GEOTECHNICAL ENGINEERING REPORT
PROPOSED RESIDENCE
2003 82nd AVENUE SE
MERCER ISLAND, WASHINGTON

Project No. 19-012
March 4, 2019

Credit: Google Earth

Prepared for:

Nick Phillips

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March 4, 2019
PanGEO Project No. 19-012

Mr. Nick Phillips
2003 – 82nd Avenue SE
Mercer Island, Washington 98040

Subject: **Geotechnical Engineering Report**
Proposed Residence
2003 – 82nd Avenue SE
Mercer Island, Washington 98040

Dear Mr. Phillips:

As requested, PanGEO has completed a geotechnical study for the proposed single-family residence at the above address. In preparing this report, we performed a reconnaissance of the property, reviewed existing data, drilled three test borings at the site, and conducted engineering analyses. The results of our study and our design recommendations are presented in the attached report.

In summary, the proposed house footprint is underlain by competent glacially consolidated soils at shallow depths. In our opinion, the proposed development is feasible from the geotechnical standpoint, and provided that the recommendations presented in this report are incorporated into the design and construction of the project, the proposed development will not adversely affect the project site or surrounding properties. The new structure may be supported by conventional footings. A soldier pile wall represents a feasible excavation support system to allow for the construction of the proposed house basement while maintaining stability of the site.

We appreciate the opportunity to be of service. Should you have any questions, please do not hesitate to contact us.

Sincerely,

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

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MERCER ISLAND, WASHINGTON

1.0 GENERAL

PanGEO, Inc. is pleased to present the following geotechnical engineering report to assist the project team with the design and construction of the proposed residence at 2003 82nd Avenue SE, in Mercer Island, Washington. This study was prepared in general accordance with our mutually agreed scope of services outlined in our proposal dated January 8, 2019, which was approved on the same date. Our scope of services included reviewing readily available geologic and geotechnical data, conducting a site reconnaissance, advancing three test borings at the site, conducting engineering analyses, and preparing the following geotechnical report.

2.0 SITE AND PROJECT DESCRIPTION

The subject site is located at 2003 82nd Avenue SE in Mercer Island, Washington, as shown on Figure 1, Vicinity Map. The site consists of an approximately 1 acre, irregularly shaped parcel that measures a maximum of about 600 feet in the north-south direction, and up to about 200 feet in the east-west direction. The property includes about 50 feet of frontage along Lake Washington to the north. The site is surrounded by single-family homes, and is situated immediately west of the intersection of 82nd Avenue SE and 81st Avenue SE.

The site is currently occupied by a two-story single-family residence with daylight basement on the eastern portion of the site. The existing residence is accessed by a driveway from 82nd Avenue SE. The remainder of the site is undeveloped, with the exception of a north-south trending gravel access road that runs the entire length of the property, and terminates at the shoreline of Lake Washington.

The site is scarcely forested with mature native evergreen trees, and includes an understory of ferns, vines and other native plant species.

The topography of the southern portion of the site slopes down from east to west at grades of approximately 40%, with some areas slightly steeper, and some areas flatter. The northern, narrow extension of the property that extends to Lake Washington generally slopes down moderately to gently from the south to north. Based on our review of the topographic survey, prepared by Cascade Land Surveying, site grades along the eastern property line are about 144 feet (NAVD88) and site grades along the western property line are as low as 62 feet.

Plate 1 below depicts current site conditions.

Plate 1. *Looking northeast, near south end of the site, at the general location of the current and proposed residence. (01/04/2019)*

We understand that the proposed project includes the demolition of the existing structure and the construction of a three-level single-family residence, with the lower two levels daylighting to the west. The new house will be located within the approximately same footprint as the existing house, but will extend slightly further west. We understand the basement floor elevation of the new house will be around 113.5 feet (NAVD), which is deeper than the existing basement floor elevation. We anticipate that the excavation necessary to construct the basement walls of the new house will extend up to about 20 feet below existing grades.

A new driveway and auto court will also be constructed to access the new garage located at the south end of the house. The driveway may either originate from 81st Avenue SE, or may be accessed from the private asphalt drive south of the site. Retaining walls along the upslope and downslope side of the driveway and auto court will be needed to accommodate the change in grade.

A 900 square-foot accessory structure is also proposed at the site, and will be located at the far north end of the property, near the shoreline of Lake Washington.

Figure 2 depicts the approximate location of the proposed residence and accessory structure in relation to the property boundaries and existing site features.

3.0 SUBSURFACE EXPLORATIONS

3.1 CURRENT EXPLORATIONS

A subsurface exploration program was completed on January 24, 2019. The subsurface exploration program included three test borings (PG-1 through PG-3) that were advanced on the subject site. The approximate test boring locations were measured from existing site features and are indicated on the attached Site and Exploration Plan (Figure 2). Two borings (PG-1 and PG-2) were drilled to depths of about 11 to 14.5 feet below ground surface using a limited access, portable Acker drill rig. A third boring (PG-3) was advanced to about 14 feet below ground surface using an RCT 60 small track mounted rig. The drill rigs were owned and operated by Boretec 1 Inc., of Bellevue, Washington. Drill rigs were equipped with a 5-inch outside diameter hollow stem auger, and soil samples were obtained from the borings at 2½ and 5-foot intervals in general accordance with Standard Penetration Test (SPT) sampling methods (ASTM test method D-1586) in which the samples are obtained using a 2-inch outside diameter split-spoon sampler. The sampler was driven into the soil a distance of 18 inches using a 140-pound weight falling a distance of 30 inches. The number of blows required for each 6-inch increment of sampler penetration was recorded. The number of blows required to achieve the last 12 inches of sample penetration is defined as the SPT N-value. The N-value provides an empirical measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils.

A geologist from PanGEO was present during the field explorations to observe the test borings, obtain representative samples, and to describe and document the soils encountered in the explorations. The completed borings were backfilled with bentonite chips.

The soil samples retrieved from the borings were described using the system outlined on Figure A-1 of Appendix A, and the summary boring logs are included as Figures A-2 through A-4.

3.2 PREVIOUS EXPLORATIONS

Boring PG-3[2015] was advanced by PanGEO on the adjacent property to the east for a different geotechnical study in 2015, but was utilized for this study to evaluate subsurface conditions in the area of the proposed accessory structure. The approximate location of the previously advanced test boring is shown on the attached Figure 2. PG-3[2015] was advanced using a limited access, portable Acker drill rig owned and operated by CN Drilling, of Seattle, Washington. Drill rig was equipped with a 5-inch outside diameter hollow stem auger, and soil samples were obtained from the boring at 2½ and 5-foot intervals in general accordance with Standard Penetration Test (SPT) sampling methods (ASTM test method D-1586). The soil samples retrieved from the boring was described using the system outlined on Figure A-1 of Appendix A, and the summary boring log for PG-3[2015] is included in Appendix B.

4.0 SUBSURFACE CONDITIONS

4.1 SITE GEOLOGY

According to the *Geologic Map of Mercer Island, Washington* (Troost and Wisher, 2006), the project site is underlain deposits sourced from Pre-Olympia aged glacial and non-glacial deposits ranging from very dense coarse grain deposits consisting of sand and gravel to hard, fine grain deposits of silt and clay. The map indicates that the underlying older deposits are capped by Vashon Glacial Till on the east side of 82nd Avenue SE. Recent lake deposits are mapped along the shoreline of Lake Washington at the north portion of the site. Lastly, the map shows mass-wastage deposits directly west of the subject site, and on the lower slopes of the subject site.

The Pre-Olympia fine-grained glacial deposits (Q_{pogf}) and Pre-Olympia fine-grained deposits (Q_{pof}) mapped at the site are described by Troost and Wisher as hard, silt and clay with interbedded sands.

4.2 Soil Conditions

The subsurface explorations at the site generally encountered a sequence of topsoil or fill over native glacially consolidated fine-grained deposits. The deposits encountered appeared to be consistent with the mapped geology described above.

The soils encountered at each of the subsurface exploration locations are described in the boring logs presented in Appendix A and B of this report. The attached Figure 3 presents a generalized subsurface profile across the site (Section A-A') based on our interpretation of the subsurface conditions encountered in the explorations.

A summary of the generalized soil units encountered in our test borings are presented below.

> ***Topsoil:*** A thin surficial layer of topsoil or forest duff was encountered in borings PG-1 and PG-2. The organic rich soil unit was generally less than 12 inches thick, and consisted of loose silty sand with scattered to prevalent organics and rootlets.
>
> ***Fill:*** Approximately seven feet of loose to medium dense, silty sand with trace gravel and rootlets was observed in boring PG-3, which was advanced in the driveway behind the basement wall of the existing house. We interpreted this soil to be backfill placed during original construction of the house. Fill soils were not encountered in PG-1 or PG-2. In boring PG-3[2015], which was advanced near the proposed accessory structure, about 3 to 4 feet of medium dense silty sand and stiff sandy silt was encountered below the ground surface, that was interpreted to be fill.
>
> ***Pre-Olympia Fine-Grained Glacial Deposits (Q_{pogf}):*** Underlying the topsoil and fill, all test borings advanced at the site the site generally encountered medium dense to very dense and medium stiff to hard interbedded silty fine sand, sandy silt, and silty clay with some gravel, that we interpreted to be the mapped Pre-Olympia Fine-Grained Glacial Deposits (Q_{pogf}). This unit was encountered to the termination depth of about 14½ feet, 14 feet, and 19 feet in borings PG-1, PG-3 and PG-3[2015] respectfully. In boring PG-3, refusal was reached in this unit on a large cobble or boulder. In boring PG-2, this unit terminated about 9 feet below the ground surface.

Pre-Olympia Fine-Grained Deposits (Q_{pof}): Underlying the glacial deposits in boring PG-2, a hard, blue-grey sandy silt with trace gravel was encountered. This unit was interpreted to be Pre-Olympia Fine-Grained Deposits (Q_{pof}). PG-2 terminated in this unit at a depth of 11 feet.

4.3 GROUNDWATER CONDITIONS

At the time of our subsurface investigations (January 2019 and July 2015), groundwater was not encountered in test borings PG-1 through PG-3 and PG-3[2015]. Based on the observed soil conditions, we anticipate that groundwater may become perched within the fill soils on top of the underlying very dense or hard native deposits during certain times of the year. It should be noted that groundwater elevations and seepage rates are likely to vary depending on the season, local subsurface conditions, and other factors. Generally, the water level is higher and seepage rates are greater in the wetter, winter months (typically October through May).

5.0 GEOLOGIC HAZARDS ASSESSMENT

5.1 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 indicates that slopes of 15% or more and slopes between 40-79% are present at the site. The map also indicates that mass wasting deposits exist over the lower western slopes of the site, and a landslide scarp is mapped along the eastern property line of the subject site. According to the map, a previously documented landslide is located several parcels to the north of the subject site.

Site Reconnaissance: A site reconnaissance was conducted on January 24, 2019. As part of our site reconnaissance we traversed the slope to look for evidence of past or on-going slope instability, including the mapped scarp on the site, and mass wasting deposits. During our site reconnaissance we did not observe evidence of past instability in the project area, such hummocky terrain, obvious slide scarps, uneven topography, or tension cracks. Along the top of the slope, where the geologic hazard map indicated the presence of a scarp, we did not observe evidence of a scarp, but did observe a slightly over-steepened slope which we infer was created by filling for the 81st Avenue SE roadway alignment above. No evidence of mass wasting deposits were noted in the immediate vicinity of the proposed developed area, nor were mass wasting deposits encountered in our test borings.

We observed the mature trees on the site to have generally straight trunks, and no evidence of significant soil creep was observed. The slopes below the existing residence were uniform in nature, and appeared to slope at an angle of about 2H:1V to 2.5H:1V, which is consistent with our review of the topographic survey. Plates 2 and 3 below depict slope conditions downslope of the existing and proposed developed area.

	
Plate 2. *Looking southeast at slope below existing/proposed residence.*	***Plate 3.*** *Looking south along west side of house at slope below existing residence.*

During our site reconnaissance we also observed the condition of the existing residence, to look for signs of settlement and distress, which may indicate slope movement. No significant foundation cracks, evidence of tilting, or displacement was noted in the exposed portion of the existing house foundation.

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, small, shallow surficial slides are the likely type of failure that could occur on the steepest portions of the site. However, due to the limited amount of surficial loose soils encountered in our test borings, the lack of observed evidence of recent shallow slides, and the relatively thick vegetation cover which protects the surface of the slope from erosion, in our opinion the potential for a shallow slides at the site is relatively low.

It is our opinion that the proposed development as currently planned is 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 development is properly design and constructed. Our recommendations include the use of a soldier pile wall shoring wall to provide temporary support for the proposed basement excavation, and adequate embedment of the house foundation below surficial soils that may be prone to downslope movement.

5.2 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 the very stiff to hard glacial soils underlying the proposed building sites, as well as the lack of groundwater, in our opinion, the potential for soil liquefaction during an IBC-code level earthquake is considered minimal, and special design considerations associated with soil liquefaction are not required.

It is also our opinion that the potential for significant seismic-induced land sliding is relatively low at the site due to the dense and hard glacial soils underlying the slope, and lack of steep slopes greater than 2H:1V. Shallow slides within over-steepened portions of the slope could have the potential to be triggered by a seismic event. However, provided the design of the new development considers the potential of shallow slides triggered by a seismic event, such as adequate foundation embedment, in our opinion the potential shallow slides will not negatively impact the proposed development. It may also be noted that the site retaining walls will be designed to consider the seismic loading.

5.3 Erosion Hazards

The subject site is mapped within a potential erosion hazard area according to the City of Mercer Island's Geologic Hazards Map. Based on soil conditions encountered in the borings, the near-surface site soils are likely to exhibit moderate to low erosion potential. In our opinion, the erosion hazards at the site can be effectively mitigated with the best management practice 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 silt fencing at the construction perimeter, limiting removal of vegetation to the construction area, placing gravel or hay bales at the disturbed/traffic areas, covering stockpile soil or cut slopes with plastic sheets, constructing a temporary drainage pond to control surface runoff and sediment trap, and placing quarry spalls at the construction entrance.

Permanent erosion control measures should include establishing vegetation, landscape plants, and hardscape established at the end of project, and reducing surface runoff to the minimum extent possible.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 SEISMIC DESIGN PARAMETERS

The 2015 International Building Code (IBC) seismic design section provides a basis for seismic design of structures. Table 1 below provides seismic design parameters for the site that are in conformance with the 2015 IBC, which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps. The spectral response accelerations were obtained from the SEAOC/OSHPD website http://seismicmaps.org for the project address.

Table 1 – Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec. [g]	Spectral Acceleration at 1.0 sec. [g]	Site Coefficients		Design Spectral Response Parameters	
	S_S	S_1	F_a	F_v	S_{DS}	S_{D1}
D	1.36	0.523	1.0	1.5	0.906	0.523

Liquefaction Potential: Liquefaction is a process that can occur when soils lose 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, loose to medium dense, and must be saturated. Because the proposed building sites are not underlain by saturated silt or loose to medium dense sand, but instead generally very stiff to hard silt, in our opinion the liquefaction potential below the proposed structures is low, and design considerations related to soil liquefaction are not necessary for this project.

6.2 Spread Footings

Based on our understanding of the subsurface conditions at the site, in our opinion the proposed residence may be supported by conventional spread and strip footings. Footings should be founded on the medium dense to dense sandy soils or very stiff to hard sandy silt anticipated to be present at the proposed foundation elevation.

6.2.1 Foundation Embedment

Due to the sloping site grades, we recommend that the portion of the foundation along the downslope side of the structure have a minimum embedment of four feet below the existing ground surface.

6.2.2 Allowable Bearing Pressure

We recommend a maximum allowable soil bearing pressure of 3,000 pounds per square foot (psf) be used to size the footings for the main house. For the foundation design of the accessory structure, we recommend using a maximum allowable soil bearing pressure of 2,000 psf to size the footings, as we anticipate the footings for this structure will bear on stiff sandy silt and silty sand. The recommended allowable bearing pressure is for dead plus live loads. For allowable stress design, the recommended bearing pressure may be increased by one-third for transient loading, such as wind or seismic forces. Continuous and individual spread footings should have minimum widths of 18 and 24 inches, respectively.

Total and differential settlements are anticipated to be within tolerable limits for footings designed and constructed as discussed above. Footing settlement under static loading conditions is estimated to be less than about ¾-inch. We anticipate differential settlement across the footprint of the structure should be less than about ½-inch. Most settlement will occur during construction as loads are applied.

6.2.3 Lateral Resistance

Lateral loads on the structure may be resisted by passive earth pressure developed against the embedded portion of the foundation system and by frictional resistance between the bottom of the foundation and the supporting subgrade soils. Footings bearing on the stiff to hard sandy silt may be designed using a frictional coefficient of 0.3 to evaluate sliding resistance developed between the concrete and the subgrade soil. Passive soil resistance may be calculated using an equivalent fluid weight of 300 pcf, assuming foundations are backfilled with structural fill. The above values include a factor of safety of 1.5. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches of soil should be neglected.

6.2.4 Perimeter Footing Drains

Footing drains should be installed around the perimeter of the structures, at or just below the invert of the footings. Under no circumstances should roof downspout drain lines be connected to the footing drain systems. Roof downspouts must be separately tightlined to appropriate discharge locations, and must not be allowed to discharge onto slopes. Cleanouts should be installed at strategic locations to allow for periodic maintenance of the footing drain and downspout tightline systems.

6.2.5 Footing Subgrade Preparation

Footing subgrades should be in a firm and stable condition prior to setting forms and placing reinforcing steel. Any loose or softened soil should be removed from the footing excavations. The adequacy of the footing subgrade soils should be verified by a representative of PanGEO, prior to placing forms or rebar.

If loose or disturbed soil is encountered at the footing elevation, the footing may be lowered to bear on the undisturbed soils, or the unsuitable soils should be removed and replaced with properly compacted structural fill, or lean-mix concrete.

6.3 FLOORS SLABS

We anticipate that competent, native soils will be encountered at the slab level. Structural fill placed below the slab should be properly compacted in accordance with the structural fill recommendations presented in this report. The exposed subgrade should be compacted to a firm condition prior to placing the backfill or capillary break layer. Conventional slab on grade construction may be used for the floor slabs in the house and accessory structure. The floor slab design may be accomplished using a modulus of subgrade reaction of 125 pci.

Interior concrete slab-on-grade floors should be underlain by a capillary break consisting of at least of 4 inches of pea gravel or compacted 5/8-inch, clean crushed rock (less than 3 percent fines). The capillary break material should meet the gradational requirements provided in Table 2, below.

Table 2 – Capillary Break Gradation

Sieve Size	Percent Passing
¾-inch	100
No. 4	0 – 10
No. 100	0 – 5
No. 200	0 – 3

The capillary break should be placed on the subgrade that has been compacted to a dense and unyielding condition.

We recommend that a 10-mil polyethylene vapor barrier should also be placed directly below the slab. Construction joints should be incorporated into the floor slab to control cracking.

6.4 Basement Wall Design Parameters

Below-grade walls should be properly designed to resist the lateral earth pressures exerted by the soils behind the wall. Proper drainage provisions should also be provided behind the walls to intercept and remove groundwater from behind the wall. Our geotechnical recommendations for the design and construction of the below-grade walls are presented below.

6.4.1 Lateral Earth Pressures

We anticipate that a temporary soldier pile wall will be used for shoring around the majority of the basement perimeter. The below grade portions of basement walls cast against the shoring walls may be designed for an earth pressure based upon an equivalent fluid weight of 35 pcf, assuming a level backslope. For a basement wall that is constructed in an open cut and then backfilled, the wall may be designed for an earth pressure based upon an equivalent fluid weight of 35pcf for a wall that is allowed to yield, and 50 pcf for a wall that is restrained (assuming level backslope). The recommended lateral pressures assume that the backfill behind the wall consists of a free draining and properly compacted fill with adequate drainage provisions.

A uniform pressure of 8H psf should be added to all basement walls to reflect the increase loading for seismic conditions, where H corresponds to the buried depth of the wall.

If surcharge loads or building foundations will be located within a horizontal distance equal to the height of the backfilled wall, lateral earth pressures will need to be increased based upon the type and magnitude of surcharge.

6.4.2 Lateral Resistance

Lateral forces from wind or seismic loading may be resisted by the combination of passive earth pressures acting against the embedded portions of the foundations and by friction acting on the base of the foundations. Passive resistance values may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf). This value includes a factor of safety of at least 1.5 assuming that a properly compacted structural fill will be placed adjacent to the sides of the footings. A coefficient friction of 0.30 may be used to determine the frictional resistance at the base of the footings. This coefficient includes a factor of safety of approximate 1.5.

6.4.3 Wall Backfill

Based on the results of our test borings, the on-site soils consist of sandy silt and silty sand. The silty soils would not be suitable to be re-used as wall backfill. For budgeting purpose, we recommend that wall backfill consist of imported free draining granular soils such as Seattle Mineral Aggregate Type 17 or Gravel Borrow as defined in Section 9-03.14(1) of the WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT, 2016). In areas where the space is limited between the wall and the face of excavation, clean crushed 5/8-inch rock may be used as backfill without compaction.

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. Within 5 feet of the wall, the backfill should be compacted to 90 percent of the maximum dry density.

6.4.4 Wall Drainage & Damp Proofing

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 pea 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.

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 through weep holes inside the building beneath the floor slab and tight-lined to an appropriate outlet.

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

6.5 TEMPORARY & PERMANENT SOLDIER PILE WALLS

We anticipate that a temporary shoring wall will be needed along the upslope side of the new house to allow construction of the deep daylight basement. In our opinion a soldier pile wall represents a feasible temporary shoring system to maintain stability of the excavation, and protect adjacent properties. We anticipate that the temporary shoring wall may need to be about 20 feet tall.

In addition, we anticipate that a permanent soldier pile wall would be a feasible wall type for site walls along the proposed driveway, and above or below the proposed auto court.

We offer the following geotechnical design recommendations for the proposed temporary and potentially permanent soldier pile walls utilized for this project.

6.5.1 Soldier Pile Wall

A soldier pile wall consists of vertical steel beams, typically spaced from 6 to 8 feet apart along the proposed wall alignment, spanned by timber lagging. Prior to the start of excavation, the steel beams are installed into holes drilled to a design depth and then backfilled with lean mix or structural concrete. As the excavation proceeds downward and the steel piles are subsequently exposed, timber lagging is installed between the piles to further stabilize the walls of the excavation. For a permanent wall, a variety of facing schemes, including cast-in-place concrete, can be applied to the face of the wall to give the wall a desired aesthetic appearance.

Due to the height of the proposed walls, one level of tie-backs will most likely be required to maintain stability of the soldier pile walls. In general, tiebacks are typically used for wall heights greater than about 12 to 15 feet to achieve a more economical design. To reduce the wall height of temporary walls, it may be possible to incorporate a temporary cut slope above the wall.

Design Lateral Pressures – For a cantilevered soldier pile wall or a soldier pile wall with one level of tiebacks, the earth pressures depicted on Figure 4 should be used for design of wall. Above the bottom of excavation, the recommended active earth, surcharge and seismic pressures should be applied over the full width of pile spacing. Below the bottom of excavation, the active and surcharge pressures should be applied over one pile diameter, and the passive resistance should be applied over two times the pile diameter.

Vertical Capacity – We recommend the vertical capacity of the soldier piles be determined using an allowable skin friction value of 500 psf for the portion of the pile below the bottom of the excavation, and an allowable end bearing value of 20 ksf.

Groundwater Seepage and Caving Soil Conditions - The drilling of soldier piles behind the existing basement wall may encounter fill soils that have the potential to cave. As a result, the drilling contractor should be prepared to use temporary casings to stabilize the drill holes, if needed. Significant groundwater is not anticipated within the depth of the soldier piles, but if groundwater seepage is encountered, we recommend that the lean concrete or structural concrete backfill be placed with tremie pipes if more than one foot of water is present at the bottom of the holes at the time of concrete placement.

6.5.2 Tiebacks

If tiebacks will extend beyond the property boundaries, temporary or permanent easements will be needed from the neighboring property owners.

Tieback Location – Because excessive pile top deflections can occur before the first row of tiebacks is installed, it may be necessary to limit the first row of tiebacks to no more than 6 to 8 feet below the pile top unless steel beams of sufficient size will be used to limit the magnitude of the cantilever deflection.

Corrosion Protection – For permanent walls with tiebacks, the tiebacks are an integral component of wall support, and therefore tiebacks with double corrosion protection should be utilized. For temporary shoring walls that utilize tiebacks, the tiebacks do not need corrosion protection.

No-Load Zone - Tieback bond length should be located behind a no-load zone as indicated in Figure 4. The tiebacks should have a minimum bond length of 15 feet beyond the no-load zone in the load zone.

Assumed Capacity – The manner in which the tieback anchors carry load will depend on the type of anchor selected, the method of installation, and the soil conditions surrounding the anchor. Accordingly, we recommend use of a performance specification requiring the tieback contractor to install anchors capable of satisfactorily achieving the design structural loads, with a pullout resistance factor of safety of 2.0. For planning purposes, however, the anchors may be sized for an allowable skin friction value of 2.5 kips per lineal foot of anchor bond length, assuming that small diameter (about 6 inches) pressure-grouted tiebacks will be used. Multiple post-grouting may be needed in order to achieve the design capacity, especially if initial pressure grouting is not utilized. We recommend that the allowable tieback loads be limited to about 120 kips per anchor. Anchors should have a minimum bond length of 15 feet.

The actual capacity of the anchors should be checked with 200 percent verification tests. At least two 200-percent tests should be performed prior to installing production anchors. 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)

- Prior to installing production anchors, perform a minimum of two tests each on each anchor type, installation method and soil type with the tested anchors constructed to the same dimensions as production anchors.
- Test locations to be determined in conjunction and approved by the geotechnical engineer.

- 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 200% 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.
- 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 or less between 6 and 60 minutes and 24 and 240 minutes, and all time increments 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
- 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.

<u>Groundwater and Caving Soil Conditions</u> – Although not anticipated, if layers of wet sand are encountered during drilling of the tiebacks, we recommend the use of temporary casing

during installation to keep the drilled holes open, and to minimize the risk of potential ground loss.

Installation Considerations - The tiebacks for this project should be installed by experienced personnel. The use of compressed air to flush the drill cuttings must be properly controlled as the use of excessive amount of compressed air while drilling tiebacks could lead to reduction of soil strength and ground movements.

Performance Monitoring – The retaining wall should be designed to limit lateral and vertical deflection to about 1 inch. Ground settlements behind the wall are expected to be less than 1 inch.

Because some ground deformations will likely occur due to the excavation (open cut or shored), we recommend that existing conditions on the adjacent private properties and public right-of-way 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 completed to determine potential deformations. 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.

6.5.3 Lagging

Lagging design recommendations for general conditions are presented on Figure 4. If the retaining wall will be a permanent structure, the lifespan of treated timber lagging should be considered in design. Typically, the useable life of timber lagging is on the order of 25 years before repair and/or replacement is necessary. To prolong the life of the lagging, other materials such as concrete (shotcrete) could be considered.

6.5.4 Drainage

Adequate drainage provisions should be incorporated into the design of the permanent soldier pile retaining walls. If concrete lagging or a concrete facing over timber lagging is used, 3-inch diameter weep holes should be installed at the bottom of each soldier pile bay to allow drainage at the base of the wall. The discharged water from the weep holes, or seepage from the lagging, should be collected and discharged at an appropriate outlet, as allowing seepage to flow over the driveway could lead to slippery pavement conditions.

6.6 SITE RETAINING WALLS

We anticipate that retaining walls up to about 6 to 8 feet tall may be needed along the proposed driveway, and walls up to about 14 feet tall may be needed along the downslope side of the proposed auto court. In our opinion, a number of wall types would be feasible from the geotechnical perspective, including soldier pile walls, cast-in-place concrete cantilever walls, gravity walls, and MSE (Mechanically Stabilized Earth) walls.

6.6.1 Geofoam for Wall Backfill

Where large fills are needed behind the walls, such as in the auto court, lightweight fill, such as geofoam, maybe be utilized to backfill the upper portion of the wall to reduce the lateral earth pressure on the wall. If used, the geofoam should extend back from the wall a horizontal distance equal to the intersection with a 1H:1V projection from the bottom of the wall. Provided the geofoam is installed in this configuration, the effective height of the wall used to calculate the lateral earth pressure may be reduced by the thickness of the geofoam backfill.

6.6.2 Soldier Pile Wall

If soldier piles will be utilized along the upslope or downslope side of the driveway and auto court, the recommendations for permanent soldier pile walls presented above in *Section 6.5.1* through *Section 5.6.4* may be used. The advantage of a soldier pile wall is that temporary open cuts are not required, so earthwork (i.e. cuts and fills) is reduced. In addition, because soldier piles will likely be installed for the temporary shoring wall around the house, the installation equipment will already be on-site.

6.6.3 Cast-in-Place Concrete Wall

Cast-in-place cantilevered concrete walls may also be used along the driveway and auto court. If cast-in-place site walls are utilized, the same recommendations presented above in *Section 6.2 and 6.4* above may be used for design. While cast-in-place walls are typically less expensive than soldier pile walls, the cost-saving is often reduced when wall heights become increasingly tall, such as over 10 feet. Temporary cuts are also required to construct the walls, followed by placement and compaction of backfill, which increases earthwork costs and time.

6.6.4 Gravity Wall

The principal advantage of a gravity wall is the ease and speed of construction, and the typically low construction cost. In our opinion gravity walls would be feasible for this project in in areas where the maximum exposed wall height is less than about 6 feet tall. If a gravity wall will be used for this project, we recommend that either a concrete block wall or a rock-filled gabion wall be used.

Precast concrete blocks of various sizes may be used for this project. One commonly used product is Ultra Block (www.ultrablock.com), which has a typical dimension of 2½ feet by 2½ feet by 5 feet. Blocks made of returned concrete, and have dimensions of 2 feet by 2 feet by 6 feet (i.e. ecology blocks) should not be used. Concrete blocks can be made with various finishes or texture to provide the desired aesthetics. All concrete block walls should be battered no steeper than 6V:1H.

Gabion walls should be constructed in general accordance with WSDOT Standard Plan Sheet D-6, and Section 8-24.3(3) Gabion Cribbing of the 2016 *WSDOT Standard Specifications*. Each gabion basket should be placed horizontally and with a minimum of 6 inches of setback from the basket below, hence creating an average wall face inclination of no steeper than 6V:1H. Dimensions of gabion baskets may vary depending on the suppliers.

Minimum Width – In general, as a minimum, gabion basket walls and concrete block walls on this project should have a minimum base width equal to at last one-half the wall height.

Minimum Embedment & Subgrade Improvement - Gravity walls should have a minimum of one foot of embedment along the upslope side of the driveway or auto court, and a minimum embedment of 4 feet below the ground surface along the downslope side of the driveway and auto court.

All walls should be founded on competent native soils or properly compacted fill. If needed, a 6-inch layer of granular structural fill such as crushed rock may be placed as a leveling course before placing the base course of blocks or baskets.

Geotechnical Design Parameters – We recommend that the following geotechnical parameters be used for design of gravity walls:

Active earth pressure:	35 pcf
Allowable Passive Pressure:	300 pcf
Allowable Friction Coefficient:	0.30
Allowable Bearing Capacity:	3000 psf

Once the wall alignments and heights have been determined, PanGEO can provide appropriate block wall or gabion basket design configurations.

Wall Backfill and Drainage Considerations **-** Where backfill is needed behind gravity walls, free draining granular material is recommended. A drainage system should be provided behind the base of all walls to prevent buildup of hydrostatic pressures. As a minimum, the drain should consist of 4-inch diameter perforated PVC pipe, encased in washed drain rock wrapped in filter fabric. The footing drain should discharge to a storm drain or appropriate outlet.

6.6.5 MSE Wall

In areas where large amounts of fill will be needed, MSE walls may represent a feasible and cost-effective wall type. An MSE wall consists of properly placed and compacted granular structure fill between layers of geosynthetic reinforcement (i.e. geogrid). The reinforced fill creates a stable mass that resists the retained soil. As a rule of thumb, the geogrid reinforcements needs to typically extend behind the wall face a horizontal distance equal to about 70% to 80% of the wall height. A variety of wall faces can be constructed along the face of the reinforced soils mass, such as small concrete blocks, pre-cast concrete panels, or wire baskets filled with rock. If an MSE walls is considered for this project, PanGEO will provide geotechnical design recommendations specific to the wall location and configuration. MSE wall systems are preoperatory, and therefore the walls are typically designed by the wall manufacture. However, PanGEO would review the final wall design, and verify global stability of the wall.

6.7 On-Site Infiltration Considerations

Based on our review of the City of Mercer Island Low Impact Development (LID) infiltration feasibility map, the project site is located in an area were infiltrating LID is not permitted.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Temporary Unsupported Excavations

Temporary excavations should be constructed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. It is our opinion temporary excavations at the site parallel to the overall slope angle may be cut at a maximum 1.5H:1V inclination, to remain stable, and reduce the potential of destabilizing the site. Temporary excavations perpendicular to the overall slope angle (i.e. excavations that will not be surcharged by a backslope), may be cut at a maximum of 1H:1V.

Temporary excavations should be evaluated in the field during construction based on actual observed soil conditions. If seepage is encountered, excavation slope inclinations may need to be reduced. During wet weather, the cut slopes may need to be flattened to reduce potential erosion and should be covered with plastic sheeting.

7.2 Temporary Excavation Shoring

See Section 6.5 above for temporary soldier pile shoring wall recommendations.

7.3 Groundwater Control

Perched groundwater seepage may be encountered within the foundation excavations. Groundwater seepage, which is expected to be relatively minor, can likely be controlled by sloping the base of the excavation to a low point and removing the water using a sump and pump.

7.4 Material Reuse

The native soils underlying the site are moisture sensitive, and will become disturbed and soft when exposed to inclement weather conditions. For planning purposes, we do not recommend reusing the native soils as structural fill. If it is planned to use the native soil in non-structural areas, the excavated soil should be stockpiled and protected with plastic sheeting to prevent it from becoming saturated by precipitation or runoff.

7.5 Structural Fill and Compaction

During dry weather, some native soils that are compactable and non-organic may be suitable as non-structural fill, but in our opinion should not be utilized for structural fill.

The native soils contain a high percentage of fines and will degrade if exposed to excessive moisture, and compaction and grading will be difficult or impossible if the moisture content increases above the optimum condition.

Imported fill should consist of well graded granular material having a maximum grain size of three inches and no more than 7 percent fines passing the US No. 200 sieve based on the minus 3/4-inch fraction.

Structural fill should be placed in 6- to 12-inch thick loose lifts and compacted to at least 95 percent maximum dry density, per ASTM D-1557 (Modified Proctor). In non-structural areas, the recommended compaction level may be reduced to 90 percent. Heavy compaction equipment should operate directly over utilities until a minimum of 2 feet of backfill has been placed.

The procedure to achieve proper density of a compacted fill depends on the size and type of compaction 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.

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.

7.6 WET WEATHER CONSTRUCTION

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below. The following procedures are best management practices recommended for use in wet weather construction:

- 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 the 0.75-inch sieve. The fines should be non-plastic.
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water.
- Geotextile silt fences should be installed at strategic locations around the site to control erosion and the movement of soil.
- Excavation slopes and soils stockpiled on site should be covered with plastic sheeting.

7.7 Erosion Considerations

Surface runoff can be controlled during construction by careful grading practices. Typically, this includes the construction of shallow, upgrade perimeter ditches or low earthen berms in conjunction with silt fences to collect runoff and prevent water from entering excavations or to prevent runoff from the construction area leaving the immediate work site. Temporary erosion control may require the use of hay bales on the downhill side of the project to prevent water from leaving the site 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 a positive and permanent discharge system.

Permanent control of surface water should be incorporated in the final grading design. Adequate surface gradients and drainage systems should be incorporated into the design such that surface runoff is collected and directed away from the structure to a suitable outlet. Potential issues associated with erosion may also be reduced by establishing vegetation within disturbed areas immediately following grading operations.

8.0 STATEMENT OF RISK

The site is mapped as a geologic hazard area by the City of Mercer Island, as documented above. Per Mercer Island City Code, development within geologic hazard areas and critical slopes may occur if the geotechnical engineer provides a statement of risk with supporting documentation indicating that one of the following conditions can be met:

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; or

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

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.

It is our opinion that Criterion A and B can be met provided that the development is designed and constructed in accordance with the recommendations in this report. The house design will utilize a soldier pile wall to support the temporary cuts into the slope for the proposed daylight basement. Permanent walls will be utilized to support soils adjacent to the proposed auto court and driveway, and the walls will be designed to accommodate the code-level seismic loading. In addition, the proposed house foundation and walls will be designed with proper embedment such that they bear on the glacially consolidated native soils. Permanent erosion control measures, including landscape and hardscape installations, will effectively mitigate the risk of erosion to disturbed areas of the site in the long term. As such, in our opinion, the development will not negatively affect the stability of the slope, or the surrounding properties.

In addition, in our opinion Criterion B can be met through best management practices during construction, including the proper use of a silt fence, minimize earthwork activities during periods heavy precipitation, minimize exposed areas in the wet season, and other appropriate temporary erosion control measures. Permanent erosion control measures, as described above, will effectively mitigate the risk of erosion in the long term.

9.0 ADDITIONAL SERVICES

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

10.0 CLOSURE

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

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

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

This report 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.

JON C. REHKOPF
STATE OF WASHINGTON
40412
REGISTERED
PROFESSIONAL ENGINEER

3/4/2019

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11.0 REFERENCES

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.

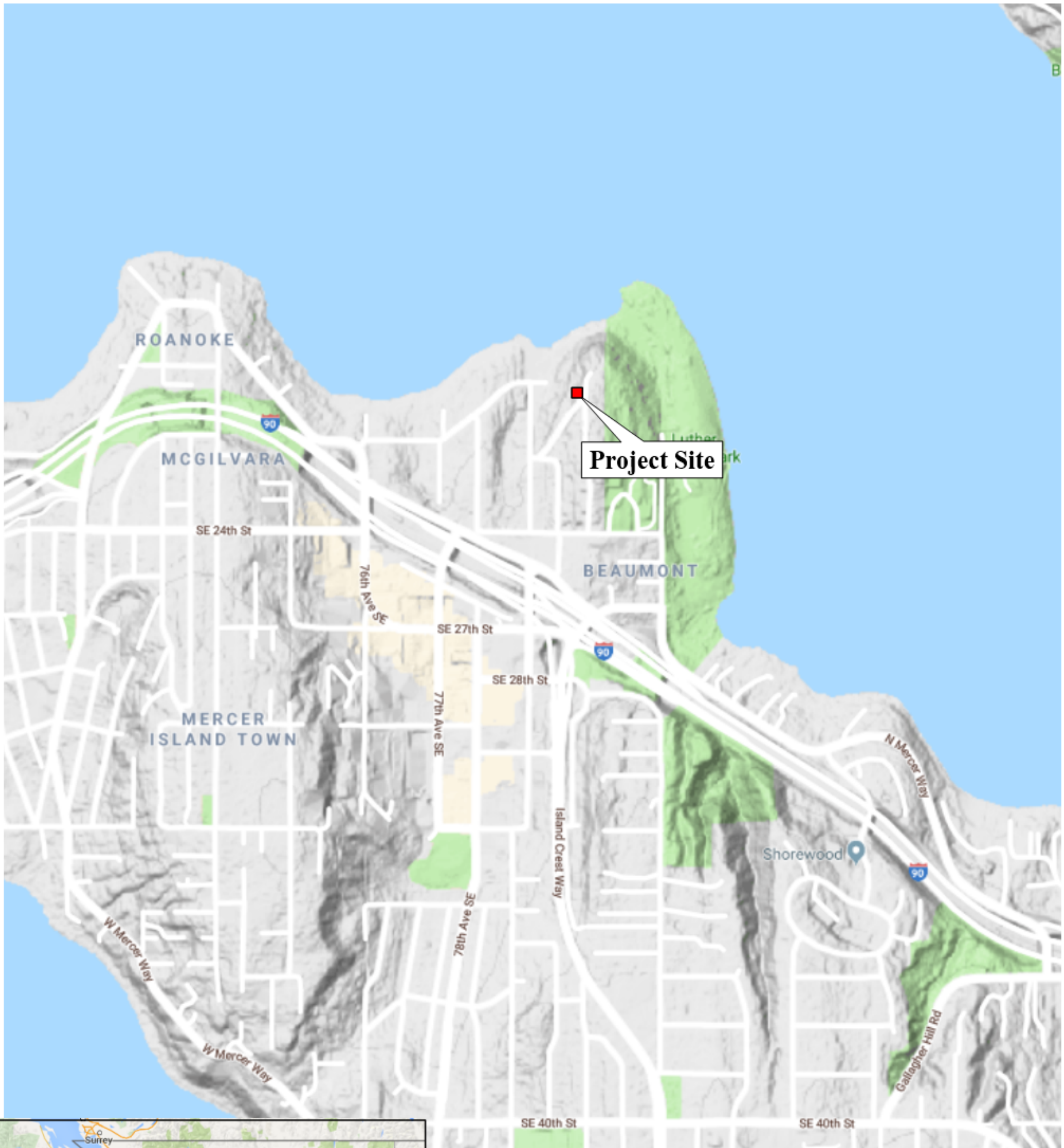
Geotechnical Engineering Report, Proposed Residence, 1935 82nd Avenue SE, Mercer Island, WA. Prepared by PanGEO, Inc. dated, March 15, 2016.

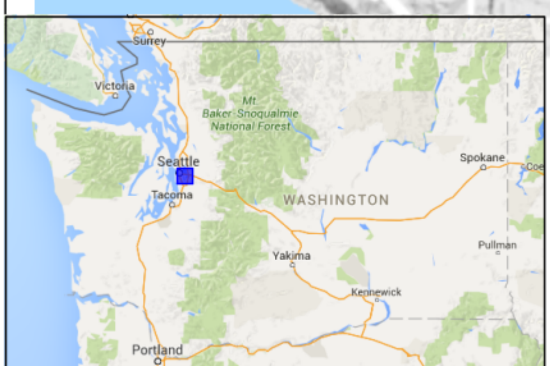
International Code Council, 2015, *International Building Code (IBC), 2015*.

Troost, K.G., and Wisher, A.P, 2006, *Geologic Map of Mercer Island, Washington*, scale 1:24,000.

Washington State Department of Transportation (WSDOT), 2018, *Standard Specifications for Road, Bridges, and Municipal Construction*, Olympia, Washington.

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





Base Map: Google Terrain

NORTH

Not to Scale

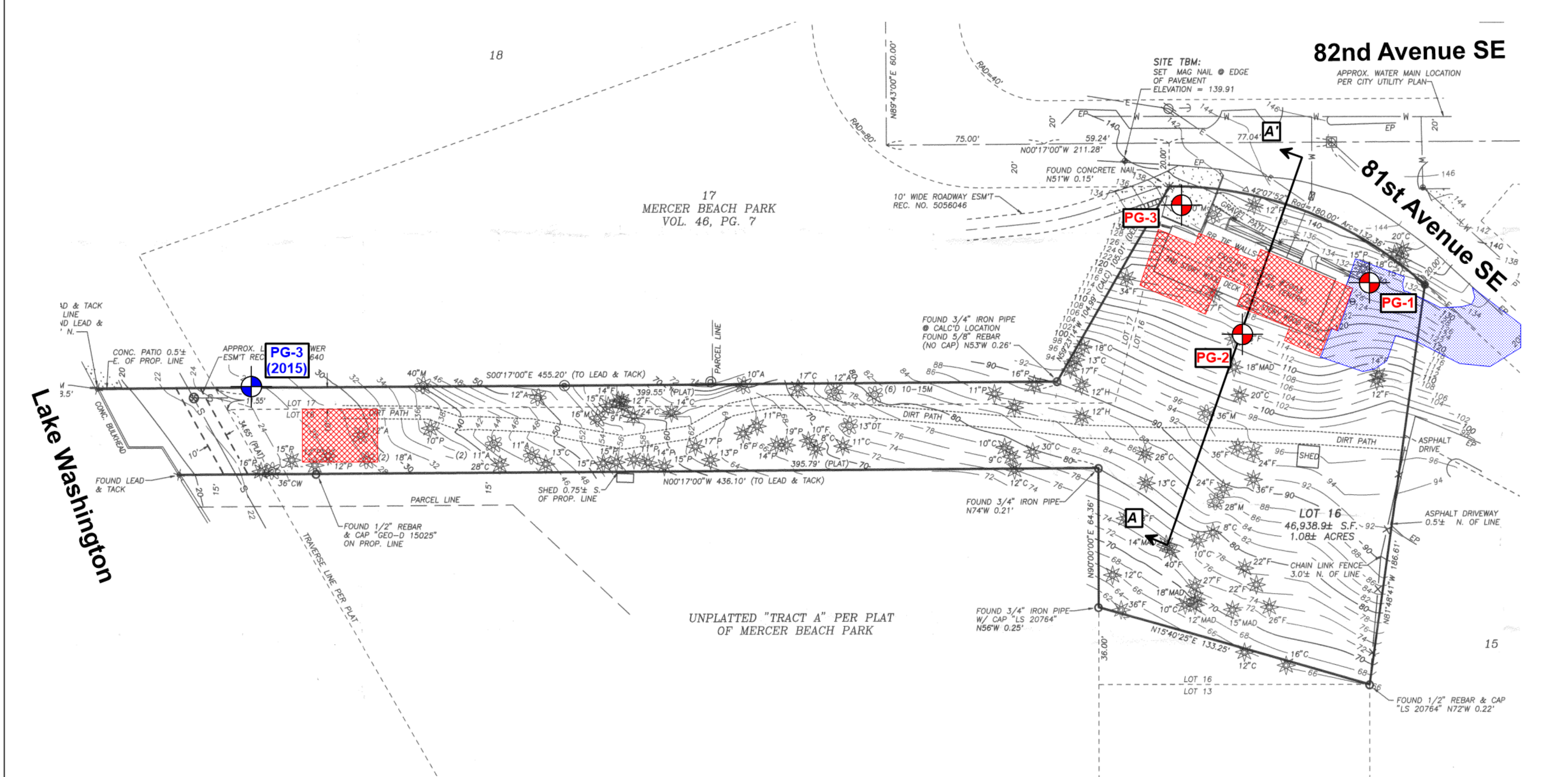
PanGEO INCORPORATED

Proposed Residence
2003 82nd Avenue SE
Mercer Island, Washington

VICINITY MAP

Project No.	19-012	Figure No.	1

19-012 - Vicinity Map.gpj 2/7/19 (09:05) SPS



Base map modified from survey by Cascade Land Surveying, dated December 9, 2016

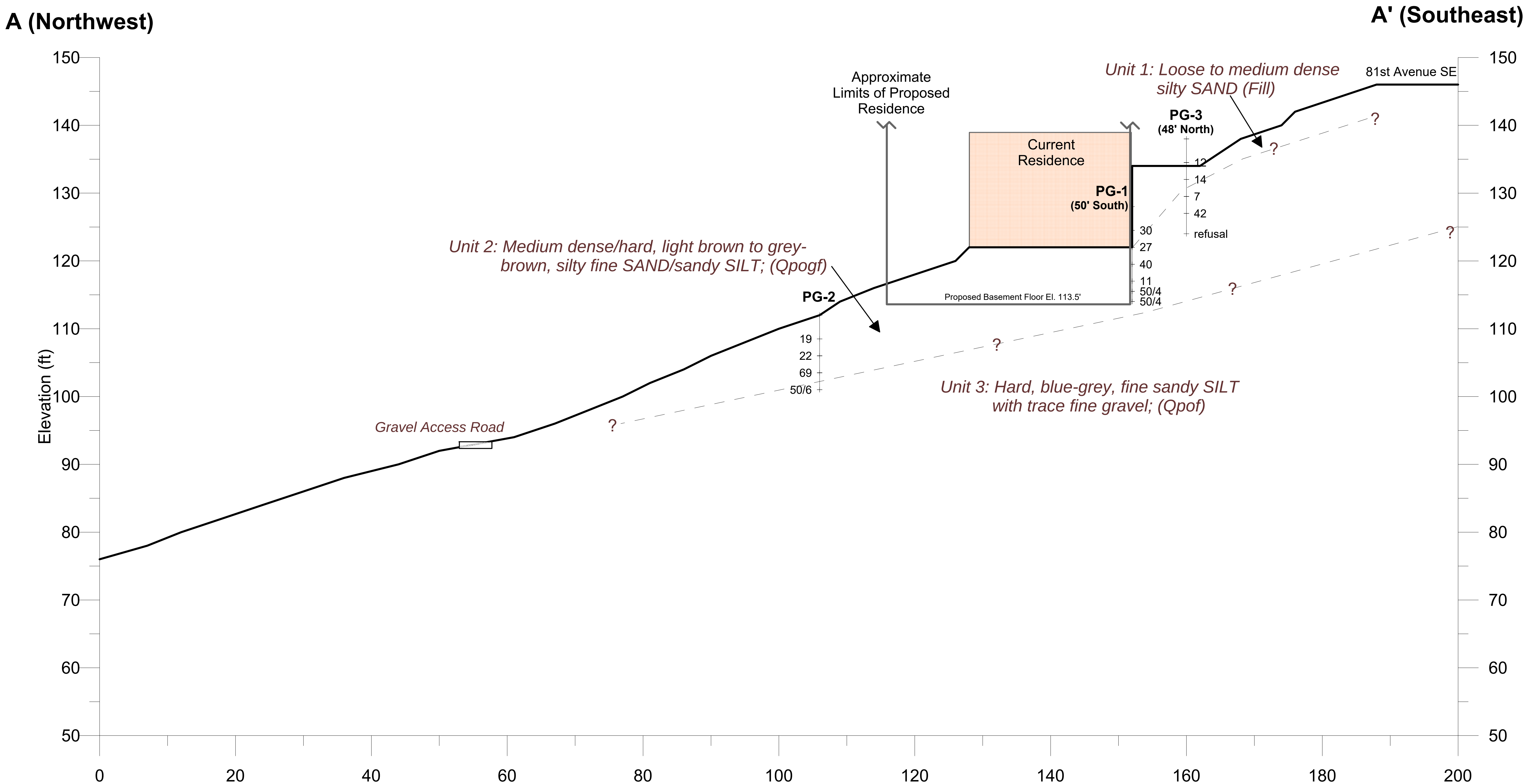
Approx. Scale:
1 inch = 50 feet

Legend:

- Approx. Proposed Structures
- Approximate Boring Locations *PanGEO, Inc., January 2019*
- Previous Boring Locations *PanGEO, Inc., June 2015*
- Approx. Proposed Driveway
- Subsurface Profile

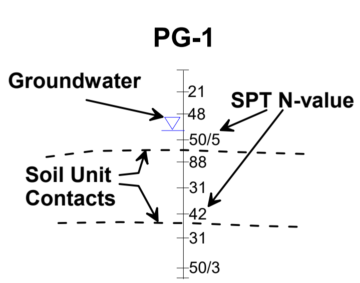
PanGEO INCORPORATED	Proposed Residence 2003 82nd Avenue SE Mercer Island, Washington	SITE AND EXPLORATION PLAN
		Project No. 19-012 / Figure No. 2

19-012 Fig 2 Site and Exploration Plan.grf 2/13/19 (12:38) SPS



Approx. Scale
1" = 15' (H)
1" = 15' (V)
(No vertical exaggeration)

Graphics Legend:



Notes:
1. Ground profile based on site topography by Cascade Land Surveying (12/9/16).
2. See Figure 2 for location of Section A-A'.
3. See report text for a detailed explanation of the subsurface profile across the site.
4. The generalized soil profile is based on widely-spaced explorations. Soil conditions may vary over a small distance, and the actual subsurface conditions may be different from the generalized soil profile depicted in this figure.

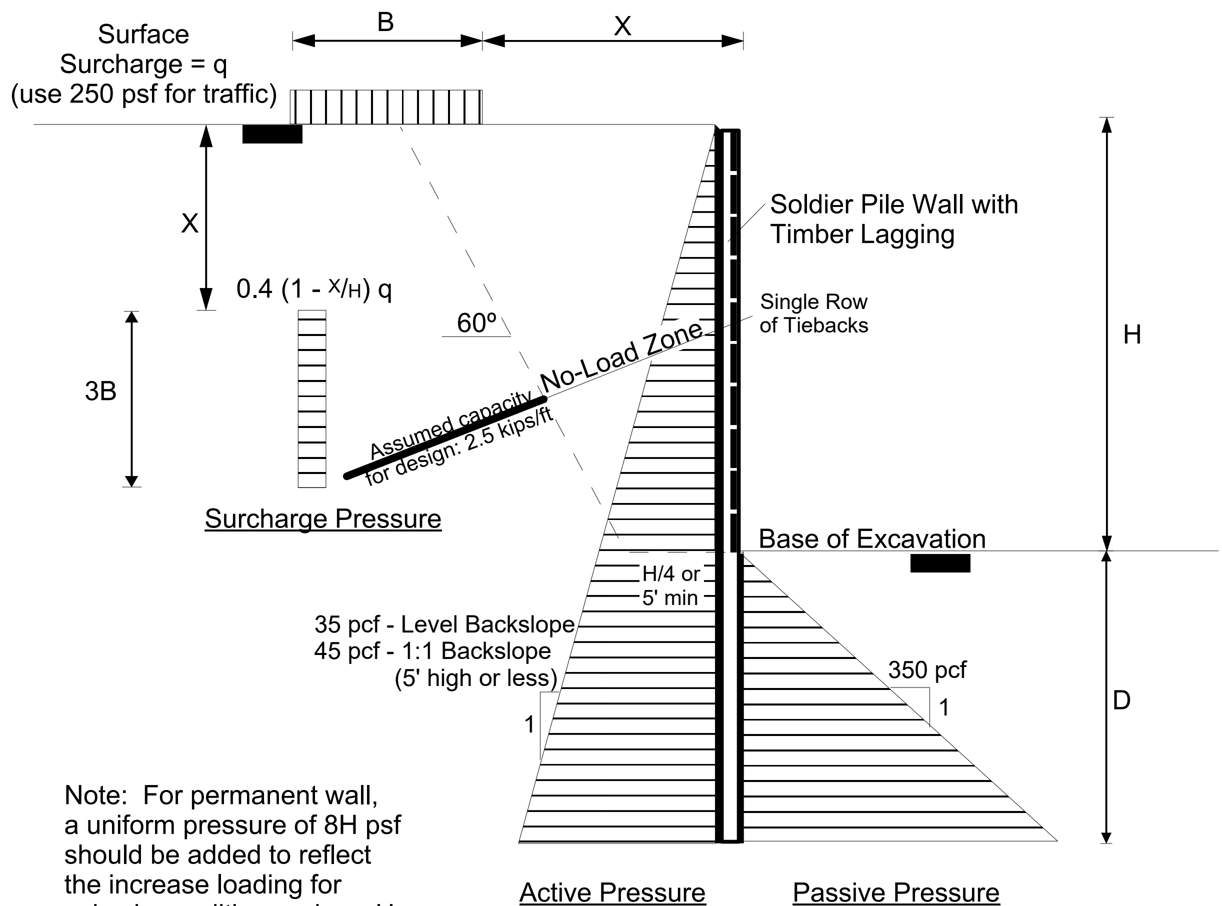
PanGEO INCORPORATED

Proposed Residence
2003 82nd Avenue SE
Mercer Island, Washington

GENERALIZED SUBSURFACE PROFILE
SECTION A-A'

Project No.	Figure No.
19-012	3

19-012 Figure 3 Cross Section A-A'.grf 3/4/19 (17:09) SPS



Note: For permanent wall, a uniform pressure of 8H psf should be added to reflect the increase loading for seismic conditions, where H corresponds to the exposed height of the wall.

Notes:
1. Embedment (D) should be determined by summation of moments at the bottom of the soldier piles. Minimum embedment should be at least 10 feet.
2. A factor of safety of 1.5 has been applied to the recommended passive earth pressure value. No factor of safety has been applied to the recommended active earth pressure values.
3. Active and surcharge pressures should be applied over the full width of the pile spacing above the base of the excavation, and over one pile diameter below the base of the excavation.
4. 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 with soldier piles spaced at 8 feet or less.
6. Refer to report text for additional discussions.

Fig 4 EP diagram.grf 3/4/19 (12:45) JCR

PanGEO INCORPORATED	**Proposed Residence** **2003 82nd Avenue SE** **Mercer Island, Washington**	**SOLDIER PILE WALL DESIGN** **CANTILEVERED / SINGLE TIEBACK WALL** Project No. **19-012** Figure No. **4**

APPENDIX A

TEST BORING LOGS

RELATIVE DENSITY / CONSISTENCY

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

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Notes:

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

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

DESCRIPTIONS OF SOIL STRUCTURES

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

COMPONENT DEFINITIONS

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

TEST SYMBOLS

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

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

SYMBOLS

Sample/In Situ test types and intervals

2-inch OD Split Spoon, SPT (140-lb. hammer, 30" drop)

3.25-inch OD Spilt Spoon (300-lb hammer, 30" drop)

Non-standard penetration test (see boring log for details)

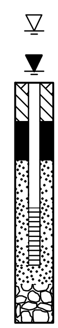
Thin wall (Shelby) tube

Grab

Rock core

Vane Shear

MONITORING WELL



Groundwater Level at time of drilling (ATD)

Static Groundwater Level

Cement / Concrete Seal

Bentonite grout / seal

Silica sand backfill

Slotted tip

Slough

Bottom of Boring

MOISTURE CONTENT

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

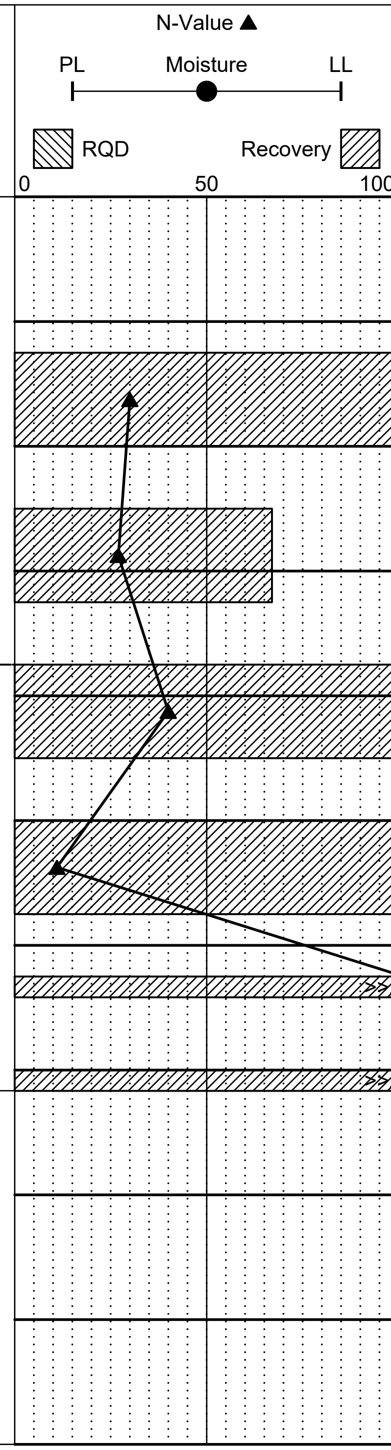
PanGEO INCORPORATED
Phone: 206.262.0370

Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	2003 82nd Avenue SE	Surface Elevation:	128.0ft
Job Number:	19-012	Top of Casing Elev.:	N/A
Location:	2003 82nd Avenue SE, 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						*- Approximately 6 inches of topsoil and mulch.*
	S-1					Medium dense, moist, light brown, silty fine SAND; trace gravels and cobbles, trace iron oxide staining, trace rootlets.
2						**[PRE-OLYMPIA FINE-GRAINED GLACIAL DEPOSITS - Qpogf].**
	S-2		13 15 15			
4						
6	S-3		13 14 13			
						- Water added at 7 feet to aid drilling.
8	S-4		13 18 22			Hard, moist, grey-brown, sandy lean SILT; trace gravel.
10	S-5		6 4 7			*- Pocket of slightly elastic silt at about 10 feet.*
12	S-6		50/4			*- Increase in gravel in sample.*
						- Water added at 14 feet to aid drilling.
						- Trace carbon at bottom of hole.
14	S-7		50/4			
16						Boring terminated at approximately 14.3 feet below ground surface. No groundwater was observed at time of drilling.
18						
20						



Completion Depth:	14.3ft	Remarks: Boring drilled using an acker portable drill rig. Standard penetration test (SPT) sampler driven with a 140 lb safety hammer. Hammer operated with a rope and cathead mechanism. Surface elevations (NAVD88) estimated from Survey by Cascade Land Surveying, dated December 2016.
Date Borehole Started:	1/24/19	
Date Borehole Completed:	1/24/19	
Logged By:	S. Scott	
Drilling Company:	Boretec 1	

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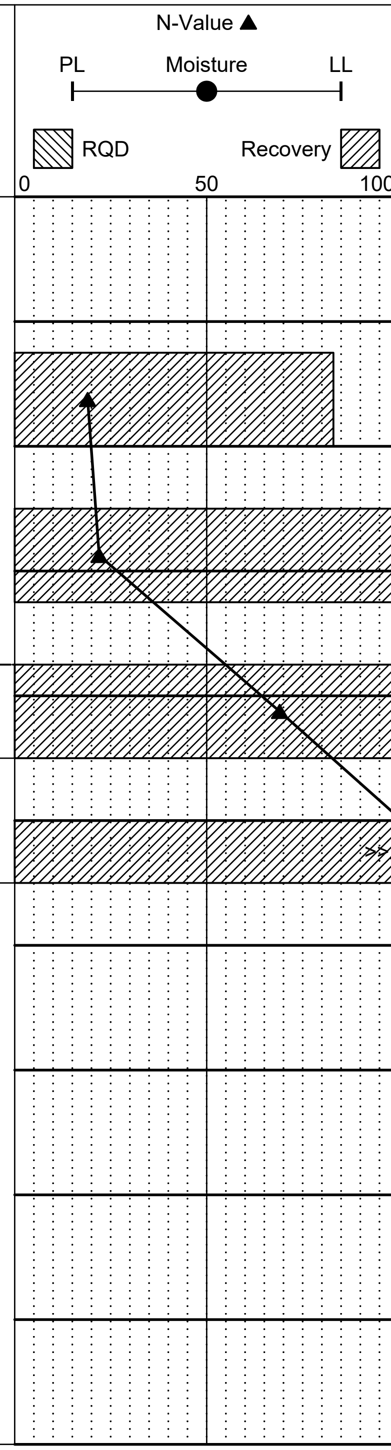
LOG OF TEST BORING PG-1

Figure A-2

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

Project:	2003 82nd Avenue SE	Surface Elevation:	112.0ft
Job Number:	19-012	Top of Casing Elev.:	N/A
Location:	2003 82nd Avenue SE, 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						*- Approximately 6 inches of topsoil and mulch.*
	S-1					Very stiff, moist, grey to light brown, sandy slightly elastic SILT; trace gravel, trace mottling, roots.
2						**[PRE-OLYMPIA FINE-GRAINED GLACIAL DEPOSITS - Qpogf].**
	S-2		5 8 11			
4						
6	S-3		11 11 11			
8	S-4		28 38 31			Very dense to hard, moist, grey-brown, sandy SILT to silty SAND; trace gravel, trace mottling. *- Water added at 9 feet to aid drilling.*
						Hard, moist, blue-grey, fine sandy SILT; trace gravel; potential hydrocarbon odor observed.
10	S-5		35 50/6			**[PRE-OLYMPIA FINE-GRAINED DEPOSITS - Qpof].**
12						Boring terminated at approximately 11 feet below ground surface. No groundwater was observed at time of drilling.
14						
16						
18						
20						



Completion Depth:	11.0ft
Date Borehole Started:	1/24/19
Date Borehole Completed:	1/24/19
Logged By:	S. Scott
Drilling Company:	Boretec 1

Remarks: Boring drilled using an acker portable drill rig. Standard penetration test (SPT) sampler driven with a 140 lb safety hammer. Hammer operated with a rope and cathead mechanism. Surface elevations (NAVD88) estimated from Survey by Cascade Land Surveying, dated December 2016.

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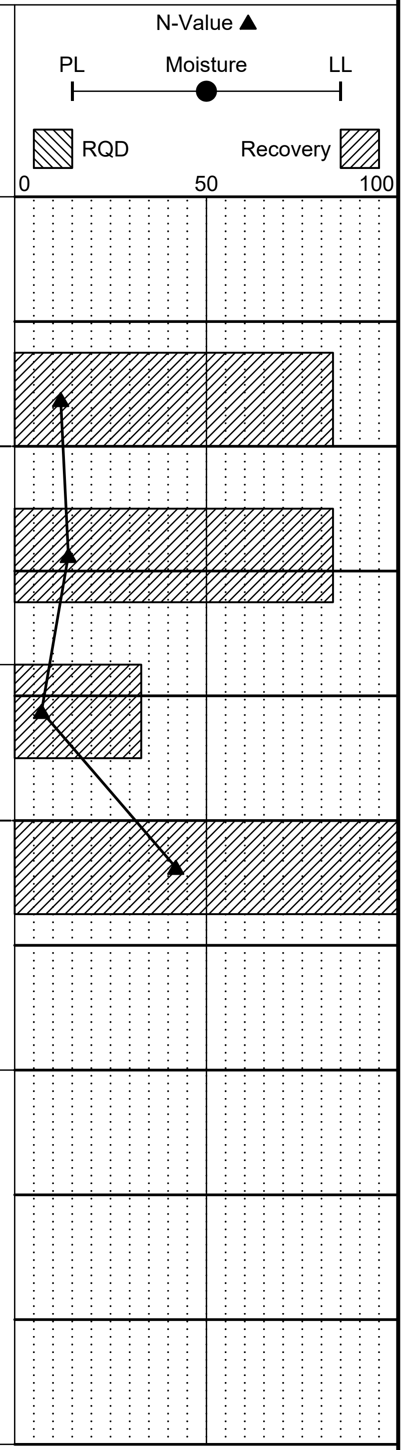
LOG OF TEST BORING PG-2

Figure A-3

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

Project:	2003 82nd Avenue SE	Surface Elevation:	138.0ft
Job Number:	19-012	Top of Casing Elev.:	N/A
Location:	2003 82nd Avenue SE, 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						*- Approximately 4 inches of concrete over gravel.*
						Loose to medium dense, moist, brown, silty SAND; trace gravel. **[FILL].**
2						
	S-1		3 3 9			
4						Medium dense, moist, light brown to grey-brown, silty fine SAND; trace gravel, trace rootlets, trace iron-oxide staining.
	S-2		6 7 7			
6						
8	S-3		4 3 4			Medium stiff, moist, light brown, sandy lean SILT; trace gravel, trace mottling, reworked. **[PRE-OLYMPIA FINE-GRAINED GLACIAL DEPOSITS - Qpogf].**
10	S-4		5 9 33			Hard, moist, light brown to grey brown, silty CLAY; trace carbon.
12						
14						*- Refusal at 14 feet due to boulder (?).*
						Boring terminated at approximately 14 feet below ground surface. No groundwater was observed at time of drilling.
16						
18						
20						



Completion Depth:	14.0ft
Date Borehole Started:	1/24/19
Date Borehole Completed:	1/24/19
Logged By:	S. Scott
Drilling Company:	Boretec 1

Remarks: Boring drilled using an RCT60 small track mounted drill rig. Standard penetration test (SPT) sampler driven with a 140 lb safety hammer. Hammer operated with a rope and cathead mechanism. Surface elevations (NAVD88) estimated from Survey by Cascade Land Surveying, dated December 2016.

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LOG OF TEST BORING PG-3

Figure A-3

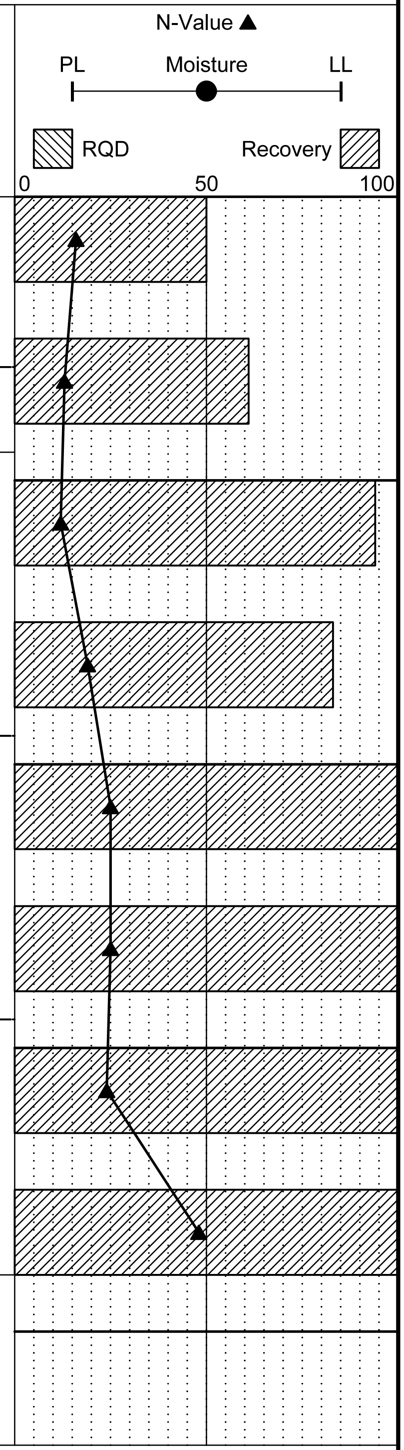
The stratification lines represent approximate boundaries. The transition may be gradual.

APPENDIX B

PREVIOUS TEST BORING LOG

Project:	Proposed Residence	Surface Elevation:	24.0ft
Job Number:	07-105.300	Top of Casing Elev.:	
Location:	1935 82nd Avenue SE, Mercer Island, WA	Drilling Method:	Hollow Stem Auger
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0	S-1		4, 7, 9			Medium dense, damp to moist, brown, silty fine SAND with trace gravel, some roots (Fill).
	S-2		4, 6, 7			Stiff, moist, dark brown to gray, sandy SILT to SILT, non-plastic, trace roots and brown stains (Fill/Disturbed Soil).
5	S-3		4, 5, 7			Stiff to very stiff, moist, brown-gray, clayey SILT, low to medium plasticity, some fracture, homogeneous (pre-Olympia Glaciolacustrine Deposit).
	S-4		4, 8, 11			- very moist.
10	S-5		8, 11, 14			Very stiff, moist, gray, clayey SILT, low plasticity, some fracture, massive, homogeneous.
	S-6		4, 9, 16			- some laminated.
15	S-7		6, 11, 13			Very stiff to hard, moist, gray, clayey SILT to SILT with some fine sand, low to non plasticity, massive to some laminated, homogeneous.
	S-8		13, 15, 33			
20						Bottom of boring at about 19 feet below ground surface. No groundwater was encountered during drilling.



Completion Depth:	19.0ft
Date Borehole Started:	7/31/15
Date Borehole Completed:	7/31/15
Logged By:	J. Chen
Drilling Company:	CN Drilling

Remarks: Boring was drilled with a Acker portable drill rig. Standard Penetration Test (SPT) sampler driven with a 140 lb hammer using a rope and cathead dropping 30 inches per stroke.

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LOG OF TEST BORING PG-3

Figure A-4

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