# GEOTECHNICAL DESIGN REPORT
# 9167 SE 64th Street
# Mercer Island, Washington

**PROJECT NO. 19-062.300**
**October 7, 2024**


Image Credit: King County imap

Prepared for:

**Benjamin Altman**

PanGEO INCORPORATED

*Geotechnical & Earthquake*
*Engineering Consultants*

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October 7, 2024
File No. 19-063.300

Mr. Benjamin Altman
10365 Honcho Place
San Diego, CA 92124-1219

**Subject**: **Geotechnical Design Report**
**Proposed Single-Family Residence**
**9167 | SE 64th Street, Mercer Island, Washington**

Dear Mr. Altman,

Please find attached our geotechnical design report for the proposed single-family residence at the above address in Mercer Island, Washington. This report documents the subsurface conditions at the site and presents our geotechnical engineering recommendations for the proposed development.

PanGEO previously prepared a geotechnical report for three lots, including the subject lot, dated April 16, 2019. We subsequently prepared a supplemental addendum, dated May 7, 2020, for the subject lot, to address pin pile foundations and temporary shoring, which was followed by a second addendum, dated December 28, 2021, to update our recommendations for temporary shoring. The attached design report is based on two recently advanced test borings drilled at the site, as required by the City of Mercer Island via plan review comments, as well as the currently proposed design with a basement finished floor elevation of about 168½ feet. The attached report is intended to supersede our original report for the three lots, and all previous addendums, and should be used for final design of the project. The conclusions and recommendations in the attached report are, in general, consistent with our previous recommendations, however, some revisions and additions were required due to the results of the most recent test borings, and current project design.

*Soil Conditions* - In summary, the site is underlain by about 9 to 15 feet of weak soils, consisting of fill or mass-wastage deposits, over medium stiff to hard clay, underlain by very dense sand.

*Foundation Recommendations* - Based on the currently proposed design, the majority of the proposed house foundations will not reach the underlying medium stiff to hard clay, and would be bearing on the upper, weak soils. As such, to provide adequate support for the new residence, we recommend that the entire residence be supported by small diameter driven pipe piles, commonly referred to as pin piles.

*Driveway Walls* - Due to the thick layer of weak soils in the proposed driveway area, we recommend that the proposed cast-in-place concrete driveway retaining walls be supported by small diameter driven pipe piles. While supporting the proposed driveway walls, which may have a maximum height of about 8 feet, would be feasible with spread footings bearing on improved soils, in our opinion the risk of long-term settlement and cracking of the walls would be moderate to high, and not likely desirable from an aesthetic standpoint.

*Site Stabilizing Measures* – Based on the results of our slope stability analysis, the upper weak soils (i.e. fill and mass-wastage deposits), are only marginal stable in the static condition, and prone to movement during the design earthquake. As such, to provide adequate stability to the developed portion of the lot, and to meet the current design codes, we recommend that the site be stabilized with a row of drilled concrete piles, such as soldier piles, with a minimum diameter of 24-inches, spaced a minimum of 6 feet on-center, along the downslope side of the proposed house. The piles should have a minimum tip elevation of 144 feet, or deeper, if required by the structural design. Due to observed variations in the existing ground surface along the downslope side of the proposed development (i.e. south side of the house and deck), the stabilizing soldier piles may also be used to retain fill soils placed to raise grade to match the basement finished floor elevation.

*Temporary Shoring* – The current design calls for temporary excavations up to about 19 feet deep into the hillside for the proposed basement. In addition, up to about 8 feet of fill will be placed for the new driveway, resulting in the need to retain up to about 25 feet of soil for the final condition. We recommend that temporary shoring consist of a soldier pile wall along the north wall of the basement, and along portions of the east and west basement walls. Where wall heights exceed about 10 to 12 feet, for a more efficient design, we recommend that drilled and grouted tiebacks be utilized to anchor the soldier piles.

*Critical Area Considerations* – Provided that the recommendations presented in this report are incorporated into the project plans and construction of the project, in our opinion the proposed project is feasible from the geotechnical standpoint, and will not adversely affect the steep slopes at and adjacent to the site.

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

Sincerely,

Stephen H. Evans, L.E.G.
Senior Engineering Geologist
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Encl.: Geotechnical Design Report

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**MERCER ISLAND, WASHINGTON**

## 1.0 INTRODUCTION

This report presents the results of a geotechnical engineering study that was undertaken to support the design of the proposed single-family residence (SFR) at 9167 SE 64th Street, in Mercer Island, Washington. This study was performed in general accordance with our mutually agreed scope of services outlined in our proposal for the current study dated April 24, 2024, which was subsequently authorized by you. Our scope of services included reviewing readily available geologic and geotechnical data, including our previous studies on this and other properties owned by you, conducting a site reconnaissance, drilling two supplemental geotechnical test borings and installing a groundwater monitoring well, measuring a field developed surficial profile, performing engineering analyses, and developing the conclusions and recommendations presented in this report.

## 2.0 SITE AND PROJECT DESCRIPTION

The project site consists of an irregularly shaped parcel located at 9167 SE 64th Street, on the west side of Mercer Island, Washington (*see Figure 1, Vicinity Map*). The 18,635 square-foot parcel (#302405-9213) is located south of the alignment of SE 64$^{th}$ Street, which in this location is within private property owned by the New Hope Church. The Church property flanks the project site on the north, a single-family residence is located to the east, and Engstrom Open Space (part of the Mercer Island Park system) lies to the west, which is undeveloped.

The property slopes down moderately to steeply to the south, with nearly 60 feet of relief down to the level of East Mercer Way at elevation about 130 feet (NAVD88). At the level of East Mercer Way, there is a City of Mercer Island sewer easement and a soft surface trail that gives access to Pioneer Park. From the trail, the topography slopes down another 15 feet or more to a stream bed at elevation about 113 to 115 feet. The property continues across the stream and starts up the other side of the ravine. The site plan and topography of the project site and surrounding parcels is shown in *Figure 2A, Site and Exploration Plan*.

There is a moderately sloping bench area at the subject site at about elevation 170 feet (NAVD88), which is about 28 feet below SE 64$^{th}$ Street. The current project plans call for the development of the area above the bench with a single-family residence and new driveway (see Figure 2A & 2B, showing the footprint of the proposed house and driveway).

The project parcel is predominantly covered with trees, shrubs, bushes, forest duff, leaf litter and other dense wild vegetation. The existing condition of the site is shown below in Plate 1.


***Plate 1 – General conditions of the heavily vegetated steep slope below SE 64th Street, looking south from SE 64th Street. (6/11/2024)***

We understand that the proposed project, as currently envisaged, will consist of constructing a new multi-level single-family residence with daylight basement in the upper portion of the site (*see Figure 2A & 2B).* The northern portion of the proposed single-family residence will be benched into the steep slope (*see Plate 2, below*). Additionally, an access driveway is planned to connect to SE 64th Street above the subject site, through the property of the New Hope Church. The proposed project will also include other site improvements, such as retaining walls around the driveway and light wells on the north side of the building, and a stormwater retention vault under the driveway.

The property is mapped within a landslide hazard area by the City of Mercer Island. As such, any development will need to consider the steep slopes and landslide hazards. To that end, this study includes a quantitative slope stability analysis, and provides recommendations to adequately stabilize the developed portion of the property in accordance with current building codes.

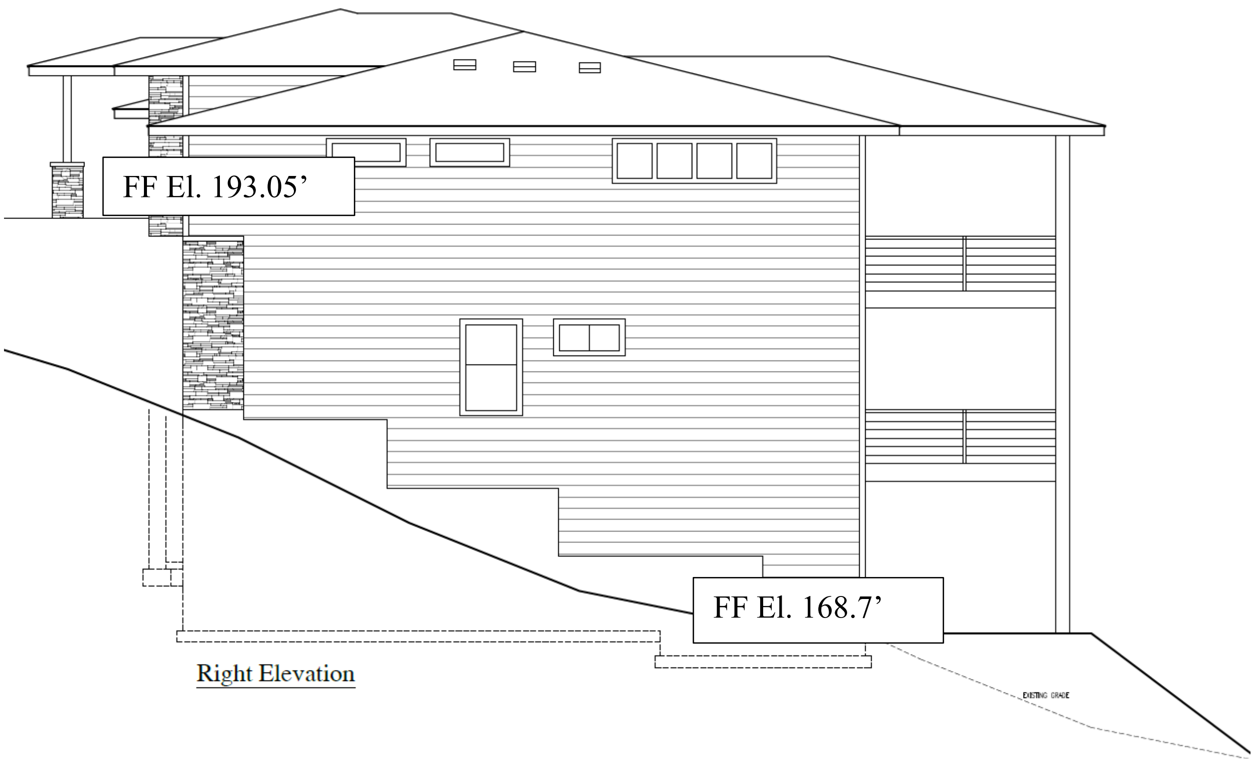



***Plate 2 –Schematic N-S Building Section, dated 10/9/23, by Mcleod Home Designs.***

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.

## 3.0 SUBSURFACE EXPLORATIONS

### 3.1 Current Subsurface Exploration

PanGEO completed two (2) test borings (PG-1 and PG-2) at the subject site on June 27, 2024. The borings were advanced approximately 41½ feet below the existing grade using a D-50 tracked drill rig owned and operated by Holocene Drilling of Seattle, Washington. The approximate boring locations were measured from on-site features, and are shown on the attached *Figures 2A and 2B.*

The D-50 is equipped with 8-inch O.D. hollow stem augers. Standard Penetration Tests (SPT) were performed at 2½ and 5-foot depth intervals using a 2-inch diameter split-spoon sampler. The sampler was driven with a 140-pound drop hammer falling a distance of 30 inches for each strike, in general accordance with *ASTM D-1586, Standard Test Method for Penetration Test and Split Barrel Sampling of Soils*. The hammer was operated using a hydraulic lift mechanism. 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. The completed borings were backfilled with bentonite chips, as required by the Washington State Department of Ecology.

A geologist from PanGEO was present on a full-time basis to observe the drilling, assist in sampling, and to describe and document the soil samples obtained from the borings. The soils were logged in general accordance with the system summarized on *Figure A-1, Terms and Symbols for Boring and Test Pit Logs*. Summary boring logs are included as Figures A-2 and A-3 in Appendix A. The stratigraphic contacts indicated on the boring log represent the approximate depth to the boundaries between soils units. Actual transitions between soil units may be more gradual or occur at different elevations. The descriptions of groundwater conditions and depth are likewise approximate.

### 3.2 Previous Subsurface Explorations

In addition to our test borings completed for the current study, we also completed one test boring on the subject site during an earlier study that included two other properties. Specifically, we drilled the previous test boring PG-7 in on March 21, 2019, to a depth of 16½ feet below the ground surface, in the area of the planned driveway access. The approximate location of PG-7 is also indicated on Figures 2A and 2B, and the log is included in Appendix A along with the current logs.

In addition to the borings by PanGEO, we reviewed borings previously drilled on the property by AMEC Earth & Environmental, Inc., in 2001. Location information on the logs and in the body of the AMEC report was conflicting, hence the logs could not be located accurately enough to be used in this report. However, the logs provided useful confirmation of general subsurface conditions.

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 SITE GEOLOGY

According to *The Geologic Map of Mercer Island (Troost and Wisher,2006)*, the entire subject parcel is underlain by Lawton Clay (Qvlc) with overlying mass-wastage deposits (Qmw). The geologic map also indicates that pre-Olympia non-glacial depots (Qpon) are mapped underlying the Lawton Clay, at about the level of East Mercer Way. As with much of the east facing bluffs of Mercer Island, old landslide scarps are mapped at the top of the slope above the project site. The following is a brief description of each relevant geologic soil unit mapped in the vicinity of the site, from youngest to oldest.

- **Mass-wastage Deposits (Qmw) –** Mass-wastage deposits consist of surficial deposits transported downslope in mass by gravity (landslides, colluvial soil movement, and other gravitational processes). Mass-wastage deposits typically consists of intermixed, very loose to medium dense, coarse-grained deposits and soft to stiff fine-grained deposits with voiding.

  This geologic unit typically exhibits moderate to high compressibility and low to moderate strength characteristics due to the highly variable composition and the nature in which this unit was deposited. Additionally, the presence of this unit can have significant implications for slope stability and erosion hazards, particularly in areas with steep terrain, or that are susceptible to landslides.

- **Vashon Lawton Clay (Qvlc)** – This deposit typically consists of sediment deposited in proglacial lakes, and is laminated to massive, very stiff to hard, silt, clayey silt to silty clay. Lawton Clay deposits typically exhibit low compressibility and high strength characteristics in an undisturbed state.

- **Pre-Olympia Non-Glacial Deposits (Qpon)** – This geologic unit is described by Troost and Wisher as generally consisting of very dense and hard, sand, gravel, silt, clay and organics of non-glacial origin. The unit may contain tephra beds, paleosols, and iron oxidized layers. These pre-Olympia deposits also typically exhibit low compressibility and high strength characteristics in an undisturbed state.

### 4.2 Soil Conditions

The current and previous test borings advanced at and near the project site generally confirmed the mapped geologic stratigraphy; however, we infer that the fine grained, lean clay deposits may be pre-Olympia fine-grained deposits, based on experience with similar strata, and considering the upper limits of Lawton deposition as mapped in the Seattle area. However, for the purposes of this report, the fine-grained lean clay deposits are referred to as the mapped Lawton Clay.

In general, the test borings encountered a sequence of disturbed, soft to medium stiff, lean clay (mass wasting deposits), overlying undisturbed, glacially consolidated, stiff to hard lean clay and clayey silt which we refer to in this report as Lawton Clay (Qvlc). At depth in PG-2, we encountered very dense, fine sand with a trace of silt, which we interpret as the mapped nonglacial deposit (Qpon).

For the purposes of this report, we have grouped the soils encountered in the borings into four engineering soil units (ESUs) based in part on their engineering properties, the composition of the soils, and anticipated engineering behaviors. The interpreted subsurface conditions are depicted in *Figure 3 – Generalized Subsurface Profile A-A'* and brief descriptions of the generalized soil conditions encountered at the locations of the test borings advanced at the site are presented below. Please refer to the summary boring logs in *Appendix A* for more details.

**<u>Engineering Soil Unit (ESU)</u>**

***ESU 1 | Fill (Hf)*** – Fill was encountered below the leveled bench at elevation 170 feet in PG-2 to a depth of roughly 7½ feet. The fill encountered generally consisted of very loose to medium dense sand and silty sand with organics. This unit is of limited extent, and we infer was previously placed at the site for an access road, or potentially as part of the previous AMEC subsurface exploration program, to create a level drilling platform.

***ESU 2 | Colluvium / Mass-wasting deposits (Qmw)*** – Mass wasting deposits are mapped as covering most of the slopes over the entire stream drainage in which the project in situated. This unit was encountered in all borings, and in all the AMEC borings on this and properties to the east. The unit generally consists of soft to stiff, silty to sandy, lean clay. In PG-7, the mass wasting material consists of sandy silt to silty sand. The material is characterized by mixed textures, and iron oxide staining. The thickness of the unit varies by position on the slope and how much surficial grading has been done. The unit appears to be roughly 9 feet thick in PG-7, up to 15 feet thick in PG-1, and 14 feet thick with fill in PG-2.

***ESU 3 | Lawton Clay (Qvlc)*** – Underlying the mass wasting / colluvium deposits above (ESU 2), the test borings encountered stiff to hard silt and lean clay, which is laminated to massive, with low plasticity. Based on the mapping in the area, we interpret this unit as Lawton Clay (Qvlc) though based on the elevation of the site and mapping experience in Seattle (Booth, et al, 2009), the unit may be pre-Fraser. This unit was the deepest soil encountered in test borings PG-1 and PG-7. This unit is characterized by its interbedded soil structure and high SPT N-values. We anticipate this unit will exhibit high strength characteristics in its undisturbed state.

***ESU 4 | Pre-Olympia Non-Glacial Deposits (Qpon)*** – The deepest soil unit encountered on the site was penetrated in PG-2 at a depth of 26½ feet below the ground surface (approximate elevation (144 feet, NAVD88). This unit is comprised of very dense, fine sand with a trace of silt and, fine gravel.

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.

Selected Sample Photos: *Plates 3 through 6* below depict select soil samples obtained from our recent test borings. For reference purposes, the split-soon samplers pictured below have an outside diameter of 2 inches.

***Plate 3 – ESU 1: Hf | Very loose to loose, silty sand | PG-2, S-2 @ 5 – 6½ feet.***

***Plate 4 – ESU 2: Qmw | Medium stiff, lean CLAY (CL) | PG-1, S-3 @ 7½-9 feet.***

***Plate 5 – ESU 3: Qvlc | Very stiff to hard, silty, lean CLAY to clay SILT | PG-2, S-6 @ 20 – 21½ feet.***

***Plate 6 – ESU 4: Nonglacial Deposits Qpon | Very dense, fine SAND, trace silt| PG-2, S-9 @ 35 – 36½ feet.***

### 4.3 GROUNDWATER CONDITIONS

A static phreatic groundwater table was not encountered in the current or previous test explorations conducted at and near the site which extended up to 41½ feet below the ground surface. However, the current and previous test borings encountered thin zones of perched groundwater seepage, generally along the contact of the mass wasting deposits and the Lawton Clay (ESU 2 and 3).

A groundwater monitoring well was installed in boring PG-1, and a groundwater well was also previously installed in the AMEC boring B-3, although it could not be found in the field for our current study. Table 1, below, shows the elevations of the groundwater readings in the wells. It should be noted that, while the location of B-3 is uncertain, there is no reason to dispute the elevation data for boring B-3 provided by AMEC. The elevation given for B-3 is 168 feet, placing the boring on the same bench as our boring PG-2.

**Table 1 – Summary Groundwater Readings in Monitoring Wells**

| Boring/Well | Date | Ground Elevation | Depth to GW | GW Elev. |
|---|---|---|---|---|
| **PG-1** | 7/19/2024 | 186 | 15.3 | 170.7 |
| **B-3 (AMEC)** | 4/12/2001 | 168 | 6.5 | 161.5 |

Please note that there will be fluctuations in seepage and groundwater levels, depending on the season, amount of rainfall, surface water runoff, local subsurface conditions and other factors. Generally, the groundwater levels are higher and seepage rates are greater in the wetter winter months (typically October through May).

### 4.4 Laboratory Testing

The following laboratory tests were performed on select soil samples collected from the test borings:

- Moisture Content (ASTM D 2216)
- Atterberg Limits (ASTM D 4318)
- Passing #200 sieve (ASTM D 1140)

The test results are noted on the test boring logs in Appendix A, where appropriate, and the Atterberg limits test results are included in Appendix B.

## 5.0 GEOLOGIC HAZARDS EVALUATION

As part of our study, we conducted an assessment of potential geologic hazards within the subject site as defined in Mercer Island City Code Chapter 19.07.160, Geologically Hazardous Areas. Mercer Island City Code identifies three different types of Geologic Hazards: Erosion Hazards, Potential Landslide Hazards, and Seismic Hazards. The City's criteria for the various hazard areas and our assessment of the hazard areas with respect to the planned improvements are provided in the following sections of this report.

### 5.1 Erosion Hazards

The site is mapped as a potential erosion hazard area in accordance with the City of Mercer Island's Geologic Hazards Map. Based on the Web Soil Survey data, the mapped site soils (Kitsap Silt Loam KpD) have an Erosion Factor K of 0.37 to sheet and rill erosion. Factor K values range between 0.02 and 0.69, with the higher number indicating higher vulnerability. As such, we interpret the site soils to have a moderate susceptibility to erosion.

***Conclusions:*** In our opinion, the erosion hazards at the site can be effectively mitigated with best management practices during construction and with properly designed and implemented landscaping for permanent erosion control. During construction, the temporary erosion hazard can be effectively managed with an appropriate erosion and sediment control plan, including, but not limited to, installing a silt fence at the construction perimeter, placing quarry spalls or hay bales at the disturbed and high traffic areas, covering stockpiled soil or cut slopes with plastic sheets, constructing a temporary drainage pond, if needed, to control surface runoff and trap sediment, and by maintaining a stabilized construction entrance.

Permanent erosion control measures should be applied to the disturbed areas of the site as soon as feasible. These measures may include, but not limited to, planting and mulching. The use of permanent erosion control mats may also be considered in conjunction with planting/mulching to protect the soils from erosion.

### 5.2 Potential Landslide Hazards

The subject site is mapped within a potential landslide hazard area according to the City of Mercer Island's Geologic Hazards Map. The map also indicates that steep slopes are present at the site.

Review of the Geologic Map of Mercer Island (Troost and Wisher, 2006) shows that the drainage basin of the creek within which the project area is situated is lined by slide scarps, and the slopes mantled with mass wasting deposits. This data is in accordance with those found on the Washington State landslide inventory mapping for the site area, compiled in 2022 for the south portion of Mercer Island, which also includes several large prehistoric landslides. The landslide inventory in particular maps several smaller slide scarps within the mass wasting deposits on the south side over the stream valley.

The City of Mercer Island GIS mapping identifies several landslides within 500 feet of the subject site, including one on the property, and several to the east. Three of these, including the one on the subject property, have no information associated with them. The location marker on the City of Mercer Island GIS map for the on-site landslide coincides generally with a feature we observed and interpreted as a possible slide scarp, as described below. The other features were located on the lots at 9179 and 9185 SE 64th Street, east of the project site. Other identified slides are located along the property boundary between 9170 SE 64th Street (New Hope Church) and 6191 93rd Avenue SE. Documentation consists of plans for the repair of the slope, with no description of the slide itself.

**Slope Reconnaissance and Observations:** We conducted several reconnaissance visits to the site. The majority of the site contains heavily vegetated, southwest-facing slopes. The purpose of our reconnaissance was to review the condition of the steep slope and identify indications of potential historical slope instability, which may include:

- Bowl-shaped topography;
- Irregular or hummocky topography;
- Tension cracks, scarps, or other indicators of ground movement;
- Leaning or pistol-butted trees;
- Distressed vegetation;
- Vegetation of markedly different ages or types (i.e., a swath of young alders and blackberries in an otherwise mature forest);
- “Fresh” looking soil deposited at the base of steep slopes;
- Disturbed or destroyed anthropogenic features, such as fence lines that have been displaced;
- Hillside seeps or springs; and
- Ponding water/sag ponds.

As with most steep slopes, the surficial material is loose, and tends to slowly move downslope due to gravity over time, which is often referred to as “soil creep”. Our observation of the general conditions of the area suggests that there is some evidence of ongoing soil creep on the subject slope, in the form of slightly leaning trees, or trees with bent trunks.

During our site visits, we did not observe evidence of recent slope instability such as slide scarps or tension cracks within the subject property, or on the slopes immediately to the south.

We did observe areas of over-steepened slopes which could potentially be former slide scarps that have been partially eroded and/or buried by organic debris, as well as undulating terrain, which could represent former slumps. One such potential scarp was located just north of the northern property line. This over-steepened slope is in approximately the location of the identified landslide recorded by the City of Mercer Island, as described above. It is not clear if this over-steepened slope is associated with an old scarp, or the result of fill placed for the church parking lot.

Other small slide scarps are mapped on properties to the east, and presumably have been erased by grading work during construction of the homes currently occupying those properties. Further scarps are located along the property boundary between the New Hope Church and the house upslope at 6191 93rd Avenue SE.

Our test borings encountered soil deposits that we interpreted to be colluvium or mass wasting deposits, as described above, which is consistent with the City of Mercer Island Landslide Hazard Maps, and we infer that the previous slides and scarps are associated with relatively shallow slides within the colluvium or mass-wastage deposits.

***Conclusions:*** Based on our reconnaissance and our understanding of subsurface conditions at the site, in our opinion a large, deep-seated type of slope failure is unlikely on the subject property. In our opinion, landslides at the site have the potential to occur within the fill and mass wasting deposits on the steepest portions of the site. As such, as described below, we performed a detailed slope stability analysis of the site to determine the risk of landsliding, and to provide recommendations to adqeuatly stabilize the site.

It is our opinion that the proposed improvements are feasible from a geotechnical engineering standpoint, and in our opinion will not adversely affect the overall stability of the site or adjacent properties provided the recommendations outlined herein are followed, and the proposed improvements are properly designed and constructed. Our recommendations include the use of permanent stabilizing piles along the downslope side of the developed area to provide adequate support for the existing residence in the static and seismic condition, per the current standard of practice. In addition, the proposed basement walls of the house, and driveway fill, will provide stabilization for the potential unstable soils between the north property line and the church parking lot.

### 5.3 Seismic Hazards

Based on our review of the City of Mercer Island's Geologic Hazards Maps, the project site is mapped in a seismic hazard area. The City of Mercer Island Code defines seismic hazard areas as those areas subject to risk of damage as a result of earthquake-induced ground shaking, slope failure, soil liquefaction or surface faulting.

Based on our subsurface explorations, the site is underlain by primarily fine-grained silt and clay, with very dense sand at depth. Based on these conditions, in our opinion the liquefaction potential of the soils underlying the site is low, and design considerations related to soil liquefaction are not necessary for this project.

It is also our opinion that the potential for significant deep-seated seismic-induced land sliding is relatively low at the site due to the underlying hard silt and clay, and very dense sand within the core of the slope. Landslides within the fill and mass-wastage deposits do have the potential to occur during a seismic event. However, provided the proposed project is designed and constructed in accordance with the recommendations in the report, the developed portion of the site should not be adversely effected during the code-level seismic event.

## 6.0 SLOPE STABILITY

Due to the mapped geologic hazards and steep slopes at the site, as well as the results of our test borings, which encountered up to about 15 feet of fill and mass-wastage deposits, which are prone to downslope movement, we performed a slope stability analysis to evaluate the stability of the proposed development. Our analysis was intended to evaluate if stabilizing measures would be needed to meet the current building code requirements.

### 6.1 Slope Stability Analysis

Our evaluation was based on our understanding of the subsurface conditions as described above, the topographic data derived from the topographic survey by Informed Land Survey (1/6/2020), a field developed surface profile as described below, the results of our site reconnaissance, and our understanding of the planned improvements.

The stability of a slope depends on a variety of factors, including the geometry of the slope, the subsurface stratigraphy, material properties of the soils, the location of groundwater, and the effects of surface loads. Based on our understanding of the subsurface conditions at the site, the topography shown on Figure 2A, and site features observed during our site reconnaissance, we developed a generalized subsurface profile through the site as shown on Figure 2A, and presented in Figure 3.

***Field Developed Slope Profile –*** To develop a more accurate slope profile for our analysis, and to identify indications of slope instability such as possible slide scarps or minor undulations in the ground surface that may be important to our analysis, PanGEO prepared a field developed slope profile. This consisted of establishing a line-of-sight section down the fall line of the subject property, then measuring a series of closely spaced points along that section. Distances to selected points were measured from one or more base station locations, using a highly accurate laser rangefinder that measures both distance and angle, and a reflective target. The measured points were selected based on observed site features, such as changes in slope angle, or to maintain close spacing of data points. Distance and angle data were converted to delta-X and delta-Y values from the base point. The delta-Y values are converted to elevations based on known elevations derived from the topographic survey. The delta-X values are converted into distance values for the profile. The results of our field profile were similar to a section developed based on the topographic survey, with some moderate variations downslope of the proposed development area, where more ground surface undulations were measured in our field section than shown on the survey.

***Soil & Groundwater Parameters -*** Soil parameters utilized in the stability analyses are shown on the attached Figures 4 and 5. Soil parameters were selected based on observed soil types, the results of standard penetration tests performed in the test borings, published correlations for soil units in the Puget Sound Area, and our experience.

Due to the presence of perched water encountered in the test borings near the contact between the mass-wastage deposits and the underlying Lawton Clay, we assumed the groundwater level was present approximately 1-foot above the top of the Lawton Clay, and, conservatively, that the entire Lawton Clay deposit was saturated.

***Surcharge Loads –*** Because the proposed house will be supported by pin piles, foundation surcharge loads were not included in the model. However, we did include a surcharge for slab areas, in the event a slab-on-grade is utilized instead of a pile-supported slab. The estimated backfill below the proposed driveway, and potentially below the deck, was also included in the model.

***Analysis Method -*** 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. Search routines were used to identify the potential circular and non-circular failure surface having the lowest static factor of safety using the Spencer method of analysis.

The seismic stability was analyzed using a pseudo-static approach, where the effect of earthquake ground shaking is added to the static analysis in the form of an additional horizontal force. The seismic coefficient used in the pseudo-static stability analysis shall correspond to some fraction of the anticipated peak ground acceleration associated with a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years). Based on the current IBC, a seismic coefficient of 0.342g was used for this project, which corresponds to one-half of the expected peak $PGA_M$ of 0.684g.

### 6.2 STABILITY ANALYSIS RESULTS

The City of Mercer Island Building Code states that, *Alteration of landslide hazard areas and seismic hazard areas and associated buffers may occur if the critical area study documents find that the proposed alteration (a) Will not adversely impact other critical areas; (b) Will not adversely impact the subject property or adjacent properties; (c) Will mitigate impacts to the geologically hazardous area consistent with best available science to the maximum extent reasonably possible such that the site is determined to be safe; and (d) Includes the landscaping of all disturbed areas outside of building footprints and installation of hardscape prior to final inspection.* In our opinion, to meet criteria (c) above, the static and seismic factors of safety against global instability of 1.5 and 1.1, respectively, should be achieved, to be consistent with the local standard of practice.

As such, to determine if the proposed development on the slope will have an acceptable factor of safety in accordance with the criteria stated above, we evaluated the stability of the proposed condition and found that the factor of safety against global instability did not meet the requirements of 1.5 for the static condition, or 1.1 for the seismic condition. As such, we recommend that mitigating measures be incorporated into the proposed project to adequately stabilize and protect the developed area of the site in accordance with the current building code.

### 6.3 RECOMMENDED STABILIZING MEASURES

To improve the stability of the site and provide adequate factors of safety against future instability of the developed portion of the site, we recommend that a stabilizing wall be constructed along the downslope side of the proposed development. In our opinion, a feasible wall type consists of drilled soldier piles (steel beams inserted into concrete shafts) with a minimum diameter of 24-inches, and a minimum spacing of 6 feet on-center. As shown in the attached Figure 4 and 5, if the stabilizing piles have a minimum tip elevation of 144 feet (NAVD88), the developed area of the site will exceed the required factor of safety of 1.5 and 1.1 for the static and seismic condition, respectively. However, the minimum tip elevation of the stabilization piles should be determined by the structural engineer, which may need to extend below elevation 144 feet. Our analysis assumed that each pile, spaced 6-foot on-center, has an allowable shear capacity of 175 kips. See *Section 7.5.3 Permanent Stabilization Piles* for additional discussion and details. As described below, we recommend that the stabilizing soldier piles be designed for an exposed face of 12 feet to account for erosion and/or future slope movements downslope of the wall, particularly during the code-level seismic event.

### 6.4 Stability Risks and Qualifications

Based on the results of our study, it is our opinion that the proposed building and site improvements as planned will have adequate factors of safety against potential future slope instability and will not have adverse impacts on the subject and surrounding properties, provided the project is properly designed and constructed. However, it should be noted that any development on or near a steep slope or a potential landslide area always involves some level of risk. In addition, future activities on and off the site could also affect the stability of the subject site. This may include but is not limited to the proper maintenance of surface drainage, and adequate protection of the side slopes from erosion.

## 7.0 GEOTECHNICAL RECOMMENDATIONS

### 7.1 Seismic Design Considerations

#### *7.1.1 Site Class*

We anticipate that the project will be designed in accordance with the 2018/2021 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). For design purposes, Site Class D (Stiff Soil) is considered appropriate for the seismic design for the project site.

#### *7.1.2 Liquefaction*

Liquefaction is a process that can occur when soil loses its shear strength for short periods of time during a seismic event. Ground shaking of sufficient strength and duration results in the loss of grain-to-grain contact and an increase in pore water pressure, causing the soil to behave as a fluid.

Soils with a potential for liquefaction are typically cohesionless, predominately silt and sand sized, must be loose to medium dense, and be below the groundwater table.

Based on the presence of very dense / hard glacially overridden soils underlying the site, and the presence of predominantly clay soils at shallow depths with limited amounts of perched water, it our opinion that the potential for earthquake-induced liquefaction at the site is low, and special design considerations for soil liquefaction are not necessary.

### 7.2 Building Foundation – Driven Pin Piles

In general, our test borings encountered a layer of soft to medium stiff, disturbed material that extended a maximum of about 15 feet below the anticipated foundation elevation of the basement along the downslope side of the house. The approximate depth to the bottom of this unsuitable bearing layer encountered at each boring location is shown in *Figure 3*.

Based on the planned finish floor elevation of about 168½ feet, we anticipate that the majority, if not all, of the building foundation will bear on the weak mass-wastage deposits. Foundations bearing on these soils will have a high risk of long-term settlement, and therefore we do not recommend that conventional spread and strip footings be used to support the proposed residence unless they bear on the underlying very stiff to hard silt.

To avoid excessive long-term and differential building settlement due to the mass-wasting deposits, for planning purposes we recommend vertically supporting the proposed residence on small diameter driven pipe piles (pin piles). If competent soils are encountered at the foundation level during construction, PanGEO will work with the structural engineer, as desired, to reduce the required number of pin piles and revise the foundations to spread footings.

Small diameter pin piles are utilized to transfer the structure loads through the weak and marginal soils to the underlying competent bearing layer. Pin piles of 3- to 4- inches in diameter are typically utilized for projects such as the subject residence. However, 6-inch diameter piles may also be used, which have a higher vertical capacity. Three- to six-inch pin piles are typically installed using small to large hammers (600 to 4,700 lbs) mounted on small to medium-sized excavators.

***Pin Pile Sizes -*** We have provided recommendations for 3-, 4-, and 6-inch diameter pipe piles. The structural engineer should evaluate the pile sizing and spacing based on the design loads and pile capacities.

***Pin Pile Capacity*** – The following allowable axial compression capacities can be used per pile assuming a factor of safety of at least 2.0:

- 6 tons (12 kips) per 3-inch diameter pile
- 10 tons (20 kips) per 4-inch diameter pile
- 15 tons (30 kips) per 6-inch diameter pile

Penetration resistance required to achieve the capacities will be determined based on the hammer used to install the pile. The tensile capacity of pin piles should be ignored in design calculations.

***Pin Pile Specifications -*** We recommend that the following specifications be included on the foundation plan:

1. 3-inch, 4-inch, and 6-inch diameter piles should consist of Schedule-40, ASTM A-53 Grade "A" pipe.

2. 3-inch piles shall be driven to refusal with a minimum 600-lb hydraulic hammer. We recommend the following refusal criteria based on the size of hammer utilized:

| Hammer Size | Blow per Minute | Refusal Criteria (3-inch pile) |
|---|---|---|
| 600 lbs | 1000 | 12 seconds per inch |
| 850 lbs | 900 | 10 seconds per inch |
| 1100 lbs | 900 | 6 seconds per inch |

3. 4-inch piles shall be driven to refusal with a minimum 850-lb hydraulic hammer. We recommend the following refusal criteria based on the size of hammer utilized:

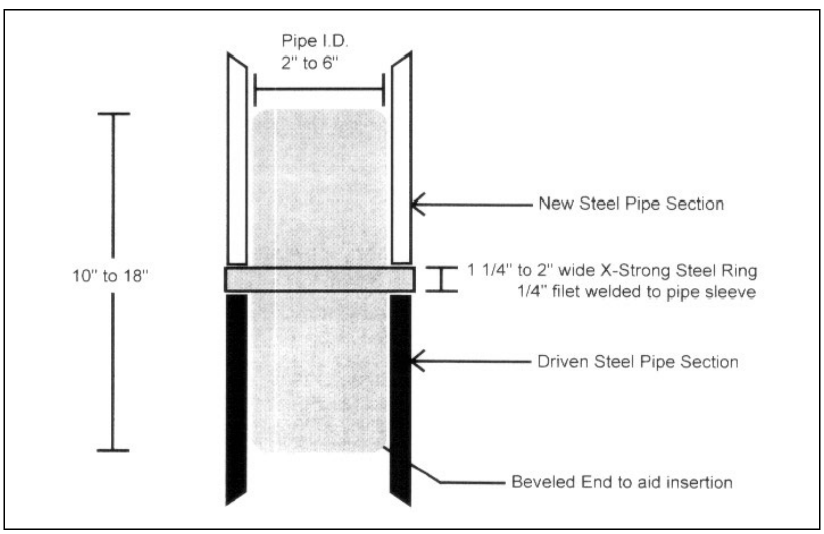
| Hammer Size | Blow per Minute | Refusal Criteria (4-inch pile) |
|---|---|---|
| 850 lbs | 900 | 16 seconds per inch |
| 1100 lbs | 900 | 10 seconds per inch |

| | | |
|---|---|---|
| 2000 lbs | 600 | 4 seconds per inch |

4. 6-inch piles shall be driven to refusal with a minimum 2000-lb hydraulic hammer. We recommend the following refusal criteria based on the size of hammer utilized:

| Hammer Size | Blow per Minute | Refusal Criteria (6-inch pile) |
|---|---|---|
| 2000 lbs | 600 | 10 seconds per inch |
| 3000 lbs | 500 | 6 seconds per inch |
| 4700 lbs | 500 | 4 seconds per inch |

5. Piles shall be driven in nominal sections and connected with compression fitted sleeve couplers (see following detail – Courtesy of McDowell Pile King, Kent, WA). We discourage welding of pipe joints, particularly when galvanized pipe is used, as we have observed welds break during driving.




6. At least 3% (but no more than 5) of the 3-inch, 4-inch, and 6-inch pin piles should be load tested to verify the driving criteria listed above. The load tests should be performed prior to installed production piles. If more than one size of pipe pile is used, each pipe size should be subject to separate testing. Contractors may elect to use a different hammer system and driving criteria, provided that the driving criteria for the selected hammer can be verified with the load test program.

7. All load tests shall be performed in accordance with the procedure outlined in ASTM D1143 - *Standard Test Methods for Deep Foundations Under Static Axial Compressive Load*. The maximum test load shall be 2 times the design load. The objective of the testing program is to verify the adequacy of the driving criteria, and the efficiency of the hammer used for the project.
8. The geotechnical engineer of record or his/her representative shall provide full time observation of pile installation and testing.

***Installation Monitoring -*** As it is not possible to observe the completed pile below the ground, judgment and experience must be used as the basis for determining the acceptability of a pile. Therefore, all piles should be installed under the full-time observation of a representative of PanGEO. This will allow us to fully evaluate the contractor's operation, collect and interpret the installation data, and verify bearing stratum elevations.

The quality of a pin pile foundation is dependent, in part, on the experience and professionalism of the installation company. We recommend that a company with experienced personnel be selected to install the piles. Furthermore, we will also understand the implications of variations from normal procedures with respect to the design criteria. The contractor's equipment and procedures should be reviewed by PanGEO before the start of construction.

***Lateral Resistance -*** The lateral capacity of pin pipes is very limited and should not be used in design. Therefore, lateral forces from wind or seismic loading should be resisted by the passive earth pressures acting against the pile caps and below-grade walls or from battered piles [batter no steeper than 3(H):12(V)]. ***Friction at the base of pile-supported footings and grade beam should be ignored in the design calculations.***

Passive resistance values may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf), assuming level ground surface in front of the footings. This value includes a safety factor of about 1.5 assuming that properly compacted granular fill will be placed adjacent to the pile caps and grade beams, and level ground surface.

***Estimated Pile Length -*** The required pile length in order to develop the recommended pile capacity is expected to vary across the footprint of the structure, depending on the actual driving conditions encountered. Based on the soil conditions encountered in our test borings, we anticipate penetrations of about 15 to 30 feet may be needed below the pile caps. A minimum pile length of 10 feet (below final basement level) should also be specified in the plans.

***Pin Pile Performance -*** It is our experience that the driven pipe pile foundations should provide adequate support with total settlements on the order of ½-inch or less.

***Obstructions -*** Obstructions may be encountered during pile installation. Where possible, the obstructions should be removed to facilitate pile driving. If obstructions cannot be removed, the structural engineer of record should be notified to revise the pile layout to accommodate moving the piles.

### 7.3 Floor Slabs

#### *7.3.1 Concrete Slab-on-grade*

A slab-on-grade may be used for the basement floor of the proposed building, however, due to the loose/soft soils that will likely be present below the slab elevation, there is a potential for some slab settlement to occur over the design life of the structure, which can result in cracks and uneven floor surfaces.

To reduce the potential of slab settlement and distress, we recommend that the floor slabs should be underlain by a sequence of at least 4 inches of capillary break material, overlying at least 12 inches of compacted structural fill. The structural fill should be placed over native subgrade soil recompacted to a firm and unyielding condition. Any soft/loose and pumping native subgrade soil observed during compaction should be removed and replaced with granular structural fill. We also recommend that a layer of geogrid reinforcement, such as Tensar TX grid, or approved equivalent, be placed on the compacted native subgrade, prior to placement of structural fill to further improve foundation soil stiffness. The geogrid panels should be overlapped a minimum of 12-inch, where needed. We also recommend that construction joints be incorporated into the floor slab to control cracking.

Depending on the contractor's construction method and sequence, more than 12 inches of structural fill may be needed in the wet season to provide a firm working surface for the contractor. It is the contractor's responsibility to determine the additional thickness of structural fill to achieve a firm working surface.

#### *7.3.2 Structural Slab*

To achieve a higher level of slab performance, a structural slab can be designed to span between the pile supported foundations. If a structural slab is utilized, the existing loose/soft soils below the slab may be left in place without re-compaction or replacement. A capillary break and vapor barrier should be placed below the slab, as described below. We recommend a pile supported structural slab when the floor surface will receive settlement sensitive floor coverings, such as tile, or if the exposed floor will be polished concrete.

#### *7.3.3 Capillary Break*

The capillary break material should consist of at least 4 inches of free-draining, clean (less than 3 percent fines) crushed rock compacted to a firm and unyielding condition. The capillary break material should have no more than 10 percent and 5 percent by weight of material passing the U.S. Standard No. 4 and No. 100 sieves, respectively. We also recommend that a 10-mil polyethylene vapor barrier be placed below the slab.

#### *7.3.4 Sub-Slab Drains*

Due to the potential of groundwater seepage near the proposed basement slab elevation, particularly on the up-slope side of the house, we recommend that sub-slab drains be installed to reduce the potential of moisture issues in the basement. We recommend sub-slab drain-pipes be spaced about 20-feet on center below the floor slab. The pipes should consist of 4-inch diameter perforated PVC pipes, be located in a minimum 16-inch-wide trench, and be surrounded with 5/8-inch clean crushed rock wrapped with a geotextile fabric. The invert of the pipe should be at least 18-inches below the bottom of the floor slab.

### 7.4 Retaining and Below-Grade Walls

Retaining and below-grade walls should be properly designed to resist the lateral earth pressures exerted by the soils behind the wall. Current design includes concrete cantilever walls supporting the driveway fills. The upslope walls, in general, will not have final exposed faces more than a few feet high. The wall west of the driveway will have final exposed face up to 10 to 12 feet high, facing west and parallel to the fall line of the slope. The retaining walls surrounding the window into the second floor office on the west side of the house (see plans) may have exposed faces up to 5 to 7 feet high. Basement walls on the upslope side of the house may be up to 25 feet high on the garage side, and less on the west side of the house.

Foundation walls may be designed in conjunction with permanent soldier pile walls, as described below.

Proper drainage provisions should be provided behind the walls to intercept and remove groundwater that may be present behind the wall. Our geotechnical recommendations for the design and construction of the retaining and below-grade walls are presented in the sections below.

***Concrete Wall Foundation Support*** – Concrete walls that are founded within the mass wasting soil unit (Qmw), which we anticipate will be the case for the driveway walls, will need to be supported on pin pile foundations to avoid excess settlement. Pin pile design requirements are provided in the sections for the house foundations, above.

***Lateral Earth Pressures*** **–** Concrete cantilever walls supporting the driveway or elsewhere should be designed for an equivalent fluid pressure of 35 pcf for level backfills behind the walls, assuming the walls are free to rotate. If walls are to be restrained at the top from free movement, such as basement walls, equivalent fluid pressures of 50 pcf should be used for level backfills behind the walls. Walls with a maximum 2H:1V backslope should be designed for an active and at rest earth pressure of 50 and 65 pcf, respectively. The recommended lateral pressures assume that the backfill behind the wall consists of free draining and properly compacted fill with adequate drainage provisions to prevent the development of hydrostatic pressure.

Permanent walls should be designed for an additional uniform lateral pressure of 10H psf for seismic loading, where H corresponds to the buried depth of the wall in feet. The recommended lateral pressures assume that the backfill behind the wall consists of a free draining and properly compacted fill with adequate drainage provisions.

***Wall Surcharge -*** The retaining and basement walls should be designed to resist surcharge pressures, if present, within the height dimension of the wall. As a minimum, for anticipated cars and delivery vans, the traffic surcharge may be considered as 80 psf of horizontal uniform pressure. Similarly, surcharge loads from construction equipment or soil/material stockpiles should be considered in the retaining and basement wall design during construction. We recommend that *Figure 6* be used to calculate the lateral pressure on the face of the wall face resulting from surcharge loading.

***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 foundation. Passive resistance values may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf). This value includes a safety factor of about 1.5 assuming that properly compacted granular fill will be placed adjacent to the pile caps and grade beams, and level ground surface. *Friction at the base of pile-supported footings and grade beam should be ignored in the design calculations.*

***Wall Drainage -*** Provisions for permanent control of subsurface water should be incorporated into the design and construction of the below-grade walls. As a minimum, 4-inch diameter perforated drainpipes should be installed behind and at the base of the wall footings, embedded in 12 to 18 inches of crushed rock or washed 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.

Under no circumstances should roof downspout drain lines be connected to the perforated footing/wall drain systems. Roof downspouts must be separately tightlined to appropriate discharge locations. Cleanouts should be installed at strategic locations to allow for periodic maintenance of the footing drain and downspout tightline systems.

If the below-grade wall will be constructed against a soldier pile wall, we recommend that prefabricated drainage mats, such as Mirafi 6000 or equivalent, be installed behind the walls (full face coverage) and the collected water should be directed inside the building beneath the floor slab and tight lined to an appropriate outlet. Additionally, a perforated footing drain should be constructed on the interior of the perimeter footing to remove any groundwater seepage.

***Wall Backfill -*** In our opinion, the on-site excavated soils are not suitable for use as wall backfill. We recommended that wall backfill should consist of free draining granular structural fill as defined in *Section 8.3* of this report.

Wall backfill should be moisture conditioned to within about 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and systematically compacted to a dense and relatively unyielding condition and to at least 95 percent of the maximum dry density, as determined using test method ASTM D-1557 (Modified Proctor). Within 5 feet of the wall, the backfill should be compacted with hand-operated equipment to at least 90 percent of the maximum dry density.

***Damp-proofing/Waterproofing -*** We recommend the designers consider utilizing a waterproofing material, such as prefabricated clay mats, or other measures, on the exterior of all below grade walls to reduce the potential for moisture intrusion into the below-grade portion of the homes. We recommend that a waterproofing or building envelope specialty consultant be retained to provide details regarding waterproofing measures, as waterproofing is beyond the scope of our work.

### 7.5 Soldier Pile Walls

Two types of soldier piles walls will be needed for this project.

- Soldier piles will be used as temporary shoring around the uphill side of the proposed residence, as approximately shown in the attached Figure 2B. Where these walls are greater than about 12 feet tall, tieback anchors or interior bracing will likely be needed to provide a more efficient design.
- As discussed in *Section 6.3* of this report, permanent stabilization piles, consisting of soldier piles, will also be needed along the south side of the developed area in order to achieve adequate stabilization of the development area. The approximate location of the stabilizing piles are also shown in the attached Figure 2B.

The following sections present our recommendations for the design of temporary or permanent soldier piles at the project site.

#### *7.5.1 Upslope Temporary or Permanent Soldier Pile and Lagging Wall*

Based on current plans, excavations up to about 19 feet below existing grades will be required to facilitate the construction of the daylight basement. Due to the presence of an unstable layer of mass wasting material and a perched water lens along the interface between the mass wasting layer and the underlying Lawton Clay, a shoring wall, with tiebacks in some cases, will be required on the upslope side of the house to provide for a safe excavation and construction. This wall may be designed either as a temporary shoring wall, or incorporated into the foundation system as a permanent element.

A soldier pile wall consists of vertical steel beams, typically spaced from 6 to 8 feet apart along the proposed wall alignment, spanned typically by timber lagging.  The steel beams are installed into holes drilled to a design depth and then backfilled with lean-mix or structural concrete. In general, tiebacks (ground anchors) are typically needed for wall heights greater than about 12 feet to achieve a more economical design. Based on the current anticipated finish floor elevations, the retained height of the soldier piles will be approximately 20 feet. Therefore, we anticipate tiebacks will likely be required for this project.

***Design Earth Pressures -*** The recommended earth pressures depicted on *Figure 7* should be used for design of cantilevered soldier pile walls, or walls with one or more level of tiebacks along the north, east and west sides of the basement excavation.

Above the bottom of excavation, the recommended active earth pressure should be applied over the full width of the pile spacing. Below the bottom of excavation, the passive resistance should be applied over two times the pile diameter, and the active pressure applied over one single pile diameter.

***Surcharge Loads -*** The lateral earth pressures shown on Figure 7 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. The surcharge loads can be calculated based on Figure 6.

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.

***Lagging –*** We recommend that the minimum lagging sizes be determined in accordance with Section 5.4.2 of FHWA *Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems* (1999). Based on the soil conditions, a minimum 4-inch-thick timber lagging may be utilized for pile spacings less than 8 feet. However, in areas where the pile spacing is larger than 8 feet or lagging will be subject to high surcharge loads such as crane outriggers or footings, the required lagging thickness should be evaluated on a case-by-case basis.

When placing timber lagging, the height of each lift may need to be limited to prevent the un-shored soil from sloughing. We recommend that the soil exposed for timber lagging installation be no more than 4 feet deep. The actual allowable vertical cut for timber lagging placement should be determined in the field, based on the actual conditions observed. The backfill behind the lagging should consist of controlled density fill (CDF).

***Vertical Capacity*** - Soldier piles may be designed using an allowable skin friction value of 0.5 ksf for the portion of the pile below the bottom front face of wall, and an allowable end bearing value of 15 ksf.

***Permanent Soldier Piles*** **-** It may be possible to incorporate the soldier piles installed for temporary shoring into the permanent structure to reduce the number of pin piles required to support the building. In this case, the soldier piles would be designed as permanent piles, and shear studs would be installed on the soldier pile flanges to transfer the vertical load of the structure into the embedded portion of the soldier pile. If the soldier piles will be utilized as part of the permanent structure, all exposed portions of the steel beams above the bottom of the excavation should be galvanized or coated to protect from corrosion. Alternatively, the steel section should be over-sized to account for corrosion.

***Wall Drainage*** **-** Where the below-grade wall will be constructed against a soldier pile wall, we recommend that prefabricated drainage mats, such as Mirafi 6000 or equivalent, be installed behind the walls (full face coverage) and the collected water should be directed inside the building beneath the floor slab and tightlined to an appropriate outlet. Additionally, a perforated footing drain should be constructed on the interior of the perimeter footing to remove any groundwater seepage.

***Performance / Pile Deflection*** **-** In general, the top of piles should be designed with one-inch deflection or less.

***Performance Monitoring*** – Because ground deformations will occur due to the excavation (open cut or shored), we recommend that existing conditions on the adjacent private properties be photo-documented prior to the start of the project. We also recommend that survey points be installed on every other soldier pile and on adjacent structures.

The survey points on the piles should be monitored at least weekly by the project surveyor until one week after the excavation has been backfilled or permanent walls are completed. The monitoring program should include changes in both the horizontal (x and y directions) and vertical deformations to the nearest 0.01-foot, and the results be promptly submitted to PanGEO for review. After the initial baseline readings, which should be taken prior to the start of pile installations, the monitoring points on the adjacent structures only need to be shot if excessive soldier pile deflections are noted. The results of the monitoring will allow the design team to confirm design parameters, and for the contractor to make adjustments if necessary.

### *7.5.2 Temporary Tiebacks*

Tiebacks will likely be needed for wall heights greater than about 12 feet to improve performance of the wall, and reduce the steel beam size. Excessive pile top deflections could occur before the tiebacks are installed. It may be necessary to limit the tiebacks to no more than 10 feet below the pile top unless steel beams of sufficient size will be used to limit the cantilever deflection.

***Tieback Easements* –** Temporary easements will be needed from the adjacent property owners to the north, east and west, to allow for tieback installations. If temporary easement cannot be obtained, the soldier pile wall would likely need to be supported with internal bracing.

***Tieback Type* –** Due to the soil conditions at the site, in our opinion drilled and grouted tiebacks should be used for anchored soldier pile walls. In our opinion driven earth anchors, or helical anchors, will not be able to achieve enough penetration into the hard clay to achieve satisfactory design loads.

***Tieback No-Load Zone* –** The recommended no-load zone for the anchor is shown in the attached Figure 7. As noted, we recommend a minimum no-load length of 15 feet to insure adequate embedment beyond the mass-wastage soils deposits.

***Tieback Adhesion Estimate* -** 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 strongly recommend the 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, for pressure grouted tiebacks with a minimum diameter of 6 inches, we recommend the following design parameters be used to size the tieback lengths:

| | |
|---|---|
| ESU 2 \| Mass Wasting Soil | We do not recommend a bond zone in the ESU2 soils |
| ESU 3 \| Lawton Clay | 2.5 kips per foot of bond length (Allowable) |

The use of pressure grouting and multiple post grouting may be needed in order to achieve the assumed capacity.

We recommend that the shoring contractor review this report and determine the appropriate design tieback adhesion, based on their experience with similar soils and their proposed methods of installation. The assumed design capacity may be revised based on contractor's input.

We recommend that the allowable tieback loads be limited to approximately 120 kips per anchor.

The actual capacity of the anchors should be checked with 200 percent verification tests. At least two 200% tests should be performed. All production anchors should be proof tested to 130% of the design load. The anchor installations should be conducted in accordance with the latest edition of the Post Tensioning Institute (PTI) "*Recommendations for Prestressed Rock and Soil Anchors*." Elements of the testing are as follows:

***Tieback Testing -*** The actual capacity of the anchors should be checked with 200 percent verification tests. At least two 200% tests should be performed. All production anchors should be proof tested to 130% of the design load. The anchor installations should be conducted in accordance with the latest edition of the Post Tensioning Institute (PTI) "*Recommendations for Prestressed Rock and Soil Anchors*." Elements of the testing are as follows:

*Verification Tests (200% Tests)*

- 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;
- Test locations to be determined in conjunction with and approved by the geotechnical engineer;
- We highly recommend the contractor perform the verification tests prior to installing production anchors to confirm their installation method can achieve the design load specified on the plans;
- Test anchors, which will be loaded to 200% of the design load, may require additional prestressing steel (steel load not to exceed 80% of the ultimate tensile strength) or reinforcing of the soldier pile;
- Load test anchors to 150% load in 25% 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 0.01inch;
- At the 150% load, the holding period shall be at least 60 minutes;

- After completion of the 150% hold, load the anchor in 25% load increments to the 200% load, which shall be held for 10 minutes; and
- A successful test shall provide a measured creep rate of 0.04 inches or less at the 150% 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% variation from the specified load).

*Proof Tests (130% load tests on all production anchors)*

- Load test all production anchors to 130% of the design load in 25% 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 0.01inch);
- At the 130% load, the holding period shall be at least 10 minutes; and
- A successful test shall provide a measured creep rate of 0.04 inches or less at the 130% load between 1 and 10 minutes with a creep rate that is linear or decreasing with time. The applied load must remain constant during the holding period (i.e., no more than 5% variation from the 130% 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 5 and 50 minutes with a linear or decreasing creep rate.

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.

In the tieback construction, a bond breaker shall be constructed in the no load zone when the installation procedures use single stage grouting.

Tiebacks will need to be designed to provide adequate clearance from utilities, if present behind the wall.

### *7.5.3 Permanent Stabilization Piles*

To adequately stabilize the developed portion of the site, we recommend the installation of soldier pile stabilizing piles along the downslope side of the house, at the locations shown in Figure 2B. Based on our field observations, the existing ground surface elevation is variable along the south side of the proposed development, and in some areas grade may need to be raised to match the design elevation of the lowest basement level. If this is the case, the stabilizing soldier piles may also be designed to retain fill used to raise grade. If grade does not need to be raised, the tops of the soldier piles could be level or slightly below the ground surface (about 1-foot) if it is not desirable for the wall to be seen.

***Minimum Pile Size and Spacing -*** To evaluate the minimum depth of embedment needed to provide adequate stabilization of the developed area, we performed slope stability analyses using limit-equilibrium methods for both static and seismic loading conditions. Based on the results of our analyses, the soldier piles along the south (downslope) side of the developed area should have a minimum hole diameter of 24 inches, a center-to-center spacing of 6 feet or less, and a minimum tip elevation of +144 feet, or deeper as determined by structural analysis. The piles should have a minimum shear capacity of 175 kips.

As described above, we recommend that the stabilizing soldier piles be designed for an exposed face of 12 feet to account for erosion and/or future slope movements downslope of the wall, particularly during the code-level seismic event.

We recommend the lagging extend a minimum of 4 feet below the ground surface on the downslope face of the wall, to prevent sloughing of soil between the piles in the event of soil loss downslope of the wall. In the event that ground movements expose soil below the lagging, we anticipate soil arching between the piles spaced at 6 feet on-center will prevent significant soil loss between the piles. We anticipate the soil loss between piles would extend less than 2 feet behind the piles, which would not result in a life-safety issue to the residence, in our opinion. If this occurred, additional lagging would be added and backfilled to repair the wall.

***Design Earth Pressures -*** The recommended earth pressures depicted on Figure 8 should be used for design of the stabilizing piles, which will function as a cantilevered wall.

Above the bottom of excavation, the recommended active earth pressure should be applied over the full width of the pile spacing. Below the bottom of excavation, the passive resistance should be applied over two times the pile diameter, and the active pressure applied over one single pile diameter.

Because the soldier pile wall is permanent, we recommended a seismic pressure of 10H (psf) be included in the pile design, where H is the exposed design height of the wall in feet.

***Corrosion Protection –*** Since the soldier pile wall will be utilized as a permanent stabilizing wall, all potentially exposed portions of the steel beams should be galvanized or coated with corrosion protection. The corrosion protection should extend at least 2 feet below the potentially exposed portion of the wall, which is equal to 2 feet more than the 12-foot design height of the wall. Alternatively, the steel section should be over-sized to account for corrosion.

***Lagging*** - As described above, we recommend the lagging be extended a minimum of 4 feet below the existing ground surface on the downslope side of the soldier piles. Lagging design recommendations for the anticipated conditions are presented on Figure 8. Lagging may consist of materials such as timber boards, cast-in-place concrete, precast concrete panels, or steel sheets. For the permanent condition, if timber lagging is utilized, treated timber should be specified, and the saw cut ends of the lagging should be treated on-site prior to lagging installation. It should be noted that even treated timber lagging will eventually deteriorate, and would need to be replaced. The lifespan on treated timber lagging may range from 15 to 25 years. The advantage of concrete or steel lagging is that they would be permanent.

***Wall Drainage Considerations -*** Due to the permeable nature of the timber lagging, drainage provisions for the stabilizing wall are not necessary in our opinion unless a concrete, watertight facing is utilized for lagging. If cast-in-place concrete lagging is utilized, we recommend weep holes be provided in each soldier pile bay to allow any accumulated water to drain through the base of the wall.

### 7.6 Permanent Cut and Fill Slopes

Based on the anticipated soil that will be exposed at the site, we recommend permanent cut and fill slopes be constructed no steeper than 2H:1V (Horizontal:Vertical). Any proposed permanent slopes with a relief of more than 8 feet should be evaluated by PanGEO on a case-by-case basis.

Cut slopes should be observed by PanGEO during excavation to verify that conditions are as anticipated. Supplementary recommendations can then be developed, if needed, to improve stability. Fill slopes must consist of properly placed and compacted structural fill, with careful compaction out to the slope face. Proper compaction may require the need to over-build the slope and then cut it back to the desired final condition. All fill must be placed on horizontal benches, and adequately keyed into the native soil. If fill slopes are proposed, PanGEO will need to assist the design team by providing specific recommendations for the fill slope proposed.

Permanently exposed slopes should be treated with permanent erosion control measures as soon as possible to improve stability of the surficial layer of soil.

### 7.7 Permanent Drainage & Infiltration Considerations

Based on currently available plans, we understand that stormwater from paved areas and roof drains will generally be collected and channeled into a stormwater detention vault located beneath the driveway area upslope of the house. Based on our understanding of the plans, excavation of the vault area will be minimal, rather the area will be filled after vault placement to achieve driveway grade. From the vault, stormwater will be conducted to an existing storm drain line that runs along the west side of the property to the sewer line at the base of the slope. Storm drainage that is not collected into the vault, and footing drains, will be captured in other facilities and likewise transmitted into the existing storm drain.

We understand from the plans that the sanitary sewer will be implaced in a new excavation parallel to the existing storm drain. Since these facilities are located in the fall line of a steep slope, we recommend that the excavations be design with periodic check dams to control drainage and erosion within the trench. Check dams should consist of fine grained, low permeability soils, spaced at intervals of not more than 20 feet. We also recommend that any new utilities down the slope consist of HDPE pipe, which are flexible, durable, and more resistant to potential slope movements, if they were to occur.

Permanent control of surface water and roof runoff should be incorporated in the final grading design. In addition to these sources, irrigation and rainwater infiltrating into landscape and planter areas adjacent to paved areas or building walls should also be controlled. All collected runoff should be directed into conduits that carry the water away from the pavement, structure, and steep slope; and into appropriate outlets. Adequate surface gradients should be incorporated into the grading design such that surface runoff is directed away from structures and steep slope.

Under no circumstances should collected surface water or downspout drains be allowed to discharge onto open slopes or behind walls. Furthermore, it is important to note that roof downspouts should be tightlined to a suitable outlet, and not discharged into the wall or perimeter footing drain system.

Due to the proximity of the steep slope and unfavorable soil conditions, infiltration of surface water should not be allowed through dispersion trenches, dry wells, or similar infiltration facilities.

### 7.8 Permanent Erosion Control Considerations

Permanent erosion control measures such as covering exposed ground surfaces with topsoil or mulch, and installing landscaping, should be performed as soon as possible after construction to limit the time the exposed surfaces are susceptible to erosion.

## 8.0 CONSTRUCTION CONSIDERATIONS

### 8.1 Site Preparation

Site preparation for the proposed project includes clearing, grubbing, shoring wall installation, and excavations to the design subgrade. All stripped surface materials should be properly disposed of off-site.

Following site excavations, the adequacy of the subgrade where structural fill, foundations, slabs, or pavements are to be placed should be verified by a representative of PanGEO. The subgrade soil in the improvement areas, if recompacted and still yielding, should be over-excavated and replaced with compacted structural fill.

### 8.2 Material Reuse

The soils at the site are extremely moisture sensitive and will become disturbed / soft when exposed to inclement weather conditions. Therefore, in our opinion, the on-site soils are not suitable to be reused as structural fill. In the context of this report, structural fill is defined as compacted fill placed under footings, pavements, concrete stairs, landings, and slabs, or other load-bearing areas. Suitable material for use as structural fill is described in *Section 8.3,* below.

The on-site soil may potentially be used as general fill in the 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.

### 8.3 Structural Fill Placement and Compaction

For planning purposes, structural fill should consist of imported, well-graded, granular material such as Seattle Type 17 Mineral Aggregate (*COS Standards and Specifications, 2023, Section 9-03.14*), WSDOT Gravel Borrow (*WSDOT Standards and Specifications, 2023, Section 9-03.14(1)*), or an approved equivalent.

Structural fill should be moisture conditioned near its optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and systematically compacted to a dense and relatively unyielding condition. The adequacy of the compaction should be verified by PanGEO. If density tests are performed, the test results should indicate at least 95 percent of the maximum dry density, as determined using test method ASTM D1557 (modified proctor). For utility backfill or backfill within 5 feet of retaining walls, the backfill should be compacted to 90 percent of the maximum dry density.

The procedure to achieve proper density of a compacted fill depends on the size and type of compacting equipment, the number of passes, thickness of the lifts being compacted, and certain soil properties. If the excavation to be backfilled is constricted and limits the use of heavy equipment, smaller equipment can be used, but the lift thickness will need to be reduced to achieve the required relative compaction. PanGEO can provide additional recommendations regarding structural fill and compaction during construction.

Generally, loosely compacted soils are a result of poor construction technique or improper moisture content. Soils with high fines contents are particularly susceptible to becoming too wet and coarse-grained materials easily become too dry, for proper compaction. Silty or clayey soils with a moisture content too high for adequate compaction should be dried as necessary, or moisture conditioned by mixing with drier materials, or other methods.

### 8.4 Temporary Excavations

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. All temporary excavations deeper than a total of 4 feet should be sloped or shored. Temporary excavations less than 4 feet along the property lines should also be sloped or shored.

For planning purposes, we recommend that temporary excavations be sloped no steeper than 2H:1V (Horizontal:Vertical) due to the weak surficial layer of mass-wasting deposits, and layers of perched groundwater. If temporary excavations extend below the fill and mass-wastage deposits, into the Lawton Clay, steeper excavations may be feasible, based on PanGEO's field observations and the configuration of the excavations.

The temporary excavations and cut slopes should be re-evaluated in the field during construction based on actual observed soil conditions. If groundwater seepage is encountered the temporary slope will likely need to be cut to shallower angles to maintain stability. During wet weather, runoff water should be prevented from entering excavations and the exposed slopes should be covered with plastic sheets.

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.

### 8.5 SOLDIER PILE INSTALLATION CONSIDERATIONS

The drilling of soldier piles is anticipated to encounter up to 15 feet of fill and mass wasting deposits over hard silt/clay and dense sand. Obstructions, such as cobbles and boulders, or other objects, may be encountered. If the obstructions are located at depths that cannot be practically removed with a backhoe/excavator or coring, the soldier pile location may be revised as directed by the structural engineer.

It is important to note that caving of the disturbed soil is possible in the upper 15 feet of the drill hole, especially if zones of seepage are encountered. The contractor should be prepared to temporarily case the holes to maintain stability during drilling. Caving in the underlying clean sand is also likely.

We recommend that the following should be incorporated into the project plans and specifications:

- The geotechnical engineer shall verify the suitability of all soldier pile holes before concrete placement;
- Tremie methods shall be used for concrete placement in all holes having 6 or more inches of accumulated water if perched ground water or heavy precipitation is encountered during construction.
- All soldier pile holes drilled shall be filled with lean concrete mix or structural concrete on the same day.
- If soldier piles are designed as a permanent foundation system, any soft/disturbed soils encountered at the bottom of the hole during drilling will need to be removed using a cleanout bucket prior to placing the steel beam and concrete.

- Caving in fill, mass-wastage deposits, and wet sand/silty sand layers could occur during drilling. As a result, the drilling contractor should be prepared to stabilize the holes by using temporary casings, hydrostatic pressures (i.e., flooding the hole), or drilling fluids.

### 8.6 TIEBACK INSTALLATION

The drilling for tiebacks may encounter soft, wet, silt and clay, or wet sand layers where caving of the drilled holes may occur. As result, the contractor should be prepared to use temporary casing during installation to keep the drilled holes open, and to minimize the risk of potential ground loss. Due to the soft/loose nature of the mass-wastage deposits, larger grout-takes than normal should be anticipated.

Tiebacks will need to be designed to provide adequate clearance from utilities, if present behind the wall.

### 8.7 TEMPORARY EROSION AND DRAINAGE CONSIDERATIONS

We recommend that the exposed temporary slopes be covered with plastic sheeting.

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 prevent water from entering excavations or to prevent turbid runoff from leaving the 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 and potential storm water detention to trap sand and silt before the water is discharged to a suitable outlet. All collected water should be directed under control to an appropriate / approve discharge point or outlet.

We recommend that the contractor should be prepared to provide temporary groundwater control methods, especially if excavation is conducted in the wet season. If present, we anticipate that the groundwater can likely be controlled with sumps and pumps.

### 8.8 WET EARTHWORK RECOMMENDATIONS

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below:

- All surfaces of the foundation subgrade should be protected against inclement weather. It is the contractor's responsibility to protect the footing subgrade from disturbance. One option is to place a 2- to 3-inch thick layer of lean-mix concrete on the footing subgrade as soon as the subgrade is exposed.

- Earthwork should be performed in small areas to minimize subgrade exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.

- 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 ¾-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 strategically located to control erosion and the movement of soil. Erosion control measures should be installed along all the property boundaries.

- Excavation slopes and soils stockpiled on site should also be covered with plastic sheets.

## 9.0 ADDITIONAL SERVICES

We anticipate the City of Mercer Island will require a plan review and geotechnical special inspections to confirm that our recommendations are properly incorporated into the design and construction of the proposed development. Specifically, we anticipate that the following construction support services may be needed:

- Review final project plans and specifications;
- Verify implementation of erosion control measures;
- Observe the stability of open cut slopes;
- Observe the installation of soldier piles and tiebacks;

- Verify the temporary tiebacks are distressed;
- Observe and monitor the installation of stabilizing soldier piles;
- Monitor pin pile installation and testing;
- Confirm the adequacy of the compaction of structural backfill;
- Observe installation of subsurface drainage provisions, and;
- Other consultation as may be required during construction.

Modifications to our recommendations presented in this report may be necessary, based on the actual conditions encountered during construction.

## 10.0 STATEMENT OF RISK

Per the Mercer Island City Code, development within geologic hazard areas requires a statement of risk. The statement of risk shall meet one of the following criteria:

a. The geologic hazard area will be modified, or the development has been designed so that the risk to the lot and adjacent property is eliminated or mitigated such that the site is determined to be safe;

b. Construction practices are proposed for the alteration that would render the development as safe as if it were not located in a geologic hazard area;

c. The alteration is so minor as not to pose a threat to the public health, safety and welfare; or

d. An evaluation of site-specific subsurface conditions demonstrates that the proposed development is not located in a geologic hazard area.

Based on our understanding of the proposed project, which will include the construction of a soldier pile shoring wall, as well as permanent stabilization piles, as discussed above, as well as best management practices to reduce potential erosion during construction, and landscaping to adequately reduce the potential of erosion for the permanent condition, in our opinion the project will be designed so that the risk to the lot and adjacent properties is eliminated or mitigated such that the site is determined to be safe. Hence, it is our opinion that criterion (a) above, will be met.

PanGEO will be available to review the final design plans to confirm our statement of risk prior to construction.

## 11.0 LIMITATIONS

We have prepared this report for use by Benjamin Altman and the project design team. Recommendations contained in this report are based on a site reconnaissance, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of work.

Variations in soil conditions may exist between 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 work 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 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.

Within the limitation of scope, schedule and budget, PanGEO engages in the practice of geotechnical engineering and endeavors to perform its services in accordance with generally accepted professional principles and practices at the time the Report or its contents were prepared. No warranty, express or implied, is made.

We appreciate the opportunity to be of service to you on this project. Please feel free to contact our office with any questions you have regarding our study, this report, or any geotechnical engineering related project issues.

Sincerely,

**PanGEO, Inc.**

10/7/24

Stephen H. Evans, L.E.G.
Senior Engineering Geologist

Jon C. Rehkopf, P.E.
Principal Geotechnical Engineer

## 12.0 REFERENCES

ASCE 2016, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI Standard 7-16.

ASTM D1557-12e1, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft3 (2,700 kN-m/m3)),* ASTM International, West Conshohocken, PA, 2012, www.astm.org

ASTM D1586-11, *Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*, ASTM International, West Conshohocken, PA, 2011, www.astm.org.

ASTM D2488-17, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedures),* ASTM International, West Conshohocken, PA, 2017, www.astm.org.

City of Seattle (COS), 2023, *Standard Specifications for Road, Bridges, and Municipal Construction*., Seattle, Washington

International Code Council, 2018, *International Building Code (IBC), 2018.*

International Code Council, 2021, *International Building Code (IBC), 2021.*

Meyerhof, G. G., *Penetration Tests and Bearing Capacity of Cohesionless Soils,* Journal of the Soil Mechanics and Foundations Division, ASCE, 1956.

NAVFAC, 1986, *Soil Mechanics, Design Manual 7.01*, Department of the Navy, Naval Facilities Engineering Command

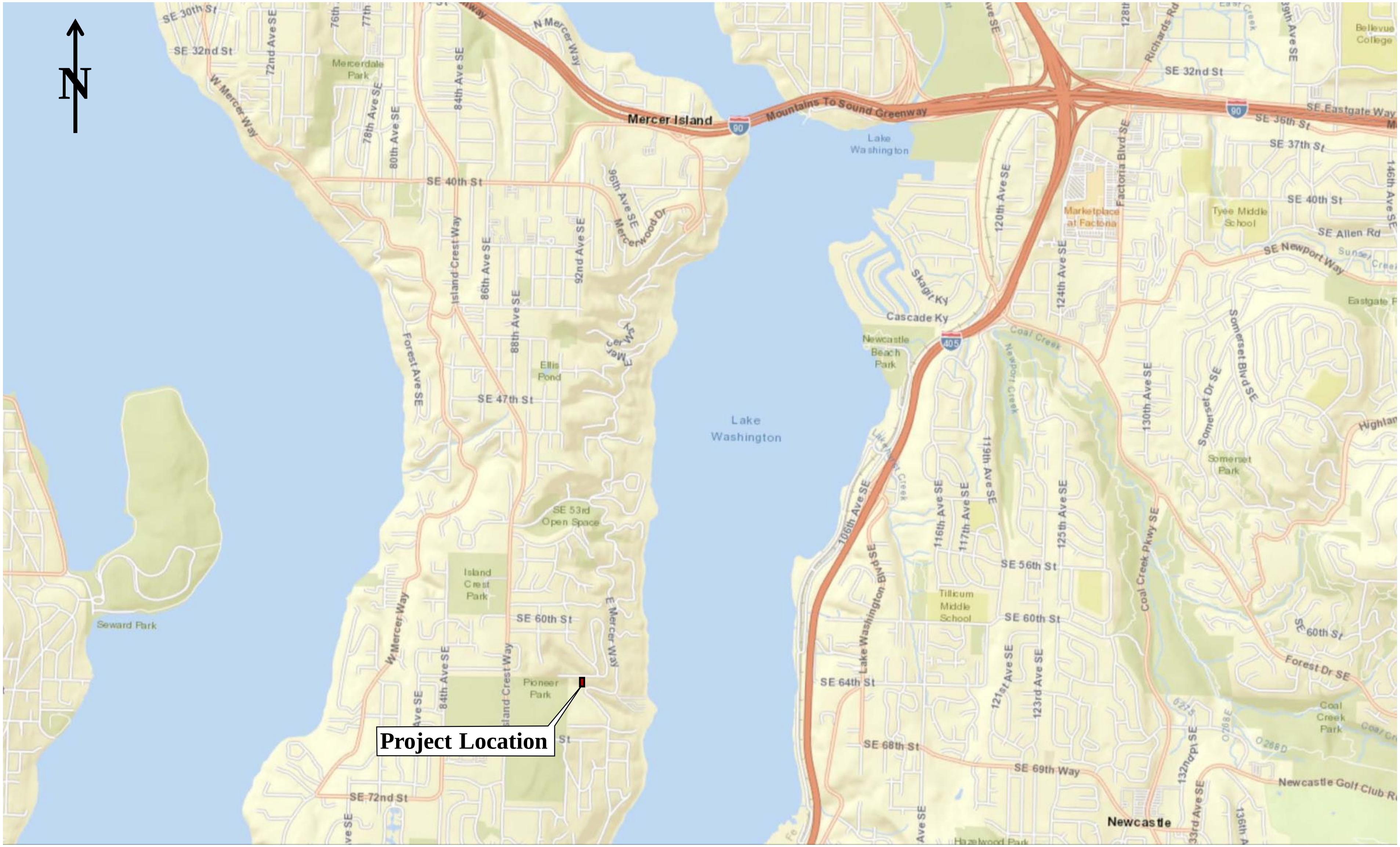
Troost, K. G. and Wisher, A.P., 2006, *Geologic Map of Mercer Island,* Geomap NW, University of Washington and the City of Mercer Island.

United States Geological Survey*, Earthquake Hazards Program, Interpolated Probabilistic Ground Motion for the Conterminous 48 States by Latitude and Longitude, 2008 Data,* accessed via: https://seismicmaps.org/

Washington Administrative Code (WAC), 2023, *Chapter 296-155 - Safety Standards for Construction Work, Part N - Excavation, Trenching, and Shoring*, Olympia, Washington

Washington State Department of Transportation (WSDOT), 2024, *Standard Specifications for Road, Bridges, and Municipal Construction*, Olympia, Washington.

19-062.300 Vicinity Fig 1.ppt 8/19/2024(10:15 AM) SHE



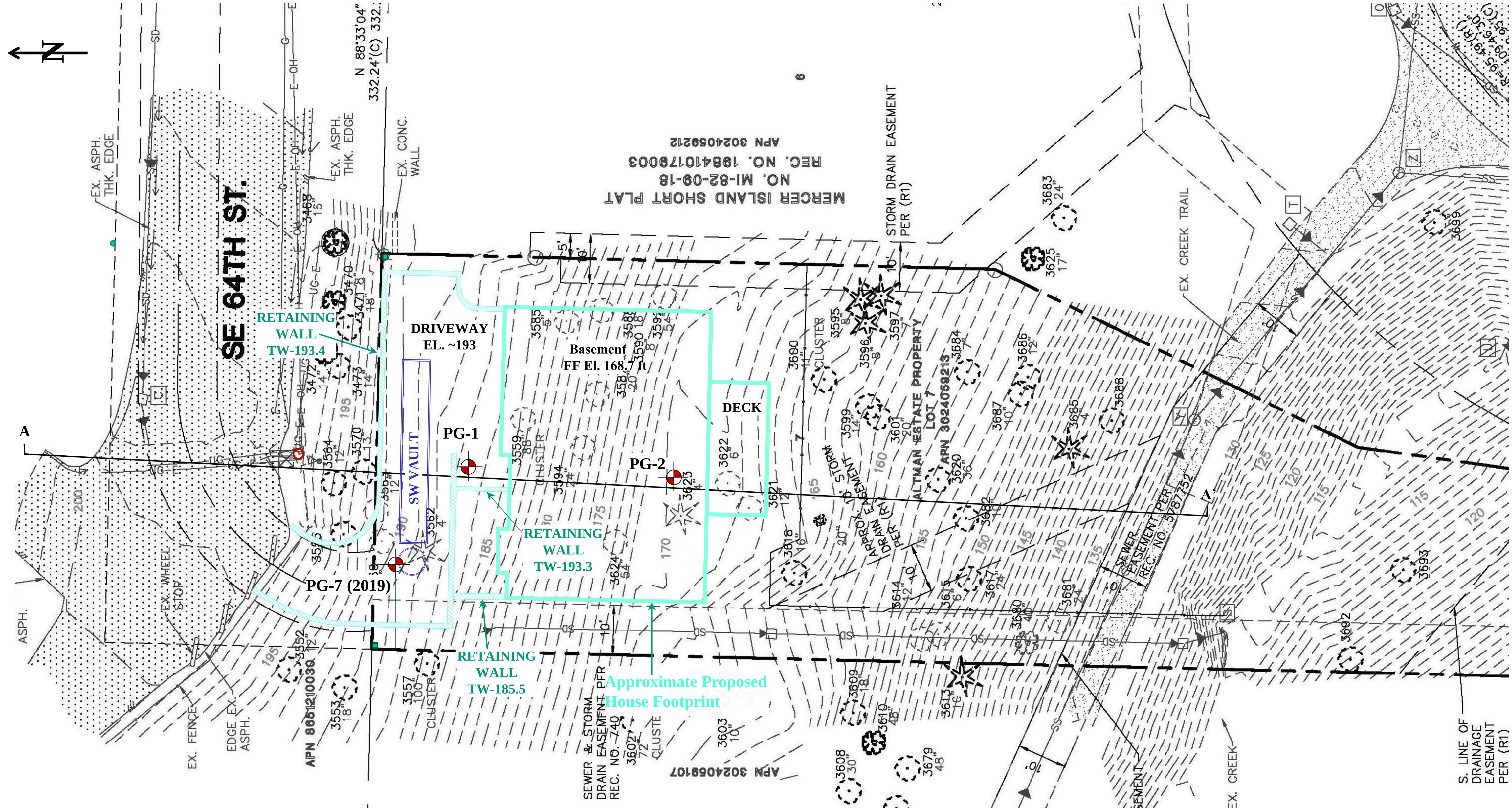

Map not to Scale
Base Map from Dept of Natural Resources Geological Information Portal

PanGEO INCORPORATED

**Proposed Single-Family Residence**
**9167 SE 64th Street**
**Mercer Island, WA**

**VICINITY MAP**

| Project No. | Figure No. |
|---|---|
| 19-062.300 | 1 |




**Approximate Scale**
**1":20'**

A — A'
Location of Profile

**Legend:**
**PanGEO Boring Location**

| PanGEO INCORPORATED | Proposed Single-Family Residence 9167 SE 64th Street Mercer Island, Washington | SITE AND EXPLORATION PLAN | |
|---|---|---|---|
| | | Project No. 19-062.300 | Figure No. 2A |

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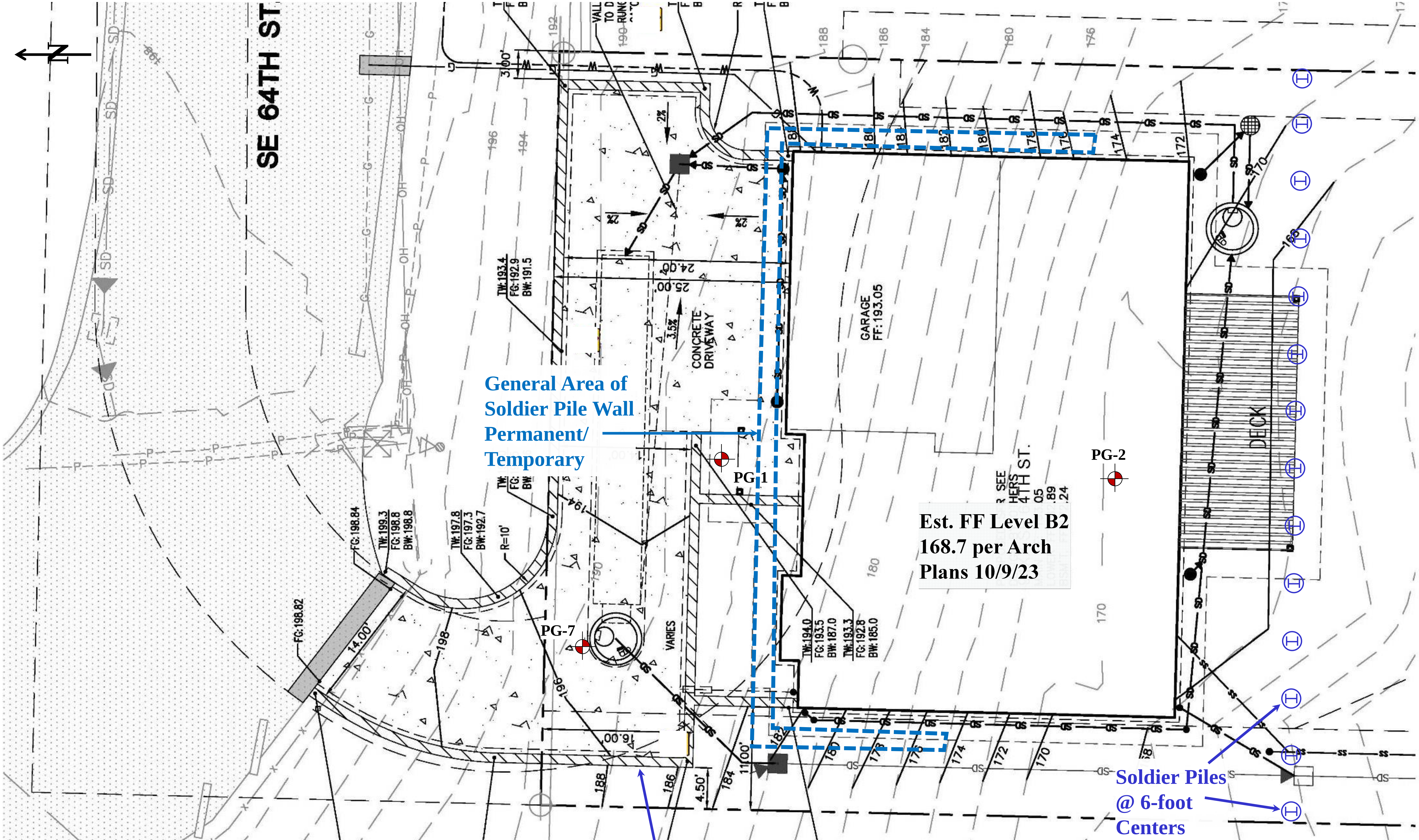


Approximate Scale
1":10'

Legend:

PanGEO Boring Location

Wall Note

Concrete Wall on pin piles

PanGEO INCORPORATED

Proposed Single-Family Residence
9167 SE 64th Street
Mercer Island, Washington

SITE PLAN WITH PROPOSED SOLDIER PILE WALLS

| Project No. | Figure No. |
|---|---|
| 19-062.300 | 2B |

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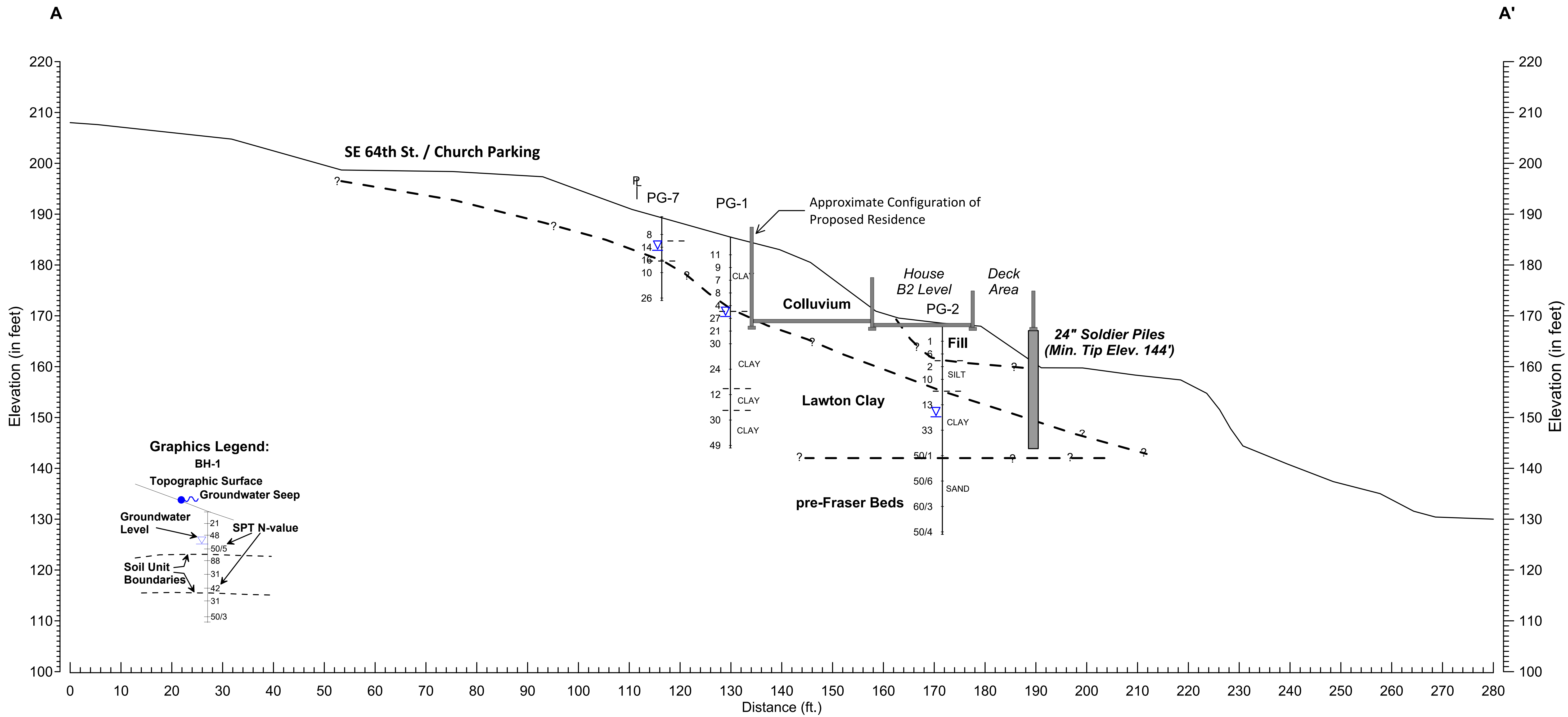
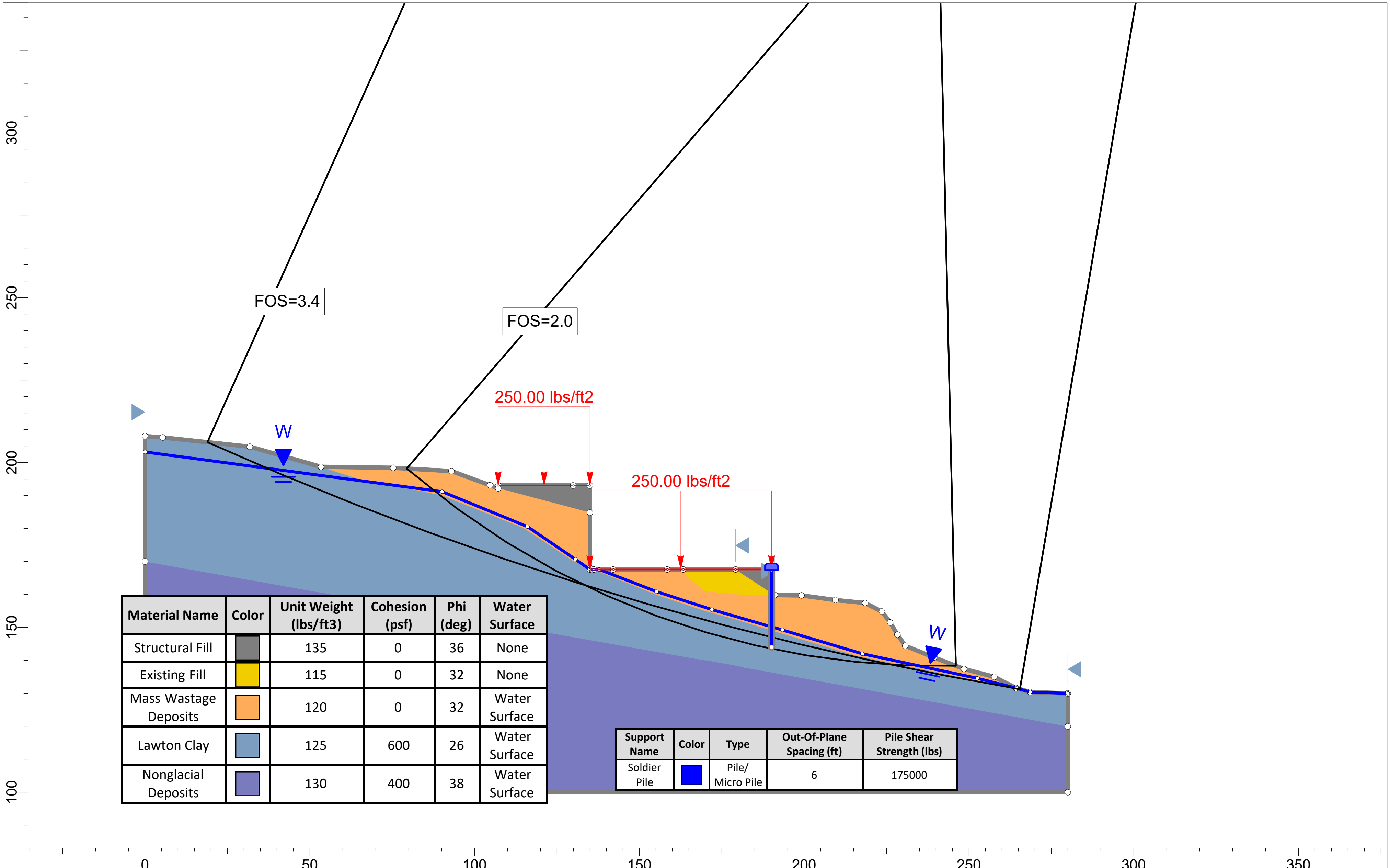



**Notes:**

1. Existing ground profile based on topographic survey by Informed Land Survey, dated January 6, 2020, supplemented with field developed section measurements.
2. Transitions between soil units and in between explorations are best estimates and may vary from the actual soil conditions.
3. See report text for detailed descriptions of subsurface conditions across site.
4. See Appendix A for detailed exploration logs.
5. See Figure 2A for Site Plan with approximate profile location.
6. Finished Floor elevation 168.7' (Level B2) from Architectural Plan dated 10/9/23.

| | | |
|---|---|---|
| PanGEO INCORPORATED | Proposed Single-Family Residence<br>9167 SE 64th Street<br>Mercer Island, Washington | GENERALIZED SUBSURFACE PROFILE - SECTION A-A'<br>Project No. 19-062 Figure No. 3 |

19-369 Stick Log and Profile Data.xls 10/7/24 (8:53:45) SHE




| Material Name | Color | Unit Weight (lbs/ft3) | Cohesion (psf) | Phi (deg) | Water Surface |
|---|---|---|---|---|---|
| Structural Fill | | 135 | 0 | 36 | None |
| Existing Fill | | 115 | 0 | 32 | None |
| Mass Wastage Deposits | | 120 | 0 | 32 | Water Surface |
| Lawton Clay | | 125 | 600 | 26 | Water Surface |
| Nonglacial Deposits | | 130 | 400 | 38 | Water Surface |

| Support Name | Color | Type | Out-Of-Plane Spacing (ft) | Pile Shear Strength (lbs) |
|---|---|---|---|---|
| Soldier Pile | | Pile/ Micro Pile | 6 | 175000 |

PanGEO INCORPORATED

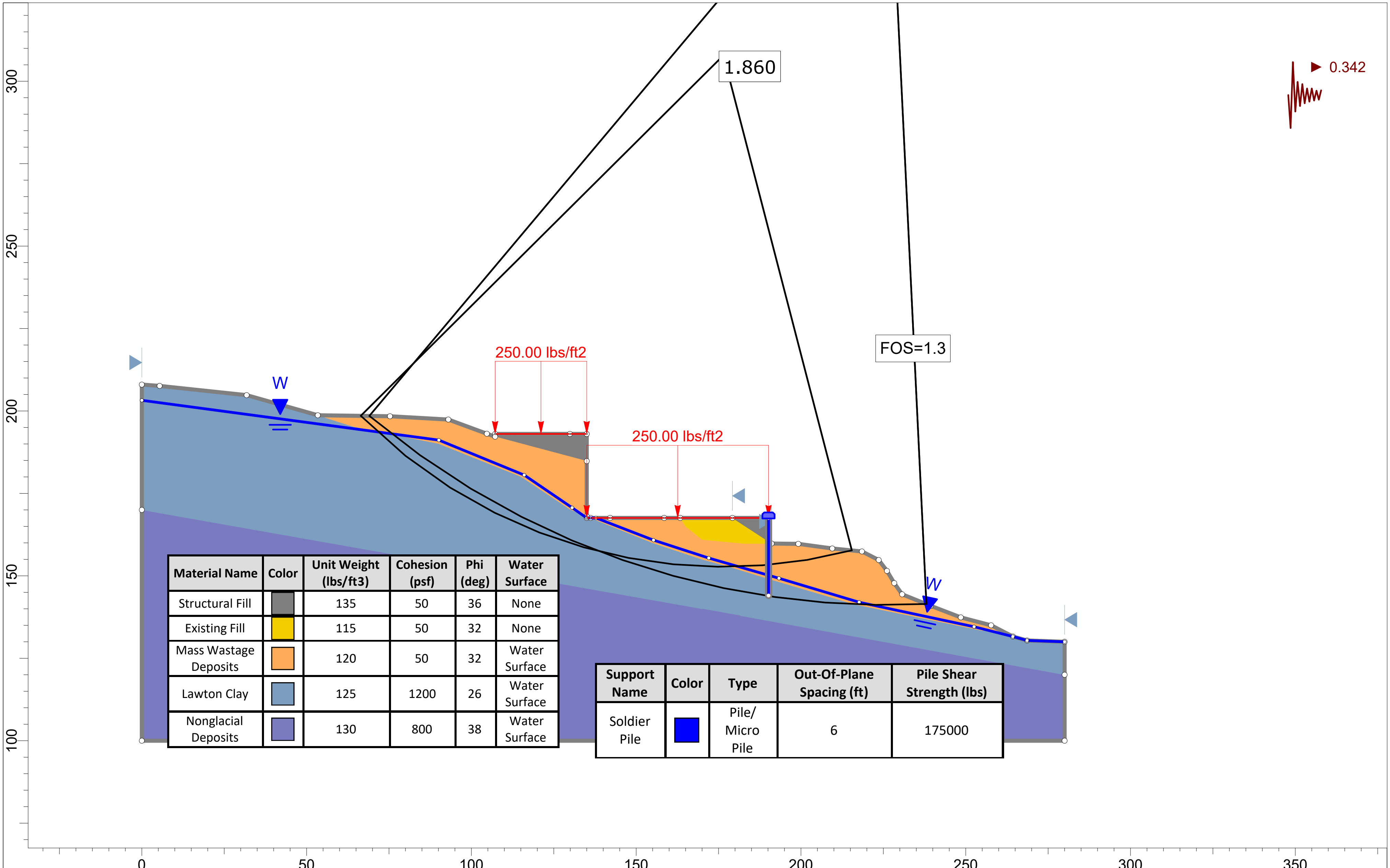
SLIDEINTERPRET 9.008

**Proposed Single-Family Residence**
**9167 SE 64th Street**
**Mercer Island, Washington**

**Static Global Stability Analysis - Stabilized Condition**

**Section A-A'**

| Scale: | Project No. | Figure No. |
|---|---|---|
| 1:480 | 19-062.300 | 4 |




| Material Name | Color | Unit Weight (lbs/ft3) | Cohesion (psf) | Phi (deg) | Water Surface |
|---|---|---|---|---|---|
| Structural Fill | | 135 | 50 | 36 | None |
| Existing Fill | | 115 | 50 | 32 | None |
| Mass Wastage Deposits | | 120 | 50 | 32 | Water Surface |
| Lawton Clay | | 125 | 1200 | 26 | Water Surface |
| Nonglacial Deposits | | 130 | 800 | 38 | Water Surface |

| Support Name | Color | Type | Out-Of-Plane Spacing (ft) | Pile Shear Strength (lbs) |
|---|---|---|---|---|
| Soldier Pile | | Pile/ Micro Pile | 6 | 175000 |

PanGEO INCORPORATED

SLIDEINTERPRET 9.008

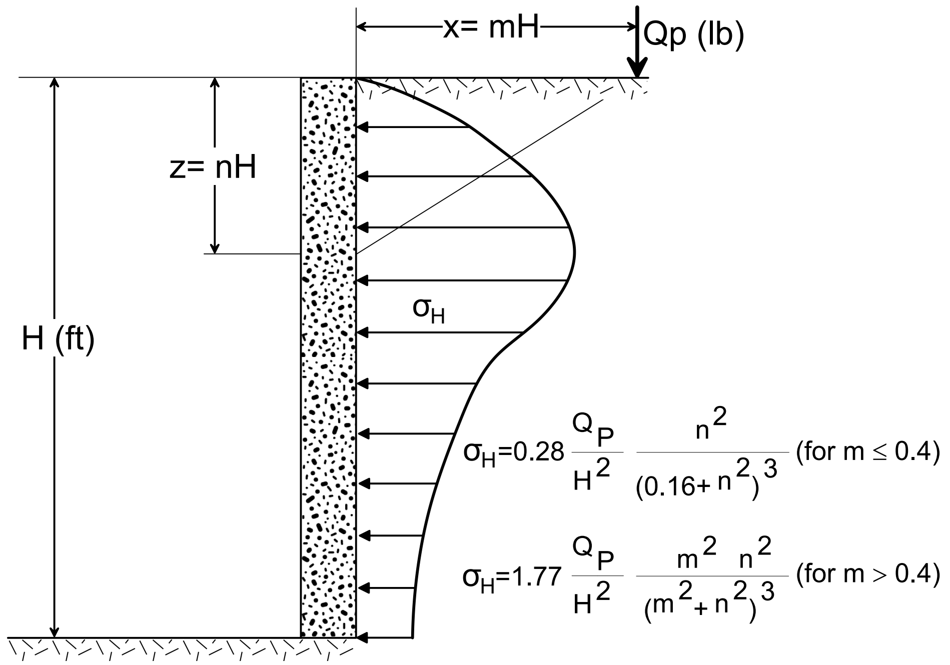
**Proposed Single-Family Residence**

**9167 SE 64th Street**

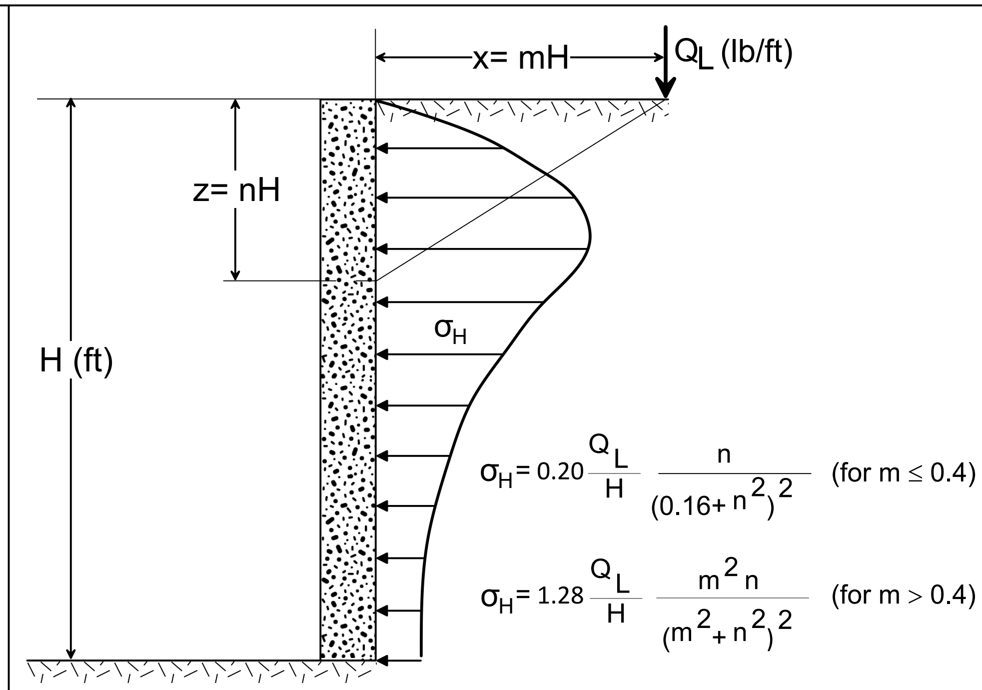
**Mercer Island, Washington**

**Pseudo-Static Global Stability Analysis - Stabilized Condition Section A-A'**

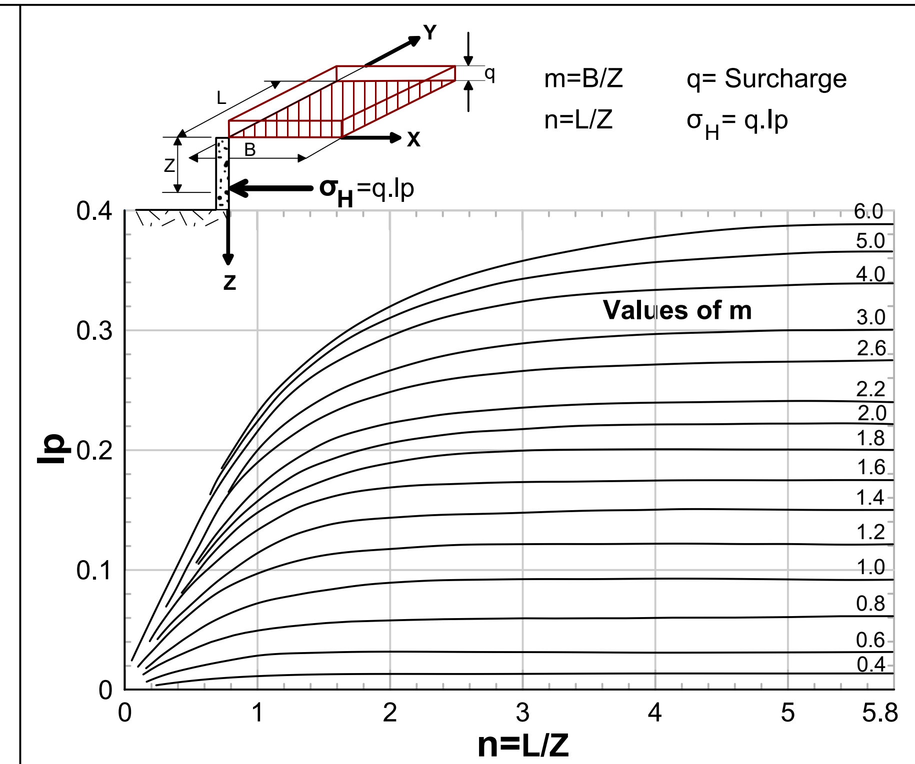
| Scale: | Project No. | Figure No. |
|---|---|---|
| 1:480 | 19-062.300 | 5 |



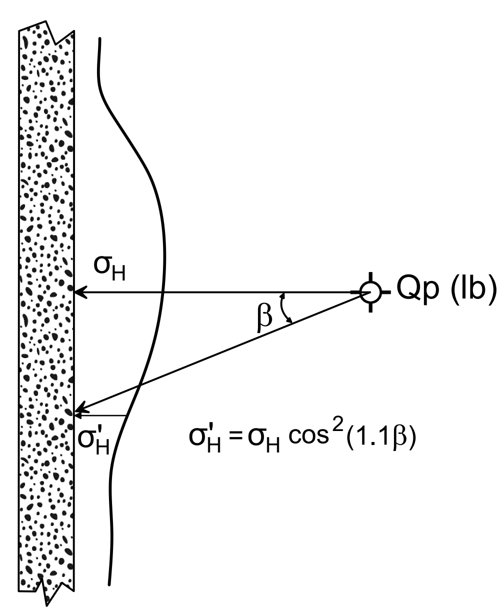

*A-1) Lateral Pressure Due to Point Load- Elevation View*



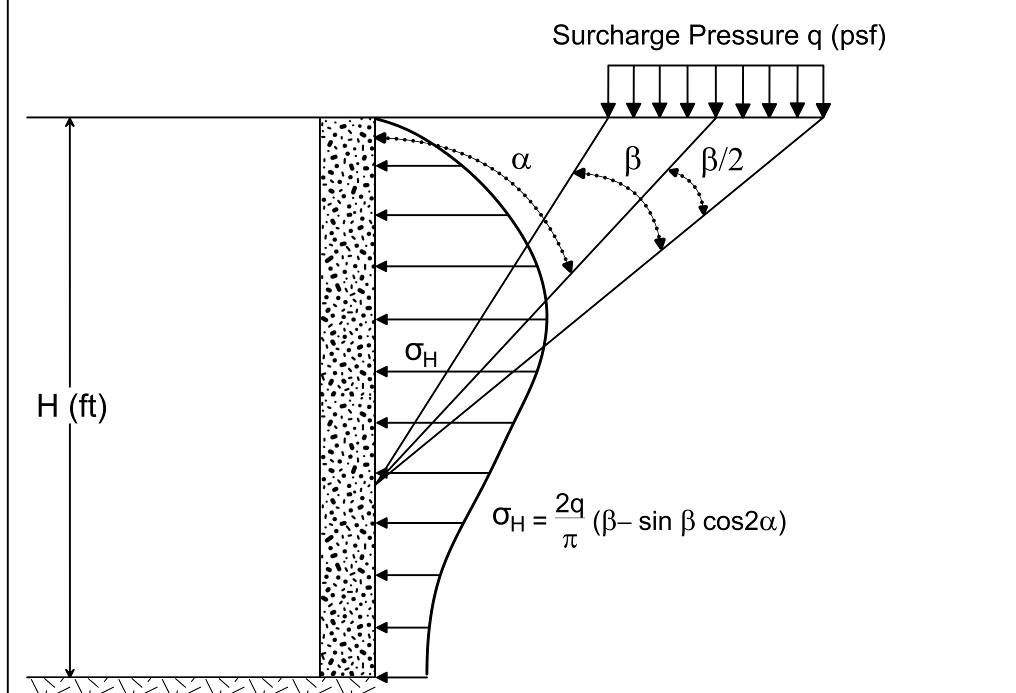

*B) Lateral Pressure Due to Line Load-Parallel to the Wall*



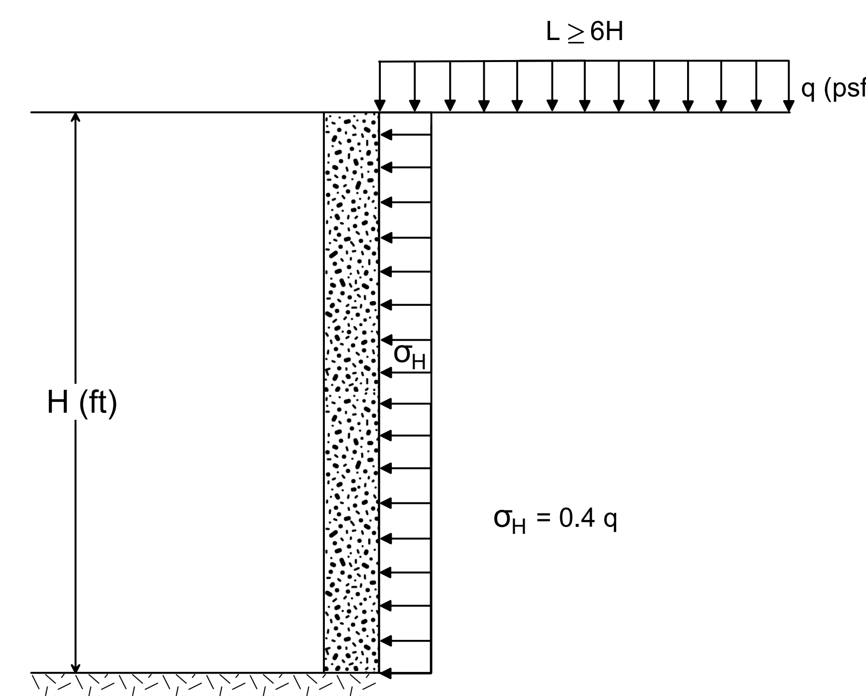

*D) Lateral Pressure Due to Adjacent Footing*




*A-2) Lateral Pressure Due to Point Load- Plan View*




*C) Lateral Pressure Due to Strip Load-Perpendicular to the Wall*




*E) Lateral Pressure Due to Uniform Surcharge.*
*(For L ≤ 6H Use Chart D Above)*

* $\sigma_H$ in psf.

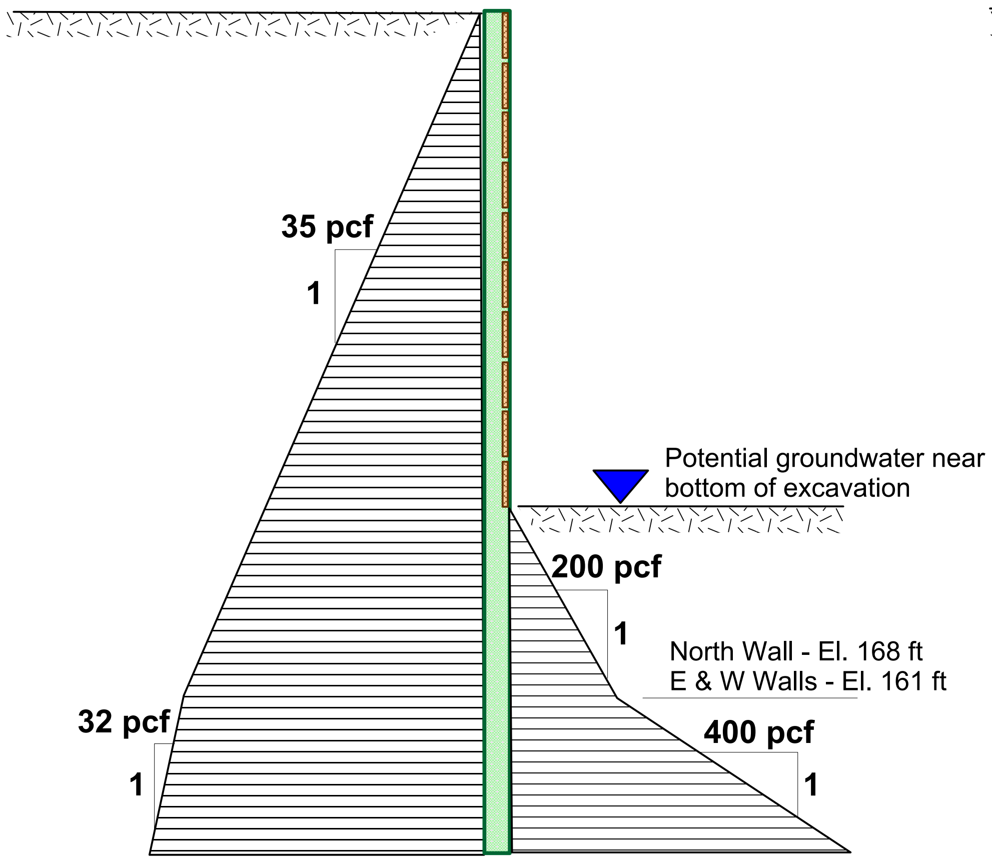
PanGEO INCORPORATED

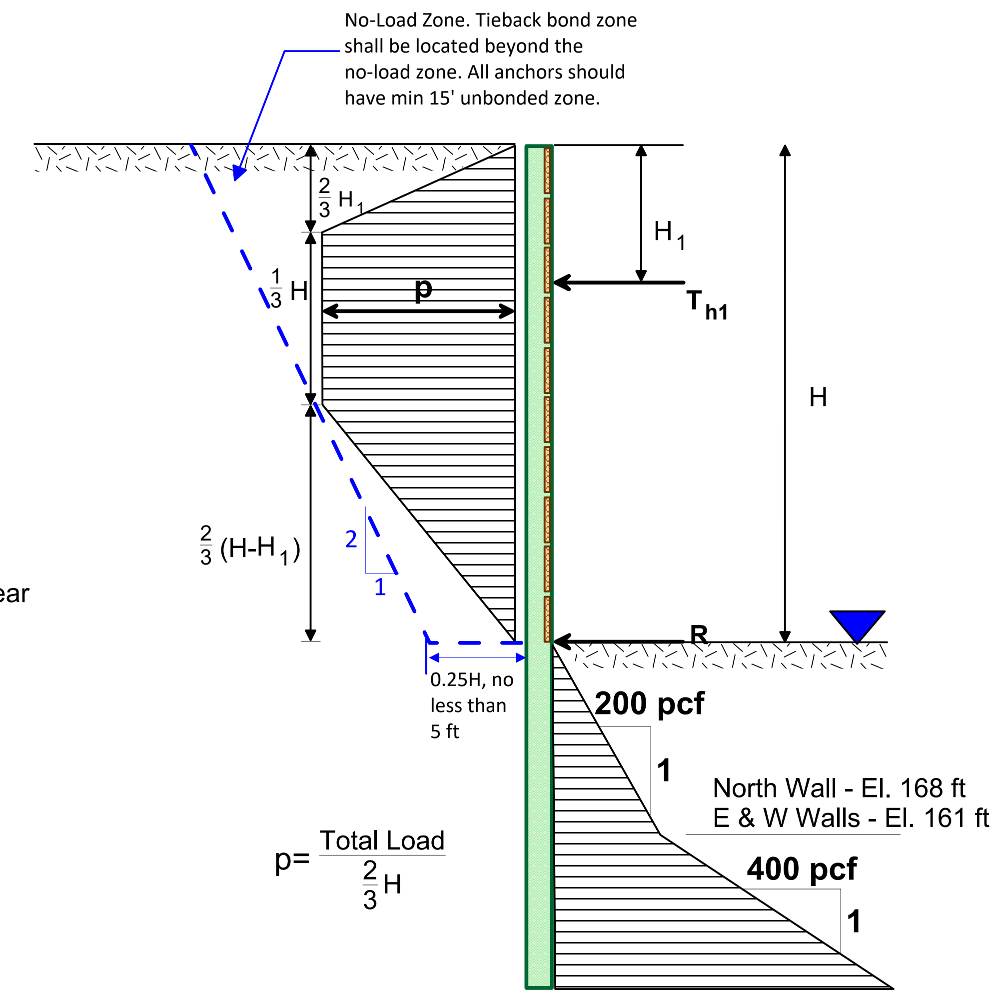
**Proposed Single-Family Residence**
**9167 SE 64th Street**
**Mercer Island, Washington**

**SURCHARGE LOADING ON WALLS**

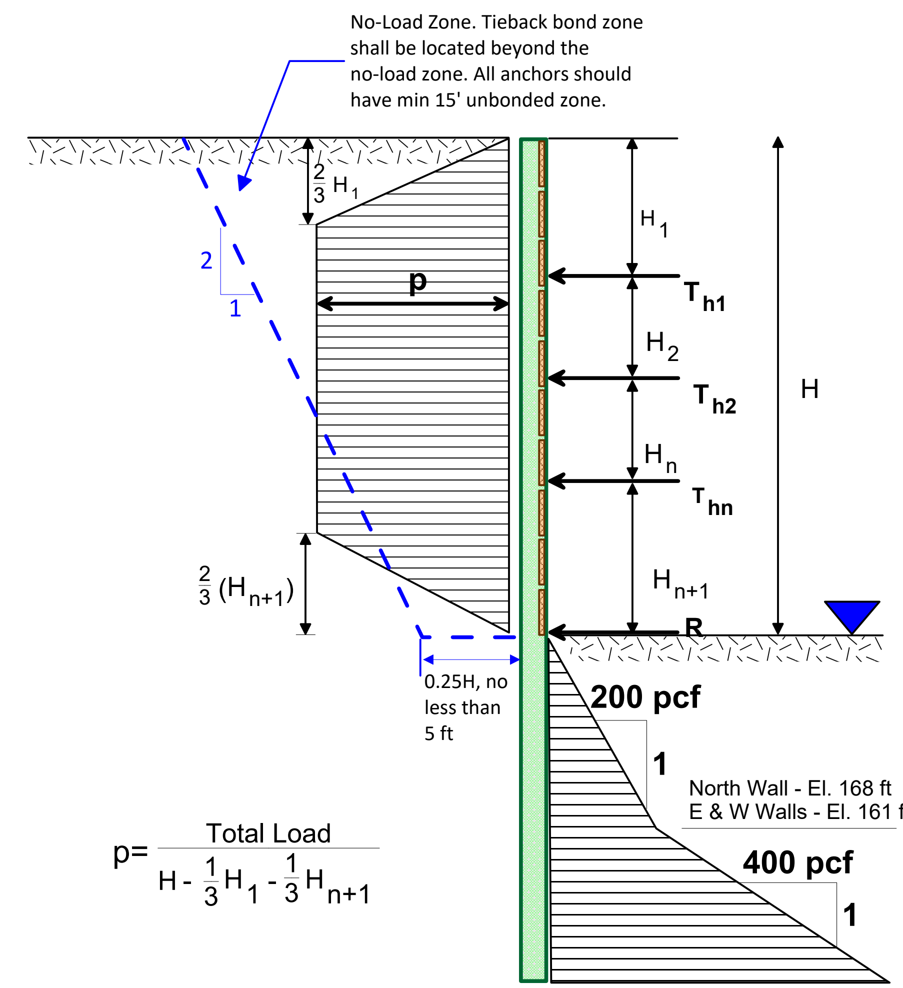
| Project No. | Figure No. |
|---|---|
| 19-062.300 | 6 |

19-062.300 Surcharge Loading for Walls 10/7/24 (8:36:24)




**Total Load= 0.65 $K_A \gamma H^2$ = $23H^2$**




**Total Load= 0.65 $K_A \gamma H^2$ = $23H^2$**

**a) Cantilever Wall**

**b) Walls with One Level of Ground Anchors**

**c) Walls with Multiple Levels of Ground Anchors**

**Legend:**

$H_1$ = Distance from the ground surface to uppermost ground anchor.

$H_{n+1}$= Distance from the base of excavation to lowermost ground anchor.

$T_{hi}$= Horizontal load in ground anchor i.

R= Reaction force to be resisted by subgrade (i.e., below the base of excavation).

p= maximum ordinate of diagram.

$K_A$= Active earth pressure coefficient.

$\gamma$ = Soil unit weight (pcf).

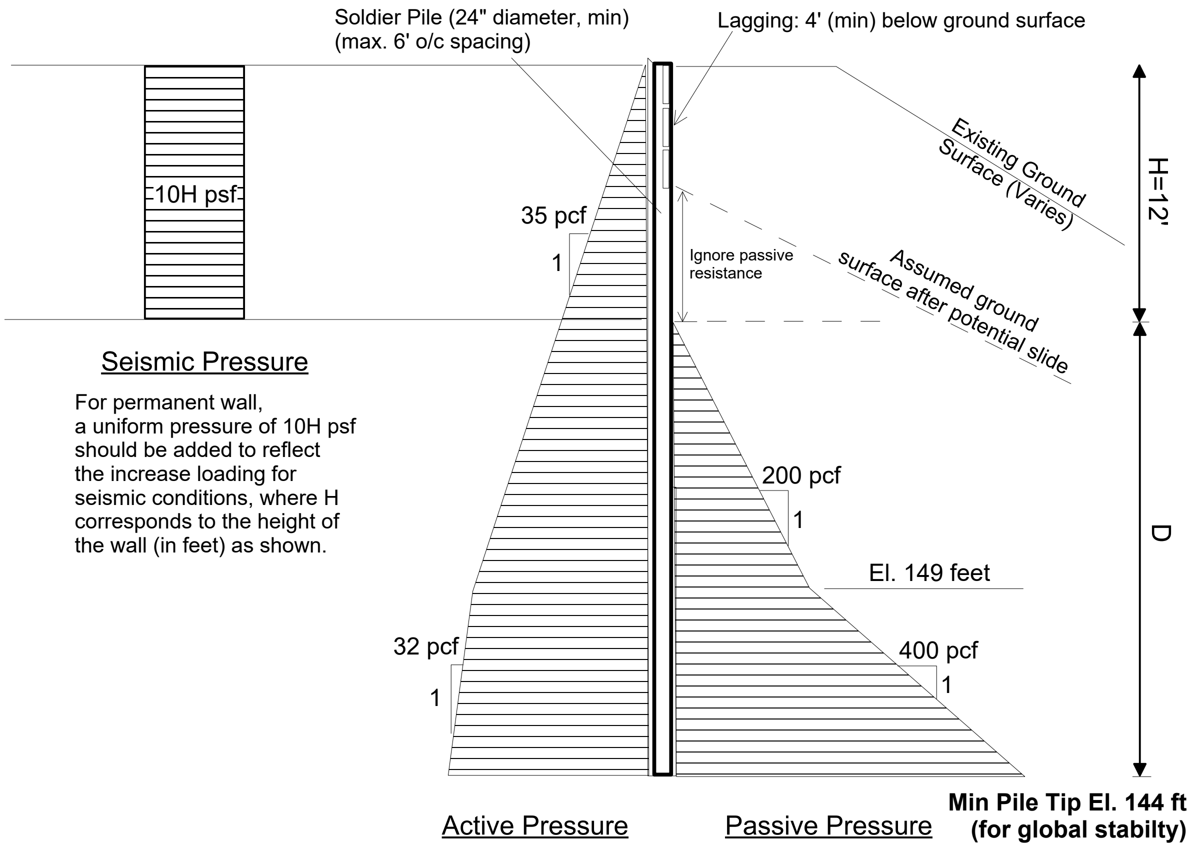
**Seismic Pressure:**
For permanent walls, a uniform pressure of 10H psf should be added to reflect the increase loading for seismic conditions, where H corresponds to the height of the wall (in feet).

PanGEO INCORPORATED

**Proposed Single-Family Residence**
**9167 SE 64th Street**
**Mercer Island, Washington**

**APPARENT EARTH PRESSURE**
**SOLDIER PILE WALLS**

| Project No. | Figure No. |
|---|---|
| 19-062.300 | 7 |




Notes:
1. Embedment (D) should be determined by summation of moments at the bottom of the soldier piles, or minimum tip elevation, as shown.
2. A factor of safety of 1.5 has been applied to the recommended passive earth pressure values. No factor of safety has been applied to the recommended active earth pressure values.
3. Active and seismic pressures should be applied over the full width of the pile spacing above the the bottom of excavation. The active pressure should be applied over one pile diameter below the bottom of excavation.
4. Passive pressure should be applied to two times the diameter of the soldier piles.
5. Use 50% of the lateral earth pressure for lagging design.
6. For permanent wall, piles should be treated for corosion protection, or oversized accordingly.
7. Refer to report text for additional discussions.

Fig 8 EP diagram.grf 10/7/24 (3:43:26) JCR

| | | |
|---|---|---|
| PanGEO INCORPORATED | **Proposed Single-Family Residence**<br>**9167 SE 64nd Street**<br>**Mercer Island, Washington** | **DESIGN LATERAL PRESSURES**<br>**SOUTH STABILIZING SOLDIER PILE WALL**<br>Project No. **19-062.300** Figure No. **8** |

# APPENDIX A

## CURRENT SUBSURFACE INVESTIGATION

**9167 SE 64th Street, Mercer Island, WA | PanGEO, Inc.**

## 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 |

| | | | |
|---|---|---|---|
| Project: | Proposed Single Family Residence | Surface Elevation: | ~186 ft |
| Job Number: | 19-062 | Top of Casing Elev.: | ~186 |
| Location: | 9167 SE 64th ST, Mercer Island, WA | Drilling Method: | HSA |
| Coordinates: | Northing: 47.54568, Easting: -122.21527 | Sampling Method: | SPT |

Depth, (ft): 0.0, 5.0, 10.0, 15.0, 20.0, 25.0, 30.0, 35.0, 40.0, 45.0, 50.0

| Sample No. | Sample Type | Blows / 6 in. | Other Tests |
|---|---|---|---|
| S-1 | ☒ | 4 5 6 | |
| S-2 | ☒ | 3 4 5 | |
| S-3 | ☒ | 4 3 4 | |
| S-4 | ☒ | 3 2 6 | |
| S-5 | ☒ | 0 0 4 | GS |
| S-6 | ☒ | 4 9 18 | |
| S-7 | ☒ | 5 7 14 | GS |
| S-8 | ☒ | 5 10 20 | |
| S-9 | ☒ | 6 9 15 | |
| S-10 | ☒ | 2 2 10 | |
| S-11 | ☒ | 8 13 17 | |
| S-12 | ☒ | 9 18 31 | |

## MATERIAL DESCRIPTION

**FILL**

Loose, dark brown, silty SAND with organics; moist.

**Mass-Wastage Deposits - [Qmw]**

Stiff, brown-gray, lean CLAY (CL) with sand, some iron oxide staining, scattered rootlets; moist.

-- becomes medium stiff.

-- becomes stiff, dark gray.

-- becomes soft - Minus #200 Mesh 81.8%.

**Lawton Clay - [Qvlc]**

Very stiff, gray, lean CLAY (CL), massive, blocky low plasticity; moist, light gray partings, hard waxy lenses with brittle breakage.

Minus #200 Mesh 97.1%.

-- becomes hard, residual sedimentary structures.

-- becomes very stiff.

Stiff, gray, sandy, lean CLAY, massive, low plasticity; wet.

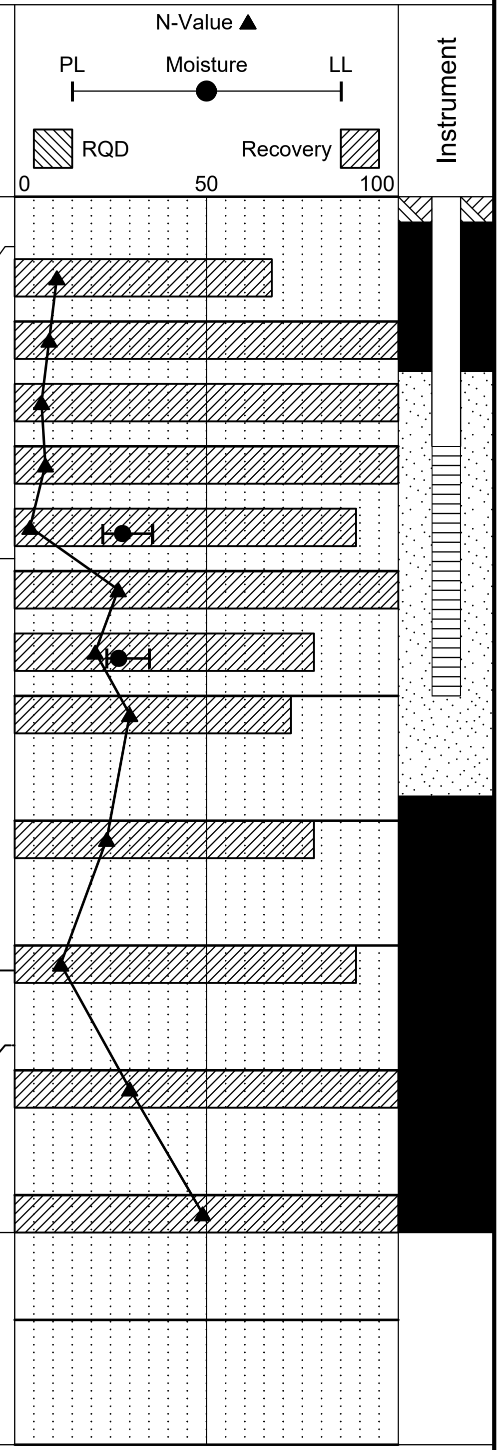
-- 1-inch-thick sand lens.

-- perched groundwater in sandy layer.

Hard, gray, lean CLAY, some silt, laminated, scattered sand partings; wet.

Boring terminated about 41.5 feet below ground surface.

Perched groundwater encountered about 18 feet below ground surface at the time of drilling, and at 15.3 BGS on 7/19/24 as measured in the piezometer installation.




| | |
|---|---|
| Completion Depth: | 41.5ft |
| Date Borehole Started: | 6/27/24 |
| Date Borehole Completed: | 6/27/24 |
| Logged By: | T. Howitz |
| Drilling Company: | Holocene Drilling |

Remarks: Borings drilled using a Diedrich D-50 tracked drill rig. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated by hydraulic mechanism. This surface elevation is estimated from topographic survey prepared by Informed Land Survey dated 1/6/2020. **Vertical Datum: NAVD 88. Horizontal Datum: WGS 84.**

PanGEO INCORPORATED

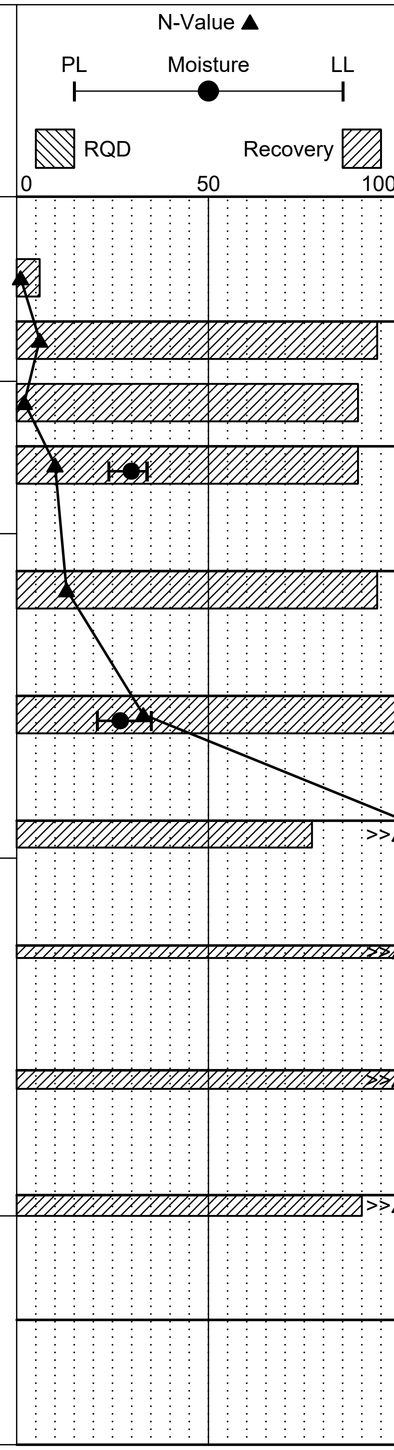
# LOG OF TEST BORING PG-1

**Figure A-2**

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

| | | | |
|---|---|---|---|
| Project: | Proposed Single Family Residence | Surface Elevation: | ~170 ft |
| Job Number: | 19-062 | Top of Casing Elev.: | na |
| Location: | 9167 SE 64th ST, Mercer Island, WA | Drilling Method: | HSA |
| Coordinates: | Northing: 47.54567, Easting: -122.21536 | Sampling Method: | SPT |

| Depth, (ft) | Sample No. | Sample Type | Blows / 6 in. | Other Tests | Symbol | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|
| 0.0 | | | | | | **FILL**<br>Loose, dark brown, silty SAND with organics; jumbled texture, woody debris,moist. |
| | S-1 | | 2 0 1 | | | |
| 5.0 | S-2 | | 2 3 3 | | | |
| | S-3 | | 0 0 2 | | | **Mass-Wastage Deposits - [Qmw]**<br>Soft to stiff, brown-gray, lean SILT (ML), some sand and sand lenses, mixed textures, some iron oxide staining, moist to very moist. |
| 10.0 | S-4 | | 2 3 7 | GS | | -- becomes stiff, Minus #200 Mesh 87.4%. |
| | | | | | | **Lawton Clay - [Qvlc]**<br>Hard, gray, lean CLAY (CL), massive to laminated, low plasticity; moist, trace shearing, broken textures. |
| 15.0 | S-5 | | 5 5 8 | | | -- partings.<br>-- encountered perched groundwater. |
| 20.0 | S-6 | | 5 13 20 | GS | | Minus #200 Mesh 88.8%. |
| 25.0 | S-7 | | 5 11 50/1 | | | |
| | | | | | | **Nonglacial Deposits - [Qpon]**<br>Very dense, fine SAND with trace silt, fine gravel in upper 3 inches of layer; moist. |
| 30.0 | S-8 | | 50/6 | | | |
| 35.0 | S-9 | | 47 50/3 | | | |
| 40.0 | S-10 | | 40 50/4 | | | |
| | | | | | | Boring terminated about 40.83 feet below ground surface.<br>Perched groundwater encountered about 18 feet below ground surface at the time of drilling. |
| 45.0 | | | | | | |
| 50.0 | | | | | | |




| | |
|---|---|
| Completion Depth: | 41.5ft |
| Date Borehole Started: | 6/27/24 |
| Date Borehole Completed: | 6/27/24 |
| Logged By: | T. Howitz |
| Drilling Company: | Holocene Drilling |

Remarks: Borings drilled using a Diedrich D-50 tracked drill rig. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated by hydraulic mechanism. This surface elevation is estimated from topographic survey prepared by Informed Land Survey, dated 1/6/2020. **Vertical Datum: NAVD 88. Horizontal Datum: WGS 84.**

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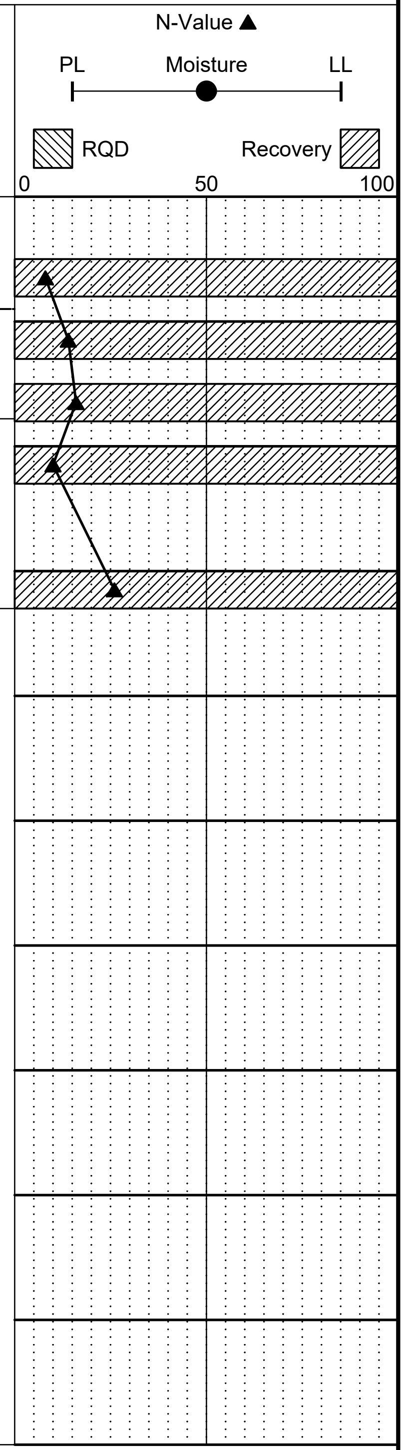
# LOG OF TEST BORING PG-2

**Figure A-3**

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

| | | | |
|---|---|---|---|
| Project: | Proposed Single Family Residence | Surface Elevation: | ~189.5 ft |
| Job Number: | 19-062 | Top of Casing Elev.: | na |
| Location: | 9167 SE 64th ST, Mercer Island, WA | Drilling Method: | HSA |
| Coordinates: | Northing: , Easting: | Sampling Method: | SPT |

| Depth, (ft) | Sample No. | Sample Type | Blows / 6 in. | Other Tests | Symbol | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|
| 0.0 | | | | | | **Mass-Wastage Deposits - [Qmw]**<br>Loose, brown, SILT with fine sand, moist, non-plastic with rapid dilatancy, homogeneous, occasional rusty pockets, laminated. |
| | S-1 | | 3 4 4 | | | |
| 5.0 | S-2 | | 4 6 8 | | | Medium dense, brown gray, silty, fine SAND to sandy SILT, very moist with wet zones, non-plastic with rapid dilatancy, homogeneous, laminated, wood at top.. |
| | S-3 | | 6 7 9 | | | Occasional rusty laminae, abundant fine sparkles - qtz grains?. |
| 10.0 | S-4 | | 5 5 5 | | | **Lawton Clay - [Qvlc]**<br>Medium stiff to very stiff, gray, silty, lean CLAY, very moist, low plasticity; no dilatancy, homogeneous, scattered rusty pockets, indistinctly laminated, rare blady organics. |
| 15.0 | S-5 | | 9 12 14 | | | |
| | | | | | | Boring terminated about 16.5 feet below ground surface.<br>Perched groundwater encountered about 7 feet below ground surface at the time of drilling. |
| 20.0 | | | | | | |
| 25.0 | | | | | | |
| 30.0 | | | | | | |
| 35.0 | | | | | | |
| 40.0 | | | | | | |
| 45.0 | | | | | | |
| 50.0 | | | | | | |




| | |
|---|---|
| Completion Depth: | 16.5ft |
| Date Borehole Started: | 3/21/19 |
| Date Borehole Completed: | 3/21/19 |
| Logged By: | S. Evans |
| Drilling Company: | Boretec, Inc. |

Remarks: Borings drilled using an Acker drill rig. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated by rope and Cathead mechanism. This surface elevation is estimated from topographic survey prepared by Informed Land Survey, dated 1/6/2020. **Vertical Datum: NAVD 88. Horizontal Datum: WGS 84.**

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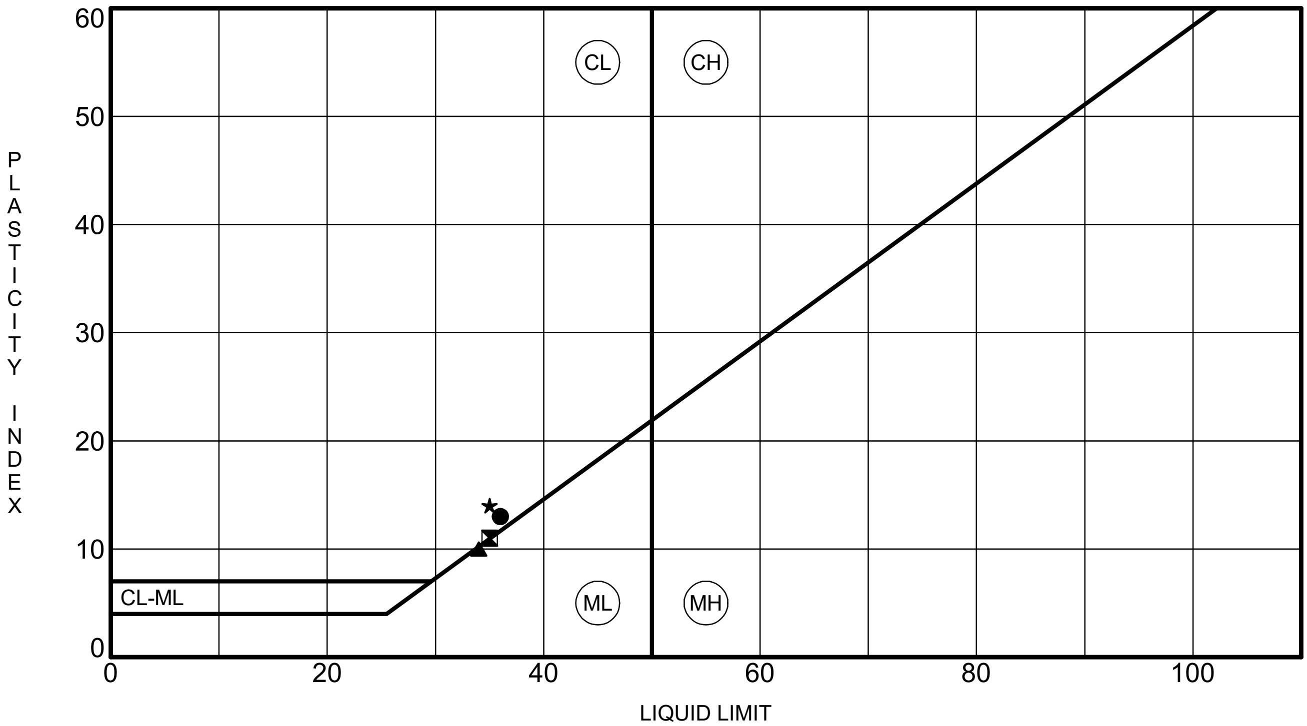
# LOG OF TEST BORING PG-7-19

**Figure A-4**

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

# APPENDIX B

## LABORATORY TEST RESULTS




| | Specimen Identification | | M | LL | PL | PI | Fines | Classification |
|---|---|---|---|---|---|---|---|---|
| ● | **PG-1** | **13.5** | **28** | **36** | **23** | **13** | **82** | **LEAN CLAY with SAND(CL)** |
| ⧗ | **PG-1** | **18.5** | **27** | **35** | **24** | **11** | **97** | **LEAN CLAY(CL)** |
| ▲ | **PG-2** | **11.0** | | **34** | **24** | **10** | **87** | **SILT(ML)** |
| ★ | **PG-2** | **21.0** | **27** | **35** | **21** | **14** | **89** | **LEAN CLAY(CL)** |


PanGEO INCORPORATED
Phone: 206.262.0370


# ATTERBERG LIMITS

Project: E Mercer Way 4-Lot Dev West Parcel
Job Number: 19-062
Location: 9167 SE 64th ST, Mercer Island, WA

**Figure B-1**