rounded cobbles, and non-plastic fines. Percent passing the No. 200 sieve for samples taken at 5 feet bgs in TP-1 and TP-2 were 1% fines.
POORLY GRADED SAND WITH SILT (SP–SM):	Poorly graded sand with silt was encountered in all borings: below 25 feet bgs in borings B-1 and B-3, below 15 feet bgs in boring B-2, and below 10 feet bgs in boring B-3, to termination depths of 31.5 feet bgs in borings B-1, B-2, and B-4, and to 71.5 feet bgs in boring B-3. The material was medium dense, light to dark brown with areas of olive brown, moist to wet, with fine- to medium-grained sand and non-plastic fines. Percent fines passing the No. 200 sieve for this unit were between 7 to 11%.

2.5 Groundwater

The depth of groundwater was not directly measured during our explorations. Piezometers were installed in borings B-2 and B-4 to obtain depths to groundwater during different times of the year. Measured groundwater depths are included in Table 1. The piezometer installed in B-2 was not accessible or did not exist during the second round of measurements, completed in 2024.

Table 1. Measured Groundwater Depths

Date	Depth to Groundwater (ft bgs)	
	B-2	B-4
March 1, 2019	16.9	17.2
March 26, 2024	N/A	9.0

Please note that groundwater levels can fluctuate during the year depending on climate, irrigation season, extended periods of precipitation, drought, and other factors.

2.6 Infiltration Testing

PBS previously completed two infiltration tests in test pits TP-1 and TP-2 at a depth of approximately 5 feet bgs using the small pilot infiltration test (PIT) method procedures of the 2019 Stormwater Management Manual for Western Washington (SWMMWW). Following the saturation period, water was added to the excavation to achieve a head (water height) of 1 foot. The flow rate was adjusted to maintain a relatively constant head in the test pit (static head). The flow rate and volume of water used were recorded; the water was then turned off and allowed to drain while the water level was recorded at regular time intervals. Results of our field infiltration testing are presented in Table 2.

Table 2. Infiltration Test Results

Test Location	Depth of Infiltration Test (ft bgs)	Steady State Infiltration Rate (in/hr)	Falling Head Infiltration Rate (in/hr)	Recommended Infiltration Rate[1] (in/hr)	Soil Classification
TP-1	5	81	39	12	Well-graded SAND (SW) with cobbles
TP-2	5	52	34	10.5	Well-graded SAND (SW) with cobbles

[1] Based on a total correction factor CF_T = 0.315; CF_v = 0.7; CF_t = 0.5; CF_m = 0.9

The infiltration rates listed in Table 2 are not permeabilities/hydraulic conductivities, but field-measured rates. The recommended rates include correction factors related to long-term infiltration rates. In accordance with the SWMMWW, the infiltration rate obtained in the field must be reduced using correction factors that are appropriate for the design situation and the appropriate correction factors should account for the planned level of pre-treatment, maintenance, vegetation, siltation, etc. These factors account for site variability, number of tests conducted, uncertainty of the test method, and the potential for long-term clogging.

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

3 SEISMIC HAZARD

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. 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 Snohomish County Liquefaction Susceptibility Map (Palmer et al., 2004), the site is shown as mapped in a zone of low to moderate liquefaction hazard. Estimates of liquefaction settlement that could occur at the site as the result of a code-based earthquake are presented in section 4.2.2.

3.2 Seismicity and Faulting

3.2.1 Seismic Sources

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

3.2.2 Crustal Earthquakes and Faults

Review of the US Geological Survey Quaternary Fault and Fold Database and WADNR Geologic hazards portal indicates the site is within close proximity (less than 25 km or 15.5 miles) to numerous faults. Due to their proximity, the crustal faults are considered potentially significant seismic sources for severe ground motion in Snohomish County.

Table 3. Faults Within the Site Vicinity

Fault Zone Name	Fault ID	Approximate Distance to Site (Surface Projection in km)
Mount Washington fault zone	35	12
Devils Mountain fault	26	17
Utsalady Point fault	2120	24
Southern Whidbey Island fault zone	2210	25

3.2.3 Cascadia Subduction Zone (CSZ) – Interface Earthquakes

The CSZ represents the boundary between the subducting Juan de Fuca tectonic plate and the overriding North American tectonic plate. Recurrence intervals for subduction zone earthquakes are based on studies of the geologic record, with studies estimating a recurrence interval between 500 to 530 years. Geologic evidence and written records from Japan suggest the most recent earthquake occurred in January 1700. The 1700 earthquake probably ruptured much of the approximate 620-mile (1,000 km) length of the CSZ and was estimated at moment magnitudes of M_W 9.0. The horizontal distance from the edge of the CSZ megathrust is located approximately 321 km (200 miles) west of Arlington, Washington, with an estimated depth to slab of approximately 50 km. The current US Geological Survey risk-based maximum credible earthquake for CSZ megathrust is M_W 9.0±0.2.

3.2.4 Intraslab Earthquakes

Intraslab earthquakes occur within the subducting slab. They are problematic in the sense that they do not have a surface expression or rupture the ground surface, and their seismicity generates deformation along many faults within the slab. The CSZ has generated significant intraslab destructive earthquakes including the 2001 M_W 6.8 Nisqually earthquake in the Puget lowland.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 Geotechnical Design Considerations

The project site is underlain by zones of medium dense, 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, foundation support on shallow foundations is not feasible without some consideration of earthquake risk or mitigation. For our evaluation, we have considered three options for foundation support—each having different levels of risk associated with damage during an earthquake:

- Conventional spread footings over ground improvement
- Mat slab foundations on non-liquefiable crust
- Stiffened conventional foundations over reinforced fill on non-liquefiable crust.

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

4.2 Seismic Design Considerations

4.2.1 Code-Based Seismic Design Parameters

The current seismic design criteria for this project are based on the 2018 International Building Code (IBC) with State of Washington amendments. According to the Site Class Map of Snohomish County, Washington (Palmer, 2004), the site is located within an area classified as Site Class D, characterizing the profile as stiff soil. Due to the potential for liquefaction of site soils, the site should be considered Site Class F. However, in accordance with ASCE 7-16, for structures having a fundamental period of less than 0.5 second, a site-response analysis is not required to determine the spectral accelerations of liquefied soils and seismic design parameters can be determined using the pre-liquefaction site class. Based on subsurface conditions encountered in our explorations combined with SPT blow counts, Site Class D is appropriate for use in design. The seismic design criteria, in accordance with the 2018 IBC, are summarized in Table 4.

Table 4. 2018 IBC Seismic Design Parameters

Parameter	Short Period	1 Second
Maximum Credible Earthquake Spectral Acceleration	S_s = 1.039 g	S_1 = 0.371 g
Site Class	D[1]	
Site Coefficient	F_a = 1.084	F_v = 1.929[2,3]
Adjusted Spectral Acceleration	S_{MS} = 1.127 g	S_{M1} = 0.716 g[4]
Design Spectral Response Acceleration Parameters	S_{DS} = 0.751 g	S_{D1} = 0.477 g
Adjusted Peak Ground Acceleration	PGA_M = 0.511 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 second. If fundamental period is larger than 0.5 second, 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 = 1.929 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 will likely occur in layers below depths of 25 to 35 feet bgs at the site during a code-based earthquake and probably result in 2 to 7 inches of total liquefaction-induced settlement, with less than 1 inch of differential liquefaction settlement over 30 feet.

The risk of surface manifestation of liquefaction is reduced at the site by the presence of the existing non-liquefiable layer at the surface (i.e., "crust"). This crust is approximately 20 to 40 feet thick (represented by the unsaturated and non-liquefiable soil). Using the estimated ground surface acceleration associated with a code-based earthquake, methods developed by Ishihara (1985), and the liquefiable layer thickness at the site, the crust would need to be on the order of 30 feet thick.

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.

4.3 Foundation Alternatives

Potential seismically induced settlement would affect the proposed building foundations; however, the presence of the existing, non-liquefiable crust would likely mask these effects. The magnitude of liquefaction-induced settlement could be reduced and/or mitigated with ground improvement, mat slab foundations, or by placing stiffened/structurally enhanced foundations over reinforced fill. A more detailed discussion of these alternatives is included in the paragraphs that follow.

4.3.1 Shallow Foundations over Ground Improvement

Shallow spread footings bearing on site soils treated with ground improvement could potentially be used to support loads associated with the proposed development. Ground improvement techniques such as stone columns could be implemented to address the liquefaction hazards that are present at the site. However, due to the required depth of treatment and anticipated expense, PBS assumes ground improvement will not be economically feasible for this project. If requested, PBS can provide additional detailed ground improvement recommendations under a separate scope of services. The detailed design for ground improvement is typically completed by a design-build contractor. Depending on the settlement limitations of the new structures, improving all the potentially liquefiable soils at the site may not be necessary.

4.3.1.1 Stone Columns

Stone columns could be designed to provide suitable static support for building foundations and possibly reduce the risk of liquefaction, but may not be as effective where sands contain large amounts of fine-grained silt or in predominantly silt soils.

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

Depending on the application, stone columns can be 2 to 3 feet in diameter and installed in a grid at about 6 to 10 feet on-center. The actual diameter and spacing is typically determined by a specialty subcontractor, with the design reviewed by the geotechnical engineer of record. The extent beyond the intended area of improvement is typically one-third the depth of improvement. Stone columns can be used in conjunction with appropriately designed building foundation systems, including spread footings or mats.

4.3.2 Mat Foundations on Non-Liquefiable Crust

Use of mat foundations for the proposed structures would help reduce the impacts of possible differential settlement. The presence of the non-liquefiable crust does not reduce the risk of liquefaction in the deeper soils; however, our opinion is that settlement at the ground surface would be within tolerable limits for lightly loaded wood-frame structures. Mat foundations should be designed to span a distance of at least 10 feet in the case of surface manifestation of liquefaction and localized loss of ground support. Specific recommendations for design and construction are included in the following sections.

This alternative requires an acceptance of risk by the owner that some settlement is likely to occur during a code-based seismic event and is representative of design and construction practices in the area. If the owner is unwilling to accept the risks of post-liquefaction settlement, ground improvement or a deep foundation system should be considered.

4.3.2.1 Design Bearing Pressure

We recommend that the mat foundations be designed for an average allowable bearing pressure of 1,000 psf for combined dead plus long-term live loads, with a maximum localized bearing pressure of 1,500 psf. These pressures could be increased by up to one-third for seismic and wind loads. Subgrades should be compacted to a dense and unyielding condition prior to placement of at least 18 inches of compacted structural fill below the mat foundations.

Foundations will settle in response to column and wall loads. Based on our evaluation of the subsurface conditions and our analysis, mat foundations designed and constructed as recommended in this report are estimated to undergo total post-construction settlement on the order of 1 inch under static loading conditions. Differential settlement will be on the order of one-half of the total settlement.

4.3.2.2 Foundation Embedment Depth

PBS recommends that the perimeter of mat foundations be founded a minimum of 18 inches below the lowest adjacent grade. This can be accomplished with a thickened edge if the mat thickness is less than 18 inches. The foundations should be founded below an imaginary line projecting upward at a 1H:1V (horizontal to vertical) slope from the base of any adjacent, parallel utility trenches or deeper excavations.

4.3.2.3 Subgrade Preparation

Excavations for foundations should be carefully prepared to a neat and undisturbed state. A representative from PBS should confirm suitable bearing conditions and evaluate all exposed foundation subgrades. Observations should also confirm that loose or soft materials have been removed from new foundation excavation areas. Localized deepening of the excavations may be required to penetrate loose, wet, or deleterious materials. Any overexcavation should be backfilled with granular structural fill, such as clean crushed rock. The overexcavation width should extend at least one-half the overexcavation depth beyond the edges of the foundations. We suggest recompacting the exposed native subgrade prior to placing structural fill.

Satisfactory subgrade support for building mat foundations can be obtained from the on-site soil 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 18-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, 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 1½ inch, and has less than 5% by dry weight passing the US Standard No. 200 Sieve.

4.3.2.4 Modulus of Subgrade Reaction

We recommend designing the mat foundations using a preliminary modulus of vertical subgrade reaction of 100 pounds per cubic inch (pci). This value was calculated based on the allowable bearing pressure and estimated static settlement. The dimensions of the loaded area and load concentrations across the mat should be considered in evaluating the modulus of subgrade reaction. The structural engineer should consult with PBS to refine the subgrade modulus values once the structural design is finalized.

4.3.2.5 Lateral Resistance

Lateral loads can be resisted by passive earth pressure on the sides of the mat and by friction on the base of the mat. A passive earth pressure calculated using an equivalent fluid weight (EFW) of 300 pounds per cubic foot (pcf) may be used for foundations confined by native soils and new structural fills. The allowable passive pressure has been reduced by a factor of two to account for the large amount of deformation required to mobilize full passive resistance. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent unpaved areas should not be considered when calculating passive resistance. For foundations supported on new structural fills, use a coefficient of friction equal to 0.35 when calculating resistance to sliding. This value includes a factor of safety (FS) of 1.5.

4.3.3 Stiffened Conventional Foundations over Reinforced Fill

As an alternative to mat foundations, the proposed lightly loaded wood-frame structures may be supported on conventional foundations bearing on the reinforced structural fill, provided the structures can be designed to accommodate anticipated static and seismic settlements.

PBS recommends that the structural engineer incorporate an interconnected, grade-beam-supported floor system or include additional reinforcing steel to “stiffen” the conventional foundations (footings and slabs) so that if settlement occurs, the foundation can settle as a unit and reduce the amount of differential settlement that can occur under seismic conditions. The inclusion of an interconnected, grade-beam-supported floor system or the addition of structural steel will not address the liquefaction potential that underlies the site but it will help to reduce the damage to floor slabs and foundations that can occur during a seismic event. Considering the presence and contribution of the reinforced fill section, the stiffened footings and slabs should be designed to span a distance of at least 4 feet to accommodate potential localized loss of ground support.

This alternative requires an acceptance of risk by the owner that some settlement is likely to occur during a code-based seismic event and is representative of design and construction practices in the area. If the owner is unwilling to accept the risks of post-liquefaction settlement, ground improvement or a deep foundation system should be considered.

4.3.3.1 Reinforced Fill

Due to the potential for surface manifestations of liquefaction (sand boils) at the site, PBS recommends including layers of geosynthetic reinforcement (geogrid) within the proposed structural fill if conventional (non-mat) foundations are used to support the proposed structures without ground improvement. The reinforcement should be placed in the lower portion of the fill to avoid conflict with utilities below the structures. The geosynthetic reinforcement will provide an economical settlement mitigation solution that creates a composite reinforced zone (reinforced soil mass) to resist liquefaction surface manifestations and provide more uniform bearing pressure to the building foundations.

The reinforced zone should be at least 2.5 feet thick and extend at least 3 feet beyond the edges of structure foundations. The reinforced zone should be fully encapsulated in nonwoven separation geotextile per the Washington Department of Transportation Standard Specification (WSDOT SS) 9-33.2(1), Table 3. The separation geotextile shall be placed with a 3-foot minimum overlap at the edges between adjacent sheets.

The interior of the reinforced fill zone should consist of alternating layers of compacted backfill and four equally spaced layers of geogrid reinforcement. Backfill within the reinforced zone shall consist of crushed surfacing base course (CSBC) per WSDOT SS 9-03.9(3). The CSBC should be compacted to a minimum of 95% of the maximum dry density (MDD) as determined by test method ASTM D1557 (modified proctor).

The function of the geogrid reinforcement within the reinforced zone is to provide lateral confinement of the aggregate and to enhance soil arching. The tensile load in the reinforcement is estimated based on tension membrane theory and is a function of the amount of strain in the reinforcement. The initial strain of the reinforcement was limited to 2% in design. A minimum of four layers of Tensar Biaxial Geogrid BX-1100 geosynthetic reinforcement (or equivalent) should be installed with a vertical spacing of 6 inches. Figure 3 below shows the conceptual layout of separation geotextile and geogrid reinforcement in the lower portion of reinforced fill zone. Table 5 below provides additional geogrid reinforcement requirements.

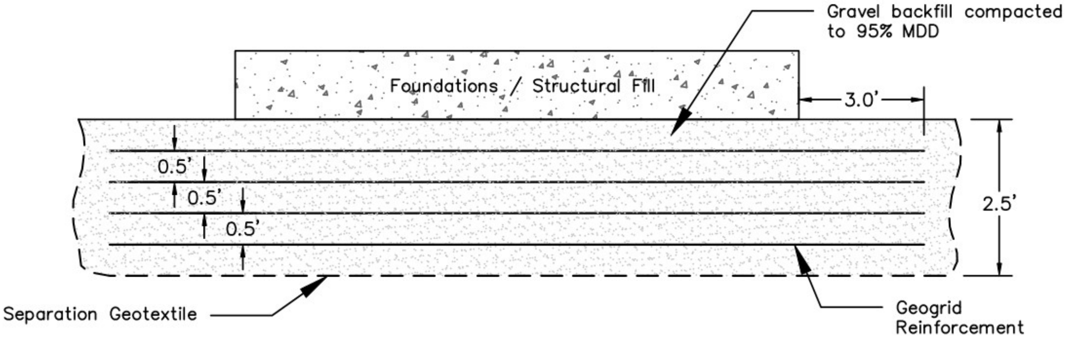

Figure 3. Schematic of Reinforced Fill Zone

The end result would be a “stiff” reinforced section of structural fill that could better tolerate total settlement and would limit differential settlements across the structural fill section.

Table 5. Geogrid Reinforcement Requirements

Property	Value	Test Method
Ultimate tensile strength (both directions)	850 lb/ft	ASTM D6637-10 A
Tensile strength at 5%	580 lb/in	ASTM D6637-10 A
Mass/unit area	Min. 8 oz/sq.yd	ASTM D5261
Aperture size	1.0 inch	ASTM D4759-02
Resistance to long-term and UV degradation	100%	EPA 9090 and ASTM D4355-05

Notes: lb/ft – pounds per foot, lb/in – pounds per inch, oz/sq.yd – ounces per square yard

4.3.3.2 Minimum Footing Widths and Design Bearing Pressure

Continuous wall and isolated spread footings should be at least 18 and 24 inches wide, respectively. Footings should be sized using a maximum allowable bearing pressure of 2,000 pounds per square foot (psf). This is a net bearing pressure and the weight of the footing and overlying backfill can be disregarded in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads. Allowable bearing pressures may be increased by one-third for seismic and wind loads.

Footings founded on the proposed reinforced fill section will still experience static settlement in response to column and wall loads. Based on our evaluation of the subsurface conditions and our analysis, we estimate post-construction static settlement under the allowable bearing pressure will be less than 1 inch for foundation loads. Differential settlement will be on the order of one-half of the total settlement.

4.3.3.3 Footing Embedment Depths

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

4.3.3.4 Footing Preparation

A representative from PBS should confirm suitable bearing conditions and evaluate the reinforced fill and footing subgrades. Observations should also confirm that loose or soft materials have been removed from excavations and concrete slab-on-grade areas. Localized deepening of footing excavations may be required to penetrate loose, wet, or deleterious materials. Any overexcavation should be backfilled with granular structural fill, such as clean crushed rock. The overexcavation width should extend at least one-half the overexcavation depth beyond the edges of the foundations. We suggest recompacting the exposed native subgrade prior to placing structural fill.

PBS recommends a minimum 4-inch-thick-layer of compacted, crushed rock be placed over the footing subgrades to help protect them from disturbance due to foot traffic and the elements. The footing subgrade (reinforced fill section) should be in a dense and undisturbed condition prior to pouring concrete.

4.3.3.5 Lateral Resistance

Lateral loads can be resisted by passive earth pressure on the sides of footings and grade beams, and by friction at the base of the footings. A passive earth pressure calculated using an EFW of 300 pcf may be used for footings confined by the new structural fill. The allowable passive pressure has been reduced by a factor of two to account for the large amount of deformation required to mobilize full passive resistance. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent unpaved areas should not be considered when calculating passive resistance. For footings supported on structural fills, use a coefficient of friction equal to 0.35 when calculating resistance to sliding. This value includes an FS of 1.5.

4.3.3.6 Floor Slabs

PBS recommends the inclusion of structural elements and/or reinforcement to add an element of rigidity to the foundations and concrete slabs of the structures. The structural engineer should incorporate additional reinforcing steel to connect the footings to the slabs so that the structure can settle as a unit. The additional structural steel will help to reduce the amount of damage to concrete slabs.

Floor slabs supported on the reinforced fill section prepared in accordance with the preceding recommendations may be designed using a modulus of subgrade reaction (k) of 150 pci.

4.4 Retaining Walls

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 EFW of 35 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 seismic loading on retaining walls over 6 feet in exposed (retained) height, we recommend using a triangular distribution (seismic surcharge) equivalent using an EFW of 11 pcf. Walls should be designed by applying the active earth pressure plus the seismic loading, or applying at-rest earth pressures, whichever is greater. If vertical surcharge loads, q, are present within 0.5H of the wall, a lateral surcharge of 0.3q (for walls allowed to rotate) and 0.5q (for restrained walls) should be applied as a uniform horizontal surcharge active over the full height of the wall. These values assume that the wall is vertical and the backfill behind the wall is horizontal. Seismic lateral earth pressures were computed using the Mononobe-Okabe equation. Recommended lateral earth pressure distributions are shown on Figure 4, Retaining Wall Earth Pressure Diagram. Additional lateral pressures due to surcharge loads can be estimated using the guidelines shown on Figure 5, Lateral Surcharge Detail.

Lateral loads can also be resisted by a passive resistance calculated using an EFW of 300 pcf acting against retaining walls and foundations, and by friction acting on the base of spread footings or mats using a friction coefficient of 0.35.

4.4.1 Wall 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.

4.5 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 disturbed native soils, not be steeper than 1.5H:1V, and temporary slopes made in undisturbed native and glacially consolidated soils not be steeper than 1H:1V. 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 periodically observed by the geotechnical engineer.

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

4.6 Ground Moisture

4.6.1 General

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

4.6.2 Perimeter Footing Drains

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

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

4.6.3 Vapor Flow Retarder

A continuous, impervious vapor flow retarder must be installed over the ground surface in the crawl space and 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.

4.7 Pavement Design

The provided pavement recommendations were developed based on our experience with similar developments and references the associated Washington Department of Transportation (WSDOT) specifications for construction.

The minimum recommended pavement section thicknesses are provided in Table 6. 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 6. Minimum AC Pavement Sections

Traffic Loading	AC (inches)	Base Course (inches)	Subgrade
Pull-in Car Parking Only	3	8	Firm subgrade as verified by PBS personnel*
Drive Lanes and Access Roads	4	8	

* 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 structures 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 6 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 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.

5.1.2 Wet/Freezing Weather and Wet Soil Conditions

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

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

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

5.1.3 Compacting Test Pit Locations

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

5.2 Excavation

The near-surface soils at the site can be excavated with conventional earthwork equipment. Sloughing and caving should be anticipated. All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. The contractor is solely responsible for adherence to the OSHA requirements. Trench cuts should stand relatively vertical to a depth of approximately 4 feet bgs, provided no groundwater seepage is present in the trench walls. Open excavation techniques may be used provided the excavation is configured in accordance with the OSHA requirements, groundwater seepage is not present, and with the understanding that some sloughing may occur. Trenches/excavations should be flattened if sloughing occurs or seepage is present. Use of a trench shield or other approved temporary shoring is recommended if vertical walls are desired for cuts deeper than 4 feet bgs. If dewatering is used, we recommend that the type and design of the dewatering system be the responsibility of the contractor, who is in the best position to choose systems that fit the overall plan of operation.

5.3 Structural Fill

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

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

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

5.3.1 On-Site Soil

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

5.3.2 Imported Granular Materials

Imported granular material used during periods of wet weather or for haul roads, building pad subgrades, staging areas, etc., should be pit or quarry run rock, crushed rock, or crushed gravel and sand, and should meet the specifications provided in WSDOT SS 9-03.14(2) – Select Borrow. In addition, the imported granular material should be fairly well graded between coarse and fine, and of the fraction passing the US Standard No. 4 Sieve, less than 5% 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 spread footings, slabs-on-grade, and other below-grade structures should be clean, crushed rock or crushed gravel and sand that is fairly well graded between coarse and fine. The granular materials should contain no deleterious materials, have a maximum particle size of 1½ inch, and meet WSDOT SS 9-03.12(1)A – Gravel Backfill for Foundations (Class A). The imported granular material should be placed in one lift and compacted to not less than 95% of the maximum dry density, as determined by ASTM D1557.

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, footing subgrades, and/or pile installation. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

7 LIMITATIONS

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

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

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

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

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

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Important Information about This

Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> 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 <u>Not</u> 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

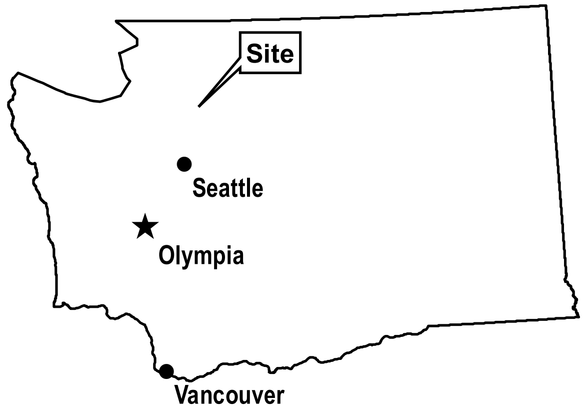
While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> 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 <u>not</u> building-envelope or mold specialists.***

GBA GEOPROFESSIONAL BUSINESS ASSOCIATION

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

Figures

Source: ESRI Topographic



VICINITY MAP

HARMONY AT ARLINGTON
ARLINGTON, WASHINGTON

DATE: APR 2024 · PROJECT: 73672.000

PBS

FIGURE

1

L:\Gis\GEOTECH\project\41391.004\fig_01_WA.mxd

EXPLANATION

⊕ B-1 - Boring name and approximate location

⊕ B-2 - Boring name and approximate location with piezometer

▨ TP-1 - Test pit name and approximate location

☐ Approximate site boundary

SOURCES: Google Earth 2018

1 inch = 100 feet

0 50 100 200 Feet

SITE PLAN

HARMONY AT ARLINGTON
ARLINGTON, WASHINGTON

DATE: APR 2024 · PROJECT: 73672.000

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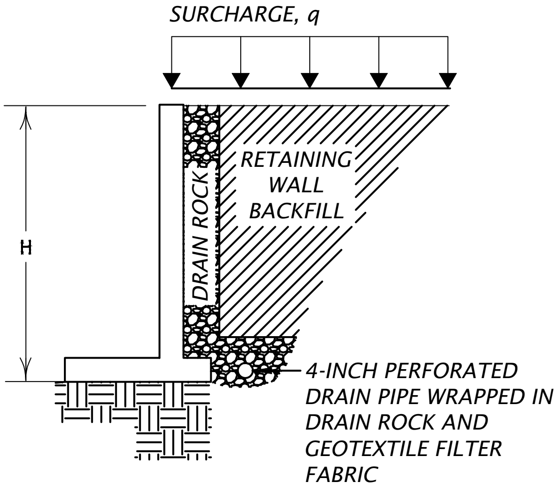
FIGURE

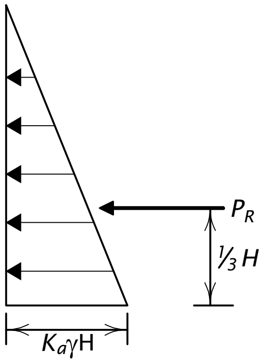
2

L:\Gis\GEOTECH\project\41391.004\fig_02_landscape_11x17_legend_right.mxd



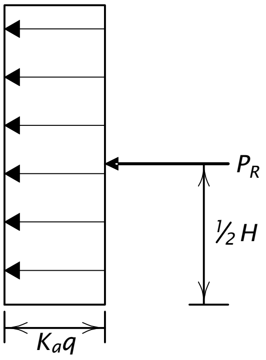
PARAMETER	VALUE
K_a	0.30
K_o	0.50
ΔK_{ae}	0.096
γ	115 pcf

ACTIVE EARTH PRESSURE



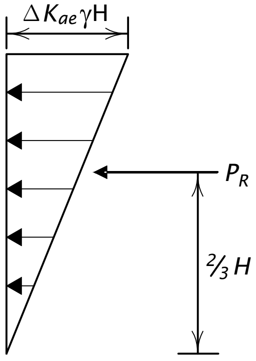
$P_R = K_a \gamma \frac{H^2}{2}$

SURCHARGE PRESSURE (ACTIVE)



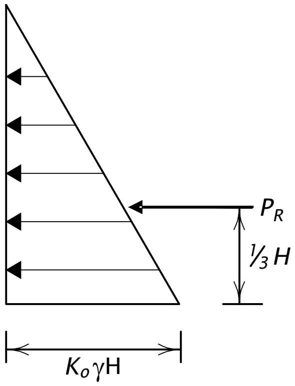
$P_R = K_a q H$

SEISMIC SURCHARGE PRESSURE



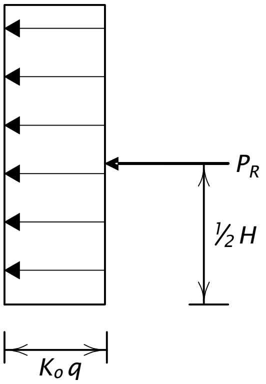
$P_R = \Delta K_{ae} \gamma \frac{H^2}{2}$

AT-REST EARTH PRESSURE



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

SURCHARGE PRESSURE (AT-REST)



$P_R = K_o q H$

PBS

RETAINING WALL EARTH PRESSURE DIAGRAM

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ARLINGTON, WASHINGTON

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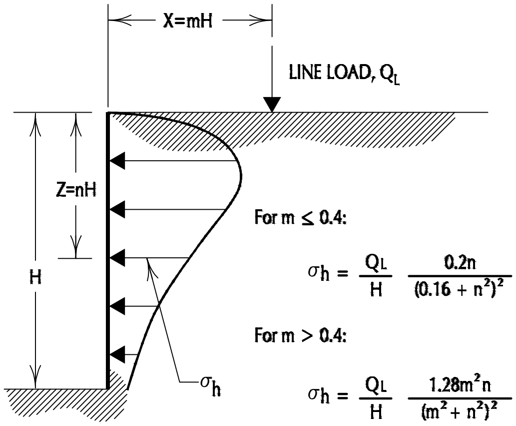
73672.000

FIGURE
4



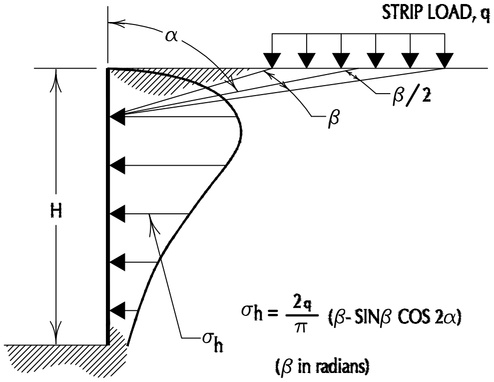
For m ≤ 0.4:

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

For m > 0.4:

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

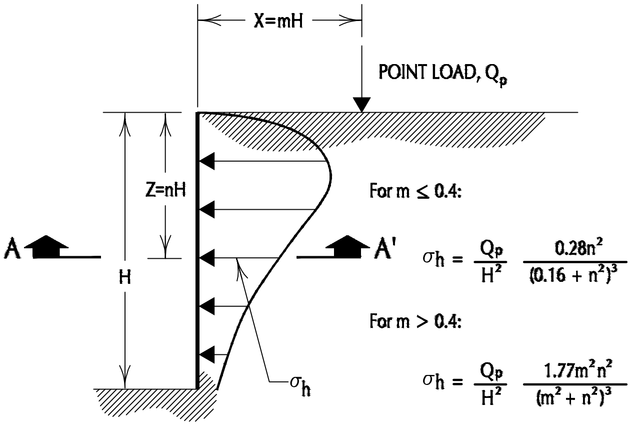
LINE LOAD PARALLEL TO WALL



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

(β in radians)

STRIP LOAD PARALLEL TO WALL

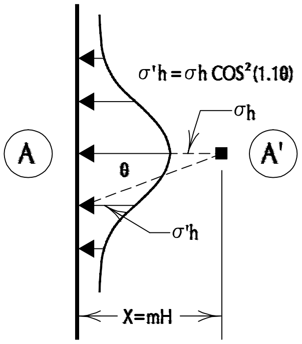


For m ≤ 0.4:

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

For m > 0.4:

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



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.

PBS

LATERAL SURCHARGE DETAIL

HARMONY AT ARLINGTON
ARLINGTON, WASHINGTON

APR 2024
73672.000

FIGURE
5

Appendix A

Field Explorations

Table A-1
Terminology Used to Describe Soil

Soil Descriptions

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

Fine-Grained Soils (50% or greater fines passing 0.075 mm, No. 200 sieve)

The primary soil type, i.e., SILT or CLAY is designated through visual-manual procedures to evaluate soil toughness, dilatency, dry strength, and plasticity. The following outlines the terminology used to describe fine-grained soils, and varies from ASTM D2488 terminology in the use of some common terms.

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

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

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

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

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

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

Table A-1
Terminology Used to Describe Soil

Soil Descriptions

Coarse - Grained Soils (less than 50% fines)

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

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

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

Example: Coarse-Grained Soil Descriptions with Fines

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

Additional descriptive terminology applied to coarse-grained soils follow.

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

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

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

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

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

Appendix A: Field Explorations

A1 GENERAL

PBS previously explored subsurface conditions at the project site by advancing four borings to depths of up to approximately 71.5 feet bgs on February 25 and 26, 2019. PBS also previously explored subsurface conditions at the project site by excavating two test pits to depths of up to 10 feet bgs on February 27, 2019, for the purpose of infiltration testing. The approximate locations of the explorations are shown on Figure 2, Site Plan. The procedures used to advance the borings and test pits, collect samples, and other field techniques are described in detail in the following paragraphs. Unless otherwise noted, all soil sampling and classification procedures followed engineering practices in general accordance with relevant ASTM procedures. "General accordance" means that certain local drilling/excavation and descriptive practices and methodologies have been followed.

A2 BORINGS

A2.1 Drilling

Borings were advanced using a truck-mounted CME-85 drill rig provided and operated by Holt Services, Inc., of Seattle, Washington, using mud rotary drilling techniques. The borings were observed by a member of the PBS geotechnical staff, who maintained a detailed log of the subsurface conditions and materials encountered during the course of the work.

A2.2 Sampling

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

A2.3 Boring Logs

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

A3 TEST PITS

A3.1 Excavation

Test pits were excavated using a using a track-mounted Komatsu 170LC excavator and a 36-inch bucket provided and operated by Taylor's Excavations, Inc., of Seattle, Washington. The test pits were observed by a member of the PBS geotechnical staff, who maintained a detailed log of the subsurface conditions and materials encountered during the course of the work.

A3.2 Sampling

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

A3.3 Test Pit Logs

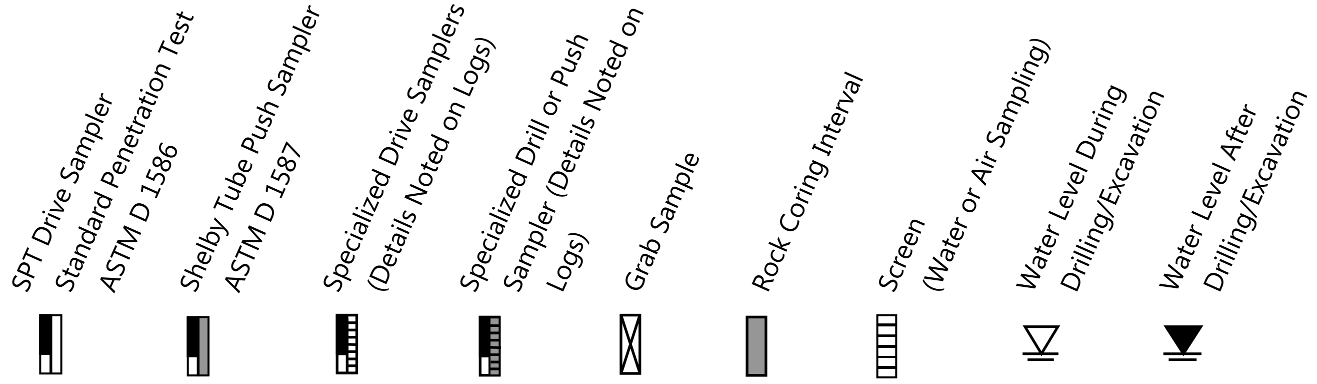
The test pit logs show the various types of materials that were encountered in the excavations and the depths where the materials and/or characteristics of these materials changed, although the changes may be gradual. Where material types and descriptions changed between samples, the contacts were interpreted.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

A4 MATERIAL DESCRIPTION

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

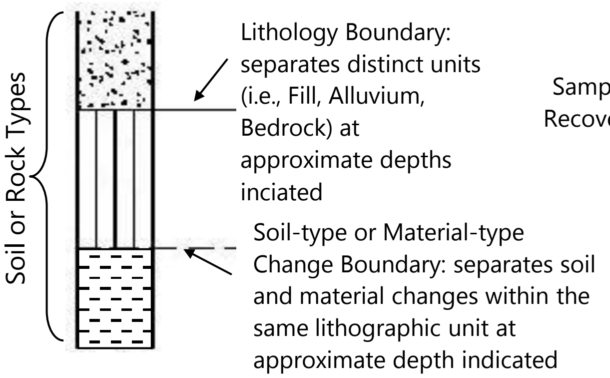
Table A-2
Key To Test Pit and Boring Log Symbols

SAMPLING DESCRIPTIONS

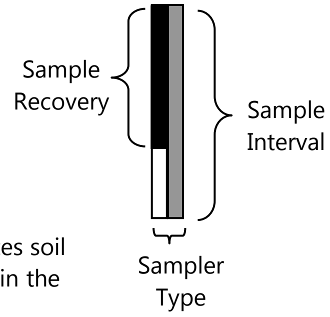


LOG GRAPHICS

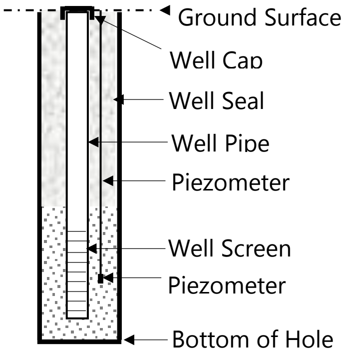
Soil and Rock



Sampling Symbols



Instrumentation Detail



Geotechnical Testing Acronym Explanations

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

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

Rev. 02/2017

PBS
AN APEX COMPANY

HARMONY AT ARLINGTON
ARLINGTON, WASHINGTON

BORING B-1

PBS PROJECT NUMBER:
73672.000

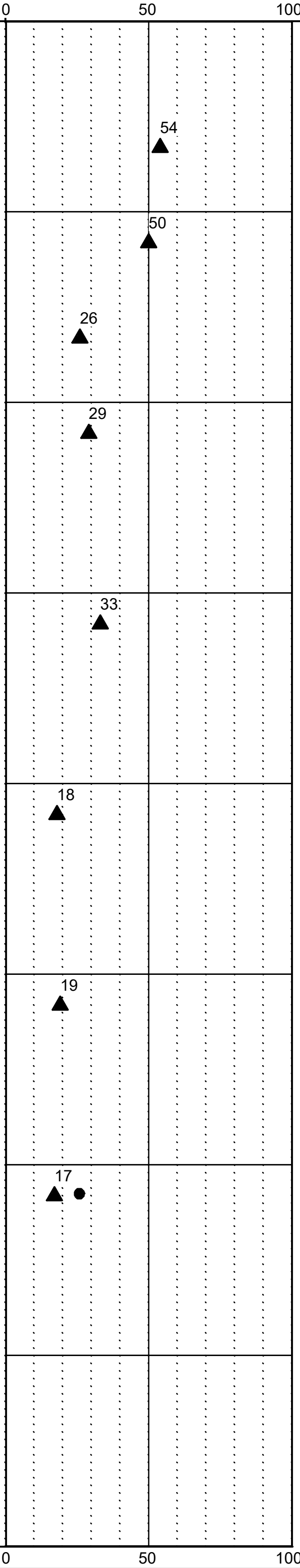
APPROX. BORING B-1 LOCATION:
(See Site Plan)
Lat: --- Long: ---

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

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

Surface Conditions: Gravel

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
0.0	Very dense, brown, silty GRAVEL (GM) with sand; low plasticity; fine to coarse sand; fine to coarse, subrounded gravel; moist	0.0		S-1	54	
5.0				S-2	50	
	Medium dense, brown, silty SAND (SM); non-plastic; fine to medium sand; trace, fine gravel; moist	7.5		S-3	26	
10.0				S-4	29	
15.0	Dense, brown, well-graded SAND (SW-SM) with silt; non-plastic; fine to coarse sand; trace, fine gravel; moist	15.0		S-5	33	
20.0	becomes medium dense			S-6	18	
25.0	Medium dense, dark brown, poorly graded SAND (SP-SM) with silt; non-plastic; fine sand; moist	25.0		S-7	19	
30.0	becomes wet		P200	S-8	17	P200 = 8%
	Final depth 31.5 feet bgs; boring backfilled with bentonite. Groundwater not measured at time of exploration due to mud rotary drilling.	31.5				
35.0						
40.0						



DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: M. Bagley

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 90
LOGGING COMPLETED: 2/26/2019

FIGURE A1

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HARMONY AT ARLINGTON
ARLINGTON, WASHINGTON

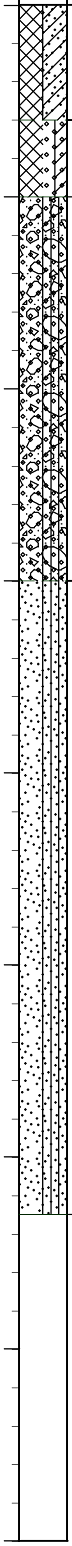
BORING B-2

PBS PROJECT NUMBER:
73672.000

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

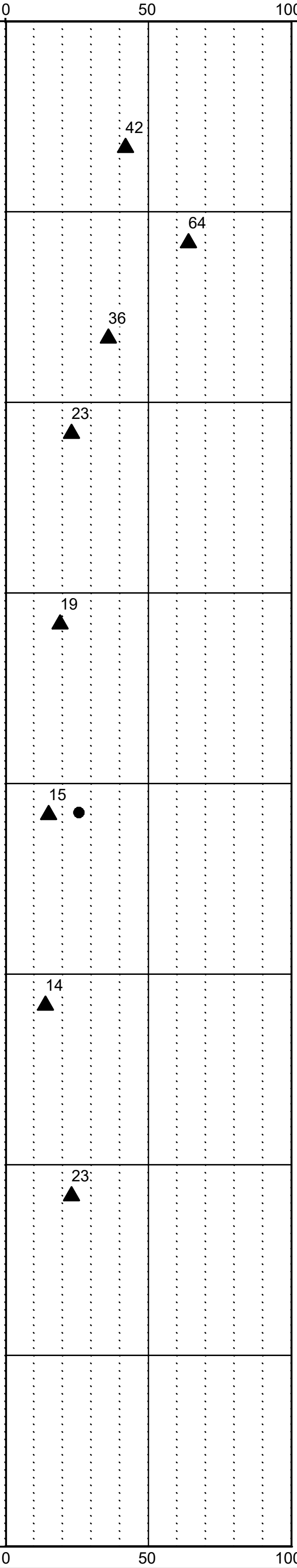
DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID
0.0	Dense, brown, clayey SAND (SC); medium plasticity; fine to coarse sand; moist **FILL**	0.0		
3.0	Dense, gray-brown, well-graded SAND (SW-SC) with clay and gravel; non-plastic; fine to coarse sand; fine to coarse, subrounded gravel; moist	3.0		S-1
5.0	Dense, brown with gray mottles, well-graded SAND (SW-SM) with silt and gravel; non-plastic; fine to coarse sand; fine to coarse, subrounded gravel; moist	5.0		S-2
				S-3
10.0	becomes medium dense			S-4
15.0	Medium dense, light brown, poorly graded SAND (SP-SM) with silt; non-plastic; fine to medium sand; trace, fine gravel; moist	15.0		S-5
20.0	becomes dark brown; wet		P200	S-6
25.0				S-7
30.0	becomes olive-brown			S-8
	Final depth 31.5 feet bgs. Piezometer installed to 20 feet bgs; boring backfilled with bentonite from 20 to 31.5 feet bgs. Groundwater not measured at time of exploration due to mud rotary drilling.	31.5		
35.0				
40.0				

GRAPHIC LOG

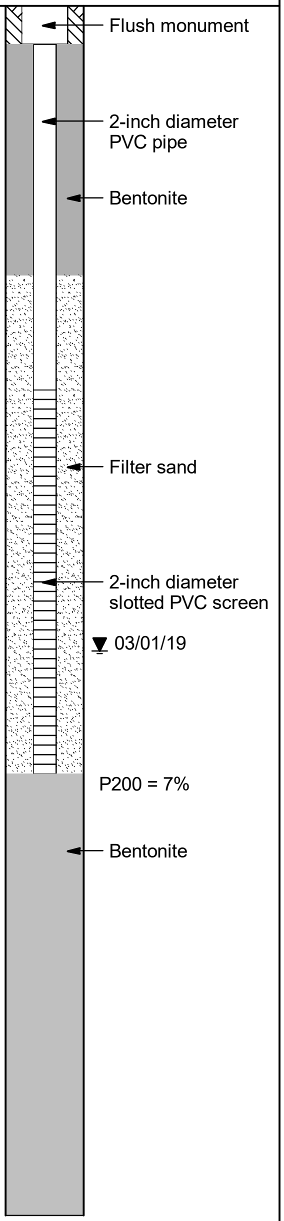


NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

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



INSTALLATION AND COMMENTS
Surface Conditions: Gravel



DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: M. Bagley

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 90
LOGGING COMPLETED: 2/25/2019

FIGURE A2
Page 1 of 1

HARMONY AT ARLINGTON
ARLINGTON, WASHINGTON

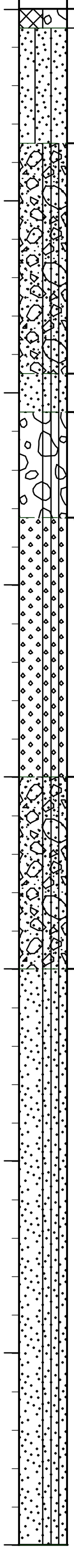
BORING B-3

PBS PROJECT NUMBER:
73672.000

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

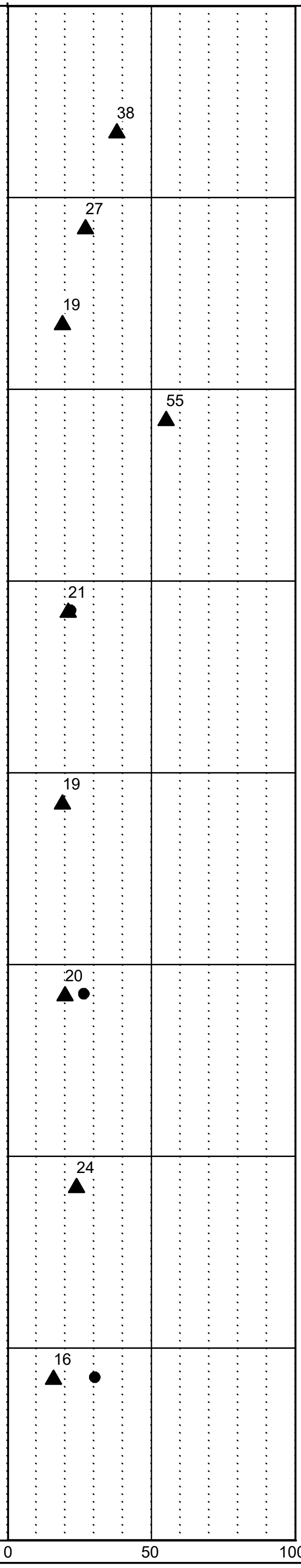
Surface Conditions: Gravel

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



DEPTH (FEET)	MATERIAL DESCRIPTION
0.0	GRAVEL FILL (~6 inches)
0.5	Dense, brown, silty SAND (SM); non-plastic; fine to medium sand; moist
3.5	Dense, gray, poorly graded SAND (SP-SM) with silt and gravel; non-plastic; fine to coarse sand; fine to coarse, rounded gravel; moist becomes medium dense, brown-gray; fine to medium sand
9.5	Very dense, brown, poorly graded SAND (SP-SM) with silt; non-plastic; fine to medium sand; moist
10.5	Very dense, brown-gray, poorly graded GRAVEL (GP-GM) with silt and sand; non-plastic; fine to medium sand; fine to coarse, rounded gravel; moist to wet
13.3	Medium dense, brown, well-graded SAND (SW-SM) with silt; non-plastic; fine to coarse sand; moist
20.0	Medium dense, brown, poorly graded SAND (SP-SM) with silt and gravel; fine to medium sand; fine, rounded gravel; moist to wet
25.0	Medium dense, brown poorly graded SAND (SP-SM) with silt; non-plastic; fine to medium sand; wet

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



TESTING	SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
	S-1	38	
	S-2	27	
	S-3	19	
	S-4	55	
P200	S-5	21	P200 = 8%
	S-6	19	
P200	S-7	20	P200 = 7%
	S-8	24	
P200	S-9	16	P200 = 8%

DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: D. Eibert

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 90
LOGGING COMPLETED: 2/25/2019

FIGURE A3

BORING LOG 73672.000 B1-4 20240426.GPJ PBS DATATMPL GEO.GDT PRINT DATE: 4/26/24:RPG

PBS
AN APEX COMPANY

HARMONY AT ARLINGTON
ARLINGTON, WASHINGTON

PBS PROJECT NUMBER:
73672.000

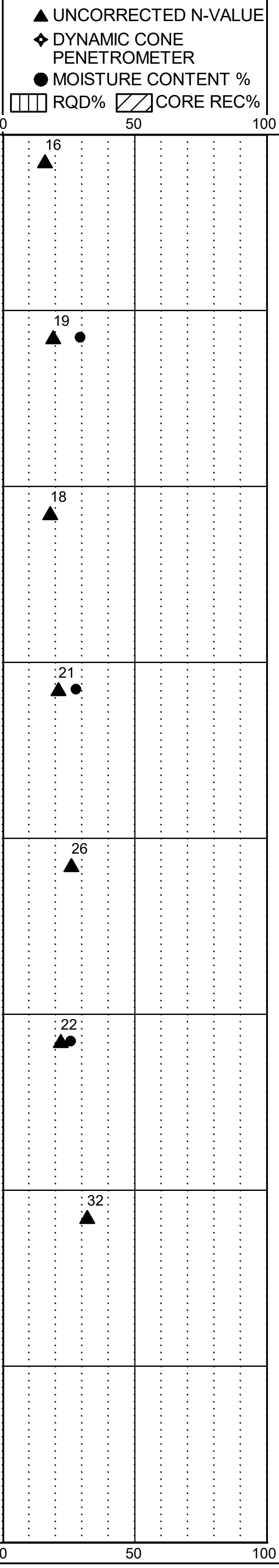
BORING B-3
(continued)

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

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: Gravel

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	INSTALLATION AND COMMENTS
40.0		Medium dense, brown poorly graded SAND (SP-SM) with silt; non-plastic; fine to medium sand; wet	40.0		S-10	
45.0				P200	S-11	P200 = 11%
		becomes gray				
50.0					S-12	
55.0				P200	S-13	P200 = 9%
60.0					S-14	
65.0				P200	S-15	P200 = 8%
						Rig chatter
70.0		becomes dense			S-16	
		Final depth 71.5 feet bgs; boring backfilled with bentonite. Groundwater not measured at time of exploration due to mud rotary drilling.	71.5			
75.0						
80.0						



DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: D. Eibert

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 90
LOGGING COMPLETED: 2/25/2019

FIGURE A3

HARMONY AT ARLINGTON
ARLINGTON, WASHINGTON

BORING B-4

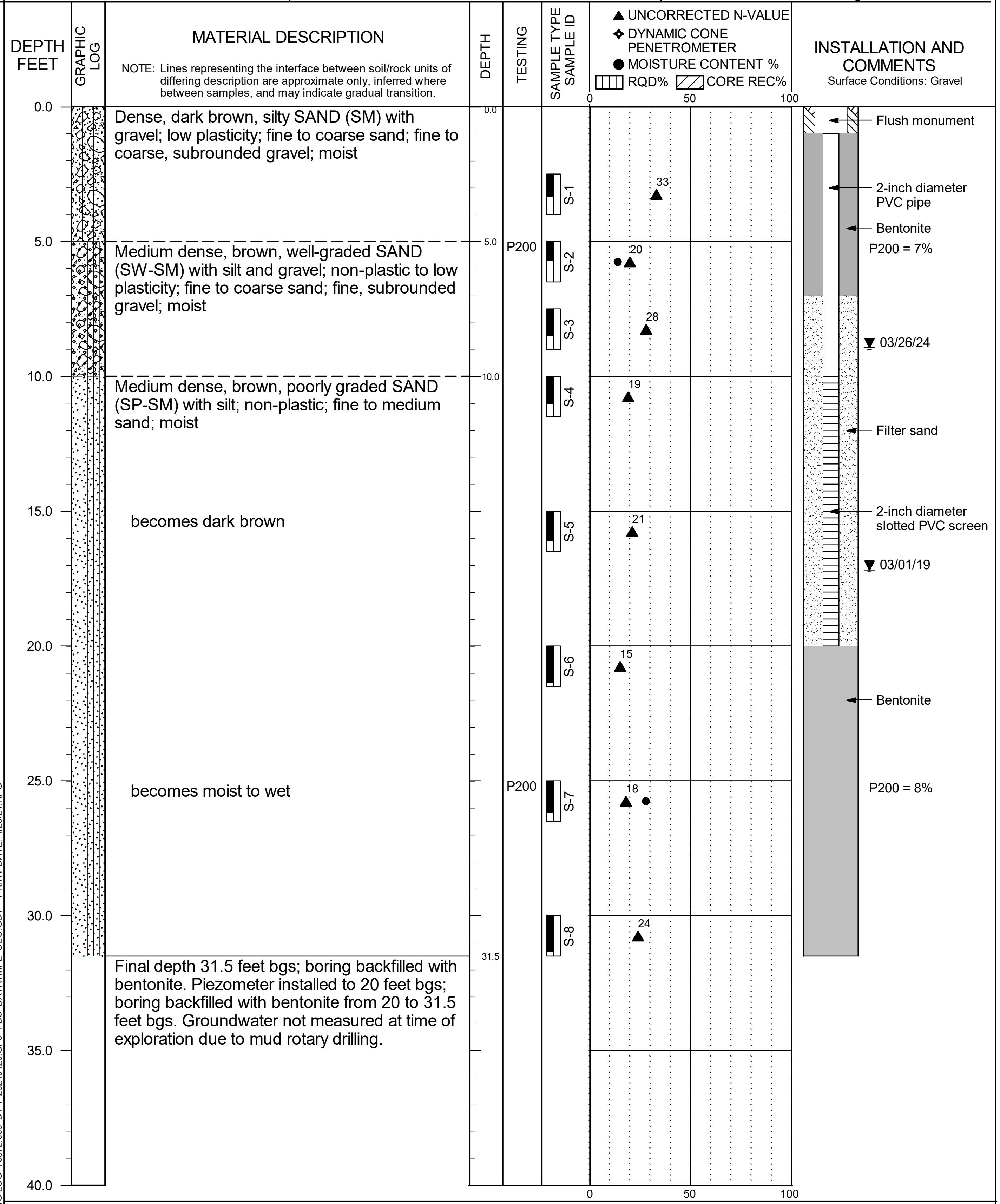
PBS PROJECT NUMBER:
73672.000

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

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

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

Surface Conditions: Gravel



0.0: Dense, dark brown, silty SAND (SM) with gravel; low plasticity; fine to coarse sand; fine to coarse, subrounded gravel; moist

5.0: Medium dense, brown, well-graded SAND (SW-SM) with silt and gravel; non-plastic to low plasticity; fine to coarse sand; fine, subrounded gravel; moist

10.0: Medium dense, brown, poorly graded SAND (SP-SM) with silt; non-plastic; fine to medium sand; moist

becomes dark brown

becomes moist to wet

31.5: Final depth 31.5 feet bgs; boring backfilled with bentonite. Piezometer installed to 20 feet bgs; boring backfilled with bentonite from 20 to 31.5 feet bgs. Groundwater not measured at time of exploration due to mud rotary drilling.

TESTING	SAMPLE ID	UNCORRECTED N-VALUE
	S-1	33
P200	S-2	20
	S-3	28
	S-4	19
	S-5	21
	S-6	15
P200	S-7	18
	S-8	24

DRILLING METHOD: Mud Rotary - Tricone
DRILLED BY: Holt Services, Inc.
LOGGED BY: M. Bagley

BIT DIAMETER: 3 7/8 inches
HAMMER EFFICIENCY PERCENT: 90
LOGGING COMPLETED: 2/26/2019

FIGURE A4

PBS
AN APEX COMPANY

HARMONY AT ARLINGTON
ARLINGTON, WASHINGTON

TEST PITS

PBS PROJECT NUMBER:
73672.000

TEST PIT LOCATION:
(See Site Plan)

MATERIAL DESCRIPTION

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

◆ DYNAMIC CONE PENETROMETER
☒ STATIC PENETROMETER
● MOISTURE CONTENT %

TP-1

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	0 — 25 — 50	COMMENTS
0.0	Brown, silty SAND (SM) with gravel and cobbles; low plasticity; medium sand; coarse, subrounded gravel; moist	0.0 0.0				
2.0	Light brown, poorly graded SAND (SP) with gravel and cobbles; fine to medium sand; coarse, subrounded gravel; moist	2.0 2.0				Infiltration testing completed at 5 feet bgs P200 = 1%
4.0	Light brown, well-graded SAND (SW) with cobbles; fine to coarse sand; moist	4.0 4.0	P200	☒ S-1	●	
6.0						
8.0				☒ S-2		
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface.Groundwater not encountered at time of exploration.	10.0 10.0				
12.0						
14.0						

Date Completed: 2/27/2019 Elevation: --- Final Depth: 10 feet bgs Latitude: --- Longitude: ---

TP-2

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	0 — 25 — 50	COMMENTS
0.0	Brown, silty SAND (SM) with gravel, cobbles, and roots; non-plastic; fine to medium sand; coarse, subrounded gravel; moist	0.0 0.0				
2.0	Light brown, poorly graded SAND (SP) with gravel and cobbles; fine to medium sand; coarse, subrounded gravel; moist	2.5 2.5				Infiltration testing completed at 5 feet bgs P200 = 1%
4.0	Brown to light brown, well-graded SAND (SW) with cobbles; fine to coarse sand; moist	4.5 4.5	P200	☒ S-1	●	
6.0						
8.0				☒ S-2		
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface.Groundwater not encountered at time of exploration.	10.0 10.0				
12.0						
14.0						

Date Completed: 2/27/2019 Elevation: --- Final Depth: 10 feet bgs Latitude: --- Longitude: ---

EXCAVATION METHOD: Komatsu 170LC with 36" Bucket LOGGED BY: M. Bagley
EXCAVATED BY: Taylor's Excavators Inc.

FIGURE A5

Appendix B

Laboratory Testing

Appendix B: Laboratory Testing

B1 GENERAL

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

B2 CLASSIFICATION TESTS

B2.1 Visual Classification

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

B2.2 Moisture (Water) Contents

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

B2.3 Grain-Size Analyses (P200 Wash)

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

SUMMARY OF LABORATORY DATA

HARMONY AT ARLINGTON
ARLINGTON, WASHINGTON

PBS PROJECT NUMBER:
73672.000

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-8	30		25.7				8			
B-2	S-6	20		25.6				7			
B-3	S-5	15		21.8				8			
B-3	S-7	25		26.4				7			
B-3	S-9	35		30.2				8			
B-3	S-11	45		29.2				11			
B-3	S-13	55		27.6				9			
B-3	S-15	65		25.6				8			
B-4	S-2	5		13.9				7			
B-4	S-7	25		27.8				8			
TP-1	S-1	5	5.0	4.1				1			
TP-2	S-1	5	5.0	3.1				1			

_LAB SUMMARY 73672.000_B1-4_20240426.GPJ PBS_DATATMPL_GEO.GDT PRINT DATE: 4/26/24:RPG

FIGURE B1