PanGEO
INCORPORATED
Geotechnical & Earthquake
Engineering Consultants

September 28, 2023
Project No. 23-244

Erika Mobley
c/o **Ponting Fitzgerald Architects LLC**
19037 West 57th Drive
Golden, Colorado 80403
Attn: Mathew Fitzgerald

Subject: **Geotechnical Report**
Proposed Additions and Alterations
7244 North Mercer Way, Mercer Island, Washington

Dear Erika:

Please find attached our geotechnical report to support the design and construction of the proposed additions and alterations to your residence at the above referenced address.

In summary, competent bearing soils consisting of native stiff to hard silt were encountered within about 1 to 5½ feet below existing grades at our test boring locations. In our opinion, where needed, conventional footings bearing on the competent native silt or on properly compacted structural fill placed on competent native silt may be used to support the proposed additions. Alternatively, where new footings risk surcharging the adjacent basement walls, and deep excavations to lower the new footings are not feasible, small diameter pipe piles (often referred to as pin piles) may be utilized to support the proposed additions/alterations. Temporary unsupported excavations may be sloped as steep as 1H:1V (Horizontal:Vertical).

We appreciate the opportunity to assist you with this project. Please call if you have any questions.

Sincerely,

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

Encl.: Geotechnical Report

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GEOTECHNICAL REPORT
PROPOSED ADDITIONS AND ALTERATIONS
7244 NORTH MERCER WAY
MERCER ISLAND, WASHINGTON

1.0 INTRODUCTION

As requested, PanGEO, Inc. is pleased to present this geotechnical report for the proposed additions and alterations to the residence located at 7244 North Mercer Way in Mercer Island, Washington. This study was performed in general accordance with our mutually agreed scope of services outlined in our proposal dated July 10, 2023, which was subsequently approved by you on August 21, 2023. Our scope of services included reviewing readily available geologic and geotechnical data, conducting a site reconnaissance, drilling two test borings at the site, and developing the conclusions and recommendations presented in this report.

2.0 SITE AND PROJECT DESCRIPTION

The subject site is an approximately 12,160 square foot lot located at 7244 North Mercer Way in Mercer Island, Washington, approximately as shown on the attached *Figure 1, Vicinity Map*. The site is rectangular in shape, and is bound by North Mercer Way to the west, and by single-family residential lots in the other directions. The site is currently developed with a one-story residence with one level of daylight basement. An attached garage is located at the southwestern corner of the site. A concrete driveway provides access to the attached garage from North Mercer Way to the west. Additionally, the northern portion of the site contains an existing concrete right-of-way, which provides access to the adjacent properties to the north and east. The layout of the site is shown on *Figure 2, Site and Exploration Plan*.

Based on review of the project topographic survey, the site gradient generally slopes down from the northwest property corner towards the east at an average gradient of about 18 percent. However, the concrete driveway west of the house is at about 30 percent gradient (see topographic contours on *Figure 2*). Total vertical relief across the site is on the order of about 30 feet. Current site conditions are shown on Plates 1 and 2 on the following page.

We understand that you plan to make alterations and construct some additions to the existing residence at the site. Proposed alterations/additions include a 59 square-foot addition to the kitchen on the western side of the house, a 13 square-foot addition to the southeast corner of the house, and lifting the garage floor 14 inches and regrading of the exiting driveway in front of the garage to permit easier vehicular access into the garage. The planned alterations also include converting 166 square-feet of existing concrete paving area on the east side of the house into additional garden area. As currently planned, the alterations/additions will not significantly increase the footprint of the overall structure.

***Plate 1**. View of the west side of the existing residence and garage at the site, looking east from North Mercer Way.*

***Plate 2.** View of the landscaping and concrete paving area on east side of the subject site, looking south.*

Based on review of the City of Mercer Island's Geologic Hazards Map, the subject site is located in a potential landslide hazard area, a seismic hazard area and a potential erosion hazard area. As a result, the City of Mercer Island requires a geotechnical engineering study to evaluate the site stability and provide recommendations to improve the site stability, if needed.

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

3.0 SUBSURFACE EXPLORATION

Two test borings (PG-1 and PG-2) were advanced at the site on August 22, 2023 using a hand-portable limited access drill rig owned and operated by CN Drilling of Seattle, Washington. Test borings PG-1 and PG-2 were drilled to maximum depths of about 14 feet and 9 feet below existing grades, respectively. The approximate boring locations were determined relative to existing features and are shown on the attached *Figure 2*.

The drill rig was equipped with 5-inch outside diameter hollow stem augers. 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. The completed borings were backfilled with drill cuttings and bentonite chips.

A geologist from PanGEO was present during the field exploration to observe the drilling, to assist in sampling, and to describe and document the soil samples obtained from the borings. The summary boring logs are included in *Appendix A, Figures A-2 and A-3*. The soil samples were described using the Modified Unified Soil Classification System outlined on *Figure A-1* in *Appendix A*.

4.0 SUBSURFACE CONDITIONS

4.1 SITE GEOLOGY

Based on our review of *The Geologic Map of Mercer Island* (Troost and Wisher, 2006), the subject site is underlain by Vashon Till (Geologic Map Unit Qvt) in the southwestern portion of the site and by pre-Olympia aged fine-grained glacial deposits (Qpogf) in the northeastern portion of the site.

Vashon till (i.e., glacial till) is described by Troost et al. as a dense to very dense, heterogeneous mixture of silt, sand, and gravel laid down at the base of an advancing glacial ice sheet. Pre-Olympia aged fine-grained glacial deposits typically consist of hard, laminated to massive, silts and clays of inferred glacial origin.

The general area of the site is also identified as being underlain by mass-wastage deposits. Mass-wastage deposits are described as loose to dense and soft to stiff, colluvium, soil, landslide debris, and organic matter with indistinct morphology.

4.2 SOIL CONDITIONS

Based on the results of our test borings, the site is generally underlain by fill/mass-wastage deposits over a layer of stiff to hard silt with trace clay that we interpreted as pre-Olympia fine-grained glacial deposits. None of the test borings encountered soils consistent with the Vashon till deposits mapped at the site.

Based on the site topographic survey and subsurface conditions encountered in the test borings, we developed a generalized subsurface profile A-A' at the approximate location shown on *Figure 2*. The subsurface profile is attached as *Figure 3*.

A description of the generalized soil units encountered in the test borings completed at the site is presented below. For a more detailed description of the subsurface conditions encountered at each exploration location, please refer to our boring logs provided as *Figures A-2 and A-3*.

> **Soil Unit 1: Fill/Mass-Wastage Deposits** – Underlying approximately 12 inches of topsoil, test boring PG-1 encountered a layer of loose, non-plastic silt with varying amounts of gravel, organics, and trace fine sand that extended to about 5½ feet below existing grade.

Based on the relatively loose density and disturbed texture, we interpret this soil unit as undocumented fill or mass-wastage deposits derived from soils upslope of the subject site. This soil unit was not encountered in boring PG-2.

Soil Unit 2: Pre-Olympia fine-grained deposits (**Qpogf**) – Below Soil Unit 1 in test boring PG-1 and below approximately 9 inches of topsoil in test boring PG-2, our borings encountered stiff to hard, low- to medium-plasticity silt with trace amounts of clay that extended to the maximum drilled depths of 14 feet and 9 feet below existing grades in borings PG-1 and PG-2, respectively. We interpret this soil as the pre-Olympia fine-grained glacial deposits mapped at the site.

4.3 GROUNDWATER CONDITIONS

Groundwater was not encountered in our test borings at the time of drilling on August 22, 2023. However, the evidence of iron-oxide staining found in Soil Unit 2 in our test borings suggests that seasonal shallow perched groundwater may be present.

The designers and contractor should be aware there will be fluctuations in groundwater conditions depending on the season, amount of rainfall, surface water runoff, 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 GEOLOGICALLY HAZARDOUS AREAS CONSIDERATIONS

5.1 LANDSLIDE HAZARDS

Based on review of the City of Mercer Island's Geologic Hazards Map, the subject site is located within a potential landslide hazard area. However, to the best of our knowledge, there are no documented past known slides at the subject site. Additionally, as part of this evaluation, we conducted a site reconnaissance of the subject property on August 22, 2023. During our site reconnaissance, we did not observe obvious evidence of ongoing slope instability at the site, such as uneven topography, slumps, or tension cracks.

As part of our reconnaissance, we reviewed the landslide inventory map from the Washington Department of Natural Resources (DNR). The DNR landslide inventory, started in 2017, identifies landslide hazard areas using high quality LiDAR data and GIS, based on the methods of Slaughter et al. (2017). Based on elevation maps derived from LiDAR, this model uses an algorithm to account for slope gradient and curvature to develop areas of interest or concerns. These areas are then viewed at multiple scales, cross-referenced with geologic maps, reviewed with orthorectified aerial photos, and field verified when possible. The suspected landslides were then further analyzed using GIS to estimate properties such as the slope gradient adjacent to headscarp, headscarp height, average scarp distance, failure depth, and landslide volume.

Based on our review of the DNR landslide inventory, there is no evidence of landslide activities on the property or immediately adjacent to the property. Additionally, we reviewed a LiDAR image of the site and its vicinity. A review of LiDAR image indicates that the slopes in the immediate vicinity of the site have a consistent slope angle and have not been significantly modified by landslides or by previous construction activities. The LiDAR image shows no signs of landslide activities on the property or its immediate vicinity.

Based on our independent evaluation of the LiDAR image and our site observation, we agree with the DNR findings, i.e., there are no known slides or head scarps on or immediately adjacent to the property.

We also evaluated the existing building and observed no signs of apparent cracks or vertical settlement on the existing foundation walls. We did observe some minor cracking in the concrete paving surface on the east side of the site, but in our opinion it is likely the result of improper fill placement under the concrete and not the result of instability. In our opinion, the site is globally stable in its current condition and the existing development has been stable in its current configuration.

Based on our reconnaissance of the slope, review of DNR landslide inventory and LiDAR imagery, and our understanding of subsurface conditions at the site, in our opinion a large deep-seated type of slope failure is relatively unlikely on the subject property. In our opinion, shallow surficial slides of various sizes are the likely type of failure that could occur on the steep slope at the site. However, due to the lack of observed evidence of recent shallow slides, and the vegetation (i.e., mature trees) that protect the surface of the slope from erosion, in our opinion the potential for a large shallow slide is relatively low.

Based on our field observations, the relatively gentle topography of the site and vicinity, and the results of our field exploration, it is our opinion that the site is stable in its current configuration. Furthermore, it is our opinion that the proposed construction will not adversely impact the overall stability of the site and surrounding properties, provided that the recommendations presented in this report are properly incorporated into the design and construction of the project.

5.2 SEISMIC HAZARDS

Based on our review of the City of Mercer Island's Geologic Hazards Maps, the project site is mapped as 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 stiff to hard silt with clay underlying the site at shallow depths and the absence of a groundwater table, in our opinion, the potential for soil liquefaction is low, and design considerations associated with soil liquefaction is not needed.

It is also our opinion that the potential for seismic-induced slope failure is low within the site due to the relatively gentle topography, and the presence of competent soils (stiff to hard silt with clay) at shallow depths.

5.3 EROSION HAZARDS

The entire 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 high 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. Recommendations for controlling erosion are provided in Section 7.6 of this report.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 SEISMIC SITE CLASS

We assume the seismic design of the proposed structure will be accomplished in accordance with the 2018 or 2021 International Building Code (IBC), which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years). Based on the results of our test borings and the geology at the site, it is our opinion that Site Class D (Stiff Soils) is considered appropriate for determining the site coefficients for the seismic design of the proposed additions.

6.2 FOUNDATION DESIGN

6.2.1 Foundation Types

Based on the subsurface conditions encountered at the site and the current design plans, it is our opinion that, where needed, conventional footing bearing on the undisturbed native soil (Soil Unit

2) or compacted structural fill placed on the undisturbed native soil may be used to support the proposed additions. We anticipate that competent bearing soils will be present at about 1 to 5 feet below the existing grades. Depending on the actual footing subgrade elevation and the variations of soil conditions, over-excavation may be required in localized areas to reach competent native soils. The foundation subgrade soils should be recompacted to a firm/dense condition prior to footing construction.

Where the additions will be constructed adjacent to the existing basement walls, we recommend the new footings be located below a 1H:1V (Horizontal:Vertical) projection from the base of the existing basement footings to avoid surcharging the basement walls. If needed, the potential surcharge pressures can be evaluated using a lateral pressure coefficient of 0.35 (i.e., lateral pressure equal to about 35 percent of the vertical pressure).

Alternatively, in lieu of deep excavations to lower the footings, we understand you may consider using small diameter pipe piles (often referred to as pin piles) to support the new addition footings where they risk surcharging the adjacent basement walls, which is feasible.

If pin piles will be used, it is our opinion that driven 2- to 4-inch diameter steel pipe piles are appropriate. Two-inch diameter pin piles are typically installed using portable, handheld equipment. Three-inch and four-inch diameter pin piles are installed using a hydraulic hammer (600 to 2,000 lbs.) typically mounted to a small to medium-sized excavator.

The following sections include our recommended foundation design parameters for conventional footings and for the 2-, 3- and 4-inch diameter steel pipe piles.

6.2.2 Conventional Footings

Where used, conventional continuous and individual spread footings should bear directly on the competent native soil (Soil Unit 2) or properly compacted structural fill placed directly on the competent native soil (Soil Unit 2). All unsuitable bearing soils, such as the fill/mass wasting deposits (Soil Unit 1) should be completely removed from below the footings prior to the fill placement.

In designing the footings, the shape of footings will need to be considered regarding the available space for temporary excavations. Where space may be limited for an unsupported open cut, it may be necessary to use L-shaped perimeter footings in order to conserve space and to allow the temporary excavations to be made within the property limits.

Bearing Pressure – We recommend a maximum allowable soil bearing pressure of 2,500 pounds per square foot (psf) be used to size the footings bearing on competent native soils and structural fill placed on the competent native soils. The recommended allowable bearing pressure is for dead plus live loads. For allowable stress design, the recommended bearing pressure may be increased by one-third for transient loading, such as wind or seismic forces.

Lateral Resistance – Lateral forces from wind or seismic loading may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundations and walls, and by friction acting on the base of the foundations:

- Passive resistance may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf) for level backfill. This value includes a factor safety of at least 1.5 assuming that properly compacted structural fill will be placed adjacent to the sides of the footings, and level ground surface adjacent to the footings.
- A friction coefficient of 0.35 may be used to determine the frictional resistance at the base of the footings. This coefficient includes a factor of safety of approximate 1.5.

Foundation Performance – Total and differential settlements are anticipated to be within tolerable limits for footings designed and constructed as discussed above. Footing settlement under static loading conditions is estimated to be less than approximately one inch, and differential settlement between adjacent columns should be less than about ½ inch. Most settlement will occur during construction as loads are applied.

Perimeter Footing Drains – Footing drains should be installed around the perimeter of the buildings, at or just below the invert of the footings. The footing drains should consist of 4-inch diameter, schedule 40 PVC or SDR 35, perforated pipe embedded in washed drain rock/pea gravel and wrapped in filter fabric. 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.

6.2.3 Driven Pipe Piles (Pin Piles)

It is our opinion that driven small diameter steel pipe piles (pin piles) may be utilized to support the proposed additions foundation, especially where the foundations risk surcharging the adjacent basement walls, and deep excavations to lower the footings may not be feasible. The principal advantages of driven pipe piles are that the pile lengths can be easily adjusted in the field, the speed of installation, no spoils to be disposed of, and eliminate the needs for over-excavations.

Pin Pile Sizes and Capacity – If pin piles will be used, it is our opinion that driven 2- to 4-inch diameter steel pin piles are appropriate. Two-inch diameter pin piles are typically installed using portable, handheld equipment. Three-inch and four-inch pin piles are installed using a hydraulic hammer (600 to 2,000 lbs.) typically mounted to a small to medium-sized excavator.

The number of piles required depends on the magnitude of the design load. Table 1 shows the recommended capacities for pin piles with an approximate factor of safety of at least 2.0.

Table 1 – Pin Pile Capacities

Pile Diameter (in)	Allowable Axial Compression (tons)
2	3
3	6
4	10

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.

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

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

1. 2-inch diameter piles should consist of Schedule-80, ASTM A-53 Grade “A” pipe.
2. 3-inch and 4-inch diameter piles should consist of Schedule-40, ASTM A-53 Grade “A” pipe.
3. 2-inch piles shall be driven to refusal with a minimum 90-lb jackhammer or a 140-lb Rhino Hammer. Refusal is defined as no more than 1 inch of penetration for 1 minute of continuous driving. Please note that the City requires load testing if a different driving criteria is used for a different hammer size.

4. 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:

Table 2 – Three-Inch Pile Refusal Criteria

Hammer Size	Approx. Blows 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

The driving criteria recommended in the table above will be verified by a static load test program (see discussion in Item 7).

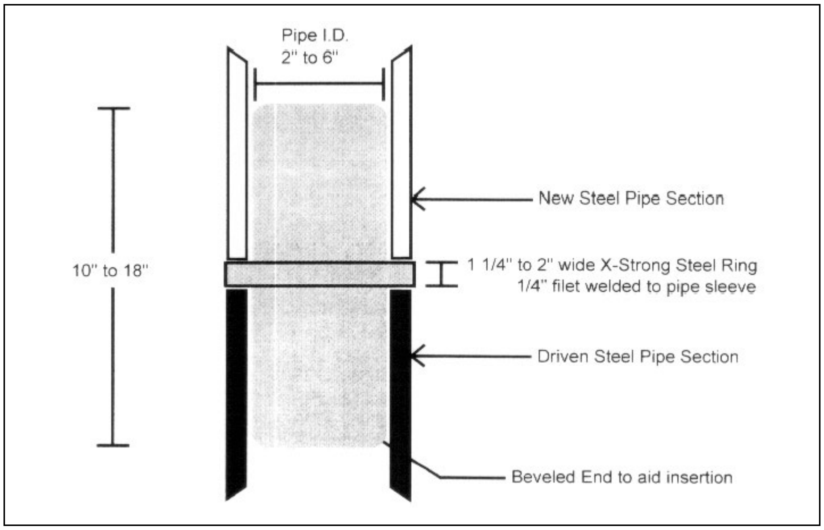
5. 4-inch piles shall be driven to refusal with a minimum 850-lb hydraulic hammer. We recommend the following refusal criteria based on the size of hammer utilized:

Table 3 – Four-Inch Pile Refusal Criteria

Hammer Size	Approx. Blows 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

The driving criteria recommended in the table above will be verified by a static load test program (see discussion in Item 7).

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



7. At least 3 percent (but no more than 5) of the 3-inch and 4-inch pin piles should be load tested. All load tests shall be performed in accordance with the procedure outlined in ASTM D1143. 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 to verify that the pile has been driven to adequate refusal within the anticipated bearing stratum.

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.

Lateral Resistance – The capacity of pin pipes to resist lateral loads is very limited and should not be used in design. Therefore, lateral forces from wind or seismic loading should be resisted by the passive earth pressures acting against the pile caps and below-grade walls or from battered piles (batter no steeper than 3(H):12(V)). ***Friction at the base of pile-supported concrete 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). This value include a safety factor of about 1.5 assuming that properly compacted granular fill will be placed adjacent to and surrounding the pile caps and grade beams, and level ground surface adjacent to the pile caps and grade beams.

Estimated Pile Length – The required pile length in order to develop the recommended pile capacity is expected to vary across the footprint of the structures, depending on the actual driving conditions encountered. For planning and cost estimating purposes, we suggest that a 5- to 10-foot penetration into the underlying hard silt is an appropriate estimate. As such, we estimate that average pile lengths of about 13 to 18 feet will be needed. We recommend a minimum pile length of 10 feet.

Pile Installation Monitoring – The quality of a pipe pile foundation is dependent, in part, on the experience and professionalism of the installation company. We recommend that a company with personnel experienced in the successful installation of pipe piles be selected to install the piles.

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.

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.

6.3 RETAINING AND BASEMENT WALL DESIGN PARAMETERS

Retaining and basement walls should be properly designed to resist the lateral earth pressures exerted by the soils behind the wall. Adequate drainage provisions should also be provided behind the walls to intercept and remove groundwater that may be present behind the walls. Our geotechnical recommendations for the design and construction of retaining/basement walls are presented below.

6.3.1 Lateral Earth Pressures

Basement walls should be designed for an equivalent fluid unit weight of 50 pcf with level backslope and 65 pcf with a backslope no steeper than 2H:1V. For cantilevered walls, 35 pcf should with level backslope and 45 pcf with a backslope no steeper than 2H:1V.

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. The recommended lateral pressures assume that the backfill behind the wall consists of a free draining and properly compacted fill with adequate drainage provisions.

6.3.2 Wall Surcharge

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

6.3.3 Wall Drainage

We recommend that footing drains be installed behind walls. As a minimum, 4-inch diameter perforated drainpipes should be installed next to the base of the footings and embedded in 12 to 18 inches of pea, washed gravel or clean crushed rock. The gravel should be wrapped in a geotextile filter fabric to prevent the migration of fines into the drain system.

For site retaining walls, if needed, weep holes may be used in lieu of perforated pipes discussed above. If used, the weep holes should be located near the base of the wall, at least 1½ inch in diameter, and spaced no more than 8 feet apart. A layer of geotextile such as Mirafi 140N should be placed behind the weep holes to prevent soil loss through the openings.

For basement walls constructed against temporary shoring, a composite drainage material, such as Miradrain 6000, should be installed between the basement walls and the shoring walls. The drain mat should be hydraulically connected to a 4-inch perforated drainpipe located along the interior of the footings and directed to an appropriate outlet.

6.3.5 Wall Backfill

Wall backfill should consist of free draining granular soils In our opinion, the on-site soils have a high fines content, and are not suitable to be re-used as wall backfill. Imported wall backfill such as City of Seattle Type 17 Mineral Aggregates (Section 9.03.10 (1) of the 2023 Seattle Standard Specifications) or Gravel Borrow (Section 9.03.14 (1) of the 2023 WSDOT Standard Specifications) should be assumed for this project.

The fill should be moisture conditioned to near its optimum moisture content, placed in loose, horizontal lifts less than about a foot 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 will be performed, the test results should indicate at least 95 percent of the maximum dry density, as determined using test method ASTM D 1557. Within 5 feet of the wall, the backfill should be compacted to 90 percent of the maximum dry density.

6.4 CONCRETE SLAB-ON-GRADE

Floor slabs, where used, may be constructed using conventional concrete slab-on-grade floor construction. The floor slabs should be supported on competent native sandy soils or compacted structural fill. Any loose sand at the slab subgrade should be either recompacted to a firm/dense condition or over-excavated to expose dense native soils. Over-excavation should be replaced with compacted structural fill.

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 ¾-inch, clean crushed rock (less than 3 percent fines). The capillary break material should also have no more than 10 percent passing the No. 4 sieve and less than 5 percent by weight of the material passing the U.S. Standard No. 100 sieve. The capillary break should be placed on the subgrade that has been compacted to a dense and unyielding condition. A 10-mil polyethylene vapor barrier should also be placed directly below the slab. We also recommend that construction joints be incorporated into the floor slab to control cracking.

6.5 PERMANENT SURFACE DRAINAGE CONSIDERATIONS

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.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 FOUNDATION SUBGRADE

All footing subgrades should be carefully prepared. Footing excavations should be observed by PanGEO to confirm that the exposed footing subgrade is consistent with the expected conditions and adequate to support the design bearing pressure. The footing subgrade at the foundation level should be in a dense condition prior to concrete pour. Any over-excavations in the footing areas should be backfilled with compacted structural fill.

The foundation subgrade will need to be protected from moisture-related disturbances if works will be performed during wet weather. This may be accomplished with at least 2 to 3 inches of lean-mix concrete, or 4 to 6 inches of crushed surfacing base course (CSBC). Alternatively, the reinforcing steel can be prefabricated, and the placement of the steel and concrete can be placed immediately after the footing excavation is completed. This will reduce the exposure of the footing subgrade to moisture.

7.2 TEMPORARY EXCAVATIONS

Where space is available, an unsupported slope cut may be incorporated into the excavation design. 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.

In general, temporary excavations deeper than a total of 4 feet should be sloped or shored. However, excavations less than 4 feet deep, if located along or near property lines, will also need to be sloped or supported if sufficient space is not available to lay back the excavations without encroaching into neighboring properties.

Where space is available for sloped open cuts, for planning purposes, the temporary unsupported excavation may be sloped as steep as 1H:1V (Horizontal:Vertical). Where space may be limited, the use of L-shaped footings may be required to conserve space for temporary cuts.

The temporary excavations and cut slopes should be re-evaluated in the field during construction based on actual observed soil conditions and may need to be flattened in the wet seasons and should be covered with plastic sheets. The cut slopes should be covered with plastic sheets in the raining season. We also recommend that heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a distance equal to 1/3 the slope height from the top of any excavation.

7.3 STRUCTURAL FILL AND COMPACTION

In the context of this report, structural fill is defined as compacted fill placed under footings, concrete stairs and landings, and slabs, or other load-bearing areas. In our opinion, the on-site soils are not suitable to be reused as structural fill. The structural backfill within the footing areas should consist of imported, granular fill such as the City of Seattle Type 17 Mineral Aggregate (Section 9.03.10 (1) of the 2023 Seattle Standard Specifications), Gravel Borrow (Section 9.03.14 (1) of the 2023 WSDOT Standard Specifications), or approved equivalent.

Structural fill 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.

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

7.4 MATERIAL REUSE

The native soils underlying the site are moisture sensitive and can become disturbed and soft when exposed to inclement weather conditions and construction traffic. 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 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 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 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.

7.6 EROSION CONSIDERATIONS

Surface runoff can be controlled during construction by careful grading practices. The erosion control plan should include measures for reducing concentrated surface runoff and protecting disturbed or exposed surfaces by mulching and revegetation. The temporary erosion and sediment control (TESC) plan should include the following:

- Construction activity should be scheduled or phased as much as possible to reduce the amount of earthwork that is performed during the wet season – October through May.
- The TESC plan should include adequate ground cover-measures, access roads, and staging areas. The contractor should be prepared to implement and maintain the TESC measures to maximize the effectiveness of the TESC elements.
- Where practical, a buffer of vegetation should be maintained around cleared areas.
- The TESC measures should be installed in conjunction with the initial ground clearing. The recommended sequence of construction within a given area after clearing would be to install silt fences and straw waddles around the site perimeter prior to starting mass grading.
- In areas where grading is complete, hydroseed or straw mulch should be placed.
- During the wet season, or when large storm events are predicted during the summer months, work areas should be stabilized so that if showers occur, the work area can receive the rainfall without excessive erosion or sediment transport. Areas that are to be left un-worked for more than two days should be covered with straw mulch or plastic sheeting.
- Soils that are to be stockpiled on-site should be covered with plastic sheeting staked and sandbagged in place.

The erosion control measures should be reviewed, adjusted and maintain on a regular basis to verify they are functioning as intended.

8.0 ADDITIONAL SERVICES

To confirm that our recommendations are properly incorporated into the design and construction of the proposed development, PanGEO should be retained to conduct a review of the final project plans and specifications, and to monitor the construction of geotechnical elements. PanGEO can provide you a cost estimate for construction monitoring services at a later date.

9.0 LIMITATIONS

We have prepared this report for use by Erika Mobley and the project team. Conclusions and 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 work.

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 work specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances.

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

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

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use this report.

We appreciate the opportunity to be of service.

Sincerely,

SIEW LAN TAN
30342
September 28, 2023

Shawn M. Harrington, G.I.T.
Project Geologist
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Siew L. Tan, P.E.
Principal Geotechnical Engineer
STan@pangeoinc.com

10.0 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.

City of Seattle, 2023, *Standard Specifications for Road, Bridges, and Municipal Construction*.

International Building Code (IBC), 2018 and 2021, *International Code Council.*

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

Washington Administrative Code (WAC), 2019, Chapter 296-155 - *Safety Standards for Construction Work, Part N - Excavation, Trenching, and Shoring*, Olympia, Washington.

WSDOT, 2023, *Standard Specifications for Road, Bridge and Municipal Construction, M 41-10, Washington State Department of Transportation.*

Base Map: King County iMap (Esri Topographic)

Not to Scale

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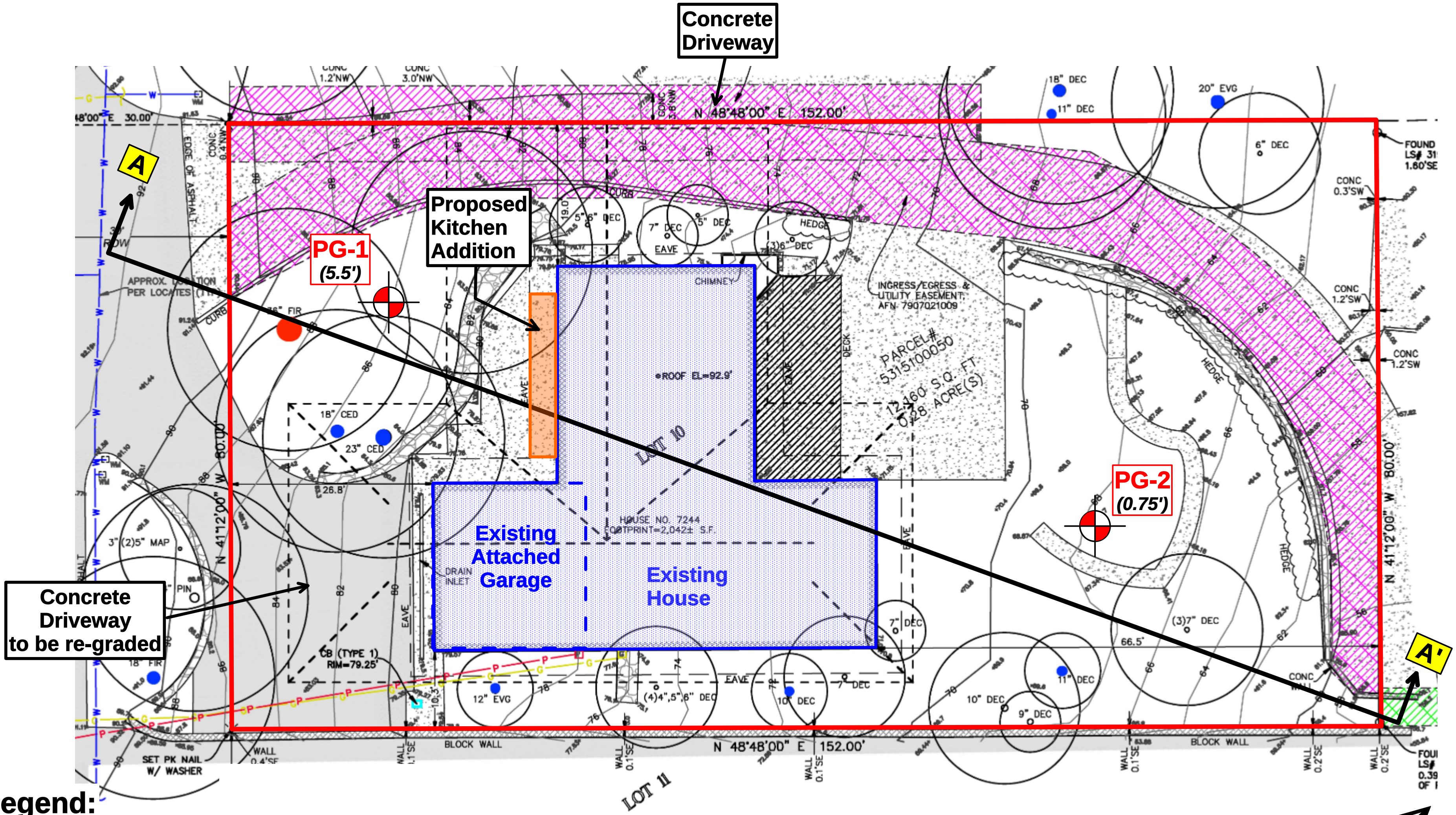
Proposed Additions and Alterations
7244 North Mercer Way
Mercer Island, WA

VICINITY MAP

Project No.	23-244	Figure No.	1



Legend:

- 🟥 Property Boundary
- 🟦 Footprint of Existing Structures
- 🟧 Footprint of Proposed Additions/Alterations
- ⊕ **PG-1** *(5.5')* Approx. Test Boring Location *(Depth to native soil in ft)*
- ↑↑ Study Cross-Section *see Figure 3 for subsurface profile*

Note:
Base map modified from boundary and topographic survey of the site prepared by Terrane, dated March 26, 2021. Elevations based on the NAVD88 Datum.

Approx. Scale
1" = 20'

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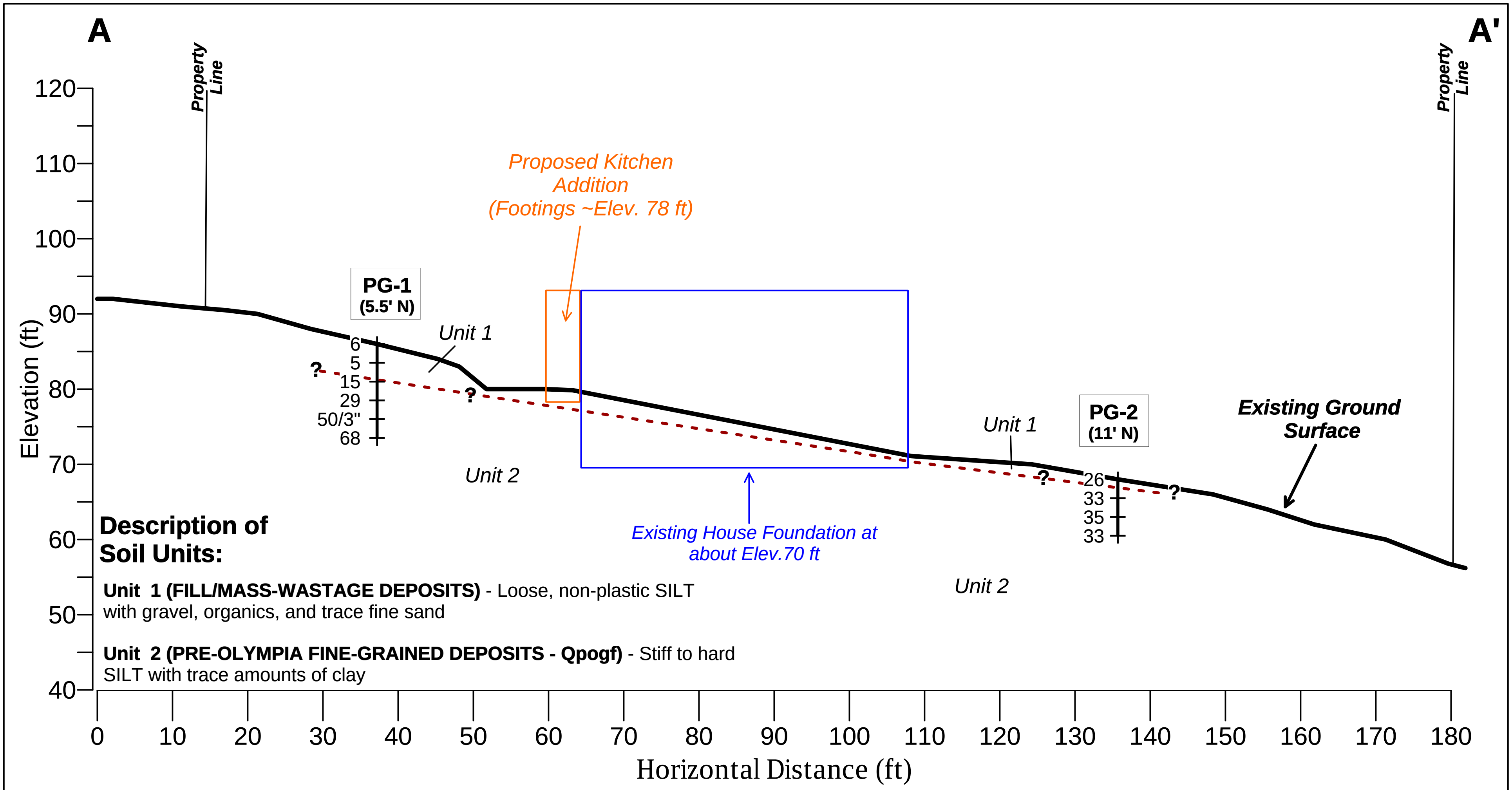
Proposed Additions and Alterations
7244 North Mercer Way
Mercer Island, WA

SITE AND EXPLORATION PLAN

Project No. **23-244**

Figure No. **2**

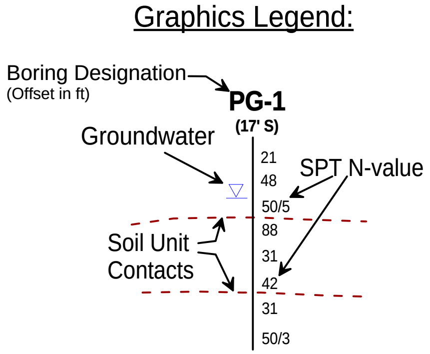


Description of Soil Units:

Unit 1 (FILL/MASS-WASTAGE DEPOSITS) - Loose, non-plastic SILT with gravel, organics, and trace fine sand

Unit 2 (PRE-OLYMPIA FINE-GRAINED DEPOSITS - Qpogf) - Stiff to hard SILT with trace amounts of clay

Graphics Legend:



Notes:
1. Ground profile based on *Topographic & Boundary Survey* by Terrane dated March 26, 2021.
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.

Approx. Scale
1" = 20' (H)
1" = 20' (V)
No vertical exaggeration

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Proposed Additions and Alterations
7244 North Mercer Way
Mercer Island, WA

GENERALIZED SUBSURFACE PROFILE
SECTION A-A'

Project No.	**23-244**	Figure No.	**3**

APPENDIX A

SUMMARY BORING LOGS

RELATIVE DENSITY / CONSISTENCY

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

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

DESCRIPTIONS OF SOIL STRUCTURES

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

COMPONENT DEFINITIONS

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

TEST SYMBOLS

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

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

SYMBOLS

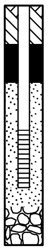
Sample/In Situ test types and intervals

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

MONITORING WELL

∇ Groundwater Level at time of drilling (ATD)

▼ Static Groundwater Level



Cement / Concrete Seal

Bentonite grout / seal

Silica sand backfill

Slotted tip

Slough

Bottom of Boring

MOISTURE CONTENT

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

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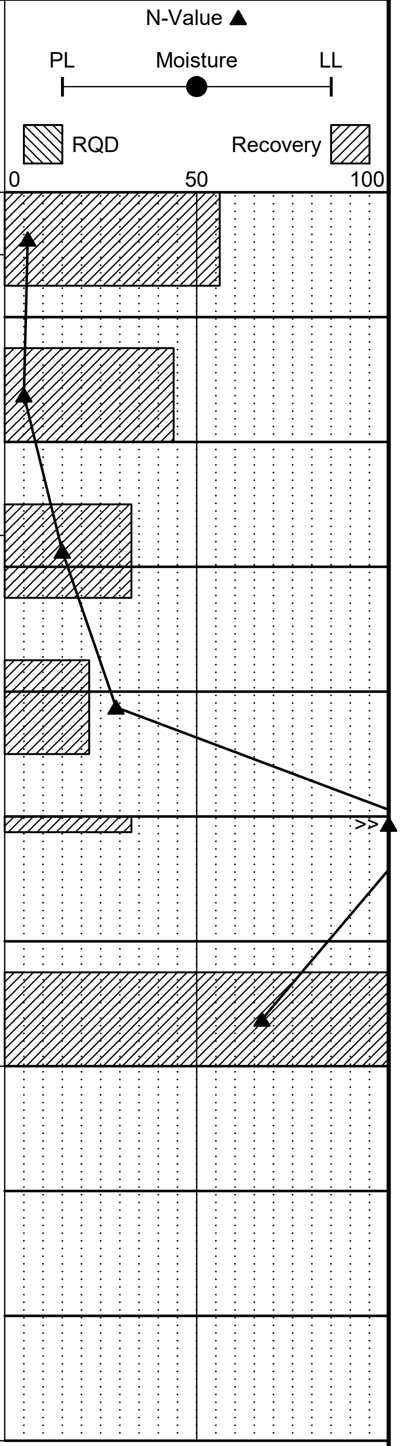
Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project: Proposed Additions and Alterations
Job Number: 23-244
Location: 7244 North Mercer Way, Mercer Island, Washington
Coordinates: Northing: 47.59319, Easting: -122.24166

Surface Elevation: ~86 ft
Top of Casing Elev.: n/a
Drilling Method: Hollow Stem Auger
Sampling Method: SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0	S-1	☒	1 1 5			Approximately 12 inches of topsoil (dark brown silty sand with organics).
						FILL/MASS-WASTAGE DEPOSITS Loose, gray-brown, non-plastic SILT with gravel, trace fine sand; moist, occasional roots.
2						
	S-2	☒	2 2 3			
4						-- Approximately 5-inch-thick root encountered at about 5 feet below grade. Driller added water to assist drilling.
6	S-3	☒	12 7 8			**PRE-OLYMPIA FINE-GRAINED GLACIAL DEPOSITS [Qpogf]** Stiff to very stiff, gray-brown, low-plasticity SILT, trace gravel and fine sand; moist.
8	S-4	☒	11 11 18			-- Becomes hard.*Poor recovery; soil description based in part on drilling action and soil cuttings.*
10	S-5	☒	50/3			*Poor recovery due to large gravel encountered. SPT N-Vaules overstated due to gravel.*
12						
	S-6	☒	21 33 34			-- Decrease in gravel content; thinly-bedded soil structure.
14						Boring terminated about 14 feet below grade. Groundwater was not encountered at the time of drilling.
16						
18						
20						



Completion Depth: 14.0ft
Date Borehole Started: 8/22/23
Date Borehole Completed: 8/22/23
Logged By: S. Harrington
Drilling Company: CN Drilling

Remarks: Boring drilled using a limited-access Acker drill rig. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. This surface elevation is estimated from topographic and bundary survey prepared by Terrane, dated March 26, 2021.**Vertical Datum: NAVD 88.**

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LOG OF TEST BORING PG-1

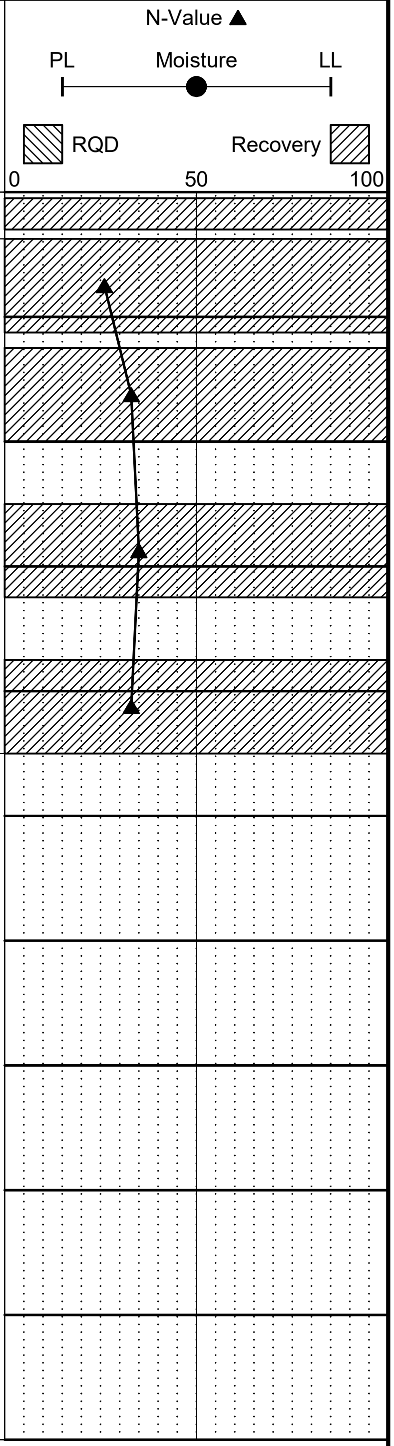
Figure A-2

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

Project: Proposed Additions and Alterations
Job Number: 23-244
Location: 7244 North Mercer Way, Mercer Island, Washington
Coordinates: Northing: 47.59332, Easting: -122.24126

Surface Elevation: ~68 ft
Top of Casing Elev.: n/a
Drilling Method: Hollow Stem Auger
Sampling Method: SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0	S-1					Bark chips over approximately 9 inches of topsoil (dark brown silty sand with organics).
2	S-2		9 12 14			**PRE-OLYMPIA FINE-GRAINED GLACIAL DEPOSITS [Qpogf]** Very stiff, brown, low- to medium-plasticty SILT, trace gravel; moist, massive, minor iron-oxide staining.
4	S-3		11 15 18			-- Becomes hard.
6	S-4		10 16 19			-- Trace clay; medium-plasticity; massive to thinly-bedded soil structure.
8	S-5		11 15 18			
10						Boring terminated about 9 feet below grade. Groundwater was not encountered at the time of drilling.
12						
14						
16						
18						
20						



Completion Depth: 9.0ft
Date Borehole Started: 8/22/23
Date Borehole Completed: 8/22/23
Logged By: S. Harrington
Drilling Company: CN Drilling

Remarks: Boring drilled using a limited-access Acker drill rig. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. This surface elevation is estimated from topographic and bundary survey prepared by Terrane, dated March 26, 2021. **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.