PanGEO
INCORPORATED
Geotechnical & Earthquake
Engineering Consultants

June 18, 2025
File No. 25-036.100

Citizen Design
Attn: Mr. Isaac Greenetz
3800 Woodland Park Avenue, #300
Seattle, WA 98103

Subject: **Geotechnical Engineering Report**
Proposed Single-Family Residence
6427 East Mercer Way, Mercer Island, Washington

Dear Isaac,

Please find attached our geotechnical engineering 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 project.

PanGEO previously prepared a geotechnical report for three lots, including the subject lot, dated April 16, 2019. We subsequently prepared supplemental addendums to address the previously proposed developments on each specific lot. The attached report is intended to supersede our original report for the three lots, and all previous addendums, and should be used for the current design of the project.

Soil Conditions - In summary, the majority of the site is generally underlain by competent soils consisting of medium dense to dense silty sand. The exception is the northeast portion of the site, where loose, unsuitable soils were encountered, which we interpreted to be undocumented fill. Perched groundwater was encountered in one test boring about thirteen feet below the ground surface.

Foundation Recommendations - Based on the currently proposed design, the majority of the proposed house foundations are expected to bear on competent soils. However, unsuitable soils are expected to be encountered along the north foundation wall, and along the approximately northern half of the east foundation wall. At these locations, the unsuitable soils would need to be over-excavated and replaced with properly compacted structural fill, or lean-mix concrete. We estimate that over-excavation depths may be up to five to six feet below the proposed footings. As an alternative to over-excavation and replacement, to avoid the large amounts of earthwork, the footings underlain by unsuitable soil may be supported by small diameter driven pipe piles, commonly referred to as pin 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 mapped critical areas at the site.

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

Sincerely,

Stephen H. Evans

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Encl.: Geotechnical Engineering Report

TABLE OF CONTENTS

LIST OF ATTACHMENTS

GEOTECHNICAL ENGINEERING REPORT
PROPOSED SINGLE-FAMILY RESIDENCE
6427 EAST MERCER WAY
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 6427 East Mercer Way (Parcel 3024059151) on 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 January 24, 2025, which was subsequently authorized by you on January 29, 2025. Our scope of services included reviewing readily available geologic and geotechnical data, including our previous studies on this site, compiling our previous engineering analyses and recommendations, performing additional engineering analyses as needed, and preparing this geotechnical engineering report for this specific property. All previous reports and memos provided for this property are superseded by this report.

2.0 SITE AND PROJECT DESCRIPTION

The project site consists of an irregularly shaped, undeveloped parcel located at 6427 East Mercer Way, on the east side of Mercer Island, Washington (see Figure 1, Vicinity Map). The 15,812 sq. ft. parcel is located above a 90-degree bend in E. Mercer Way (see Figures 2). A low ridgeline with moderately steep slopes surrounds the property to the south, west, and east, forming a semi-open, northeast-facing bowl with about 30 feet of total relief (see Plate 1 and Figure 2). The surface within the “bowl” is relatively level, and the near 90-degree angle between the south and west slopes suggest the site has been excavated and backfilled. The ridge area is covered with scattered conifer and deciduous trees, while the bowl area was grassy at the time of our explorations.

The property is mapped by the City of Mercer Island within several geological hazard areas, including an erosion hazard area, a potential landslide hazard area, and seismic hazard area. As such, the development will need to consider these hazards, which are addressed in *Section 5.0* of this report.

Current plans call for developing the house footprint in the bowl portion of the property (see Figure 2). We understand that the finished floor of the house will be around elevation 147 feet, while the finished floor of the garage will be around elevation 157 feet. As such, the lowest floor of the house will consist of a daylight basement open to the northeast. We anticipate excavations for the house foundation will be on the order of 8 feet, or less.

Plate 1 – General conditions of the property Lot bowl area, looking south. (3/1/2019)

Plate 2 –Schematic North Elevation, dated 6/6/25, by Citizen 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.

3.0 SUBSURFACE EXPLORATIONS

PanGEO completed three test borings (PG-1, PG-2 and PG-3) at the subject site on March 7, 2019. The borings were advanced to between 11½ and 26½ feet below the existing ground surface using an EC-95 track mounted drill or a hand-operated portable Acker drill rig, both owned and operated by Boretec, Inc. The approximate boring locations are shown on the attached Figure 2.

Soil samples were obtained from the borings at 2½-foot and 5-foot depth intervals in general accordance with Standard Penetration Test (SPT) sampling methods (ASTM test method D-1586) in which the samples were obtained using a 2-inch outside diameter split-spoon sampler. The sampler was driven 18-inches into the soil using a 140-pound weight freely 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 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 to A-4 in Appendix A. The stratigraphic contacts indicated on the boring logs 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.

4.0 SUBSURFACE CONDITIONS

4.1 SITE GEOLOGY

According to *The Geologic Map of Mercer Island (Troost and Wisher,2006)*, the subject parcel is underlain by pre-Olympia non-glacial deposits (Qpon), with the basal contact of the Lawton Clay

(Qvlc) located just upslope of the site. Possible mass-wastage deposits are mapped over the surface of the site.

- **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 consist 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.

- **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 test borings advanced at the project site are generally encountered soils consistent with the mapped geologic stratigraphy, though we did not encounter significant mass-wasting deposits. Rather, the disturbed surficial materials we found at the site were either interpreted to be fill, or deposits attributable to alluvial processes. Also, we consider that the fine-grained silt deposits encountered in PG-3 may be pre-Olympia fine-grained deposits. However, for the purposes of this report, the fine-grained silt deposits are referred to as the mapped Lawton Clay.

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.

Fill (Hf) – Fill was encountered in the central bowl area of the site in PG-2. The fill encountered generally consisted of loose, brown to red brown, silty fine sand with occasional organics. As the bowl area is near the low section of a shallow drainage that drains areas to the west, the bowl area may contain alluvial material in addition to fill.

Lawton Clay (Qvlc) – Test boring PG-3 encountered medium dense, brown-gray, non-plastic SILT with some fine sand, which is laminated. The unit is roughly 8 to 8½ feet thick, and is the uppermost soil layer in PG-3. We anticipate this unit will exhibit high strength characteristics in its undisturbed state.

Pre-Olympia Non-Glacial Deposits (Qpon) – The soil unit encountered at depth in all three borings was a medium dense to very dense, brown to brown-gray, fine to medium SAND unit, which we interpret as pre-Olympia non-glacial strata. The unit was laminated to bedded, with finer and coarser beds, homogeneous, slightly silty to silty, with high strength.

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 5* 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 –Hf | Loose, silty sand | PG-2, S-1 @ 2½ – 4 feet.

Plate 4 –Qvlc | Medium dense SILT | PG-3, S-1 @ 5-6½ feet.

Plate 5 – Nonglacial Deposits Qpon | Medium Dense to Very dense, SAND, trace silt | PG-1, S-2 @ 5 – 6½ feet.

4.3 GROUNDWATER CONDITIONS

Groundwater was encountered in PG-2 during drilling at a depth of roughly 13½ feet below the existing ground surface. We interpreted this as a perched lens above a thin silt bed within the overall sand deposit, as reflected in the log. 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).

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 indicates that steep slopes are not present at the site.

The City of Mercer Island GIS mapping identifies a few landslide indicators within 500 feet of the subject site. One is on East Mercer Way, north of the subject site, and one is below East Mercer Way, south of the site, above the deeply incised stream channel at that location. The indicator on East Mercer Way is likely in response to pavement settlement issues. The map also shows a landslide scarp along the north side of a drainage swale that passes along the north side of the subject property, and is more fully described in the next section. This scarp is above the project site, but is not associated with any mapped known slide.

Site Reconnaissance and Observations: We conducted a reconnaissance visit to review the condition of the sloping areas of the site, and areas adjacent to the site, and identify indications of potential historical slope instability.

The site morphology consists of a narrow descending ridge running along the south and east sides of a level bowl area, as shown in Plate 1 and Figure 2. The west side of the site rises onto a broad-backed ridge that continues ascending to the west. On the north, the bowl opens onto a west to east trending drainage swale, the axis of which is located roughly 20 to 30 feet north of the property line, per Mercer Island GIS. The drainage continues under East Mercer Way, down to discharge into Lake Washington.

The ridge line is vegetated with scattered conifer and deciduous trees, and the bowl area was generally grassy at the time of our reconnaissance, as shown in Plate 1. Overall, the site gives the appearance of having been excavated, possibly to provide sandy aggregate for construction purposes, and then backfilled. It is also possible that the area was part of the general stream drainage system, and the drainage was backfilled to provide building sites and road access.

During our site visits, we did not observe evidence of recent slope instability such as slide scarps or tension cracks within the subject property. In addition, no recent or historical slides have been mapped on or directly adjacent to the subject property. Review of the recent Lidar image of the area shows the steep road cut of East Mercer Way on the south side of the south ridge line, described above, and the bowl area, but shows no other features that may suggest slope instability in the subject area.

Conclusions: Based on our reconnaissance, review of existing data, and our understanding of subsurface conditions at the site, in our opinion neither shallow nor large, deep-seated-type slope failures are likely to adversely affect the proposed development. In addition, in our opinion the proposed development, located in the mostly gently sloping portion of the site, will not adversely affect the stability of the site, or the stability of adjacent sites.

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 dense silty sand, and a static groundwater level was not encountered in the test borings. 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 low at the site due to the underlying dense sand. 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 affected during the code-level seismic event.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 SEISMIC DESIGN CONSIDERATIONS

6.1.1 Site Class

We understand that the project will be designed in accordance with the 2021 editions of the International Building Code (IBC), and ASCE 7-16, 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.

6.1.2 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 our subsurface explorations, the site is underlain by primarily dense silty sand, and a static groundwater level was not encountered in the test borings. 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.

6.2 FOUNDATION RECOMMENDATIONS

Based on the results of our test borings and our understanding of the project design, we anticipate that the majority of the building footings will bear on competent native soils, and the remainer of the foundation will bear on loose, undocumented fill. Foundations bearing on the loose, undocumented fill are expected to experience undesirable settlements, and are therefore spread footings are not recommended when the loose soils are present at the footing elevation.

Based on our current understanding of the proposed design, the house foundations under the west, south and the southern half of the east wall will likely bear on competent bearing soils. The garage footings will also likely bear on or near competent soils. However, we anticipate the north perimeter footing, the entry way footings to the north of the house, and the northern half of the east wall footing will bear on unsuitable soils. When unsuitable soils are present, the soils will need to be over-excavated down to the competent soils, or, driven pin piles may be used to support the footings in these areas to reduce the amount of earthwork associated with over-excavations and backfill.

The following sections present our design recommendations for conventional footings and driven pin piles.

6.2.1 Conventional Footings

Footings should bear on the undisturbed native medium dense to dense soils, or on properly compacted structural fill placed on the undisturbed medium dense to dense native soils. For planning purposes, we recommend assuming that most of the perimeter footings, as well as the interior footings, will bear on suitable bearing soil. Footing locations along the north wall, and the northern half of the east wall, will most likely require over-excavation of the unsuitable soil and replacement with lean-mix concrete or structural fill, as described below. Alternatively, where over-excavation becomes too deep, pin piles, as described below, may be used in lieu of over-excavation.

6.2.1.1 Over-excavation & Replacement

The over-excavation should be backfilled with lean-mix concrete (1½ sack of cement per cubic yard, minimum) or properly compacted structural fill, such as 1¼-inch minus crushed rock, or approved equivalent. If lean-mix is used, the over-excavation should extend horizontally at least one foot beyond the edges of the footings. If structural fill is utilized, the fill should extend horizontally out from the edges of the footing a distance equal to one-half of the over-excavation depth. As such, to limit the amount of earthwork, if over-excavations more than about 3 feet are required, we recommend lean-mix be used as backfill.

6.2.1.2 Allowable Bearing Pressure

A maximum allowable soil bearing pressure of 3,000 pounds per square foot may be used to size footings bearing on the undisturbed medium dense to dense native soils, or structural fill or lean mix placed over the native soils. For allowable stress design, the recommended allowable bearing pressure may be increased by one-third for transient loading conditions such as those due to wind and seismic forces. For frost protection considerations, footings should be placed at least 18 inches below adjacent finished grade.

6.2.1.3 Lateral Resistance

Lateral loads acting on footings may be resisted by passive earth pressure developed against the embedded portion of the footings and by frictional resistance developed at the base of the footings.

- An allowable frictional coefficient of 0.4 may be used to evaluate sliding resistance.
- An allowable passive soil resistance may be calculated using an equivalent fluid pressure of 300 pcf, assuming the footings are backfilled with structural fill and level ground surface. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches of soil should be neglected.

The above values include a geotechnical factor of safety of 1.5.

6.2.1.4 Footing Drains

Footing drains should be installed around the perimeter of the building, at or just below the bottom 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. Cleanouts should be installed at strategic locations to allow for periodic maintenance of the footing drain and downspout tightline systems.

6.2.1.5 Footing Subgrade Preparation

Footing subgrades should be carefully prepared and should not contain loose, soft, or disturbed soils. The adequacy of footing subgrades should be verified by a representative of PanGEO prior to placing forms or rebar. The footing subgrades should be in a dense and unyielding condition prior to pouring concrete.

Please note that the site soils are moderately to highly moisture sensitive and can become disturbed and softened when exposed to moisture and construction traffic. Sandy soils with low fines content are also vulnerable to disturbance by general foot traffic. Protection of the foundation bearing soils should be the responsibility of the contractor.

6.2.1.6 Foundation Performance

Total and differential settlements are anticipated to be within tolerable limits for footings designed and constructed as discussed above. Footing settlement under static loading conditions is estimated to be about ¾-inch, and differential settlement across the structure should be about ½ inch or less. If driven pin piles are utilized, we estimate settlement of the pile-supported foundation to be on the order of ¼-inch. As such, differential settlements between spread footings and the pin pile supported foundation should be on the order of ½ inch or less. Most settlement will be realized during construction as the dead loads are applied.

6.2.2 Driven Pin Piles

Our test boring PG-2 encountered a layer of loose, disturbed material that extended a maximum of about four feet below the planned finished floor elevation of the basement along the north side of the house. However, based on the site topography, we anticipate that up to about 5 or 6 feet of unsuitable soils may be present near the northeast corner of the house, as projected in the attached Figure 3. As such, in lieu of over-excavation and replacement, we anticipate that utilizing a driven pin pile foundation may be more cost effective along the north wall, and northern half of the east wall. However, if competent soils are encountered at sufficiently shallow depth for over-excavation and replacement to be feasible 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.

6.2.2.1 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.

6.2.2.2 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.

6.2.2.3 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:

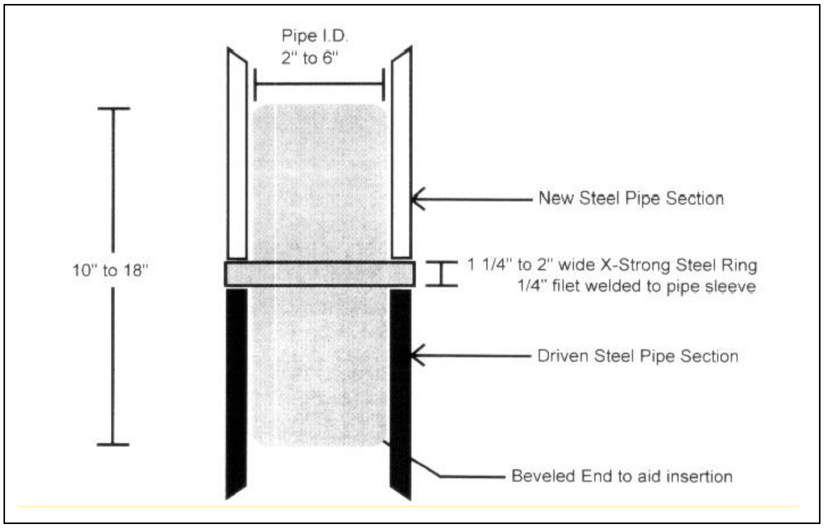
Hammer Size	Blow per Minute	Refusal Criteria (4-inch pile)
850 lbs	900	16 seconds per inch

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.

6.2.2.4 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.

6.2.2.5 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. 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.

6.2.2.6 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 20 feet may be needed below the pile caps. A minimum pile length of 8 feet (below final basement level) should also be specified in the plans.

6.2.2.7 Pin Pile Performance

It is our experience that the driven pipe pile foundations should provide adequate support with total settlements on the order of ¼- to ½-inch.

6.2.2.8 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 as needed.

6.3 FLOOR SLABS

6.3.1 Concrete Slab-on-grade

A slab-on-grade may be used for the basement floor of the proposed building. However, loose/soft soils may be present below the slab elevation in some areas of the house footprint. In these areas 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 in these areas, we recommend that the floor slab subgrade (below the base of the capillary break material, as outlined below) be over-excavated by at least 12 inches, and existing fill or 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. If more than two feet of unsuitable soils are present, to improve subgrade, we recommend that a layer of geogrid reinforcement be placed over the native subgrade prior to placement and compaction of structural fill. The geogrid should be overlapped a minimum of 12-inches. We also recommend that construction joints be incorporated into the floor slab to control cracking.

6.3.2 Structural Slab

If a higher level of slab performance is desired than described above for slab-on-grades, a structural slab can be designed to span between 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 structural slab when the floor surface will receive settlement sensitive floor coverings, such as tile, or if the exposed floor will be polished concrete.

6.3.3 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.

6.4 RETAINING AND BELOW-GRADE WALLS

Free standing retaining walls and below-grade foundation walls should be properly designed to resist the lateral earth pressures exerted by the soils behind the walls. The current design includes basement walls and low concrete cantilever walls along the north and west sides of the property. The upslope walls to the west and south will not have final exposed faces of about 3 to 5 feet high. Basement walls may be up to 9 feet high on the west side of the house.

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.

6.4.1 Concrete Wall Foundation Support

The footing recommendations outlined in Section 6.2 of this report are also applicable for the walls. For walls with fore-slopes, with a maximum height of 5 feet, we recommend that the footing be embedded a minimum of 2 feet below the finished grade in front of the wall.

6.4.2 Lateral Earth Pressures

Cantilever walls 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 9H 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.

6.4.3 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 90 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 4 be used to calculate the lateral pressure on the face of the wall face resulting from surcharge loading.

6.4.4 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, and by friction acting on the base of the footings. For pile supported walls, friction should not be accounted for at the base of the grade beam. 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 footings and level ground surface. If there is a slope descending below the wall, the passive pressure will be significantly reduced, and PanGEO can provide an acceptable value based on the specific geometry and soil conditions at the wall location. An allowable frictional coefficient of

0.4 may be used to evaluate sliding resistance at the base of a footing. This value includes a geotechnical factor of safety of 1.5.

6.4.5 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 for basement walls. 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.

6.4.6 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 7.3 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.

6.4.7 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 foundation 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.

6.5 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.

6.6 PERMANENT DRAINAGE

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 on the north side of the house. Based on our understanding of the plans, excavation of the vault area will be up to 10 feet deep, will be in the loose fill and alluvium, and will likely require shoring (see Section 7.4.2).

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.

6.7 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.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 SITE PREPARATION

Site preparation for the proposed project includes clearing, grubbing, 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.

7.2 MATERIAL REUSE

The soils at the site are moisture sensitive and will become disturbed / soft when exposed to inclement weather conditions. 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. Material for use as structural fill is described in the following section.

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.

7.3 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, 2025, 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 at least 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. We recommend that structural fill supporting foundations be compacted with jumping jack compactors at a minimum. 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.

7.4 TEMPORARY EXCAVATIONS

7.4.1 Temporary Open Cuts

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 1H:1V (Horizontal:Vertical). If temporary excavations are not in the fill, but in the dense native soil, 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, or require shoring. During wet weather, runoff water should be prevented from entering excavations and the exposed slopes should be covered with plastic sheets.

7.4.2 Temporary Shoring

As described above, the excavation for the stormwater detention system is located at the northeast corner of the subject site, close to the north property line. The planned excavation for the control structure is up to 10 feet deep, with the vault excavation possibly being nearly as deep in places. As such, shoring will most likely be required for the installation of these structures. We anticipate that trench boxes or similar conventional temporary shoring systems should provide adequate protection for the installation of these facilities.

7.4.3 Groundwater Impacts

As described above, based on the results of PG-2, we anticipate perched groundwater may be present about 13½ feet below the ground surface. However, the field exploration was conducted in March, when groundwater levels are not expected to be at their maximum. Excavation contractors should be prepared to deal with groundwater impacts, particularly during wet times of the year. We anticipate such impacts should be relatively minimal, and groundwater seepage from building excavations can likely be controlled with sumps and pumps.

7.4.4 Surcharge Avoidance

We also recommend that heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a distance equal to 1/3 the slope height from the top of any excavation.

7.5 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.

7.6 WET EARTHWORK RECOMMENDATIONS

General recommendations relating 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 or 3 to 4 inches of clean crushed rock 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.

8.0 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;
- 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.

9.0 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, it is our opinion that criterion (a) above, is applicable. PanGEO will be available to review the final design plans to confirm our statement of risk prior to construction.

10.0 LIMITATIONS

We have prepared this report for use by Citizen Design 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.

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STEPHEN H. EVANS
Stephen H. Evans

SIEW LAN TAN
30342
June 18, 2025
/For

Stephen H. Evans, L.E.G.
Senior Engineering Geologist
sevans@pangeoinc.com

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Principal Geotechnical Engineer
jrehkopf@pangeoinc.com

11.0 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-lb./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, 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.

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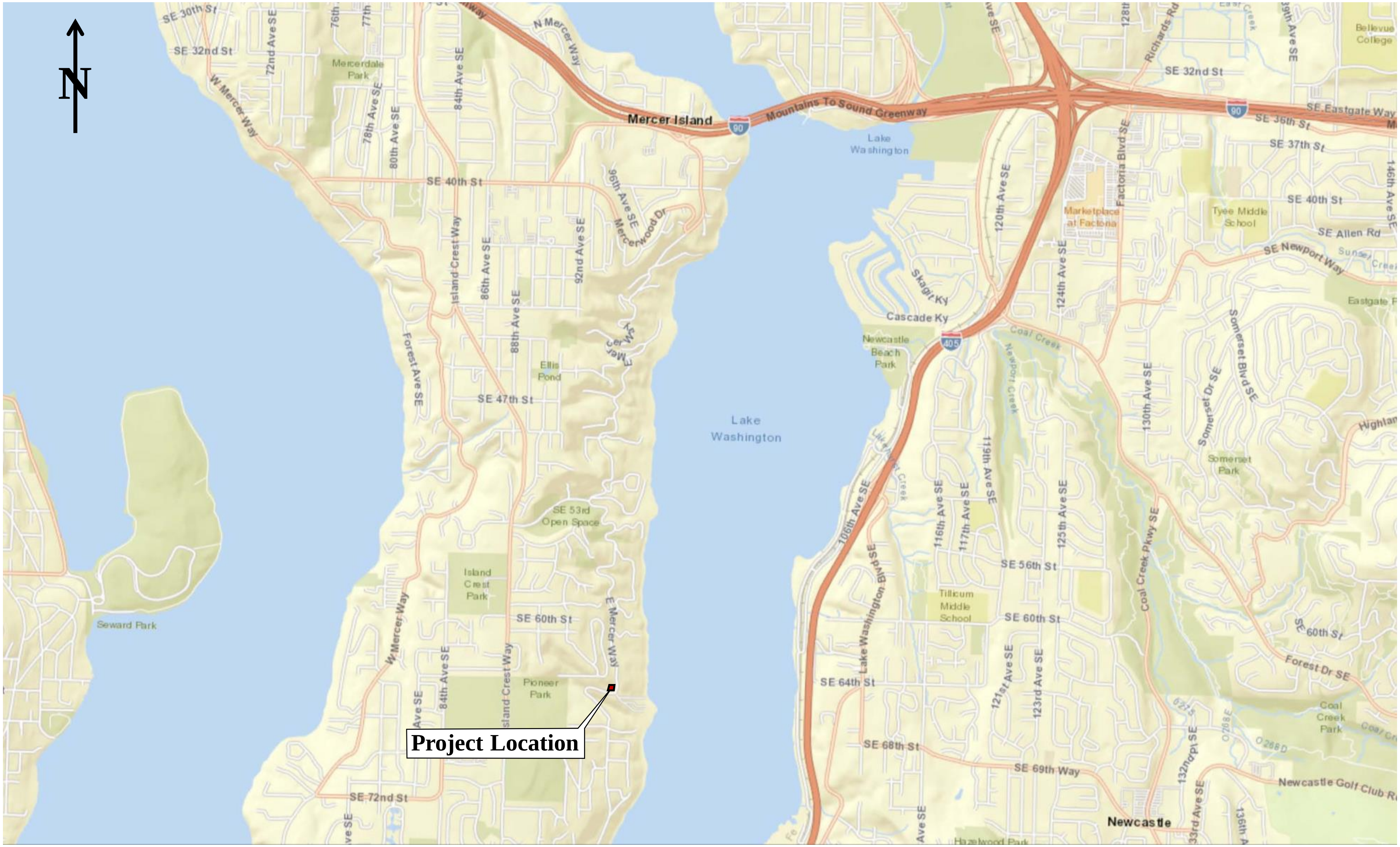
Troost, K. G. and Wisher, A.P., 2006, *Geologic Map of Mercer Island,* Geomap NW, University of Washington and the City of Mercer Island.

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), 2025, *Standard Specifications for Road, Bridges, and Municipal Construction*, Olympia, Washington.

25-036 Vicinity Fig 1.ppt 1/27/2025(4:13 PM) SHE



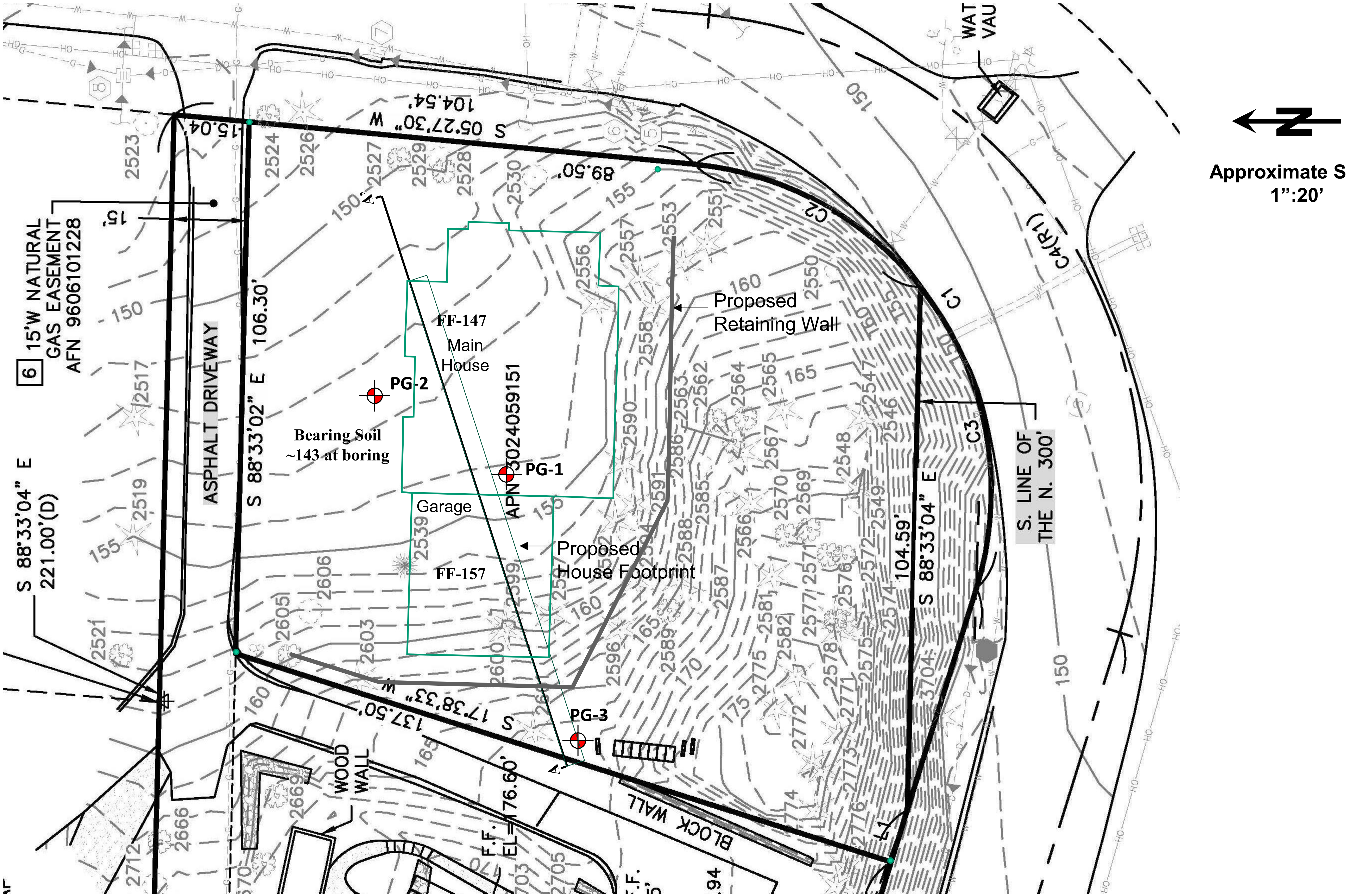
Map not to Scale
Base Map from
Dept of Natural
Resources Geological
Information Portal

PanGEO INCORPORATED

Proposed SFR Development
6423 East Mercer Way
Mercer Island, WA

VICINITY MAP

Project No.	Figure No.
25-036	1



Approximate Scale
1":20'

LEGEND:

Approximate Boring Location, PanGEO, Inc., March, 2019

Subsurface Profile (see Fig 3)

Notes:

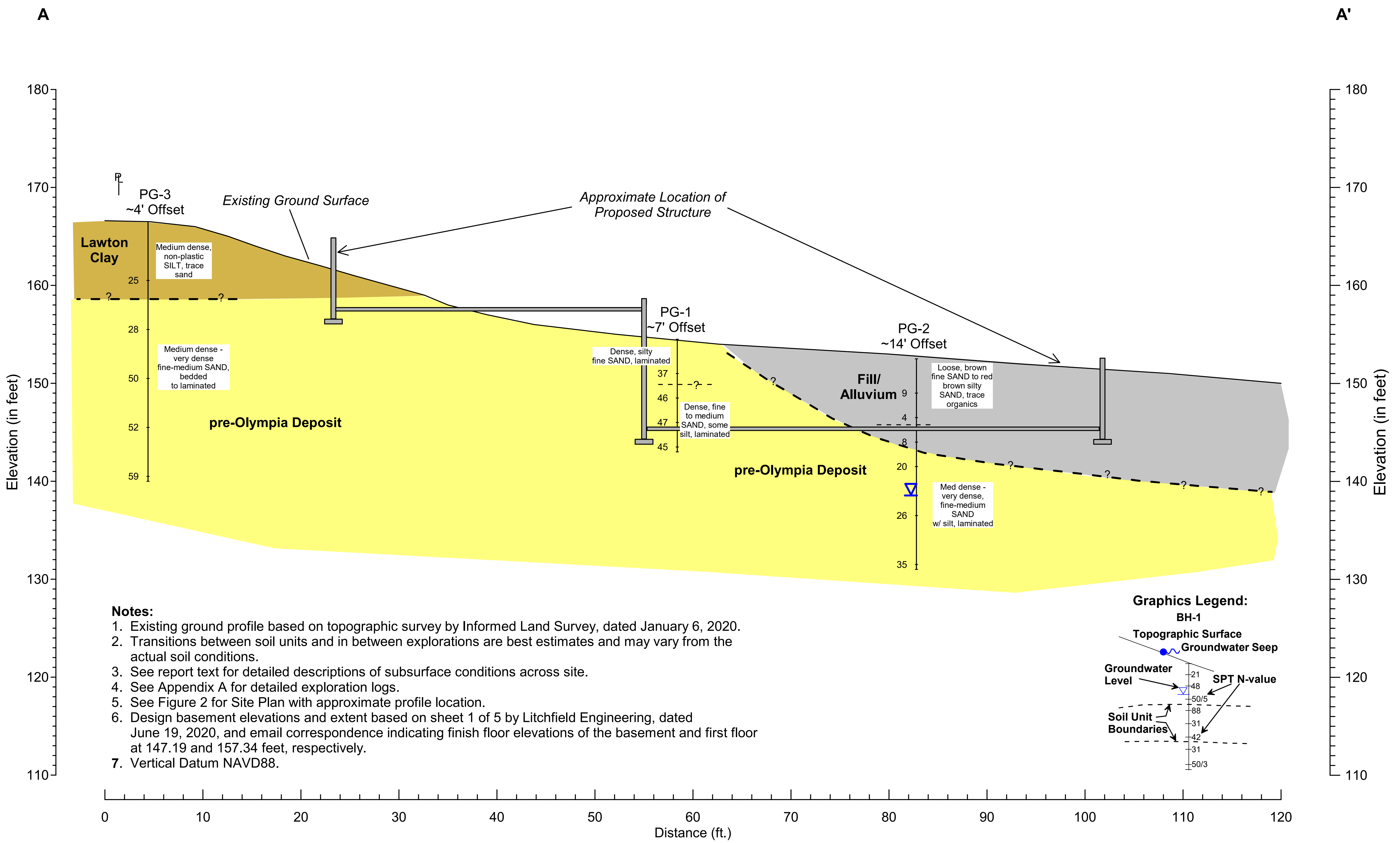
1. Topographic Survey by Informed Land Survey, dated January 6, 2020.
2. Vertical Datum NAVD 88

PanGEO INCORPORATED

Proposed Single Family Residence
6427 East Mercer Way
Mercer Island, Washington

SITE AND EXPLORATION PLAN

Project No.	Figure No.
25-036	2



Notes:
1. Existing ground profile based on topographic survey by Informed Land Survey, dated January 6, 2020.
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 2 for Site Plan with approximate profile location.
6. Design basement elevations and extent based on sheet 1 of 5 by Litchfield Engineering, dated June 19, 2020, and email correspondence indicating finish floor elevations of the basement and first floor at 147.19 and 157.34 feet, respectively.
7. Vertical Datum NAVD88.

19-369 Stick Log and Profile Data.xls 6/18/25 (3:33:56) SHE

PanGEO
INCORPORATED

**Proposed Single
Family Residence
6427 East Mercer Way
Mercer Island, Washington**

**GENERALIZED SUBSURFACE PROFILE
SECTION A-A'**

Project No.	Figure No.
25-036.100	3

APPENDIX A

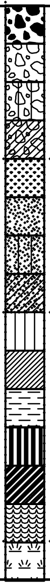
CURRENT SUBSURFACE INVESTIGATION

6427 East Mercer Way, 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

Laminated: Layers of soil typically 0.05 to 1mm thick, max. 1 cm

Lens: Layer of soil that pinches out laterally

Interlayered: Alternating layers of differing soil material

Pocket: Erratic, discontinuous deposit of limited extent

Homogeneous: Soil with uniform color and composition throughout

Fissured: Breaks along defined planes

Slickensided: Fracture planes that are polished or glossy

Blocky: Angular soil lumps that resist breakdown

Disrupted: Soil that is broken and mixed

Scattered: Less than one per foot

Numerous: More than one per foot

BCN: Angle between bedding plane and a plane normal to core axis

COMPONENT DEFINITIONS

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

TEST SYMBOLS

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

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

SYMBOLS

Sample/In Situ test types and intervals

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

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

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

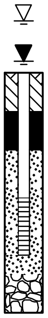
Thin wall (Shelby) tube

Grab

Rock core

Vane Shear

MONITORING WELL



Groundwater Level at time of drilling (ATD)

Static Groundwater Level

Cement / Concrete Seal

Bentonite grout / seal

Silica sand backfill

Slotted tip

Slough

Bottom of Boring

MOISTURE CONTENT

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

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Phone: 206.262.0370

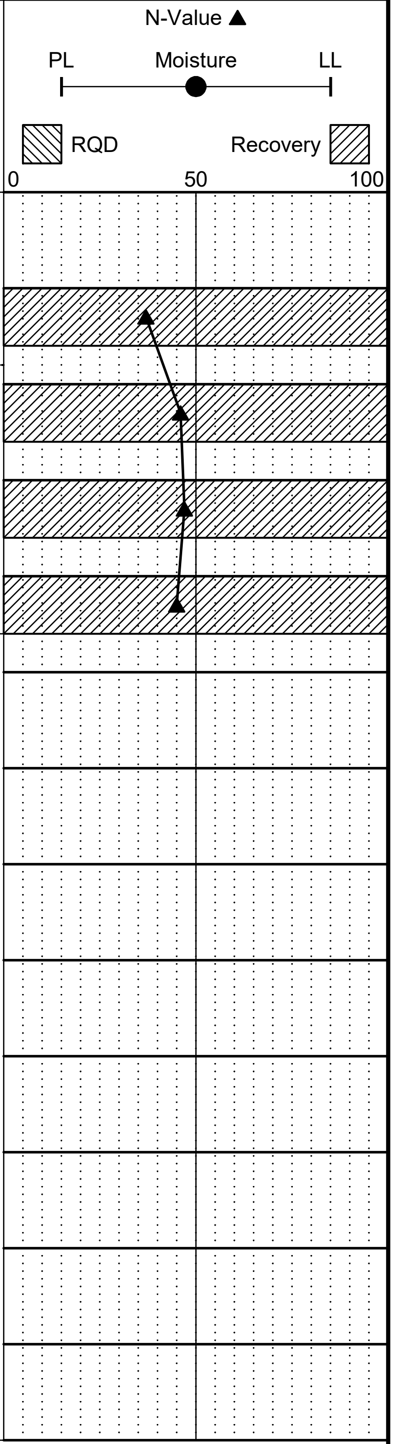
Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project: Proposed SFR
Job Number: 25-036
Location: 6427 East Mercer Way, Mercer Island, WA
Coordinates: Northing: , Easting:

Surface Elevation: 154.5ft
Top of Casing Elev.:
Drilling Method: HSA
Sampling Method: SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						Dense, brown, silty, fine SAND: slightly moist, non-plastic fines, homogeneous, laminated with slightly rusty laminae. (pre-Olympia Sand Deposit - Qpon).
2.5	S-1	☒	11 17 20			
						Dense, brown to brown gray, fine to medium SAND: slightly moist, some silt, homogeneous, laminated. (pre-Olympia Sand Deposit - Qpon).
5.0	S-2	☒	13 20 26			
7.5	S-3	☒	16 22 25			
10.0	S-4	☒	12 23 22			
						Bottom of Boring.
12.5						
15.0						
17.5						
20.0						
22.5						
25.0						
27.5						
30.0						
32.5						



Completion Depth: 11.5ft
Date Borehole Started: 3/7/19
Date Borehole Completed: 3/7/19
Logged By: S. Evans
Drilling Company: Boretec, Inc

Remarks: Boring drilled as part of previous project 19-062. No groundwater encountered during drilling. Vertical Datum NAVD 88.

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LOG OF TEST BORING PG-1

Figure A-2

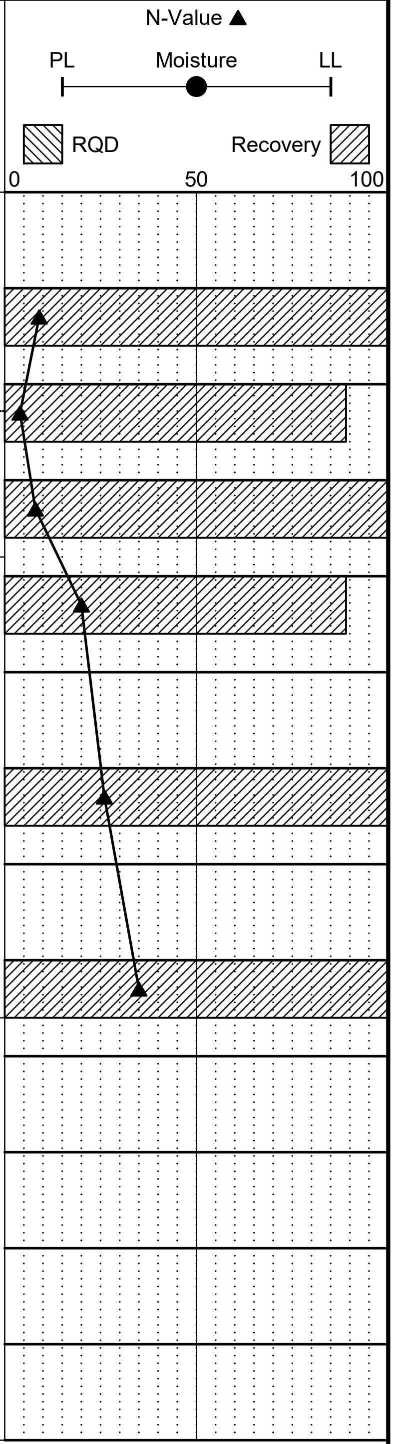
The stratification lines represent approximate boundaries. The transition may be gradual.

Sheet 1 of 1

Project: Proposed SFR
Job Number: 25-036
Location: 6427 East Mercer Way, Mercer Island, WA
Coordinates: Northing: , Easting:

Surface Elevation: 152.5ft
Top of Casing Elev.:
Drilling Method: HSA
Sampling Method: SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						Loose, brown, silty, fine SAND: moist, non-plastic fines, occasional organic bits, slightly mixed texture. (Fill - af).
2.5	S-1	☒	5 4 5			
5.0	S-2	☒	3 1 3			Loose, red brown, silty, fine SAND with silt: moist to very moist, non-plastic fines, homogeneous, massive. (Fill/Alluvium - af/Qal).
7.5	S-3	☒	4 4 4			Laminated, grading brown with reddish laminae.
10.0	S-4	☒	5 8 12			Medium dense to dense, brown to brown gray, fine to medium SAND with silt: very moist to wet, non-plastic fines, homogeneous, laminated with slightly rusty laminae at top. (pre-Olympia Sand Deposit - Qpon).
12.5					∇	
15.0	S-5	☒	9 12 14			Wet, non-plastic fines, rapid dilatancy, laminated with many yellowish laminae.
17.5						
20.0	S-6	☒	13 16 19			Massive.
						Bottom of Boring.
22.5						
25.0						
27.5						
30.0						
32.5						



Completion Depth: 21.5ft
Date Borehole Started: 3/7/19
Date Borehole Completed: 3/7/19
Logged By: S. Evans
Drilling Company: Boretec, Inc

Remarks: Boring drilled as part of previous project 19-062. Groundwater level estimated based on wetness of soil sample and water on the sampling rods. Vertical Datum NAVD 88.

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LOG OF TEST BORING PG-2

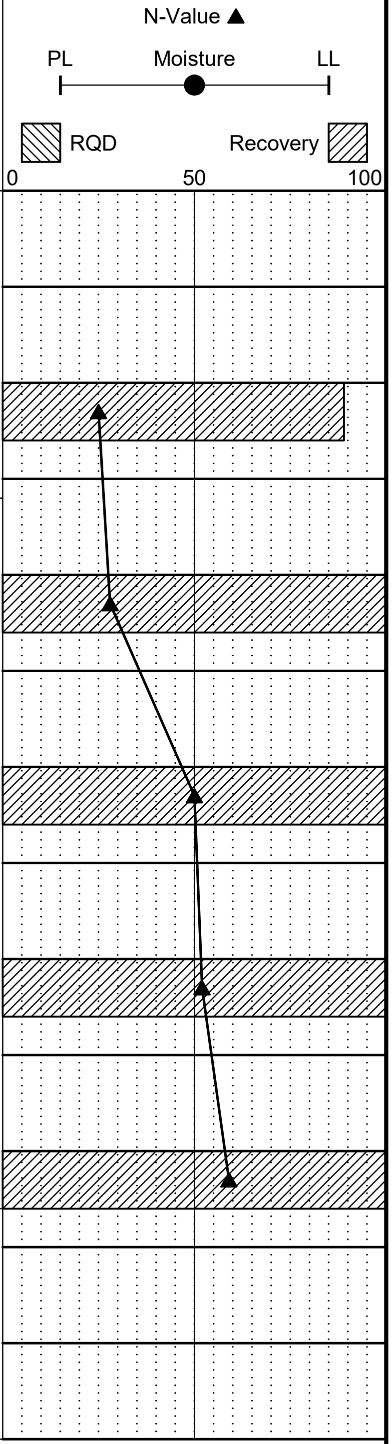
Figure A-3

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

Project: Proposed SFR
Job Number: 25-036
Location: 6427 East Mercer Way, Mercer Island, WA
Coordinates: Northing: , Easting:

Surface Elevation: 166.5ft
Top of Casing Elev.:
Drilling Method: HSA
Sampling Method: SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						Medium dense, brown gray SILT: very moist, non-plastic, trace to some fine sand, homogenous, laminated. (Lawton Clay - Qvlc).
2.5						
5.0	S-1	☒	6 10 15			Grading to silty, fine SAND.
7.5						Medium dense to very dense, brown to brown gray, fine to medium SAND: moist, non-plastic fines, fine bedded with finer / coarse beds, sharp and moderately gradational contacts, indistinctly laminated. (pre-Olympia Sand Deposit - Qpon).
10.0	S-2	☒	8 12 16			
12.5						
15.0	S-3	☒	14 25 25			Grading massive, very dense, some non-plastic silt.
17.5						
20.0	S-4	☒	15 24 28			Homogeneous, massive, fine to medium SAND.
22.5						
25.0	S-5	☒	18 29 30			Moist, trace to some silt.
27.5						Bottom of Boring.
30.0						
32.5						



Completion Depth: 26.5ft
Date Borehole Started: 3/7/19
Date Borehole Completed: 3/7/19
Logged By: S. Evans
Drilling Company: Boretec, Inc

Remarks: Boring drilled as part of previous project 19-062. No groundwater encountered during drilling. Vertical Datum NAVD 88.

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LOG OF TEST BORING PG-3

Figure A-4

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