PanGEO
INCORPORATED
Geotechnical & Earthquake
Engineering Consultants

July 13, 2022
Project No. 22-241

Mawer Brothers LLC
Attn: Mike Burke

Subject: Geotechnical Report
Proposed Residence:
6024 Southeast 22nd Street, Mercer Island, Washington

Dear Mr. Burke:

Please find attached our geotechnical report for the proposed residence at 6024 Southeast 22nd Steet, Mercer Island, Washington. This report documents the subsurface conditions at the site and our geotechnical engineering recommendations for the proposed project.

In summary, our test borings encountered up to 10 feet of gravel fill and native, loose sand and medium stiff silt, which we do not consider suitable for supporting the proposed residence, over component native, very stiff to hard silt and clay. Over-excavation to remove 10 feet of unsuitable soils does not appear practical. Hence, we recommend that a deep foundation system consisting of driven, small diameter pipe piles (often referred to as pin piles) be utilized to support the proposed residences.

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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Tel (206) 262-0370

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GEOTECHNICAL REPORT
PROPOSED RESIDENCE
6024 SOUTHEAST 22ND STREET
MERCER ISLAND, WASHINGTON

1.0 INTRODUCTION

As requested, PanGEO, Inc. is pleased to present this geotechnical report to assist the project team with the design and construction of the proposed residence at 6024 Southeast 22nd Street, Mercer Island, Washington. This study was performed in general accordance with our mutually agreed scope of services outlined in our proposal dated April 19, 2022, and was subsequently approved by you on May 16, 2022. Our scope of services included reviewing readily available geologic and geotechnical data, conducting a site reconnaissance, observing drilling of three 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 located at 6024 Southeast 22nd Street in Mercer Island, Washington, approximately as shown on the attached Figure 1, Vicinity Map. The roughly rectangular shaped site comprises about 1.31 acres (57, 175 square feet) and is bordered to the south by Southeast 22nd Street, to the east and west by existing residences, and to the north by Lake Washington. In the central portion of the site is a two-story single-family residence with a daylight basement and detached garage that was constructed in 1954. In the south portion of the site is a detached accessory dwelling unit (DADU). In the north portion of the site near the Lake Washington shoreline is a detached cabana. The partial layout of the site is shown on the attached Figure 2, Site and Exploration Plan.

Based on review of the project topographic survey and our observations while on site, the site and surrounding area generally slopes down from south to north with an average gradient of 4 to 5 percent and about 26 feet of elevation change across the length of the site. A low rockery is located north of the existing residence and provides a few feet of grade separation between the higher eastern portion of the backyard and the lower western portion of the backyard. The site is vegetated with mature Douglas fir, cedar and alder trees, lawns, ivy and landscaping trees and shrubs. Current site conditions are shown on Plates 1 and 2 on the following page.

Based on review of the City of Mercer Island's Geologic Hazards Map, the subject site is located in a potential landslide hazard area and a seismic hazard area. However, the site is not mapped within a potential erosion hazard area.

Plate 1. *View of the south side of the existing residence at the site, looking north from near existing DADU.*

Plate 2. *Looking south towards existing residence and location of proposed new residence from near the existing cabana.*

We understand it is planned to demolish the existing structures and construct a new residence with an attached garage at the site. The approximate footprint of the proposed structure is indicated on Figure 2. Based on review of the current design plans (dated June 1, 2022) provided to PanGEO by your office, we understand that the proposed residence will have one level of basement under the northern portion of the building (see Plate 3 below). We also understand that the main floor will have a finished floor elevation of 28 feet and the basement will have a slab elevation of 19.95 feet. As such, we anticipate that temporary excavations for the foundation and basement construction will be about 5 to 6 feet below existing grades, with the deepest excavations located towards the south end of the building where existing site grades are highest.

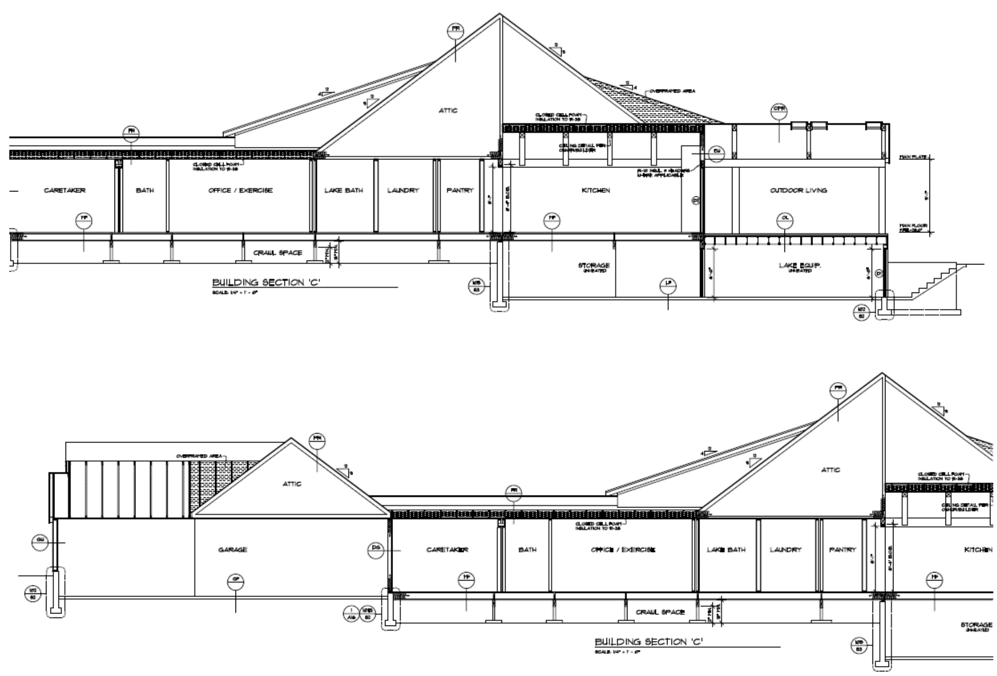


***Plate 3**. North-South sections through proposed residence from architect (Matthew Mawer Residential Design), looking west. Main floor is at Elevation 28 feet, with basement slab at Elevation 19.95 feet.*

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

Three test borings (PG-1 to PG-3) were advanced at the site on June 27, 2022 using a track mounted limited access drill rig owned and operated by Geologic Drill Partners of Fall City, Washington. Borings PG-1 and PG-3 were drilled to maximum depths of about 16½ feet below existing grades, while PG-2 was only drilled to a depth of 4½ feet below existing grades due to refusal of drilling equipment (see discussion in Section 4.2 below). The approximate boring locations were determined relative to existing features and are shown on the attached Figure 2.

The drill rig was equipped with 4-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 through A-4. 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 surficial geologic units mapped at the site include Lake Deposits (Geologic Map Unit Ql) and Vashon till (Qvt). Pre-Olympia nonglacial deposits (Qpon) are mapped about one block to the east of the site.

Lake deposits typically consist of very soft to medium stiff or very loose to medium dense silt and clay with local sand layers deposited adjacent to Lake Washington. 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 nonglacial deposits consist of sand, gravel, silt, clay and organic deposits of inferred nonglacial origin. This deposit is characterized by the presence of organics and clasts comprised of rock types that originated from local sources. Pre-Olympia nonglacial deposits are frequently present near lake level.

4.2 Soil Conditions

In general, our test borings encountered a surficial layer of fill underlain by loose sand and medium stiff silt overlying very stiff to hard silt and clay. The soils appeared consistent with the mapped geology in the vicinity of the site. A brief description of the generalized soil units encountered in our borings is presented below. Please refer to our boring logs located in Appendix A for more details.

Topsoil – At all three of our boring locations, we encountered a surficial layer of topsoil. The topsoil ranged from 3 to 4 inches thick and consisted of dark brown silt with organics.

Fill – Underlying the topsoil, all of our borings encountered a layer of loose to medium dense, silty, sandy gravel, which we interpret as undocumented fill most likely placed during construction of the exiting residence. The drilling through the gravel was difficult, and PG-2 met refusal in this layer at 4½ feet below grade after attempting to relocate the borehole several times. The fill extended to 5 feet and 7 feet below existing grade in test borings PG-1 and PG-3, respectively. It should be noted that SPT N-values are frequently overstated while sampling in gravel. As such, we do not consider this soil unit to be adequate for supporting the proposed residence.

Lake Deposits (Ql) – Below the fill in borings PG-1 and PG-3, we encountered loose sand with sandy silt interbeds that extended to about 7 feet below existing grades in PG-1 and to about 10 feet below existing grades in PG-3. The encountered soil appeared consistent with the Lake deposit mapped at the site. This soil unit is not considered adequate for supporting the proposed residence.

Pre-Olympia non-glacial deposits (**Qpon**) – Below the Lake deposits, borings PG-1 and PG-3 encountered a few feet of very stiff silty clay overlying hard clayey silt that extended to the maximum drilled depth of 16½ feet below existing grades in both boreholes. We interpret this soil as the Pre-Olympia non-glacial deposits mapped in the vicinity of the site. This soil unit is considered adequate for supporting the proposed residence.

4.3 Groundwater Conditions

Groundwater was not observed within the maximum depths of our test borings at the time of drilling. As such, we do not anticipate that groundwater will result in significant construction related issues. However, 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 mapped as being within a potential landslide hazard area. However, to the best of our knowledge, there are no documented past know slides at the subject site or immediate vicinity. Additionally, as part of this evaluation, we conducted a site reconnaissance of the subject property on May 24, 2022. During our site reconnaissance, we did not observe obvious evidence of slope instability at the site, such as uneven topography, slumps, or tension cracks. The existing building foundations were observed to be in fair condition.

Based on our field observations, the general level 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 planned 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 very stiff to hard silt and clay underlying the site 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 gentle topography, and the presence of competent soils (very stiff to hard silt and clay and very dense silty sand soils) at shallow depths.

5.3 Erosion Hazards

The subject site is not mapped within a potential erosion hazard area according to the City of Mercer Island's Geologic Hazards Map. However, in our opinion, based on soil conditions encountered in the borings, the near-surface site soils are likely to exhibit moderate 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 7.5 of this report.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 Seismic Site Class

The seismic design should be performed using the 2018 edition of the International Building Code (IBC), which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years). Based on the measured SPT in our test borings and the site geology, it is our opinion that Site Class C should be used.

6.2 Foundation Design – Driven Pipe Piles (Pin Piles)

Based on the results of our subsurface explorations conducted at the site, the location of the new residence is underlain by up to about 10 feet of fill soil and loose to medium stiff lake deposits which are not considered competent for foundation bearing. A deep foundation system consisting of driven, small diameter pipe piles (often referred to as pin piles) appears to the most appropriate foundation option. Not only will the piles provide a high level of foundation performance, but they will avoid construction challenges associated with over-excavations of as much as 10 feet of weak, unsuitable soils.

In our opinion small diameter driven steel pipe piles (pin piles) represent a feasible foundation option in terms of cost and long-term performance. Small diameter pin piles are utilized to transfer the structure loads through the marginal soils to the underlying pre-Fraser deposits. Pin piles of 2- to 4- inches in diameter are typically utilized for this purpose. However, larger diameter 6- and 8-

inch piles may also be used, which have a higher vertical capacity. 2-inch pin piles are typically installed using portable, handheld equipment and are suited for areas where limited site access exists, or in low headroom areas (i.e., inside a basement). 3-inch to 8-inch pin piles are typically installed using small to large hammers (600 to 4,700 lbs.) mounted on small to medium-sized excavators.

6.2.1 Pin Pile Sizes

Based on our understanding of the project, in our opinion 3, 4, or 6-inch diameter piles will likely be most suitable to support the proposed residences.

6.2.2 Pin Pile Capacity

The number of piles required depends on the magnitude of the design load. An allowable axial compression capacity of 3 tons (6 kips) may be used per 2-inch diameter pile, 6 tons (12 kips) per 3-inch diameter pile, 10 tons (20 kips) per 4-inch diameter piles, and 15 tons (30 kips) for 6-inch diameter piles, with an approximate factor of safety of at least 2.0.

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.

6.2.3 Pin Pile Specifications

We recommend that the following specifications be included on the foundation plan:

- 2-inch diameter piles should consist of Schedule-80, ASTM A-53 Grade “A” pipe.
- 3-inch, 4-inch, and 6-inch diameter piles should consist of Schedule-40, ASTM A-53 Grade “A” pipe.
- 2-inch piles shall be driven to refusal with a minimum 90-lb jackhammer. Refusal is defined as no more than 1 inch of penetration for 1 minute of continuous driving.
- 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

The driving criteria recommended in the table above will be verified by a static load test program (see discussion in Item 8).

- 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

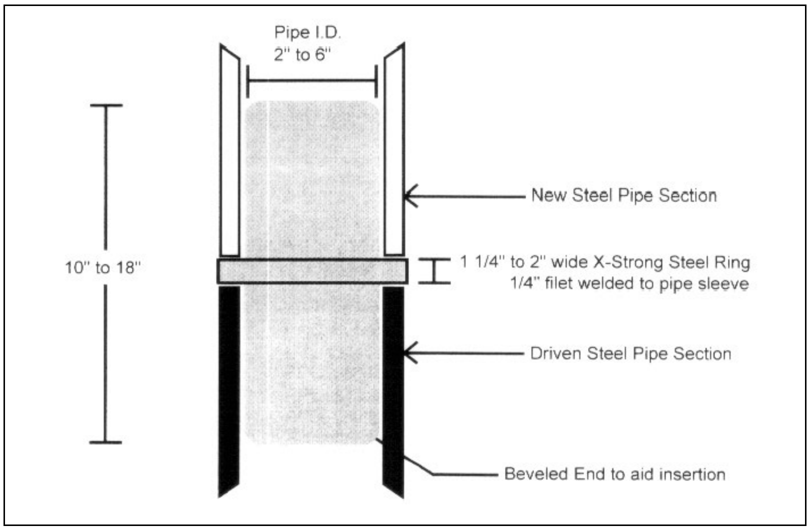
The driving criteria recommended in the table above will be verified by a static load test program (see discussion in Item 8).

- 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

The driving criteria recommended in the table above will be verified by a static load test program (see discussion in Item 8).

- 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.



- At least 3% (but no more than 5) of the 3-inch, 4-inch, and 6-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.
- As required by the City of Seattle DCI, the geotechnical engineer of record or his/her representative shall provide full time observation of pile installation and testing.

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.

6.2.4 Lateral Resistance

Lateral capacity of vertical pin piles less than 6-inches in diameter should be ignored in design calculations. Some resistance to lateral loads may be accomplished by battering the piles to a slope of 1(H):4(V), or steeper. In addition, lateral forces from wind or seismic loading may be resisted by tying the grade beams and pile caps to the soldier pile shoring wall, if it is designed as a permanent wall. Passive soil resistance values for embedded pile caps and grade beams may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf). This value includes a factor of safety of at least 1.5 assuming that a properly compacted structural fill will be placed adjacent to the sides of the pile caps and grade beams. For the seismic condition, the recommended passive pressure may be increased by one third.

6.2.5 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 and clay is an appropriate estimate. As such, we estimate that average pile lengths of about 15 to 18 feet will be needed. We recommend a minimum pile length of 10 feet.

6.2.6 Obstructions

Obstructions may be encountered within the upper fill or disturbed soils. Where possible, the obstructions should be removed to facilitate the 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.

6.3 RETAINING AND BASEMENT WALL DESIGN PARAMETERS

Retaining and basement walls should be properly designed to resist the pressure exerted by the soils behind the walls. Proper drainage provisions should also be provided behind the walls to intercept and remove any groundwater from behind the wall. Our geotechnical recommendations for the design and construction of the basement walls are presented below.

6.3.1 Lateral Earth Pressures

Cantilever walls should be designed for an equivalent fluid pressure of 40 pcf for a level backfill condition and assuming the walls are free to rotate. If the walls are restrained at the top from free movement, such as basement walls with a floor diaphragm, an equivalent fluid pressure of 50 pcf should be used for a level backfill condition behind the walls. Permanent walls should be designed for an additional uniform lateral pressure of 9H psf for seismic loading, where H corresponds to the height of the buried depth of the wall. The recommended lateral pressures assume that adequate wall drainage will be incorporated into the design and construction of the walls to prevent the development of hydrostatic pressure.

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 Lateral Resistance

Lateral forces from wind or seismic loading and unbalanced lateral earth pressures may be resisted by the lateral component of battered piles, the connection to a permanent soldier pile wall, or by the passive earth pressures acting against the embedded portions of the foundations. Passive resistance values may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf). This value includes a geotechnical factor of safety of at least 1.5 assuming that properly compacted structural fill will be placed adjacent to the sides of the footings. For transient loads such as seismic and wind loads, it is our opinion that it is appropriate to increase the design resistance values by one third.

6.3.4 Wall Drainage

Provisions for permanent control of subsurface water should be incorporated into the design and construction of basement and site retaining walls. For walls constructed with conventional free-draining backfill, a footing drain consisting of a 4-inch diameter perforated pipe embedded in at least 12 inches of washed gravel wrapped with a geotextile fabric should be placed at the base of the wall footings. We recommend that prefabricated drainage mats, such as Mirafi 6000 or equivalent, be installed behind the basement walls to promote wall drainage. The drainpipe at the base of the wall should be graded to direct water to a suitable 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 2020 Seattle Standard Specifications) or Gravel Borrow (Section 9.03.14 (1) of the 2022 WSDOT Standard Specifications) should be assumed for this project. Wall backfill should consist of free draining granular soils. It is our opinion that on-site soils are too fine-grained and should not be used as wall backfill. Imported wall backfill should consist of granular soils such as Seattle Mineral Aggregate Type 17, WSDOT Gravel Borrow, or approved equivalent.

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

6.4 Concrete Slab on Grade

Slab on Grade **–** A slab on grade may be used for the basement floor of the proposed residence, however, due to the disturbed soils that will likely be present below the slab elevation, some settlement of the floor slab, and associated distress, may potentially occur. To reduce the potential of slab settlement and distress, we recommend removing a minimum of 1-foot of existing soil below the slab, and placing 1-foot of properly compacted free-draining granular structural fill to create a firm bearing surface for the slab. If soft, wet subgrade conditions are present, we recommend placing a geotextile fabric over the exposed subgrade prior to placing the structural fill. For the subgrade improvement described above, and for slab areas bearing on native undisturbed soils, the floor slab design may be accomplished using a modulus of subgrade reaction of 100 pci.

Structural Slab **–** To achieve a high level of slab performance, a structural slab could be designed to span between the pile supported foundations. If a structural slab is utilized, the existing disturbed soils below the slab may be left in place without re-compaction or replacement.

Capillary Break **–** We recommend that the slabs be constructed on a minimum 4-inch thick capillary break. The capillary break should consist of free-draining, clean crushed rock or well-graded gravel compacted to a firm and unyielding condition. The capillary break material should 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. We also recommend that a 10-mil polyethylene vapor barrier be placed below the slab. Construction joints should be incorporated into the floor slab to control cracking.

6.5 On-Site Infiltration Considerations

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

6.6 Permanent Slope Inclinations

Permanent cut and fill slopes should be inclined no steeper than 2H:1V. Cut slopes should be observed by PanGEO during excavation to verify that conditions are as anticipated. Permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve stability of the surficial layer of soil.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Temporary Excavations

Based on our understanding of the project, we anticipate that temporary excavations for the foundation construction will be on the order of about 5 to 6 feet. We anticipate the excavations to mainly encounter gravel fill over loose/medium stiff sand and silt (Lake deposits) over very stiff clay (Pre-Olympia nonglacial deposits). Based on our observations of the site soil conditions, it is our opinion that unsupported slope cuts 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). Based on the current design, it is our opinion that sufficient space is available for unsupported open cuts. In the event that sufficient space is not available for unsupported open cuts and temporary shoring is needed, PanGEO can provide temporary shoring design recommendations if requested. 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.

7.2 Structural Fill and Compaction

The on-site soils should not be re-used as structural fill or wall backfill for the project. Structural fill should consist of a well-graded granular material having a maximum grain size of six inches and no more than 5 percent fines passing the US No. 200 sieve based on the minus ¾-inch fraction.

Structural fill should be placed in 8- to 12-inch-thick loose lifts and compacted. If the fill will be tested for compaction, the fill should be compacted to at least 95 percent maximum dry density, per ASTM D-1557 (Modified Proctor). In non-structural areas, the recommended compaction level may be reduced to 90 percent of the Modified Proctor.

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

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

7.3 Material Reuse

The native soils underlying the site are moisture sensitive will 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.

7.4 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.

7.5 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 Mawer Brothers LLC 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
STATE OF WASHINGTON
30342
REGISTERED
PROFESSIONAL ENGINEER
July 13, 2022

Shawn M. Harrington, G.I.T.
Project Geologist
SHarrington@pangeoinc.com

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.

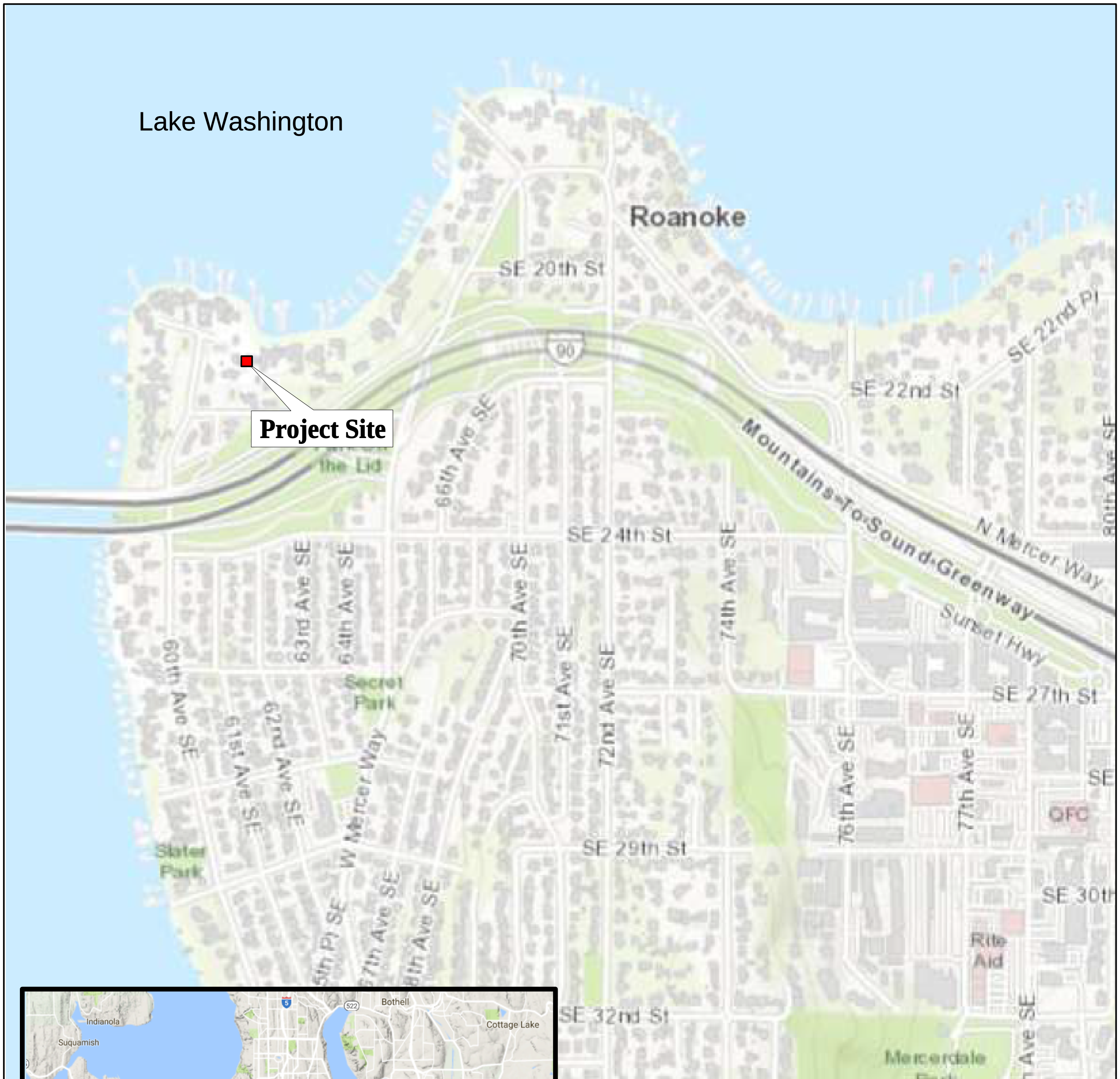
City of Seattle, 2020, *Standard Specifications for Road, Bridges, and Municipal Construction*.

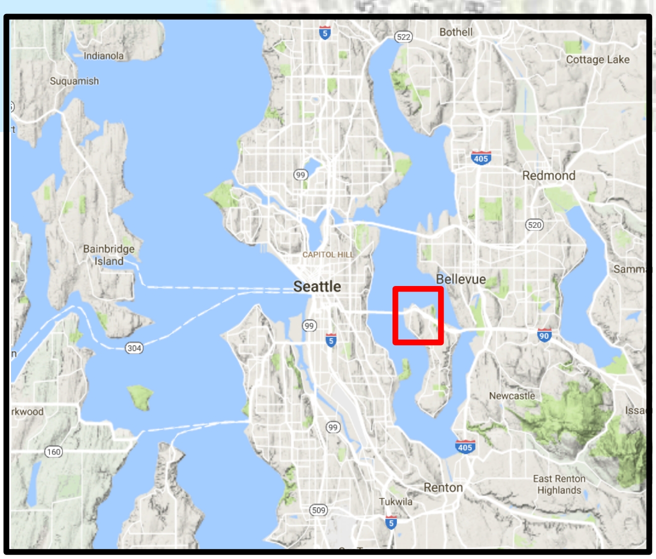
International Building Code (IBC), 2018, *International Code Council.*

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, 2022, *Standard Specifications for Road, Bridge and Municipal Construction, M 41-10, Washington State Department of Transportation.*





Base Map: King County iMap (Esri Topographic)

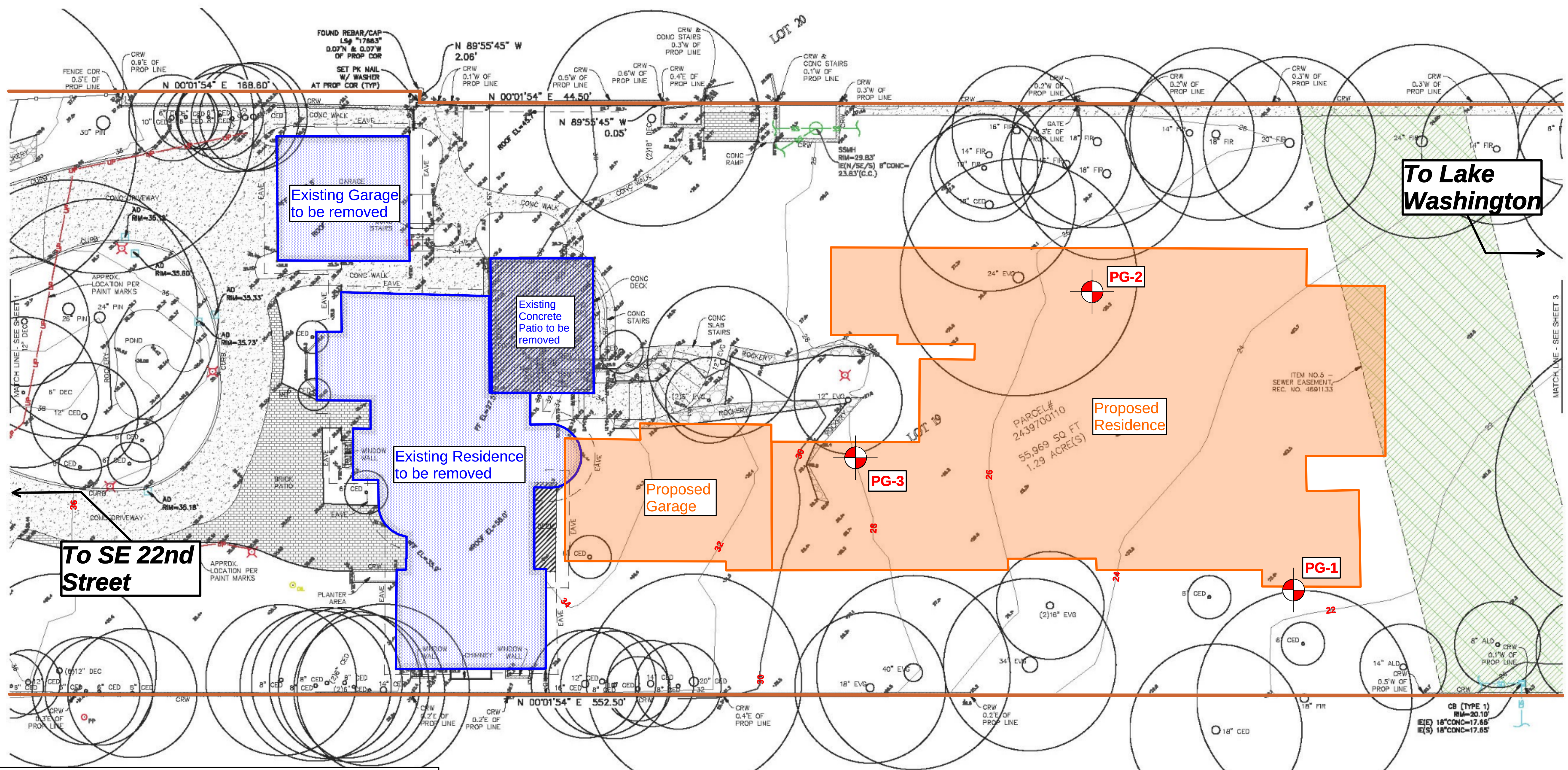
Not to Scale

PanGEO
INCORPORATED

Proposed Residence
6024 SE 22nd Street
Mercer Island, WA

VICINITY MAP

Project No.	22-241	Figure No.	1



Legend:

- Property Boundary
- Existing Structures
- Proposed Residence & Garage
- PG-# Approx. PanGEO Soil Boring Location

NORTH

Approx. Scale
1" = 20'

Base map modified from Topographic & Boundary survey prepared by Terrane, dated 10/09/2020.
Vertical Datum: NAVD 88.

PanGEO INCORPORATED	Proposed Residence 6024 SE 22nd Street Mercer Island, WA	SITE AND EXPLORATION PLAN
		Project No. 22-241 / Figure No. 2

APPENDIX A

SUMMARY BORING LOGS

RELATIVE DENSITY / CONSISTENCY

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

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

DESCRIPTIONS OF SOIL STRUCTURES

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

COMPONENT DEFINITIONS

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

TEST SYMBOLS

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

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

SYMBOLS

Sample/In Situ test types and intervals

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

MONITORING WELL

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

MOISTURE CONTENT

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

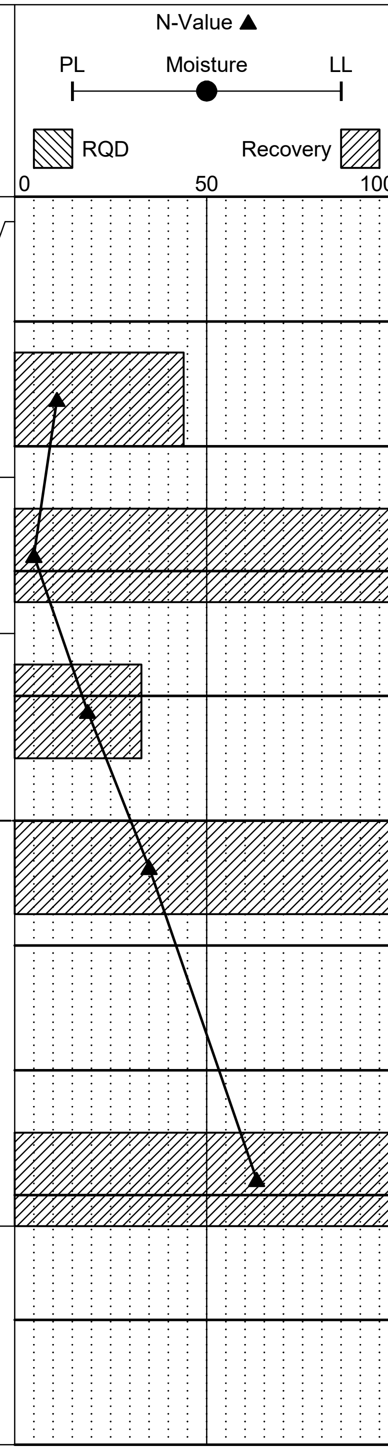
PanGEO INCORPORATED
Phone: 206.262.0370

Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	Proposed Residence	Surface Elevation:	~22 ft
Job Number:	22-241	Top of Casing Elev.:	n/a
Location:	6024 SE 22nd St, Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.59227, Easting: -122.25109	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						Grass sod over approximately 4 inches of topsoil (dark brown silty sand with organics).
						[FILL] Loose to medium dense, brown, silty, sandy GRAVEL; moist, gravel well-graded. *Hard drilling conditions in gravel from 1.5 to 3 feet below grade. Driller added water to assist drilling.*
2.5–4	S-1		20, 6, 5			
						[LAKE DEPOSITS - Ql] Loose, dark brown to gray, silty SAND to sandy SILT, occasional root; moist, silt non to low plasticity, thin organic layer in sample at about 5 feet below grade.
5–6.5	S-2		3, 2, 3			
						[PRE-OLYMPIA NONGLACIAL DEPOSITS - Qpon] Very stiff, blue-gray, silty CLAY; moist, moderate plasticity, slightly mottled.
7.5–9	S-3		6, 8, 11			
10						Hard, brown to gray-brown, clayey SILT, trace gravel; moist, moderate plasticity, massive.
10–11.5	S-4		13, 15, 20			
15–16.5	S-5		20, 25, 38			-- Occasional thin sand lenses.
16.5						Boring terminated about 16.5 feet below grade. Groundwater was not encountered at time of drilling. SPT N-Value in gravel fill at 2.5 feet below grade possibly overstated.
20						



		Remarks
Completion Depth:	16.5ft	Remarks: Borings drilled using small tracked drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Surface elevation estimated based on Topographic & Boundary Survey by Terrane, dated 10/09/20. Vertical Datum: NAVD 88.
Date Borehole Started:	6/27/22	
Date Borehole Completed:	6/27/22	
Logged By:	S. Harrington	
Drilling Company:	Geologic Drill Partners	

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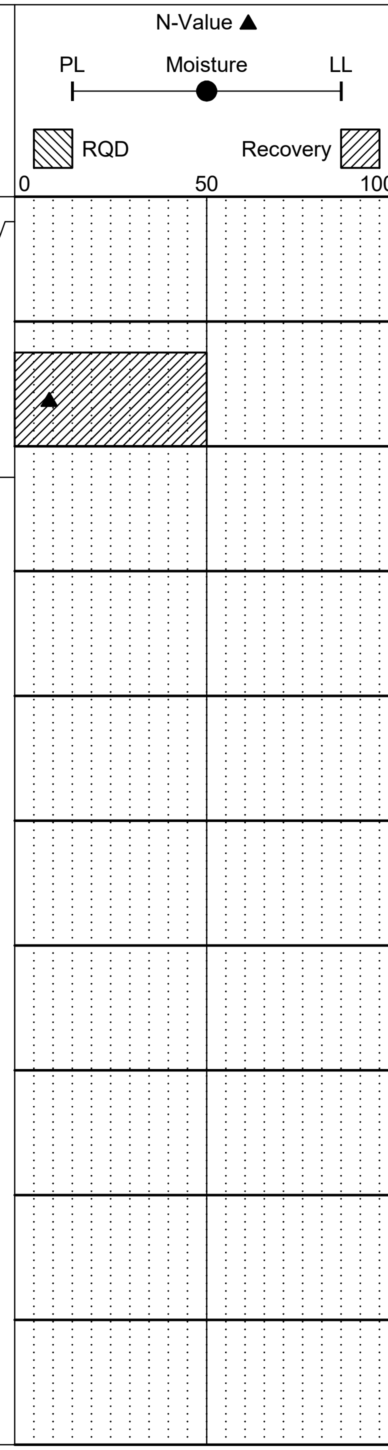
LOG OF TEST BORING PG-1

Figure A-2

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

Project:	Proposed Residence	Surface Elevation:	~25 ft
Job Number:	22-241	Top of Casing Elev.:	n/a
Location:	6024 SE 22nd St, Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.59216, Easting: -122.25129	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						Grass sod over approximately 3 inches of topsoil (dark brown silty sand with organics).
						[FILL] Loose to medium dense, brown, silty, sandy GRAVEL; moist, gravel well-graded.
2						*Hard drilling conditions in gravel from 2 to 4.5 feet below grade. Driller added water to assist drilling.*
	S-1		13, 5, 4			
4						
						Boring reached refusal in gravelly drilling conditions about 4.5 feet below grade. Groundwater was not encountered at time of drilling. SPT N-Value in gravel fill at 2.5 feet below grade possibly overstated.
6						
8						
10						
12						
14						
16						
18						
20						



Completion Depth:	4.5ft
Date Borehole Started:	6/27/22
Date Borehole Completed:	6/27/22
Logged By:	S. Harrington
Drilling Company:	Geologic Drill Partners

Remarks: Borings drilled using small tracked drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Surface elevation estimated based on Topographic & Boundary Survey by Terrane, dated 10/09/20. Vertical Datum: NAVD 88.

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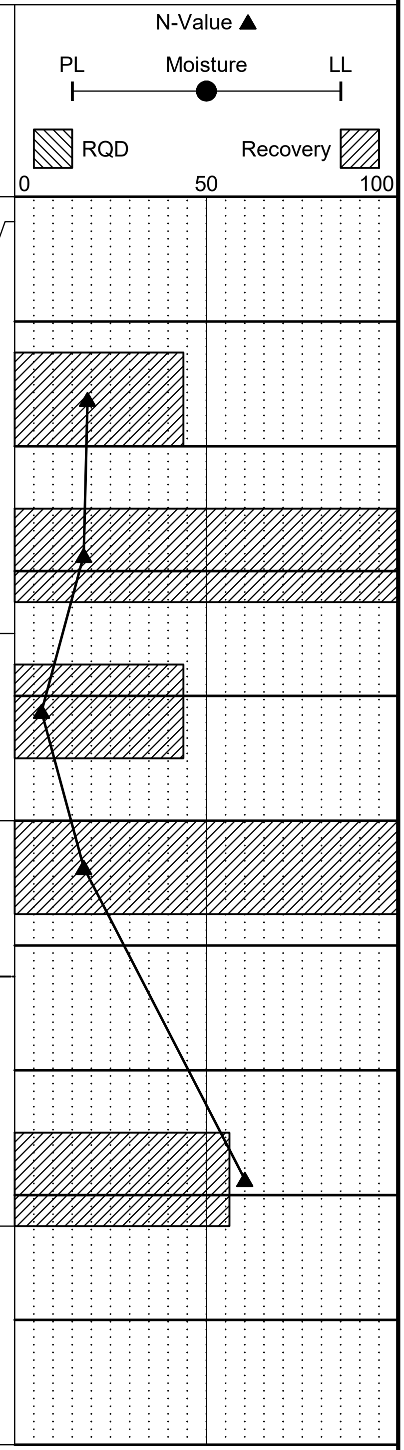
LOG OF TEST BORING PG-2

Figure A-3

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

Project:	Proposed Residence	Surface Elevation:	~28 ft
Job Number:	22-241	Top of Casing Elev.:	n/a
Location:	6024 SE 22nd St, Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.59204, Easting: -122.25117	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						Grass sod over approximately 4 inches of topsoil (dark brown silty sand with organics).
						[FILL] Loose to medium dense, brown, silty, sandy GRAVEL; moist, gravel well-graded.
2.5–4	S-1		18, 11, 8			*Hard drilling conditions in gravel from 2.5 to 7 feet below grade. Driller added water to assist drilling.*
5–6.5	S-2		13, 11, 7			
7						**[LAKE DEPOSITS - Ql]** Loose/medium stiff, brown, silty SAND and SILT interbeds; moist to very moist, low plasticity fines.
7.5–9	S-3		6, 4, 3			
10						**[PRE-OLYMPIA NONGLACIAL DEPOSITS - Qpon]** Very stiff, blue-gray, silty CLAY; moist, moderate plasticity, highly mottled.
10–11.5	S-4		6, 8, 10			
12.5						Hard, brown to gray-brown, clayey SILT, trace gravel; moist, moderate plasticity, massive.
15–16.5	S-5		18, 20, 40			
16.5						Boring terminated about 16.5 feet below grade. Groundwater was not encountered at time of drilling. SPT N-Value in gravel fill at 2.5 and 5 feet below grade possibly overstated.



Completion Depth:	16.5ft
Date Borehole Started:	6/27/22
Date Borehole Completed:	6/27/22
Logged By:	S. Harrington
Drilling Company:	Geologic Drill Partners

Remarks: Borings drilled using small tracked drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Surface elevation estimated based on Topographic & Boundary Survey by Terrane, dated 10/09/20. 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.

PanGEO
INCORPORATED
Geotechnical & Earthquake
Engineering Consultants

November 29, 2022
File No. 21-428

Mawer Brothers LLC
PO Box 52921
Bellevue, WA 98015
Attn: Matt Burke

Subject: Geotechnical Plan Review and Risk Statement
Proposed Residence:
6024 Southeast 22nd Street, Mercer Island, Washington

Dear Mr. Burke,

As requested, we completed a geotechnical review of the project plans provided by your office. The plans we reviewed included structural plans prepared by MDT Engineering dated 8/30/2022, architectural plans including pin pile layout (sheets P1 and P2) prepared by NW Lifestyle Homes dated 10/17/2022, and Civil plans prepared by BCRA dated 8/5/2022. Based on our review, it appears that the recommendations included in our report dated July 13, 2022.

We understand that the City is requesting a Statement of Risk in accordance with MICC 19.07.160.B.3. (a-d), which states the following:

> *Alteration of landslide hazard areas, seismic hazard areas and associated buffers may occur if the conditions listed in subsection (B)(2) of this section are satisfied and the geotechnical professional provides a statement of risk matching one of the following:*
>
> *a. An evaluation of site-specific subsurface conditions demonstrates that the proposed development is not located in a landslide hazard area or seismic hazard area;*
>
> *b. The landslide hazard area or seismic hazard area will be modified or the development has been designed so that the risk to the site and adjacent property is eliminated or mitigated such that the site is determined to be safe;*

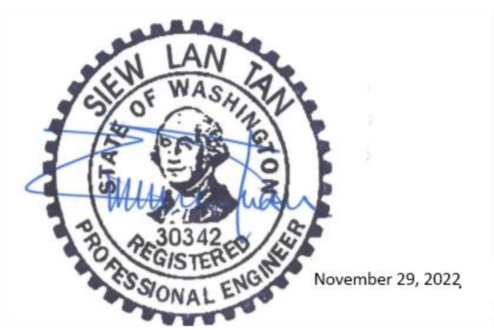
c. *Construction practices are proposed for the alteration that would render the development as safe as if it were not located in a geologically hazardous area and do not adversely impact adjacent properties; or*

d. *The development is so minor as not to pose a threat to the public health, safety and welfare.*

Based on the results of our geotechnical study as outlined in our geotechnical report, the project meets the condition stated in item (a), i.e., *An evaluation of site-specific subsurface conditions demonstrates that the proposed development is not located in a landslide hazard area or seismic hazard area.*

We trust that the information outlined in this letter meets your need at this time. Please call if you have any questions.

Sincerely,



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