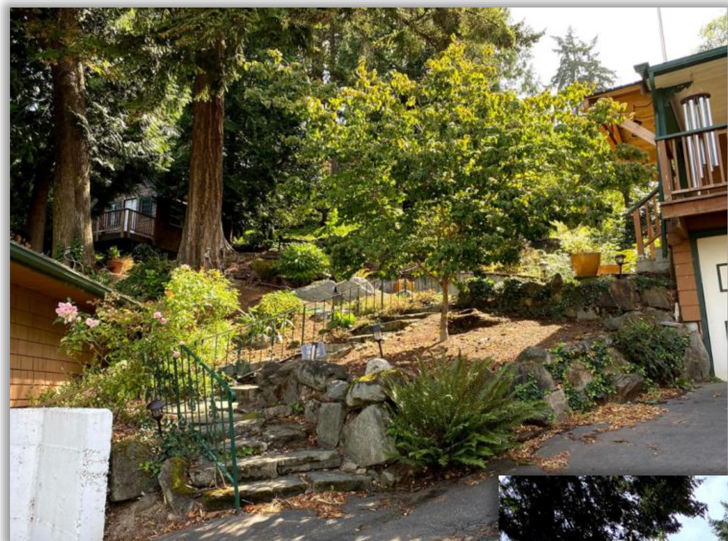
GEOTECHNICAL REPORT

PROPOSED RESIDENCE
8019 SOUTHEAST 20TH STREET
MERCER ISLAND, WASHINGTON

Project No. 20-332
November, 2020



Prepared for:

Ahbleza Pattison

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November 23, 2020
PanGEO Project No. 20-332

Ahbleza Pattison
8019 SE 20th Street
Mercer Island, WA 98040

Subject: **GEOTECHNICAL REPORT**
Proposed Residence
8019 Southeast 20th Street, Mercer Island, Washington

Dear Mr. Pattison,

Please find attached our geotechnical report for the proposed residence at 8019 Southeast 20th Street in Mercer Island, Washington. In preparing this report, we completed four test borings, reviewed readily available geologic data, and conducted our engineering analyses. In summary, at our test boring locations, we encountered a thin surficial layer of loose fill, overlying medium dense to very dense glacial till, overlying very stiff to hard sandy silt and clayey silt.

In our opinion, the proposed buildings may be supported on conventional footings bearing on the native glacial soils or on compacted structural fill placed on the native soil. Based on our understanding of the proposed excavation depths and the topography at the site, excavation shoring consisting of soldier piles and possibly tiebacks/rakers will be needed to support the temporary excavation.

We appreciate the opportunity to work on this project. Please call if there are any questions.

Sincerely,

Bryce C. Townsend, P.E.
Project Geotechnical Engineer

Encl: Geotechnical Report

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GEOTECHNICAL REPORT
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MERCER ISLAND, WASHINGTON

1.0 GENERAL

This report presents the results of our geotechnical engineering study to support the design and construction of the proposed residence. We performed our geotechnical study in general accordance with our mutually agreed scope of work outlined in our proposal dated August 27, 2020, which was subsequently approved by you on the same day. Our service scope included conducting a site reconnaissance, reviewing readily available geologic data, drilling four test borings at the site, and developing the conclusions and recommendations presented in this report.

2.0 SITE AND PROJECT DESCRIPTION

The project site is located at 8019 Southeast 20th Street in Mercer Island, Washington (see Figure 1, Vicinity Map). The subject property is a 18,701 square foot parcel and is generally trapezoidal shaped. The site is bordered by Southeast 20th Street to the north and single-family residences to the east, west, and south.

The property is occupied by an existing one-story house with a basement level generally situated near the south property line (see Plate 1 on the following page). The basement level of the existing house daylights towards the north where it roughly matches the existing grade at the north corner of the house. The house is accessed by a paved driveway from Southeast 20th Street up to the north corner of the house basement.

In addition to the house, there are two detached garages near the north property line on the east and west sides of the access driveway. There is a deck connecting the first level of the existing house to the roof of the west detached garage. The east and west detached garages are partially set back into the existing site slopes. There is also a shed near the northeast corner of the property that appears to be founded on small diameter pipe piles.

The overall property is situated on a northwest facing slope that descends about 40 feet total from the southeast corner at approximate elevation of 67 feet, to the northwest property corner at approximate elevation 27 feet (see Plate 2 on the following page). The slope continues to ascends beyond the south and east property line (see topographic survey on Figure 2).

The overall slope is landscaped with large and small trees, shrubs, shallow rockeries, and mulch. There are small concrete retaining walls on the south and east sides of the house retaining about 4 feet of soil. The grade is generally level along the north and south sides of the existing house with the grade descending along the east and west sides of the house.

Plate 1. Existing house with detached garages, looking south.

Plate 2. Existing site slope extending southward above the existing house.

We understand that you plan to demolish the existing house to construct a new residence. At this time, we understand that the finished basement floor is planned at approximate elevation 34½ feet. Both detached garages will remain with the new residence connecting to the structures. The attached Figure 2 shows the approximate proposed development layout. Based on the planned basement floor elevation and the existing topography of the site slope, we anticipate the basement excavations will be up to about 18 feet deep.

Based on our review of the City of Mercer Island GIS Portal, there are landslide, seismic, and erosion hazards mapped on the property (see Plate 3, following page).

The conclusions and recommendations in this report are based on our understanding of the proposed development, which is in turn based on the project information provided. If the above project description is incorrect, or the project information changes, we should be consulted to review the recommendations contained in this study and make modifications, if needed. In any case, PanGEO should be retained to provide a review of the final design to confirm that our geotechnical recommendations have been correctly interpreted and adequately implemented in the construction documents.

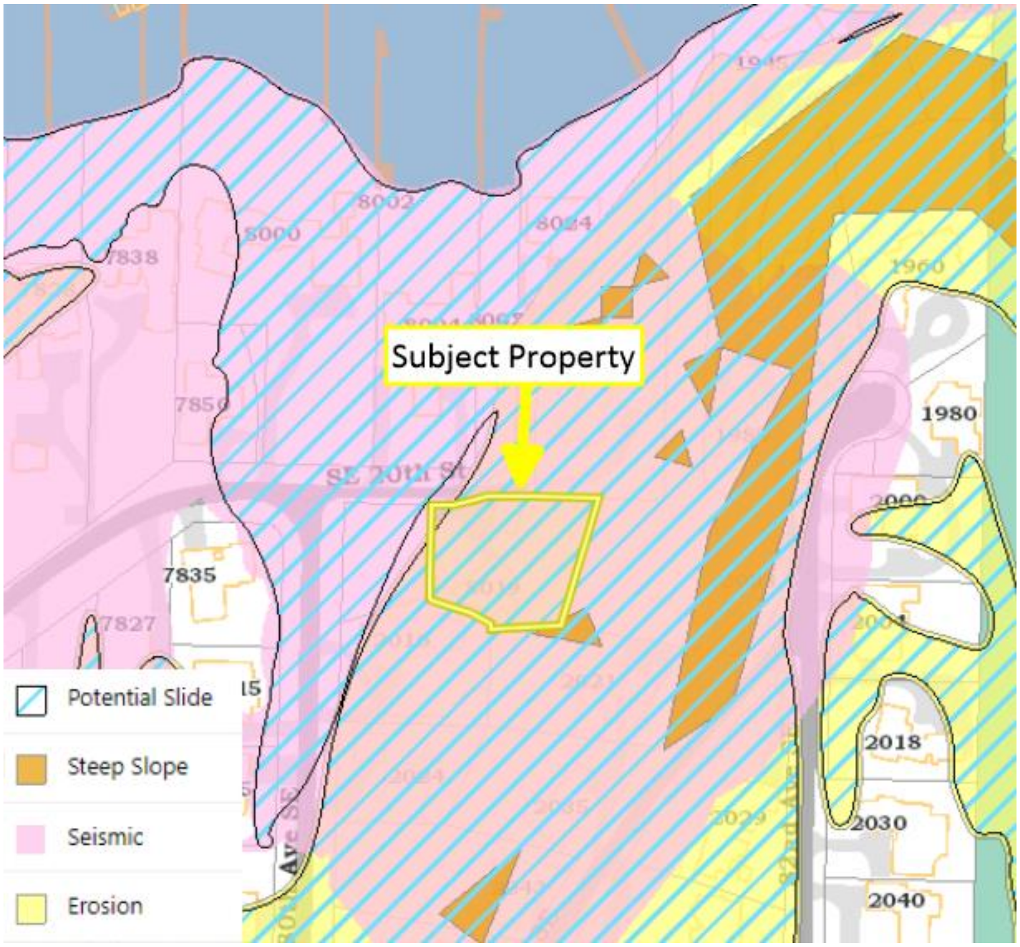


Plate 3. Geologic hazards mapped on the property (City of Mercer Island GIS Portal).

3.0 SUBSURFACE EXPLORATIONS

Four test borings (PG-1 to PG-4) were drilled at the subject site on September 4, 2020. The approximate boring locations were taped in the field from on-site features and are shown in Figure 2. Borings were drilled to depths ranging between about 11½ feet and 41½ feet below existing grades.

The drill rig was equipped with 4-inch outside diameter hollow stem augers. Soil samples were obtained from the borings in general at 2½- and 5-foot intervals in conjunction with Standard Penetration Test (SPT) sampling methods in general accordance with ASTM test method ASTM D-1586, *Standard Test Method for Penetration Test and Split Barrel Sampling of Soils*, in which the samples are obtained using a 2-inch outside diameter split-spoon sampler. The sampler was driven into the soil a distance of 18 inches using a 140-

pound weight falling a distance of 30 inches. The number of blows required for each 6-inch increment of sampler penetration was recorded. The number of blows required to achieve the last 12 inches of sample penetration is defined as the SPT N-value. The N-value provides an empirical measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils.

A geologist from PanGEO was present during the field exploration to observe the drilling, assist in sampling, and to describe and document the soil samples obtained from the borings. The soil samples were described and field classified in general accordance with the symbols and terms outlined in Figure A-1, and the summary boring logs are included as Figures A-2 through A-5.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 SITE GEOLOGY

According to the *Geology Map of Mercer Island* (Troost, et al., 2006), the surficial geologic unit mapped at the subject site is Pre-Olympia glacial till (Map Unit Qpogt) with Pre-Olympia fine-grained deposits (Map Unit Qpof) mapped immediately southeast from the site. Pre-Olympia glacial till is described by Troost et al. as a dense, heterogeneous mixture of silt, sand, and gravel laid down at the base of an advancing glacial ice sheet from the Pre-Olympia age. Pre-Olympia fine-grained deposits consist of hard, silt and clay that has been glacially overridden.

Both the pre-Olympia till and fine-grained deposits typically exhibit low compressibility and high strength characteristics in their undisturbed states.

4.2 SOIL CONDITIONS

Based on the soil conditions observed in our test borings, the site soils appear generally consistent with the mapped geology with a shallow layer of till overlying fine-grained deposits.

The following is a description of the soils observed in our test borings. Please refer to our summary test borings logs (Figures A-2 through A-5) and subsurface profile A-A’ (Figure 3) for additional details.

> **Soil Unit 1: Fill** – A surficial layer of loose to medium dense, silty, gravelly sand was encountered in PG-4 that extended to about 7 feet below existing grade. Based

on the relatively loose condition and disturbed nature of the soils encountered, we interpret this unit as undocumented fill most likely derived from the construction of the existing residence. Fill was generally 1 to 2 feet thick in test borings PG-1 and PG-2, generally consisting of loose, dark brown, silty sand with organics. Fill was not encountered in PG-3.

Soil Unit 2: Pre-Olympia Glacial Till (Qpogt) – Below the fill, test borings PG-1, PG-2 and PG-3 encountered medium dense to very dense silty sand with varying amounts of gravel, which appears to be consistent with the mapped pre-Olympia glacial till deposits. This unit extended to about 9 feet deep in PG-1 and 7 feet deep in borings PG-2 and PG-3. This unit was not encountered in boring PG-4.

Soil Unit 3 - Pre-Olympia Fine-Grained Deposits (Qpof) – Below the fill in PG-4 and glacial till in borings PG-1, PG-2 and PG-3, all four borings encountered very stiff to hard sandy silt and clayey silt. The silt was generally massive and appeared to be low to moderately plastic. Based on the massive and hard consistency, we interpret this soil unit as the mapped pre-Olympia fine-grained deposits. This unit extended to the maximum drilled depth of about 41½ feet below grade.

Our subsurface descriptions are based on the conditions encountered and observed at the time of our exploration. Soil conditions between exploration locations may vary from those encountered. The nature and extent of variations between our exploratory locations may not become evident until construction. If variations do appear, PanGEO should be requested to reevaluate the recommendations in this report and to modify or verify them in writing prior to proceeding with earthwork and construction.

4.3 GROUNDWATER

Groundwater was not encountered within the maximum exploration depth of our test borings during drilling. It should be noted that groundwater elevations may vary depending on the season, local subsurface conditions, and other factors. Groundwater levels are normally highest during the winter and early spring (typically October through May).

5.0 GEOLOGIC HAZARD ASSESSMENT

5.1 Seismic Hazard Review

Based on our review of the City of Mercer Island GIS Map, the property location is mapped as a seismic hazard.

Based on the presence of dense Pre-Olympia glacial deposits near the ground surface and the lack of groundwater observed in all four of our test borings, in our opinion, the potential for soil liquefaction is considered low. As such, it is our opinion that special design considerations associated with soil liquefaction are not needed for this project.

We also evaluated the site stability during the design earthquake. Details of our seismic stability are discussed in Section 5.2.2 of this report. In summary, the results of our analysis indicate that a minimum factor of safety of 1.1 can be achieved if the recommendations outlined in this report are implemented.

5.2 Landslide Hazard Review

According to the City of Mercer Island GIS Map, the property is located in a potential landslide area. The following sections detail our assessment of the overall site stability, including our visual observations, a quantitative slope stability analysis of the site slope, and recommendations for maintaining stability during and post-construction.

5.2.1 Existing Site Conditions

During our site reconnaissances, we did not observe evidence of recent instability such as slide scarps, hummocky ground surface, or tension cracks within the subject property. The site slopes south of the existing house appears well landscaped with trees and small shrubs with no visible signs of instability. The site retaining walls along the south side of the existing house appears vertical, indicating the site retaining walls are stable with no sights of creep or leaning. Based on our onsite observations, the overall site appears to be stable in the existing condition.

5.2.2 Quantitative Slope Stability Analysis

We performed a quantitative slope stability analysis of the site based on the soil profile shown in Figure 3. The soil profile was generated through the middle of the existing house and perpendicular to the site slope where we believe the most critical section is. Our analysis includes models for two cases: the static slope stability during the temporary excavation condition with shoring (Figure 4a), and the seismic (pseudo-static) condition with the permanent structure in place (Figure 4b). The post-condition static case is not as critical as the during-construction case and hence not included in our report.

We performed our slope stability analysis using the program SLIDE2 (Slide) published by Rocscience Inc. Slide is a two-dimensional limit equilibrium slope stability analysis program. Our analysis used the Janbu Simplified Method to determine potential failure planes as it yielded the most conservative results. The following discusses our model and analysis:

Soil Parameters: A summary of the input soil parameters is provided in Table 1 below. Input parameters were selected based on general estimates provided in USGS Open-File Report 2006-1139 (Laprade et al., 2006) and our own judgement and experience with similar soils. For the seismic condition, a cohesion of 200 psf was applied to the Pre-Olympia fine grained deposits (very stiff to hard silt and clay). According to Laprade et al., effective cohesion for pre-Olympia fine grained deposits can be estimated at about 600 psf. As such, in our opinion, a seismic induced cohesion of 200 psf is appropriately conservative.

Table 1 – SLIDE Soil Input Parameters

Soil Type	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Fill	110	28	0
Pre-Olympia Glacial Till (Qpogt)	130	40	0
Pre-Olympia Fine Grained Deposits (Qpof)	130	34	0 (static) 200 (seismic)

Groundwater: Groundwater was not observed in our subsurface explorations at the site. As such, groundwater was not modelled in our slope stability analysis.

Seismic Parameters: Seismic design parameters for the site were developed in conformance with the 2015 IBC, which specifies a design earthquake having a 2 percent probability of occurrence in 50 years (return interval of 2,475 years). A peak ground acceleration (PGA) of 0.56g was obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2008 data) for the project latitude and longitude, based on Site Class D for stiff soil. The horizontal design PGA was estimated based on taking one-half of the PGA, or 0.28g.

Results: The results of our slope stability analysis for the static and pseudo-static conditions are summarized in the attached Figures 4a and 4b, respectively.

For the static condition during the temporary excavation (Figure 4a), the computed minimum factor of safety is 1.52. A minimum soldier pile embedment of 16 feet was utilized to achieve the resulting factor of safety. Deeper pile embedment than 16 feet may be needed based on structural design.

For the seismic condition with the permanent structure in place (Figure 4b), the computed minimum factor of safety is 1.13.

Based on the results from our analysis, the global stability of the existing south slope meets the minimum factor of safety requirements of 1.5 for the static condition and 1.1 for the seismic condition.

5.3 Erosion Hazard Review

Based on our review of the City of Mercer Island GIS Map, the property is mapped as an erosion hazard area. The pre-Olympia till and fine-grained deposits near the ground surface have a relatively high fines content and may be prone to softening or erosion when exposed to surface water. However, it is our opinion that the risk for erosion can be adequately mitigated during and after construction, provided our recommendations presented in this report are incorporated into the project plans and properly implemented during construction. Our recommendations for best management practices to reduce the risk of erosion during construction can be seen in sections 8.3 Surface Erosion and Drainage Considerations and 8.4 Wet Weather Construction.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 Seismic Design Parameters

The seismic design of the building may be accomplished using the 2015 or later editions of the International Building Code (IBC), which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years). Table 1 below presents the seismic design parameters in accordance with the 2015 IBC, which are consistent with the 2008 USGS seismic hazard maps. For design purposes, a Site Class D is considered appropriate for the project site. If 2018 IBC will be used for the project, PanGEO should be contacted.

Table 2 – Summary Seismic Design Parameters per 2015 IBC						
Site Class	Spectral Acceleration at 0.2 sec. (g) S_S	Spectral Acceleration at 1.0 sec. (g) S_1	Site Coefficients		Design Spectral Response Parameters	
			F_a	F_v	S_{DS}	S_{D1}
D	1.36	0.524	1.00	1.5	0.907	0.524

6.2 Conventional Footing Recommendations

Based on the results of our test borings, dense glacial till to very stiff silt and clay are anticipated at the anticipated foundation subgrade elevations for the proposed house. As such, it is our opinion that conventional footings are appropriate to support the new foundations and site retaining walls. Our recommendations for conventional footings are presented below.

6.2.1 Allowable Bearing Pressure

Conventional footings may be sized using a maximum allowable bearing pressure of 4,000 psf, assuming the new footings will be founded on undisturbed native soils, or on compacted structural fill placed on native soils. The recommended allowable bearing pressure is for dead plus live loads. For allowable stress design, the recommended bearing pressure may be increased by one-third for transient loading, such as wind or seismic forces. Spread and continuous footings should have minimum widths of 24 and 18 inches, respectively.

6.2.2 Lateral Resistance

Lateral forces from un-balanced soil loads, wind or seismic loading may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundations and walls, and by friction acting on the base of the foundations. Passive resistance may be determined using an equivalent fluid weight of 350 pounds per cubic foot (pcf) for level backfill. This value includes a factor safety of at least 1.5 assuming that properly compacted structural fill will be placed adjacent to the sides of the footings, per Section 8.2 Structural Fill and Compaction. A friction coefficient of 0.35 may be used to determine the frictional resistance at the base of the footings. This coefficient includes a factor of safety of approximate 1.5. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches of soil should be neglected.

6.2.3 Footing Subgrade Preparation

Footings should bear directly on the native and undisturbed glacial soils expected to be encountered at the footing subgrade elevation, on compacted structural fill, or on lean-mix concrete placed on undisturbed native soils.

Based on the presence of 7 feet of fill in our boring PG-4 near the northwest side of the site, some over-excavation may be necessary to reach bearing soils along the downslope side of the development.

It should be noted that that the site soils are highly moisture sensitive, and can be easily disturbed and softened when exposed to moisture. Any loose or softened soil should be removed from the footing excavations and backfilled with structural fill or lean-mix concrete. The adequacy of the footing subgrade should be verified by a representative of PanGEO, prior to placing forms or rebar.

6.2.4 Foundation Performance

Total and differential settlements are anticipated to be within tolerable limits for footings designed and constructed as discussed above. Footing settlement under static loading conditions is estimated to be less than approximately ½ inch. Most settlement will occur during construction as loads are applied.

6.3 RETAINING WALL DESIGN PARAMETERS

Site retaining and basement walls must be designed to resist the lateral earth pressures exerted by the soils behind the walls. Adequate drainage provisions should also be provided behind the new walls to intercept and remove groundwater or surface water that may accumulate behind the wall.

Our geotechnical recommendations for the design and construction of retaining and below grade walls are presented below:

6.3.1 Lateral Earth Pressure

Cantilevered retaining walls should be designed for an active earth pressure of 35 pcf for walls with a level backslope and 45 pct for walls with a backslope (i.e. all walls retaining soils along the south slope).

Basement walls should be design for an at-rest equivalent fluid pressure of 45 pcf for walls built against shoring (i.e., soldier pile wall). These values assume the existing site slopes will remain relatively unchanged.

In addition, the walls should be designed for a uniform lateral pressure of 12H pounds square foot (psf) for seismic loading, where H corresponds to the retained height of the wall. The recommended lateral pressures assume that the backfill behind the wall consists of a free draining and properly compacted fill with adequate drainage provisions.

6.3.2 Lateral Resistance

Lateral forces from wind or seismic loading and unbalanced lateral earth pressures may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundations. See Section 6.2.2 Lateral Resistance for our recommended parameters for lateral resistance.

6.3.3 Wall Surcharge

Surcharge loads, where present, should also be included in the design of basement or retaining walls. We recommend that a lateral load coefficient of 0.35 be used to compute the lateral pressure on the wall face resulting from surcharge loads located within a horizontal distance of one-half of the wall height.

6.3.4 Wall Drainage

We recommend that perimeter wall/footing drains be installed to provide permanent control of subsurface water adjacent to the new structures. As a minimum, 4-inch diameter perforated drainpipes should be installed next to the base of the footings and embedded in 12 to 18 inches of clean gravel. The gravel should be wrapped in a geotextile filter fabric to prevent the migration of fines into the drain system. The drainpipe should be graded to direct water to a suitable outlet. New footing drains may be tied into the existing footing drain system.

For walls constructed against temporary soldier pile walls, we recommend weep pipes be placed between each soldier pile, connected to the soldier pile wall face, and tied into the perimeter footing drains.

Where applicable, in-lieu of conventional footing drains, weep holes (2-inch diameter at maximum 10 feet on center) may be used for site retaining walls. A minimum 18-inch wide zone of free draining granular soils (i.e. washed rock or equivalent) is recommended to be placed adjacent to the wall for the full height of the wall. Alternatively, a composite drainage material, such as Miradrain 6000, may be used in lieu of the clean crushed rock.

Waterproofing considerations are beyond our expertise and scope of work. We recommend that a building envelope specialist be consulted to determine appropriate damp-proofing or water-proofing measures.

6.3.5 Wall Backfill

The existing on-site soil has high fines content and is moisture sensitive. In our opinion, the on-site soils are not suitable for use as wall backfill. Wall backfill should consist of imported free draining granular soils, such as WSDOT Gravel Borrow (*WSDOT Standards and Specifications,* 2020, 9-03.14(1)), or approved equivalent.

Wall backfill should be properly moisture conditioned, placed in loose, horizontal lifts less than 8 to 12 inches in thickness, and systematically compacted to a dense and relatively unyielding condition. The adequacy of the wall backfill should be verified by PanGEO during construction.

6.4 CONCRETE SLAB

Conventional on-grade concrete slabs may be utilized for this project. Interior concrete slab-on-grade floors should be underlain by a capillary break consisting of at least of 4 inches of compacted ¾-inch, clean crushed gravel (less than 3 percent fines). The capillary break material should also have no more than 10 percent passing the No. 4 sieve and less than 5 percent by weight of the material passing the U.S. Standard No. 100 sieve. The capillary break should be placed on the dense subgrade or subgrade that has been compacted to a dense and unyielding condition. A minimum 10-mil polyethylene vapor barrier should also be placed directly below the interior slab.

Capillary break should be placed over undisturbed dense glacial soils. If soils are observed to be loose or softened, we recommend removing the disturbed soils and replacing with compacted structural fill, per Section 8.2 Structural Fill Placement and Compaction.

6.5 Permanent Slopes

It is our opinion that permanent slopes should be graded no steeper than 2H:1V. It is also our opinion that permanent slopes against the foundation or retaining walls should be graded no steeper than 3H:1V.

7.0 EXCAVATION AND SHORING RECOMMENDATIONS

7.1 Temporary Unsupported Slope Cuts

All temporary excavations deeper than a total height of 4 feet should be sloped or shored. Where space is available, it is our opinion that unsupported open cut excavations are feasible at the site. Based on the soil conditions at the site, for planning purposes, it is our opinion that temporary excavations may be sloped as steep as 1H:1V along the north, east, and west sides of the excavation. We do not recommend unsupported open cuts along the toe of the south slope due to the risk for slope instability.

Where space is limited, the use of L-shaped footings may be considered to reduce the lateral extent of the proposed excavation.

All temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. The temporary excavations and cut slopes should be re-evaluated in the field during construction based on actual observed soil conditions and may need to be flattered in the wet reasons and should be covered with plastic sheets. The cut slopes should be covered with plastic sheets in the raining season. We also recommend that heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a distance equal to 1/3 the slope height from the top of any excavation.

7.2 SOLDIER PILE SHORING WALL

Where space is not available for unsupported slope cuts, soldier piles and timber lagging are considered appropriate to support the excavation. It is our opinion that soil nails are not appropriate due to the risk of global slope instability during excavation.

A soldier pile wall consists of vertical steel beams, typically spaced from 6 to 8 feet apart along the proposed excavation wall, spanned by timber lagging. Prior to the start of excavation, the steel beams are installed into holes drilled to a design depth and then backfilled with lean mix concrete. As the excavation proceeds downward and the steel piles are subsequently exposed, timber lagging is installed between the piles to further stabilize the walls of the excavation.

In order to achieve a cost-effective design and to limit pile deflections, internal supports such as tiebacks or rakers are typically utilized for soldier piles taller than about 10 feet. Due to the height of the proposed excavation (as much as 18 feet deep), we anticipate one level of tiebacks/rakers may be needed in areas where the grade is highest.

The shoring system should be designed to provide adequate protection for the workers, adjacent structures, utilities, and other facilities. Excavations should be performed in accordance with the current requirements of WISHA. Construction should proceed as rapidly as feasible, to limit the time temporary excavations are open.

7.2.1 Design Lateral Pressures

We recommend that the earth pressures depicted on Figure 5 be used for design of soldier pile wall. Above the bottom of excavation, the active and surcharge pressures should be applied over the full width of pile spacing. Below the bottom of excavation, the active and surcharge pressures should be applied over one pile diameter, and the passive resistance should be applied over two times the pile diameter.

Lagging design recommendations are also included on Figure 5.

The lateral earth pressures shown on the figure should be increased for any surcharge loads resulting from traffic, construction equipment, building loads or excavated soil if they are located within the height dimension of the wall. Heavy point loads such as outriggers for concrete pump trucks and cranes may apply additional loads to the lagging. These loads should be individually analyzed and where appropriate should be included in the shoring design calculations.

We recommend a minimum pile embedment of 16 feet along the south wall based on the results from our slope stability analysis during the temporary excavation (see Figure 4a). Deeper pile embedment may be needed based on structural calculations.

7.2.2 Vertical Soldier Pile Capacity

We recommend the vertical capacity of the soldier piles be determined using an allowable skin friction value of 0.5 ksf for the portion of the pile below the bottom of the excavation, and an allowable end bearing value of 15 ksf.

7.2.3 Tieback Parameters

Tieback anchors may be utilized to reduce the size and length of soldier piles for excavation shoring greater than about 10 to 12 feet tall. Although soldier piles may also be internally supported by braces or rakers, such construction methods will be significantly more costly than tiebacks and will impact the construction sequence. Tiebacks are the preferred method, provided that a temporary construction easement can be obtained from your neighbors.

The manner in which the tieback anchors carry load will depend on the type of anchor selected, the method of installation, and the soil conditions surrounding the anchor. Accordingly, we recommend use of a performance specification requiring the shoring contractor to install anchors capable of satisfactorily achieving the design structural loads, with a pullout resistance factor of safety of 2.0.

For planning purposes, however, the anchors may be sized using an assumed allowable skin friction value of 2.5 kips per lineal foot of anchor bond length, assuming that small diameter (about 6 inches) pressure-grouted tiebacks will be used. Pressure grouting and multiple post-grouting may be needed in order to achieve the assumed capacity. If the contractor believes that, based on their proposed installation method in similar soil conditions, the assumed value should be revised the tieback lengths should be revised accordingly. In the tieback construction, a bond breaker shall be constructed in the no load zone when the installation procedures use single stage grouting.

The bond zone portion of the tiebacks must be located behind a no-load zone as defined in Figure 4. The tiebacks should have a minimum bond length of 15 feet beyond the no-load zone; longer tiebacks may be needed based on the design calculations.

Excessive pile top deflection could occur before the first row of tiebacks is installed. To improve the performance of the tieback wall, it may be necessary to limit the first row of tiebacks to no more than about 10 feet below pile top unless steel beams of sufficient size will be used to limit the magnitude of the cantilever deflection.

7.2.4 Tieback Testing – Verification Test

The actual capacity of the anchors should be confirmed with verification tests that test the tiebacks up to 200 percent of the design load. The anchor testing should be conducted in accordance with the latest edition of the Post-Tensioning Institute (PTI) *Recommendations for Prestressed Rock and Soil Anchors*. Verification testing procedure should adhere to the following recommendations:

- Prior to installing production anchors, perform a minimum of two tests each on each anchor type, installation method, and soil type with the tested anchors constructed to the same dimensions as production anchors. Contractor may choose to install the test anchors as part of the production anchors at its own risk.
- Test locations to be determined in conjunction and approved by the geotechnical engineer.
- Verification test anchors, which will be loaded to 200 percent of the design load, may require additional steel tendons so that the stress will not exceed 80 percent of the ultimate tensile strength.
- The verification test anchors should be loaded to a maximum 200 percent design load in 25 percent load increments, holding each incremental load for at least 5 minutes and recording deflection of the anchor head at various times within each hold to the nearest 0.01 inch.
- At the 150 percent design load, the holding period shall be at least 60 minutes.
- At the 200 percent design load, the holding period shall be for at least 10 minutes.
- An acceptable test shall provide a measured creep rate of 0.04 inches or less at the 150 percent load between 1 and 10 minutes, and 0.08 inches between 6 and 60 minutes, and both shall have a creep rate that is linear or decreasing with time. The applied load must remain constant during all holding periods (i.e. no more than 5 percent variation from the specified load).

Verification tested anchors or extended creep proof tested anchors not meeting the acceptance criteria will require a redesign by the contractor to achieve the acceptance criteria.

7.2.5 Tieback Testing – Proof Test

All production anchors should be proof tested as outlined below:

- Load test all production anchors to 130 percent of the design load in 25 percent load increments, holding each incremental load until a stable deflection is achieved (record deflection of the anchor head at various times within each hold to the nearest 0.01 inch).
- At the 130 percent design load, the holding period shall be at least 10 minutes.
- An acceptable test shall provide a measured creep rate of 0.04 inches or less at the 130 percent design load between 1 and 10 minutes. The creep rate must be linear or decreasing with time. The applied load must remain constant during the holding period (i.e. no more than 5 percent variation from the specified load). Anchors failing this proof testing creep acceptance criteria may be held an additional 50 minutes for creep measurement. Acceptable performance would equate to a creep of 0.08 inches or less between 6 and 60 minutes with a linear or decreasing creep rate.

7.2.6 Groundwater, Caving, and Obstruction Considerations

Based on the anticipated excavation depths, we do not anticipate soldier pile or tieback drilling to extend into water bearing soil layers. However, given that our subsurface investigation was conducted during the dry season, the contractor should be prepared to stabilize the holes if groundwater or caving conditions are encountered. This includes the use of drilling mud and temporary casings. Where more than 6 inches of groundwater are present in the bottom of the drilled soldier pile holes, the concrete should be placed using a tremie pipe. When placing timber lagging, the height of each lift may need to be limited if wet soils are encountered. The actual allowable vertical cut for timber lagging placement should be determined in the field, based on the actual conditions observed.

We recommend that temporary casings be used to install tiebacks to keep holes open and to mitigate the risk of ground loss beyond the excavation area.

It should also be noted that large cobbles and boulders are known to be present in till and glacial soils. As such, obstructions due to large cobbles and boulders may be encountered during drilling for soldier piles and tiebacks. If obstructions cannot be cleared with typical drilling methods, alternative locations and sizes for soldier piles and tiebacks should be considered.

7.2.7 Performance Monitoring

Ground movements will occur as a result of excavation activities. As such, adjacent building and ground surface elevations of the adjacent properties should be documented prior to commencing earthwork to provide baseline data. After installation of soldier piles but prior to mass excavation, establish monitoring points for baseline readings at the top of every other soldier pile and adjacent building house to the southwest. The monitoring points shall be monitored at least twice weekly for vertical and horizontal displacement during shoring installation and excavation. Survey data should be submitted to the project team each week to verify the performance of the shoring.

The optical survey frequency may be decreased after completion of perimeter footings, if the data indicates no or little additional movement. Surveying must continue until the permanent structure is completed up to the permanent grades.

We also recommend that the existing conditions along the city streets and the adjacent private properties be photo-documented prior to commencing on any earthworks at the site.

7.3 DEMOLITION CONSIDERATIONS

Prior to demolition activities, the structural engineer and contractor should evaluate the planned demolition sequence of the existing house basement. Removing the existing building diaphragm without adequate support of the existing basement walls could potentially destabilize the existing south slope. As such, the demolition plan should consider how to support the existing basement walls prior to the installation of the temporary soldier pile wall, such as internal bracing or soil buttresses.

8.0 EARTHWORK CONSIDERATIONS

8.1 Material Reuse

In the context of this report, structural fill is defined as compacted fill placed under footings, concrete stairs and landings, and slabs, or other load-bearing areas. In our opinion, the on-site soils contain a high fines content and are not suitable to be reused as structural fill. Suitable material for use as structural fill are described in Section 8.2 below.

The on-site soil can be used as general fill in non-structural and landscaping areas. If use of the on-site soil is planned, the excavated soil should be stockpiled and protected with plastic sheeting to prevent softening from rainfall in the wet season.

8.2 Structural Fill Placement and Compaction

For planning purpose, structural fill should consist of imported, well-grade, granular material, such as WSDOT Gravel Borrow (*WSDOT Standards and Specifications 2020*, 9-03.14(1)), or an approved equivalent. Based on the presence of perched groundwater relatively close to the ground surface, recycled crushed concrete should not be considered as a source of structural fill.

Structural fill should be properly moisture conditioned, placed in loose, horizontal lifts up to 12 inches in thickness, and systematically compacted to a dense and relatively unyielding condition, as verified by PanGEO personnel. If soil density tests will be performed, the test results should indicate at least 95 percent of the maximum dry density, as determined using test method ASTM D 1557. Within 5 feet of the basement or retaining walls, backfill should be compacted to 90 percent of the maximum dry density.

Depending on the type of compaction equipment used and depending on the type of fill material, it may be necessary to decrease the thickness of each lift in order to achieve adequate compaction. PanGEO can provide additional recommendations regarding structural fill and compaction during construction.

8.3 Surface Erosion and Drainage Considerations

Surface runoff can be controlled during construction by careful grading practices. Typically, this includes the construction of shallow, upgrade perimeter ditches or low earthen berms in conjunction with silt fences to collect runoff and prevent water from entering excavations or to prevent runoff from the construction area leaving the immediate work site.

Temporary erosion control may require the use of hay bales on the downhill side of the project to prevent water from leaving the site. Stormwater detention may be needed to trap sand and silt before the water is discharged to a suitable outlet. All collected water should be directed under control to a positive and permanent discharge system.

8.4 WET WEATHER CONSTRUCTION

It is our opinion that construction of the project can be accomplished during the wet season (October to April). However, performing earthwork activities during the wet season may be costlier than during dry weather conditions. The following procedures are the best management practices recommended for use in wet weather construction:

- All footing subgrades should be protected against inclement weather, unless the footings can be poured immediately after the subgrade is exposed. The contractor should be aware that the site soils are moisture sensitive due to its high fines content and could become disturbed and softened when exposed to inclement weather conditions. It is the contractor’s responsibility to protect the subgrade from disturbance. One option is to place 2 to 3 inches of lean-mix concrete or 4 to 6 inches of crushed surfacing base course on the newly exposed subgrade as soon as it is exposed;
- During wet weather, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight based on the portion passing the 0.75-inch sieve. The fines should be non-plastic;
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water;
- Geotextile silt fences should be installed at strategic locations around the construction area to control erosion and the movement of soil; and
- Excavation slopes and soils stockpiled on site should be covered with plastic sheeting.

9.0 ADDITIONAL SERVICES

To confirm that our recommendations are properly incorporated into the design and construction of the proposed building, PanGEO should be retained to conduct a review of the final project plans and specifications, and to monitor the construction of geotechnical elements. The City of Mercer Island, as part of the permitting process, may also require geotechnical construction inspection services. PanGEO can provide you a cost estimate for construction monitoring services at a later date.

10.0 CLOSURE

We have prepared this report for Ahbleza Pattison and the project design team. Recommendations contained in this report are based on a site reconnaissance, a subsurface exploration program, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of services.

Variations in soil conditions may exist between the locations of the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at the site that are different from those described in this report, we should be notified immediately to review the applicability of our recommendations. Additionally, we should also be notified to review the applicability of our recommendations if there are any changes in the project scope.

The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design. Additionally, the scope of our services specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances. We are not mold consultants nor are our recommendations to be interpreted as being preventative of mold development. A mold specialist should be consulted for all mold-related issues.

This report has been prepared for planning and design purposes for specific application to the proposed project in accordance with the generally accepted standards of local practice at the time this report was written. No warranty, express or implied, is made.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or other factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 months from its issuance. PanGEO should be notified if the project is delayed by more than 24 months from the date of this report so that we may review the applicability of our conclusions considering the time lapse.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use this report.

Sincerely,

PanGEO Inc.

11/23/2020

11.23.2020

Bryce Townsend, P.E.
Project Geotechnical Engineer

Siew L. Tan, P.E.
Principal Geotechnical Engineer

11.0 REFERENCES

International Code Council, 2015, *International Building Code (IBC), 2015.*

Laprade, W.T., Harp, E.L., Michael, J.A., 2006, *Shallow-Landslide Hazard Map of Seattle, Washington*, USGS Open-File Report 2006-1139, p. 6-8.

Post-Tensioning Institute, 2014, *Recommendations for Prestressed Rock and Soil Anchors.*

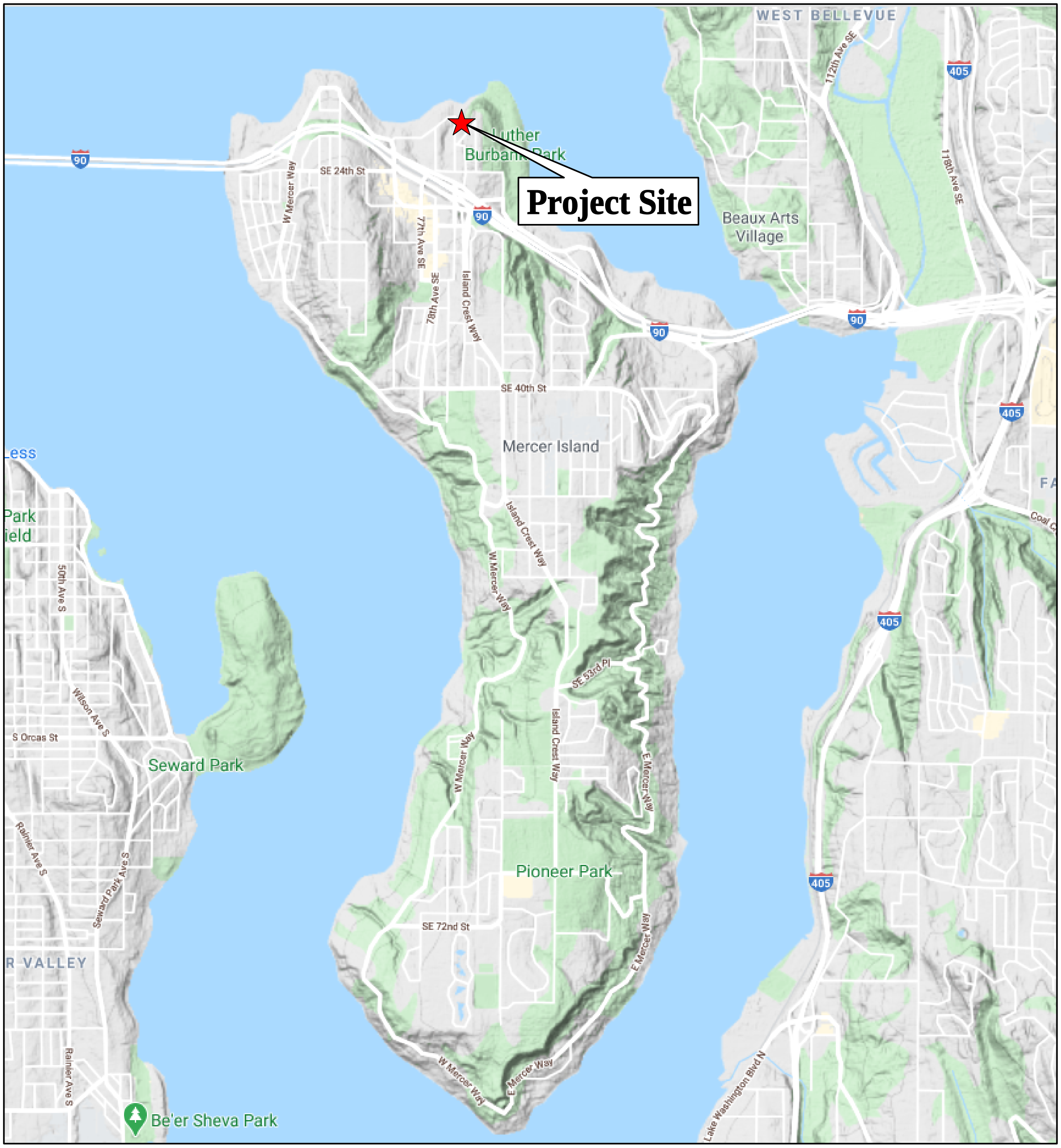
Troost, K.G., Wisher, A. P., 2006, *Geologic Map of Mercer Island, Washington.*

Troost, K.G., Wisher, A. P., 2009, *Mercer Island Landslide Hazard Assessment, Washington.*

United States Geological Survey, *Earthquake Hazards Program, Interpolated Probabilistic Ground Motion for the Conterminous 48 States by Latitude and Longitude, 2008 Data.*

Washington Administrative Code (WAC), 2013, Chapter 296-155 - *Safety Standards for Construction Work, Part N - Excavation, Trenching, and Shoring*.

WSDOT, 2020, *Standard Specifications for Road, Bridge and Municipal Construction, M 41-10.*



Base Map: Google Terrain Map

Approx. Scale:
Not to Scale

20-332 Figure1- Vicinity Map 9/14/20 (11:23:04)

PanGEO INCORPORATED	**Proposed Residence** **8019 SE 20th St.** **Mercer Island, WA**	**VICINITY MAP**	
		Project No. **20-332**	Figure No. **1**

Legend:

Approx. Borehole Location

Approx. Proposed Development Boundary

Location of Subsurface Profiles (see Figure 3)

Base Map: Topographic Survey by CHADWICK & WINTERS, 06/12/2020.

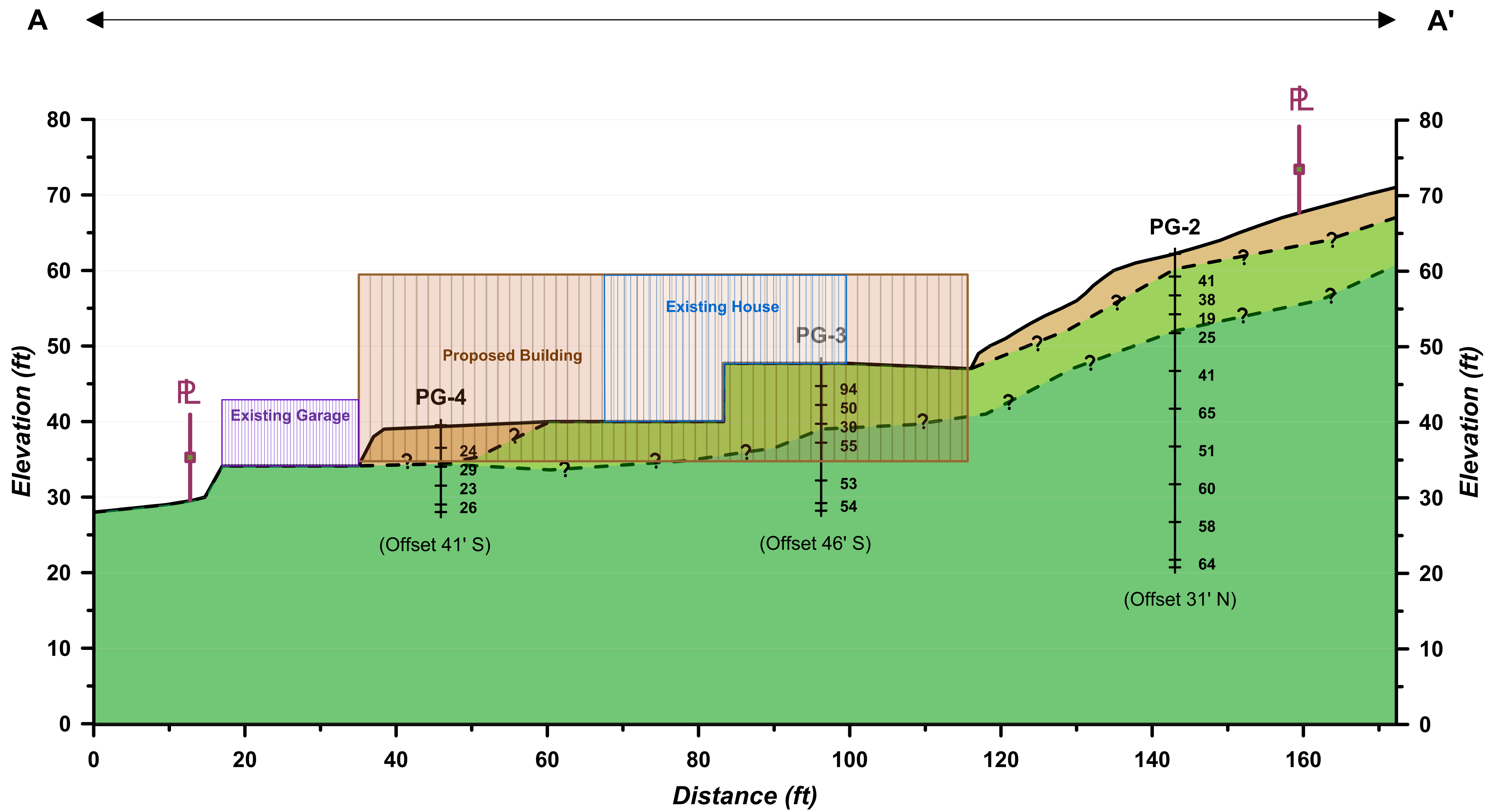
PanGEO INCORPORATED

Proposed Residence
8019 SE 20th St.
Mercer Island, WA

SITE AND EXPLORATION PLAN

Project No.	Figure No.
20-332	2

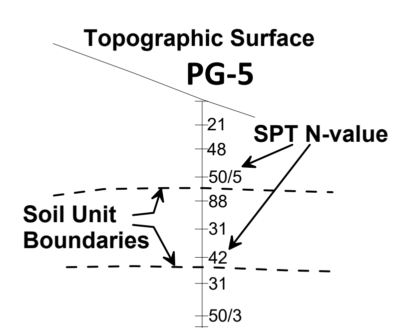
20-332 Figure 2- Site and Exploration Plan 11/19/20 (4:47:31)



Legend:

- Fill
- Pre-Olympia Glacial Till (Qpogt)
- Pre-Olympia Fine-Grained Deposits (Qpof)

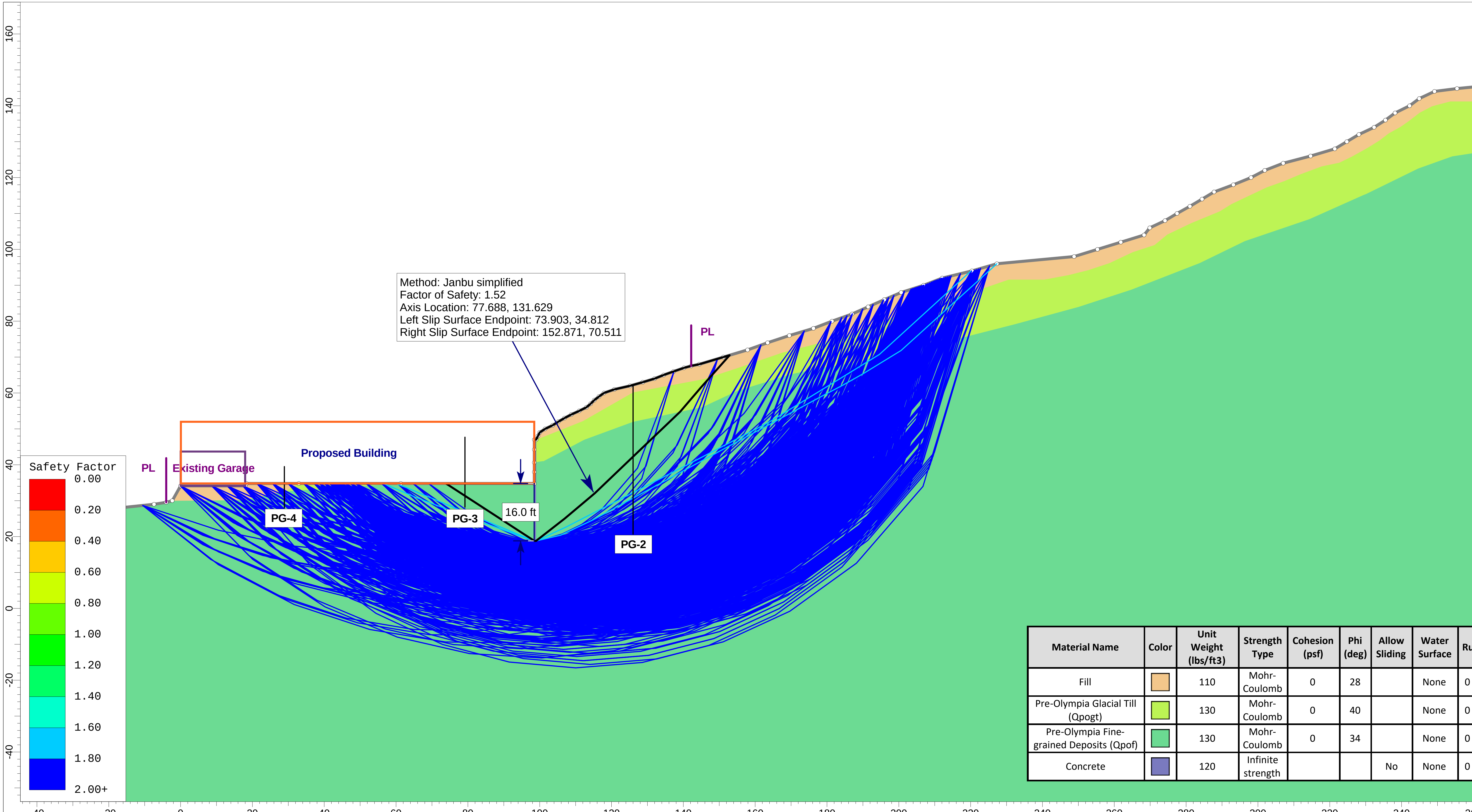
Graphics Legend



Notes:
1. Ground surface elevations based on the survey by CHADWICK & WINTERS, June 2020.
2. See Figure 2 for location of Section A-A'.
3. See report text for additional discussions on subsurface conditions.
4. The generalized soil profile is based on widely-spaced borings and probes. Soil conditions may vary over a small distance, and the actual subsurface conditions may be different from the generalized soil profile depicted in this figure.

PanGEO INCORPORATED	Proposed Residence 8019 SE 20th St. Mercer Island, WA 98040	GENERALIZED SUBSURFACE PROFILE A-A'
	Project No. 20-332	Figure No. 3

20-332 Fig.3- Subsurface Profile 11/19/20 (4:17:19)



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Ru
Fill		110	Mohr-Coulomb	0	28		None	0
Pre-Olympia Glacial Till (Qpogt)		130	Mohr-Coulomb	0	40		None	0
Pre-Olympia Fine-grained Deposits (Qpof)		130	Mohr-Coulomb	0	34		None	0
Concrete		120	Infinite strength			No	None	0

PanGEO INCORPORATED

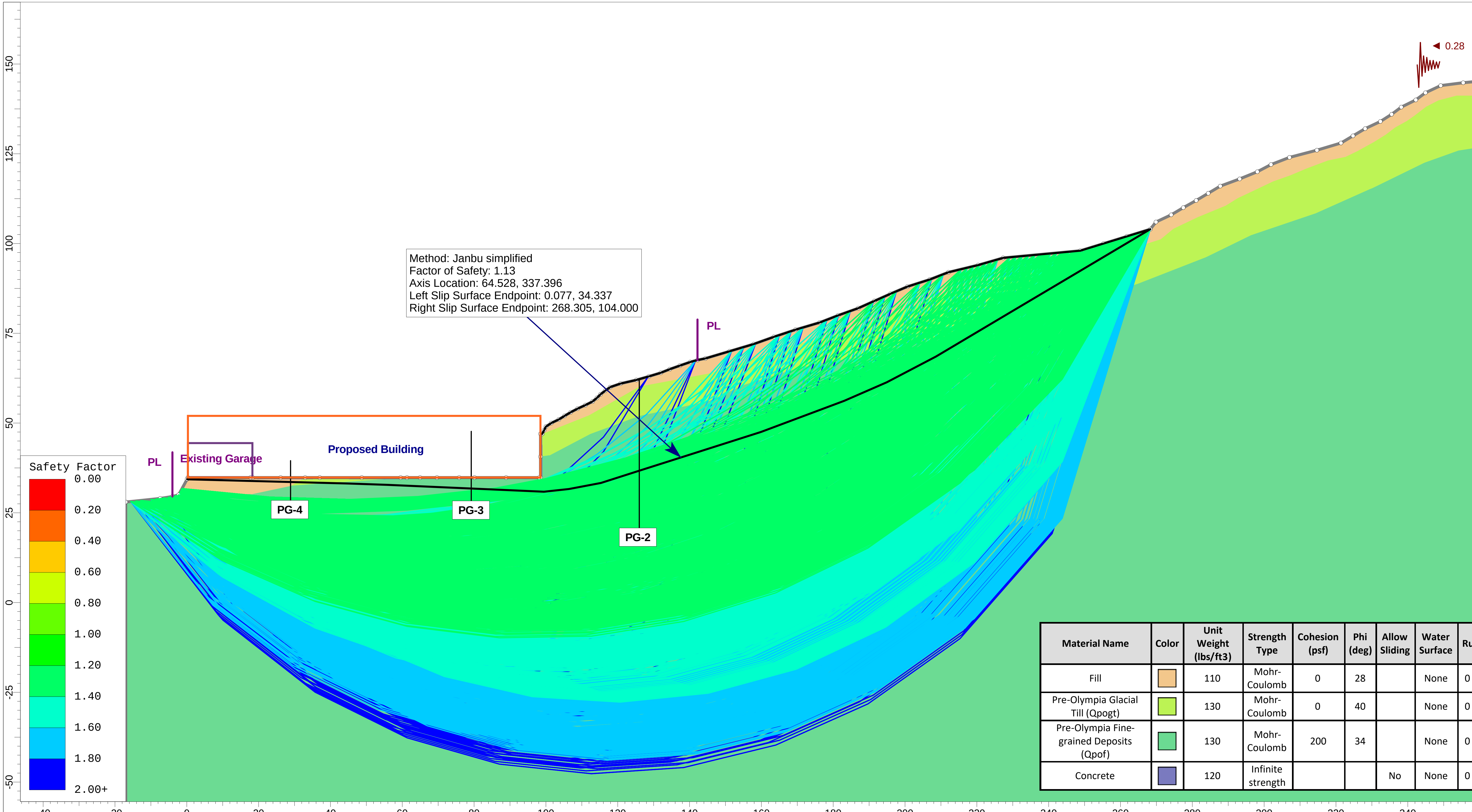
SLIDEINTERPRET 9.006

Proposed Residence
8019 SE 20th St.
Mercer Island, Washington

Static Global Stability Analysis

Section A-A'

Scale:	Project No.	Figure No.
1:300	20-332	4A



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Ru
Fill		110	Mohr-Coulomb	0	28		None	0
Pre-Olympia Glacial Till (Qpogt)		130	Mohr-Coulomb	0	40		None	0
Pre-Olympia Fine-grained Deposits (Qpof)		130	Mohr-Coulomb	200	34		None	0
Concrete		120	Infinite strength			No	None	0

PanGEO INCORPORATED

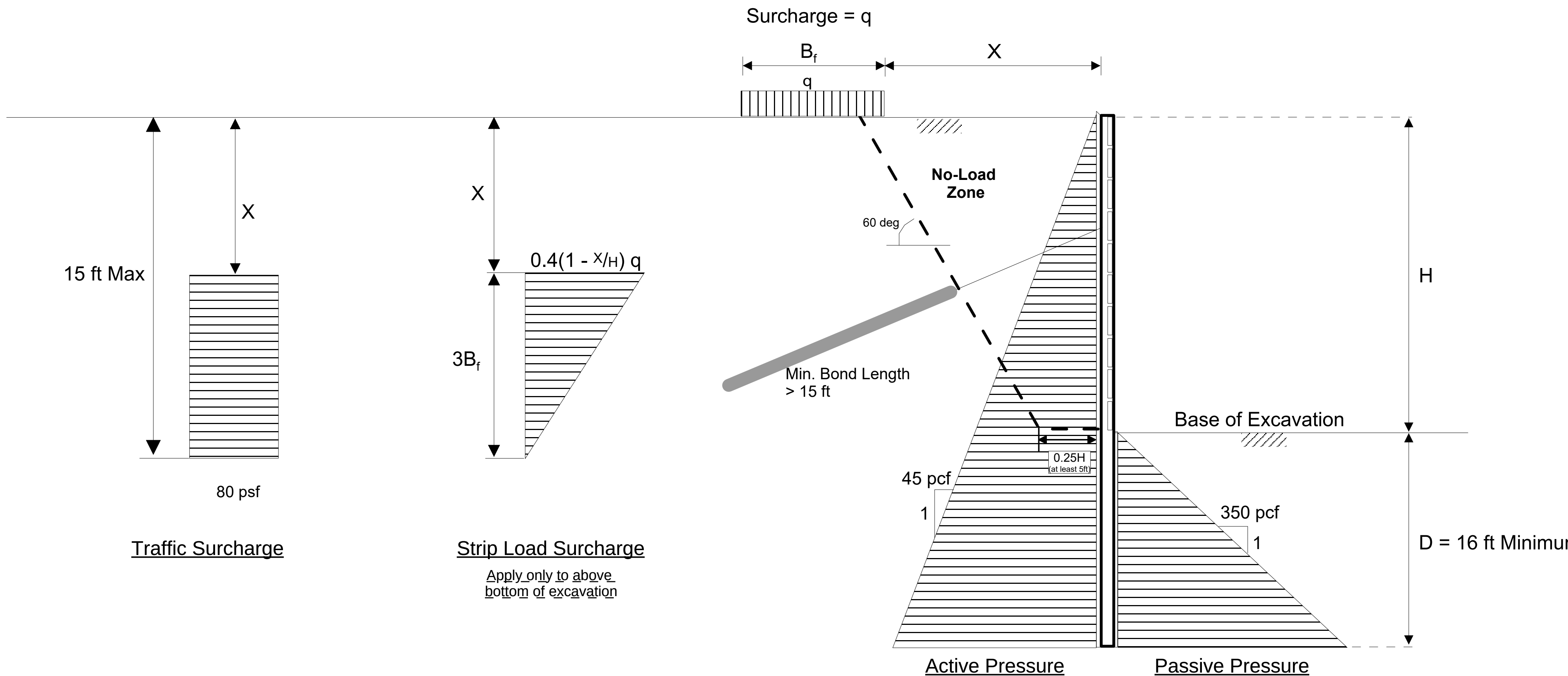
SLIDEINTERPRET 9.008

Proposed Residence
8019 SE 20th St.
Mercer Island, Washington

Pseudo-Static Stability Analysis

Section A-A'

Scale:	Project No.	Figure No.
1:300	20-332	4B



Vertical Soldier Pile Capacity: Allowable Skin Friction = 500 psf
Allowable End Bearing = 15 ksf

Notes:
1. Minumum embedment should be at least 16 feet below bottom of excavation, per results of our slope stability analysis (see Figure 5a).
2. A factor of safety of 1.5 has been applied to the recommended passive pressure values. No factor of safety has been applied to the recommended active earth pressure values.
3. Active pressures should be applied over the full width of the pile spacing above the base of the excavation, and over one pile diameter below the base of the excavation.
4. Surcharge pressures should be applied over the entire length of the loaded area.
5. Passive pressure should be applied to two times the diameter of the soldier piles.
6. Use 50% of the active and surcharge pressures for lagging design with soldier piles spaced at 8' or less.
7. Refer to report text for additional discussions.

PanGEO INCORPORATED

Proposed Residence
8019 SE 20th St.
Mercer Island, WA

DESIGN LATERAL PRESSURES
SOLDIER PILE WALL
CANTILEVERED AND ONE LEVEL TIEBACK

Project No. **20-332** Figure No. **5**

20-332 Fig 4-A Soldier Pile EP +Tieback.grf 11/23/20 (5:31:36) STS

APPENDIX A

SUMMARY BORING LOGS

RELATIVE DENSITY / CONSISTENCY

SAND / GRAVEL			SILT / CLAY		
Density	SPT N-values	Approx. Relative Density (%)	Consistency	SPT N-values	Approx. Undrained Shear Strength (psf)
Very Loose	<4	<15	Very Soft	<2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Med. Dense	10 to 30	35 - 65	Med. Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	>50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	>30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		GROUP DESCRIPTIONS	
Gravel 50% or more of the coarse fraction retained on the #4 sieve. Use dual symbols (eg. GP-GM) for 5% to 12% fines.	GRAVEL (<5% fines)	GW	Well-graded GRAVEL
		GP	Poorly-graded GRAVEL
	GRAVEL (>12% fines)	GM	Silty GRAVEL
		GC	Clayey GRAVEL
Sand 50% or more of the coarse fraction passing the #4 sieve. Use dual symbols (eg. SP-SM) for 5% to 12% fines.	SAND (<5% fines)	SW	Well-graded SAND
		SP	Poorly-graded SAND
	SAND (>12% fines)	SM	Silty SAND
		SC	Clayey SAND
Silt and Clay 50%or more passing #200 sieve	Liquid Limit < 50	ML	SILT
		CL	Lean CLAY
		OL	Organic SILT or CLAY
	Liquid Limit > 50	MH	Elastic SILT
		CH	Fat CLAY
		OH	Organic SILT or CLAY
Highly Organic Soils		PT	PEAT

Notes: 1. Soil exploration logs contain material descriptions based on visual observation and field tests using a system modified from the Uniform Soil Classification System (USCS). Where necessary laboratory tests have been conducted (as noted in the "Other Tests" column), unit descriptions may include a classification. Please refer to the discussions in the report text for a more complete description of the subsurface conditions.

2. The graphic symbols given above are not inclusive of all symbols that may appear on the borehole logs. Other symbols may be used where field observations indicated mixed soil constituents or dual constituent materials.

DESCRIPTIONS OF SOIL STRUCTURES

Layered:	Units of material distinguished by color and/or composition from material units above and below	**Fissured:**	Breaks along defined planes
Laminated:	Layers of soil typically 0.05 to 1mm thick, max. 1 cm	**Slickensided:**	Fracture planes that are polished or glossy
Lens:	Layer of soil that pinches out laterally	**Blocky:**	Angular soil lumps that resist breakdown
Interlayered:	Alternating layers of differing soil material	**Disrupted:**	Soil that is broken and mixed
Pocket:	Erratic, discontinuous deposit of limited extent	**Scattered:**	Less than one per foot
Homogeneous:	Soil with uniform color and composition throughout	**Numerous:**	More than one per foot
		BCN:	Angle between bedding plane and a plane normal to core axis

COMPONENT DEFINITIONS

COMPONENT	SIZE / SIEVE RANGE	COMPONENT	SIZE / SIEVE RANGE
Boulder:	> 12 inches	Sand	
Cobbles:	3 to 12 inches	Coarse Sand:	#4 to #10 sieve (4.5 to 2.0 mm)
Gravel		Medium Sand:	#10 to #40 sieve (2.0 to 0.42 mm)
Coarse Gravel:	3 to 3/4 inches	Fine Sand:	#40 to #200 sieve (0.42 to 0.074 mm)
Fine Gravel:	3/4 inches to #4 sieve	Silt	0.074 to 0.002 mm
		Clay	<0.002 mm

TEST SYMBOLS

for In Situ and Laboratory Tests listed in "Other Tests" column.

ATT	Atterberg Limit Test
Comp	Compaction Tests
Con	Consolidation
DD	Dry Density
DS	Direct Shear
%F	Fines Content
GS	Grain Size
Perm	Permeability
PP	Pocket Penetrometer
R	R-value
SG	Specific Gravity
TV	Torvane
TXC	Triaxial Compression
UCC	Unconfined Compression

SYMBOLS

Sample/In Situ test types and intervals

- 2-inch OD Split Spoon, SPT (140-lb. hammer, 30" drop)
- 3.25-inch OD Spilt Spoon (300-lb hammer, 30" drop)
- Non-standard penetration test (see boring log for details)
- Thin wall (Shelby) tube
- Grab
- Rock core
- Vane Shear

MONITORING WELL

- Groundwater Level at time of drilling (ATD)
- Static Groundwater Level
- Cement / Concrete Seal
- Bentonite grout / seal
- Silica sand backfill
- Slotted tip
- Slough
- Bottom of Boring

MOISTURE CONTENT

Dry	Dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

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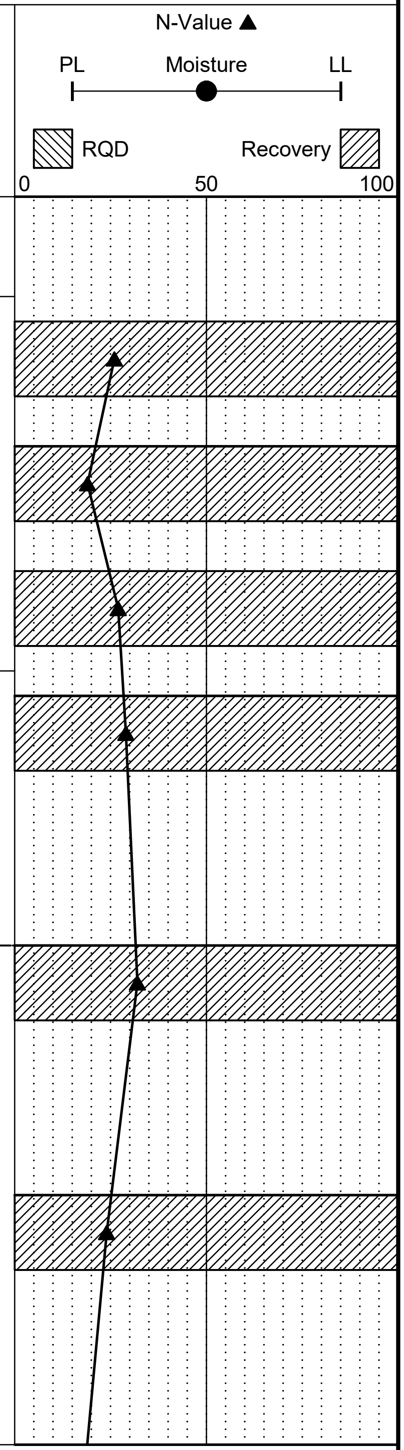
Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	Proposed Residence	Surface Elevation:	~48 ft
Job Number:	20-332	Top of Casing Elev.:	n/a
Location:	8019 SE 20th St., Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.59271, Easting: -122.23076	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						Loose, dark brown, silty SAND with organics, some gravel; moist. **[Fill].**
						Medium dense, gray-brown, silty SAND, trace gravel; moist, non-plastic fines, some iron oxide staining. **[Pre-Olympia Glacial Till - Qpogt].**
	S-1		12 11 15			
5	S-2		7 8 11			-- Increase in silt content; diamict (till-like) texture.
	S-3		7 11 16			
10	S-4		9 11 18			Very stiff, gray-brown, sandy SILT, trace clay; moist, low to moderate plasticity, minor iron oxide staining. **[Pre-Olympia Fine-Grained Deposits - Qpof].**
15	S-5		11 15 17			Very stiff to hard; gray-brown to gray, clayey SILT, trace fine sand; moist, moderate plasticity, massive.
20	S-6		8 11 13			-- Occasional fine sand partings.
25						



Completion Depth:	31.5ft
Date Borehole Started:	9/4/20
Date Borehole Completed:	9/4/20
Logged By:	S. Harrington
Drilling Company:	Geologic Drill Partners

Remarks: CAT track drill rig used. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. This surface elevation is estimated from a topographic survey by Chadwick & Winters, dated 06/12/2020. Elevations based on NAVD88.

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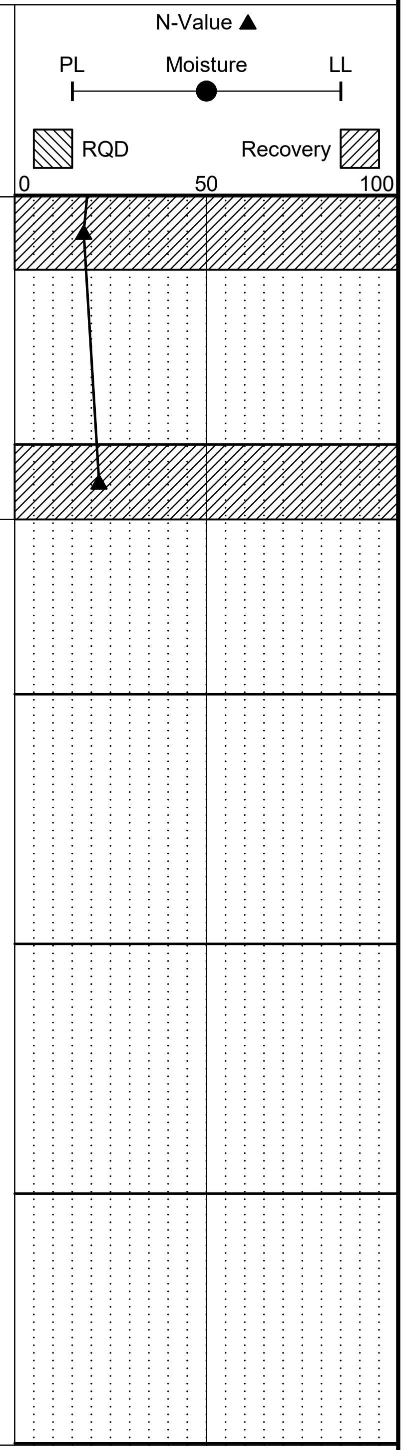
LOG OF TEST BORING PG-1

Figure A-2

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	Proposed Residence	Surface Elevation:	~48 ft
Job Number:	20-332	Top of Casing Elev.:	n/a
Location:	8019 SE 20th St., Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.59271, Easting: -122.23076	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
25	S-7		7 8 10			Very stiff to hard; gray-brown to gray, clayey SILT, trace fine sand; moist, moderate plasticity, massive. *(Continued)*
30	S-8		5 10 12			
						Boring terminated about 31.5 feet below grade. Groundwater was not encountered during drilling.
35						
40						
45						
50						



Completion Depth:	31.5ft
Date Borehole Started:	9/4/20
Date Borehole Completed:	9/4/20
Logged By:	S. Harrington
Drilling Company:	Geologic Drill Partners

Remarks: CAT track drill rig used. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. This surface elevation is estimated from a topographic survey by Chadwick & Winters, dated 06/12/2020. Elevations based on NAVD88.

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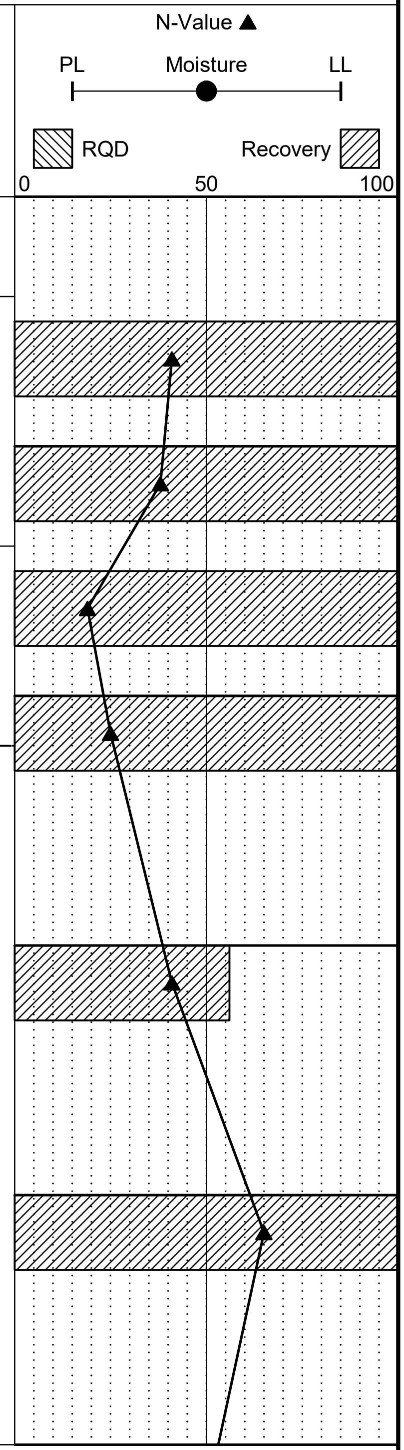
LOG OF TEST BORING PG-1

Figure A-2

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	Proposed Residence	Surface Elevation:	~60 ft
Job Number:	20-332	Top of Casing Elev.:	n/a
Location:	8019 SE 20th St., Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.59266, Easting: -122.23066	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						Loose, dark brown, silty SAND with organics, some gravel; moist. **[Fill].**
						Dense, gray-brown, silty fine SAND, tracel gravel, trace organics; moist, diamict (till-like) texture. **[Pre-Olympia Glacial Till - Qpogt].**
	S-1		15 18 23			
5	S-2		8 17 21			-- Increase in silt content.
						Very stiff, gray-brown, sandy SILT, trace clay; moist, low to moderate plasticity, minor iron oxide staining, massive to blocky texture. **[Pre-Olympia Fine-Grained Deposits - Qpof].**
	S-3		7 8 11			
10	S-4		4 9 16			
						Hard, gray, clayey SILT, trace fine sand; moist, moderate plasticity, massive to blocky texture.
15	S-5		12 21 20			
20	S-6		19 34 31			-- Very dense, silty sand interbeds; trace gravel.
25						



Completion Depth:	41.5ft
Date Borehole Started:	9/4/20
Date Borehole Completed:	9/4/20
Logged By:	S. Harrington
Drilling Company:	Geologic Drill Partners

Remarks: CAT track drill rig used. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. This surface elevation is estimated from a topographic survey by Chadwick & Winters, dated 06/12/2020. Elevations based on NAVD88.

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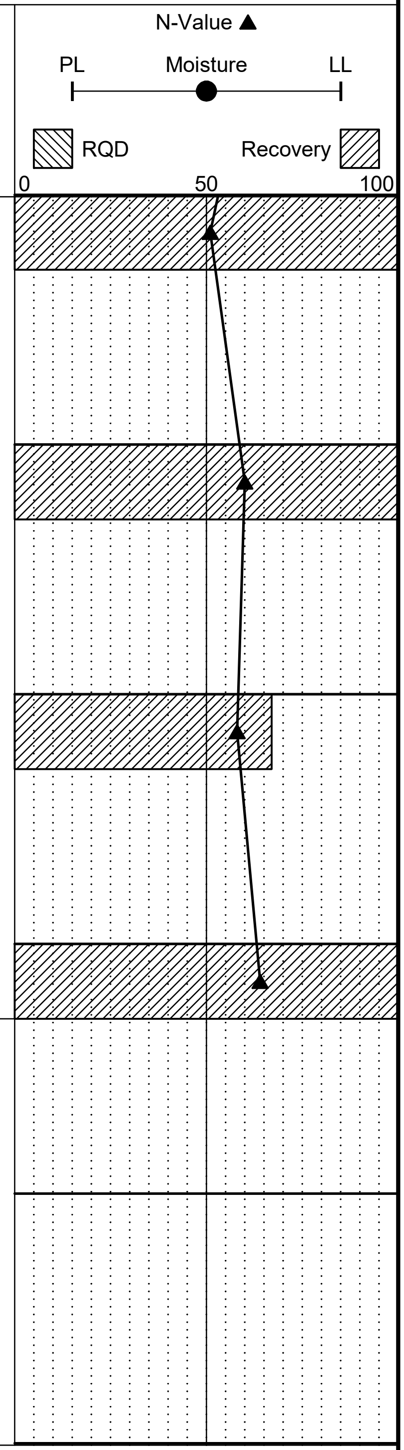
LOG OF TEST BORING PG-2

Figure A-3

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	Proposed Residence	Surface Elevation:	~60 ft
Job Number:	20-332	Top of Casing Elev.:	n/a
Location:	8019 SE 20th St., Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.59266, Easting: -122.23066	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
25	S-7		12 21 30			Hard, gray, clayey SILT, trace fine sand; moist, moderate plasticity, massive to blocky texture. *(Continued)*
30	S-8		22 28 32			
35	S-9		24 26 32			-- Occasional fine sand lenses.
40	S-10		18 24 40			
41.5						Boring terminated about 41.5 feet below grade. Groundwater was not encountered during drilling.
45						
50						



Completion Depth:	41.5ft
Date Borehole Started:	9/4/20
Date Borehole Completed:	9/4/20
Logged By:	S. Harrington
Drilling Company:	Geologic Drill Partners

Remarks: CAT track drill rig used. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. This surface elevation is estimated from a topographic survey by Chadwick & Winters, dated 06/12/2020. Elevations based on NAVD88.

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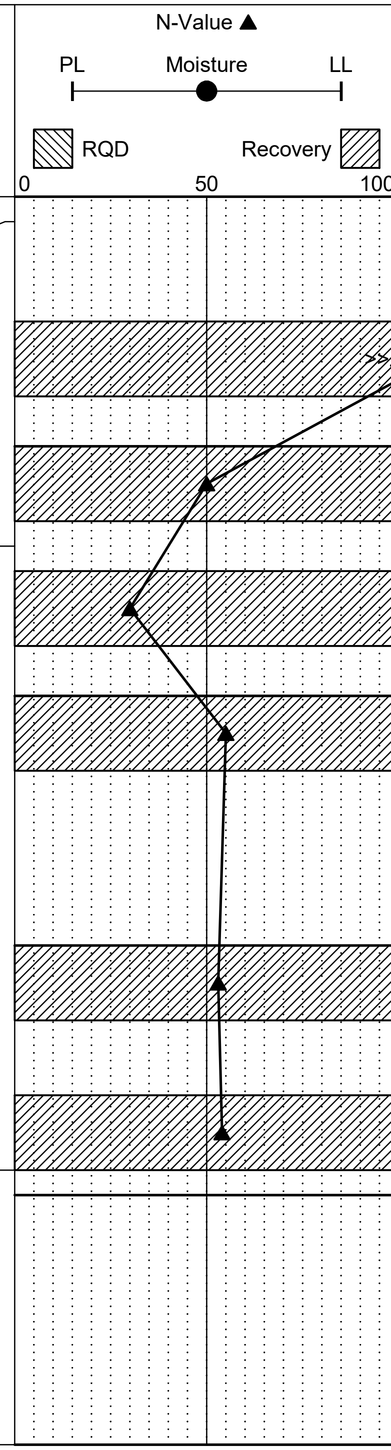
LOG OF TEST BORING PG-2

Figure A-3

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	Proposed Residence	Surface Elevation:	~46 ft
Job Number:	20-332	Top of Casing Elev.:	n/a
Location:	8019 SE 20th St., Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.59244, Easting: -122.23101	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						Approximately 6 inches of topsoil (dark brown silty sand with organics).
						Very dense, gray-brown, silty SAND with gravel; moist, diamict (till-like) texture. **[Pre-Olympia Glacial Till - Qpogt].**
	S-1		28 44 50/6			
5	S-2		19 25 25			
						Hard, gray, clayey SILT; moist, moderate plasticity, blocky texture. **[Pre-Olympia Fine-Grained Deposits - Qpof].**
	S-3		12 11 19			
10	S-4		12 22 33			
15	S-5		19 24 29			-- Occasional fine gravel.
	S-6		12 25 29			-- Becomes massive; occasional fine sand partings.
20						Boring terminated about 19.5 feet below grade. Groundwater was not encountered during drilling.
25						



Completion Depth:	19.5ft
Date Borehole Started:	9/4/20
Date Borehole Completed:	9/4/20
Logged By:	S. Harrington
Drilling Company:	Geologic Drill Partners

Remarks: CAT track drill rig used. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. This surface elevation is estimated from a topographic survey by Chadwick & Winters, dated 06/12/2020. Elevations based on NAVD88.

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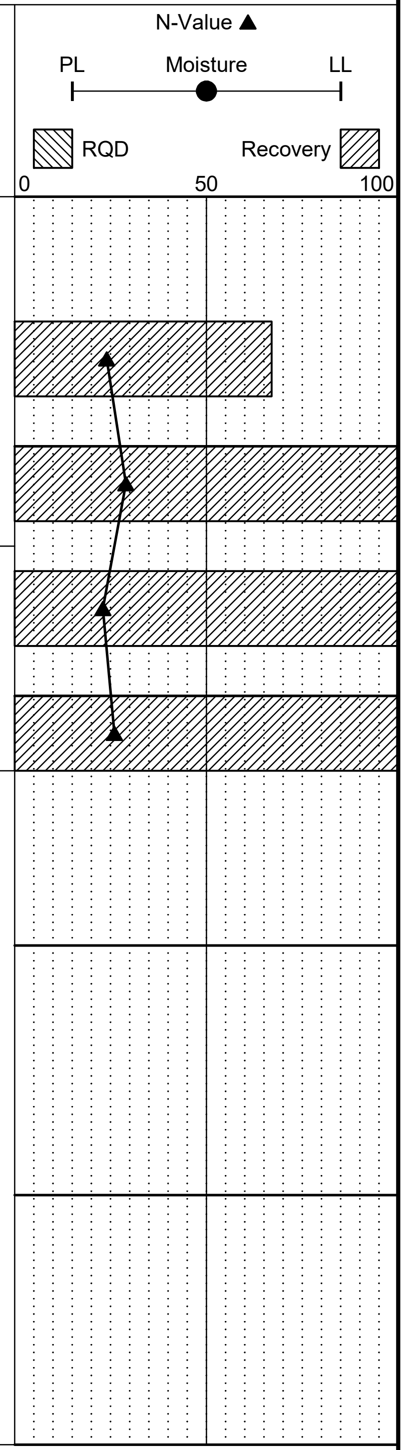
LOG OF TEST BORING PG-3

Figure A-4

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	Proposed Residence	Surface Elevation:	~42 ft
Job Number:	20-332	Top of Casing Elev.:	n/a
Location:	8019 SE 20th St., Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.59249, Easting: -122.2311	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						Loose to medium dense, light brown, silty gravelly SAND, occasional cobble; moist, minor iron oxide staining, disturbed. **[Fill].**
	S-1		16 12 12			
5	S-2		8 13 16			
						Very stiff, gray-brown, sandy SILT; moist, low plasticity, trace iron oxide staining, blocky to slightly laminated. **[Pre-Olympia Fine-Grained Deposits - Qpof].**
	S-3		6 9 14			
10	S-4		8 12 14			-- Occasional medium sand lenses.
						Boring terminated about 11.5 feet below grade. Groundwater was not encountered during drilling.
15						
20						
25						



Completion Depth:	11.5ft
Date Borehole Started:	9/4/20
Date Borehole Completed:	9/4/20
Logged By:	S. Harrington
Drilling Company:	Geologic Drill Partners

Remarks: CAT track drill rig used. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. This surface elevation is estimated from a topographic survey by Chadwick & Winters, dated 06/12/2020. Elevations based on NAVD88.

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LOG OF TEST BORING PG-4

Figure A-5

The stratification lines represent approximate boundaries. The transition may be gradual.