GEOTECHNICAL ENGINEERING STUDY

For

HIEN PHAN RESIDENCE

4102, ISLAND CREST WAY
MERCER ISALAND, KING COUNTY, WA 98040

Prepared For

SOUTHERN BIRCH MORTGAGE

6505, 186TH ST. SW
LYNWOOD, WA 98037

Prepared By

PGE Pacific Geo Engineering
Geotechnical Engineering, Consulting & Inspection

P.O. BOX 1419, ISSAQUAH, WASHINGTON 98027

PGE PROJECT NUMBER 22-631

September 13, 2022

September 13, 2022

Client: Southern Birch Mortgage
6505, 186th St. SW
Lynnwood, WA 98037

Attn.: Hien Phan
Owner/Client

Re: Hien Phan Residence
Geotechnical Engineering Study
4102, Island Crest Way
Mercer Island, King County, WA 98040
PGE Project No. 22-631

Dear Hien:

As per the request, Pacific Geo Engineering, LLC (PGE) has completed the geotechnical engineering study for the subject site in Mercer Island, Washington, which is shown in the Site & Exploration Plan, Figure 1. This study includes soil investigation, laboratory testing of native soil, and the foundation recommendations for the proposed residence.

This study is completed in accordance with the mutually agreed upon scope of services described in our proposal no. 22-06-682, dated June 27, 2022, which was authorized on July 28, 2022. The scope of services was developed based on the preliminary understanding of the proposed development obtained from the owner.

1.0 Proposed Development

The general location of the site with the existing site features are shown in the Site & Exploration Plan, Figure 1, prepared by Site Surveying, Inc. The proposed development plan calls for demolishing the existing residence and building a new, double-story residence with one basement level in the subject site.

Based on our experience with similar projects, we anticipate that wall loads will be in the range of 3 to 4 kips per lineal foot, isolated column loads in the range of 40 kips, and slab-on-grade floor loads of 150 pounds per square foot (psf).

At the time of this study, the final site grades and the basement floor grade are not available to PGE; however, for the purpose of this study, we assume that the basement floor grade will require an excavation depth of approximately 10 feet below the current grades to house the basement floor.

The conclusions and recommendations contained in this report are based upon our current understanding of the proposed development. We recommend that PGE should be allowed to review the final design grades and the actual features of the proposed development, and the final construction plan set to verify that the geotechnical recommendations provided in this report are incorporated into the final construction documents. PGE's review of the final plan set would also allow re-evaluating the recommendations, and if necessary, modifying the recommendations before the construction begins. We believe this would be helpful for the project's speedy completion and success.

2.0 Scope of Services

Based on the scope of this geotechnical study delineated in the contract agreement, the following items are accomplished - field exploration, laboratory testing, geologic literature review, laboratory soil testing, engineering evaluation of the field and laboratory data, infiltration potential evaluation, and foundation recommendations.

The scope of our work did not include any wetland study, or any environmental analysis or evaluation to find the presence of any hazardous or toxic materials in the soil, surface water, groundwater, or air in or around this site.

2.1 Engineering Evaluation

The results from the field and laboratory tests were evaluated and engineering analyses were performed to develop the design information and the engineering recommendations for the geotechnical aspect of the proposed development, which are provided in this report.

<u>Subsurface Conditions</u>

- Descriptions of the soil and the groundwater conditions;
- Soil Test Pit Log;
- Depth to water table and any sign of high water table, if encountered;
- Laboratory soil index property test results.

General Site Development & Earthwork & Grading

- Grading and earthwork including site preparation, and fill placement and compaction;
- Use of on-site soils as structural fills;
- Imported structural fill requirements;
- Temporary and permanent excavation slopes;
- Temporary construction dewatering;
- Underground utility structure trench backfilling and pipe bedding.
- Site drainage including permanent subsurface drainage system and temporary groundwater control measures, if necessary.
- Dry and wet weather construction.
- Erosion control measurements.

Geologic Hazards

- Liquefaction potential evaluation of native soil;
- Potential geologic hazards evaluation: landslide, erosion, and seismic.

Structure

- Foundation type and allowable bearing capacity value for supporting the residence;
- Estimated total and differential settlements for the recommended bearing capacity value and observed soil conditions;
- Frictional and passive values for the resistance of lateral forces;
- Subgrade preparation for footings;
- Basement retaining wall design parameters;
- Basement wall design lateral earth pressure diagrams;
- Basement retaining wall details;
- Basement floor slab-on-grade for the proposed residence;
- Subgrade preparation for slab-on-grade floor;
- Seismic design considerations, including the site coefficient per 2018 IBC code.

3.0 Surface and Subsurface Features

3.1 Site Location

The subject property (Parcel #545030-0025) is located at 4102, Island Crest Way, Mercer Island as shown in the Site & Exploration Plan, Figure 1. As per this figure, the site is bounded by Island Crest Way on west, and single-family residences on other three sides.

3.2 Site Descriptions

The subject property is located within a region dominated by densely populated single family residences. The property is a rectangular shape land, with almost 0.3 acre area. The property is currently occupied by a single-storey existing residence located in the eastern portion of the property. The site has an entrance via a concrete paved driveway from the Island Crest Way, which continues up to the residence, and then extends further in the backside of the residence. The property has vegetations comprised of mostly landscape grasses and few scattered small trees. The property has minor downward slope from its east to the west boundary adjacent to the Island Crest Way. The existing residence is located almost in the level flat ground in the uphill area of the site close to the eastern boundary of the property. The site then slowly slopes down to the west boundary of the property adjacent to the Island Crest Way. The average elevation in the flat level residence area is approximately 298 and the elevation at the bottom of the slope adjacent to the Island Crest Way is approximately 290. The above elevation difference over the horizontal distance of approximately 70 feet generates a slope gradient of approximately 11%. The above features are shown in Figure 1.

4.0 Field Investigation

Our field exploration was performed on August 25, 2022. A total of three (3) test pits were excavated in the subject property to determine the soil and groundwater conditions of the site. The test pit locations are shown in the Site & Exploration Plan, Figure 1, attached with this report.

Test pits were excavated to depths of approximately 5 to 10 feet below the existing grades as shown in the soil test pit logs (**Appendix A**). Test pits were backfilled with loosely compacted excavated soils. The locations of the test pits in Figure 1 should be considered accurate only to the degree implied by the measuring methods. The test pits were completed using a backhoe rented by PGE.

A geotechnical engineer from PGE observed the field exploration works including the test pit excavations, soil sampling, continually logging the subsurface conditions in the test pits, collecting representative bulk samples from different soil layers at different depths of the test pits, and visually-manually classifying the soil samples in the field as per the methods described in the ASTM D-2488-93 (based on soil samples' density/consistency, moisture condition, grain size, and plasticity estimations). The soil samples were designated according to the test pit number and sampling depth, stored in watertight plastic containers, and later on transported to our laboratory for further visual examination and testing.

Results of the field investigation are presented in the soil test pit logs (**Appendix A**). The final exploration logs were prepared with our observation and interpretation of the test pit excavation, and visual examination of the samples in the field and later on in the laboratory. The soils were classified according to the methods presented in figure 'Key to Exploration Logs' in Appendix A. This figure also provides a legend explaining the symbols and abbreviations used in the soil exploration logs. The soil logs indicate the depth where the soils change. It should be noted that the indicated stratification lines on the logs represent the approximate boundaries between soil types. The actual transitions of varying soil strata may be more gradual in the field.

5.0 Laboratory Testing

Laboratory tests were conducted on several selected representative soil samples to evaluate the general physical properties and the engineering characteristics of the soils encountered. The bulk samples were visually-manually classified in the laboratory following the procedure described in ASTM D-2488-17 (based on the soil samples' density/consistency, moisture condition, grain size, and plasticity estimations), and later on the soil samples' classifications were supplemented by laboratory tests data in accordance with the procedure described in ASTM D-2487-17.

Moisture content tests were conducted on the samples in accordance with ASTM D-2216-10 procedures. One (1) Sieve Analysis tests (Grain size distributions) were performed on two selected samples in accordance with ASTM D-422 procedure.

The results of the moisture content tests and the amount of percentages of minus #200 sieve passed are provided in the test pit logs, Appendix A. The grain-size distributions of the soil obtained from the Sieve Analysis test are shown in the laboratory test report B-1 in **Appendix B**.

6.0 Regional Geology

The site is in the Puget Sound Lowland, a north-south trending structural and topographic depression lying between Olympic Mountains on the west and Cascade Mountains on the east. The lowland depression experienced successive glaciation and nonglaciation activities over the time of Pleistocene period. During the most recent Fraser glaciation, which advanced from and retreated to British Columbia between 13,000 and 20,000 years ago, the lowland depression was buried under about 3,000 feet of continental glacial ice. During the successive glacial and nonglacial intervals, the lowland depression, which is underlain by Tertiary volcanic and sedimentary bedrock, was filled up above the bedrocks to the present-day land surface with Quaternary sediments, which consisted of Pleistocene glacial and nonglacial sediments. The glacial deposits include concrete-like lodgement till, lacustrine silt, fine sand and clay, advance and recessional outwash composed of sand or sand and gravel, and some glaciomarine materials. The nonglacial deposits include largely fluvial sand and gravel, overback silt and clay deposits, and peat attesting to the sluggish stream environments that were apparently widespread during nonglacial times.

7.0 Site Geology

The geologic unit of the subject site mapped on the 'Geologic Map of Mercer Island, Washington, 2006' by Kathy Goetz Troost and Aaron P. Wisher. as being underlain by Vashon Sub-glacial Till (Qvt).

The glacial till generally consists of a compact and dense, heterogeneous mixture of gravel, sand, silt, and clay that was deposited at the base of the continental ice mass and was subsequently overridden. As such, the glacial till is considered to be over consolidated and generally has high strength and low compressibility characteristics, where undisturbed.

In general, our explorations in the test pits encountered till in the form of very dense, weakly cemented, chunks of silty sand with sub-rounded to well rounded gravel (USCS classifications: SM), which conforms with the above geologic unit, Vashon Sub-glacial Till (Qvt), shown in the map.

8.0 Site Soil and Groundwater Conditions

A topsoil layer of approximately 12 inches thickness consisted of black silt with roots and organics is encountered at the test pit locations. The topsoil is in slightly moist and loose conditions.

The topsoil is underlain by existing fills consisted of light brown, silty sand with gravel up to 6 feet depth below grade in test pit TP-1 and up to 4 feet depth below grade in test pit TP-2. The fills contain brick bats and decayed wood pieces. The existing fills are in moist condition and in medium dense state. No fills are noticed in test pit TP-3.

The fills in test pit TP-1 and TP-2, and the topsoil in test pit TP-3 are underlain by native soil consisted of glacial till. The till in the exploration area is encountered at approximately 6 feet in TP-1, 4 feet in TP-2, and 2 feet below the grades in test pit TP-3. The till is comprised of lightly gray, silty sand with sub-rounded to well rounded gravel (USCS classification: SM), and occasional cobbles extends up to the bottom of the test pits. The test pit TP-1, TP-2, TP-3 were terminated at approximately 10 feet, 8 feet, and 5 feet below the existing grades. In general, the till is found in moist condition, and in very dense state. Till is noticed with partially cemented chunks. The digging through the till is difficult. No cave in was noticed across the depths of the test pits. The tills contain percentage of fines of approximately 23%.

<u>Hydrogeologic Condition</u>

No groundwater or perched groundwater seepage was noticed in the test pits within their exploration depths. However, minor, scattered signs of iron-oxide stains are visible in the fills in test pit TP-1 and TP-2.

Perched water is defined when stormwater permeates through the upper, less denser soils, and accumulates on top of the underlying denser, less permeable soils, like glacial till, which is very typical in the Puget Sound area. Typically, perched water presents in a spatial manner above the glacial till. It is to be noted that fluctuations in the perched water amount and level may be expected due to the seasonal variations in the amount of rainfall, surface runoff, and other factors not apparent at the time of our explorations. Typically, the perched water level rises higher and the flow rate increases during the wet winter months. The possibility of the fluctuations and the presence of perched water and the signs of mottling should be considered when considering any underground infiltration system in this site for managing the stormwater runoff of the proposed development.

The preceding discussion on the subsurface conditions of the site is intended as a general review to highlight the major subsurface stratification features and material characteristics. For more complete and specific information at individual test pit location, please review the Soil Test Pit Log (Figure A-1, A-2, A-3) in Appendix A. The logs include soil descriptions, stratification, and location of the samples, and the laboratory test results. It should be noted that the stratification lines shown on the logs represent the approximate boundaries between various soil strata; actual transitions may be more gradual or more severe. The subsurface conditions depicted in the soil logs are for the test pit locations indicated only, and it should not necessarily be expected that these conditions are representative at other locations of the site.

9.0 Conclusions and Recommendations

9.1 General

Based on this study, the subject site is considered suitable for the proposed development, provided the geotechnical recommendations provided in this report are properly understood and interpreted, and strictly implemented during the design and construction of the proposed development.

Based on the soil conditions in the test pits, and the assumed depth of the basement floor level at approximately 10 feet below the grades, the footings for the basement retaining walls will bear on the very dense glacial till, encountered at approximately 2 to 6 feet below the grades. The till at the bottom of the wall footings will be in very dense state hence considered as suitable to support the footings. The isolated interior columns can be supported by spread footings and the basement retaining walls can be supported by continuous strip footings. The till can be considered as ‘competent' native subgrade, which is described as the native soil unit that is to be compacted and proofrolled adequately to firm and unyielding conditions following the procedures outlined later on in Section 9.2.2, 'Subgrade Preparation' of this report. The 'competent' native subgrade described above will be able to provide an allowable bearing capacity value of 3000 psf to support the footings and the residence.

A slab-on-grade floor for the basement level can be built on the 'competent' native subgrade as described above for the footings.

The final naïve subgrades and its preparation to support any load bearing structure must be monitored and approved by the on-site geotechnical special inspector during the construction of the project.

The new structural fills to be placed behind the basement retaining walls must be placed and compacted adequately as described later on in Section 9.2.6 'Fill Placement and Compaction Requirements' of this report. The backfill to be placed behind the retaining walls should be consisted as per the recommendations provided later on in Section 9.2.5 'Structural Fill' of this report.

9.2 Site Preparation

Preparation of the site should involve clearing, stripping, subgrade preparation and proofrolling, cutting, filling, excavations, and drainage installations. The following paragraphs provide specific recommendations on these issues.

9.2.1 Clearing and Grubbing

Initial site preparation for construction of the proposed residence, driveway, parking area, any other load-bearing structure, and placing new fills on the native subgrades should include stripping of vegetation and topsoil from the construction areas. Based on the topsoil thickness encountered at our test pit locations, we anticipate topsoil stripping depths of about 12 inches, however, thicker layers of topsoil may be present in unexplored portions of the site. It should be realized that if the stripping operation takes place during wet winter months, it is typical a greater stripping depth might be necessary to remove the near-surface moisture-sensitive silty soils disturbed during the stripping; therefore, stripping is best performed during dry weather period. Stripped vegetation debris should be removed from the site. Stripped organic topsoil will not be suitable for use as structural fill but may be used for future landscaping purposes.

9.2.2 Subgrade Preparation

After the site clearing and site stripping, cut and fill operations can be initiated to establish desired final grades for the proposed structures and any new fills to be placed on the native subgrades. Any exposed subgrades that are intended to provide direct support for new construction and/or require new fills should be adequately proofrolled to evaluate their conditions and to identify the presence of any isolated soft and yielding areas and to verify that stable subgrades are achieved to support the proposed structures, and any new fills. Proofrolling should be done with a loaded dump truck or a front-end loader or a big vibratory roller under the supervision of the on-site geotechnical engineer. If it is found by the on-site geotechnical engineer that the soil is too wet near the subgrade to be proofrolled or it not feasible to proofroll the subgrade, then an alternative method (i.e., visual evaluation and probing with a 1/2-inch diameter steel T-probe) can be used by the geotechnical engineer to identify the presence of any isolated soft and yielding areas and to verify that stable subgrades are achieved to support the proposed structures and any new fills.

If any subgrade area are found in soft and moist conditions, ruts and pumps excessively, and cannot be stabilized in place by compaction the affected soils should be over-excavated completely to firm and unyielding suitable bearing materials, and to be replaced with new structural fills to desired final subgrade levels. If the depth of overexcavation to remove unstable soils becomes excessive, a geotextile fabric, such as Mirafi 500X or equivalent in conjunction with structural fills may be considered to achieve a firm bearing subgrades to support the proposed structures and any new fills.

If needed to stabilize the soft/wet base of an overexcavated area, we recommend considering a 6 to 12-inch layer of ballast rock or quarry spalls should be placed to form a base on which the structural fill needs to be placed and compacted to achieve the final grade. Ballast rock should meet the requirements for Class B Foundation Material in Section 9-03.17 and quarry spalls should meet the requirements in Section 9-13.6 of the 2014 WSDOT Standard Specifications. The ballast rock or quarry spalls should be pushed into the subgrade with the back of a backhoe bucket or with the use of a large-vibratory steel drummed roller without the use of vibration. Such decision should be made the on-site geotechnical engineer during the actual construction of the project.

The loosely backfilled soils in the areas of exploratory test pits should be overexcavated completely to the firm native soils and backfilled with adequately compacted new structural fills to the final grades. Tree stumps and large root balls should be removed completely and backfilled with new structural fills to the desired subgrade levels.

9.2.3 Reuse of Native Soils as Structural Fills

The ability to use the overexcavated existing fills as structural fills for backfilling behind the basement retaining walls will depend on the quality of the fills, i.e., the content of decayed wood debris and organics, fines content, and soil types and their gradation. Typically, soils containing decayed wood debris and organics are not considered suitable for use as structural fills. However, if seems feasible, the decayed wood debris and the organic lumps may be selectively handpicked to remove from the fills for using the remaining portion of the fills as structural fills. The on-site geotechnical engineer should inspect the process to verify if the existing fills can be used as structural fills.

The existing fills and the native soils below the fills contain higher percentage of fines (approximately 23%) compared to the typical 'imported structural fills' that contains 5% or less fines, therefore considered as moisture sensitive soils. Typically, when the fines content (that portion passing the U.S. No. 200 sieve) of soil increases, the soil becomes increasingly sensitive to small changes in moisture content, which makes the soils' compaction more difficult or impossible. Soils containing more than about 5 percent fines by weight cannot be consistently compacted to the recommend degree when the moisture content is more than about 2 percent above or below the optimum. Especially, if the soils with higher fines content are used during the wet weather period, typically between October and May, significant reduction in the soils strength and support capabilities occur. Also, when these soils become wet they may be slow to dry and thus significantly retard the progress of grading and compaction activities. These soils can be used as structural fills during the dry season, provided the optimum moisture content of the soils can be maintained during the compaction.

The suitability of using the native soils or the existing fills should be verified and approved by the on-site geotechnical engineer prior to their use. If the existing fills cannot be used after the inspection and asked by the on-site geotechnical engineer to discard the existing fills, then imported new structural fills are to be brought in to the site for backfilling behind the basement retaining walls. In the event that whether the fill materials are to be imported to the site, we recommend that the imported fill materials be verified and approved by the on-site geotechnical engineer of PGE prior to their use.

If the native soils contain higher percentages of fines then such soils would pose problems during their compaction when the moisture content of the soils become excessive in the wet months. During wet weather periods, typically between October and May, increases in the moisture content of these soils can cause significant reduction in the soils strength and support capabilities. In addition, when these soils become wet they may be slow to dry and thus significantly retard the progress of grading and compaction activities. It will, therefore, be advantageous to do the backfilling operations using the native soils as structural fills during the dry season, typically from July through September. This would significantly reduce the earthwork costs over wet weather construction.

In the event that whether the fill materials are to be imported to the site, or if on-site soils are to be reused as structural fills, we recommend that the potential fill materials be verified and approved by the on-site geotechnical engineer prior to their use.

9.2.4 Dry Weather Construction

Since the site soils have higher fines content (approximately 23%), we recommend that the proposed construction should be completed during the dry season of the year to mitigate any erosion related issues if arise during the construction activities in the wet season. Erosion particularly happens, when uncontrolled surface runoff is allowed to flow over unprotected excavation areas of the site during the wet winter months.

9.2.5 Structural Fill

If the native soils are found unsuitable for using as structural fills then we recommend that imported structural fill should be used for backfilling purposes. Structural fill is defined as non-organic soil, free of deleterious materials, and well-graded and free-draining granular material, with a maximum of 5 percent passing the No. 200 sieve by weight, and not exceeding 6 inches for any individual particle. A typical gradation for structural fill is presented in the following table.

Structural Fill	
U.S. Standard Sieve Size	Percent Passing by Dry Weight
3 inch	100
¾ inch	50 –100
No. 4	25 – 65
No. 10	10 – 50
No. 40	0 – 20
No. 200	5 Maximum*

* Based on the ¾ inch fraction.

Other materials may be suitable for use as structural fill provided they are approved by the project geotechnical engineer. Such materials typically used include clean, well-graded sand and gravel (pit-run); clean sand; various mixtures of gravel; crushed rock; controlled-density-fill (CDF, it should meet the requirements in Section 2-09.3(1)E of the 2008 WSDOT Standard Specifications); and lean-mix concrete (LMC). Recycled asphalt, concrete, and glass, which are derived from pulverizing the parent materials are also potentially useful as structural fill in certain applications. These materials must be thoroughly crushed to a size deemed appropriate by the geotechnical engineer (usually less than 2 inches). The structural fills should have a maximum 2 to 3-inch particle diameter.

9.2.6 Fill Placement and Compaction Requirements

Generally, quarry spalls, controlled density fills (CDF), lean mix concrete (LMC) do not require special placement and compaction procedures. In contrast, clean sand, crushed rock, soil mixtures and recycled concrete should be placed under special placement and compaction procedures and specifications described here.

The structural fills under structural elements should be placed in uniform loose lifts not exceeding 12 inches in thickness for a walk-behind heavy-duty vibratory plate compactor and 4 inches for hand held smaller and lighter compaction equipment. Each lift should be compacted to a minimum of 95 percent of the fill's maximum dry density as to be determined in the laboratory by ASTM Test Designation D-1557 (Modified Proctor) method, or to the applicable minimum City or County standard, whichever is the more conservative. The fill should be moisture conditioned such that its final moisture content at the time of compaction should be at or near (typically within about 2 percent) of its optimum moisture content, as determined by the ASTM method. If the fill materials are on the west side of optimum, they can be dried by periodic windrowing and aeration or by intermixing lime or cement powder to absorb excess moisture.

If field density tests indicate that the last lift of compacted fills has not been achieved the required percent of compaction or the surface is pumping and weaving under loading, then the fill should be scarified, moisture-conditioned to near optimum moisture content, re-compacted, and re-tested prior to placing additional lifts.

Care in the placement and compaction of fills behind the retaining walls must be taken in order to insure that undue lateral loads are not induced on the wall. Large equipment such as a big vibratory roller must not be allowed to compact the fills behind the wall and the slope embankment above the wall. A walk-behind big vibratory plate compactor can be used behind the wall. No large equipment should be allowed to traverse over the slopped embankment during its compaction. A hoe-pack with a large boom that can be reached to the compaction area can be used for sloped embankment compaction. However no hoe-pack should be used within the horizontal distance behind the wall equal to the retained height of the wall.

9.2.7 Site Drainage

Surface Drainage

The final site grades of the finished development must be such that surface runoff will flow by gravity away from the building and other structure, and should be directed to appropriate collection points. We recommend providing a minimum drainage gradient of about 2% for a minimum distance of about 10 feet from the building perimeter. A combination of using controlled surface drainage and capping of the building surroundings by concrete, asphalt, or low permeability silty soils will help minimize or preclude surface water infiltration around the perimeter of the building and beneath the garage basement floor slab. Paved areas should be graded to direct runoff to catch basins and or other collection facilities. Collected water should be directed to the on-site drainage facilities by means of properly sized smooth walled PVC pipe. Interceptor ditches or trenches or low earthen berms should be installed along the upgrade perimeters of the site to prevent surface water runoff from precipitation or other sources entering in to the lower area of the lot. It should be noted that surface water runoff from precipitation flows as a sheet flow over slope is considered to be the primary cause of surficial sloughing and triggering slope failure. Therefore, the surface drainage system should be designed in such a way that stormwater runoff over the finished lot must not create any sheet flow over the sloped areas of the lot, instead, the stormwater runoff must be collected in drain pipes to discharge in approved discharge points at the toe of the slope. Surface drainage system and the water collection facilities should be designed by a professional civil engineer.

Footing Excavation Drain

Water must not be allowed to pond in the foundation excavations or on prepared subgrades either during or after construction. If due to the seasonal fluctuations, groundwater seepage is encountered within footing depths, we recommend that the bottom of excavation should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff, and then direct the water to ditches, and to collect it in prepared sump pits from which the water can be pumped and discharged into an approved storm drainage system.

Basement Wall Footing Drain

Footing drains should be used where (1) crawl spaces or basements will be below a structure, (2) a slab below the outside grade, and (3) the outside grade does not slope downward from a building. The drains must be laid with a gradient sufficient to promote positive flow by gravity to a controlled point of approved discharge. The foundation drains should be tightlined separately from the roof drains to this discharge point. Footing drains should consist of at least 6-inch diameter, heavy-walled, perforated PVC pipe or equivalent. The pipe should be surrounded by at least 6 inches of free-draining gravel over the pipe and 3 inches of free-draining gravel below the pipe. The pipe and the free-draining gravel should be wrapped in a non-woven geotextile filter fabric (Mirafi 140N) to limit the ingress of fines into the gravel and the pipe. Cleanouts should be provided. The drains should be located along the outside perimeter of the spread footings of the retaining walls. Also, the drain should be placed at or below the invert of the footing. A typical footing drain detail is provided in Figure 3 of this report.

Downspout or Roof Drain

These should be installed once the building roof in place. They should discharge in tightlines to a positive, permanent drain system. Under no circumstances connect these tightlines to the perimeter footing drains.

9.2.8 Temporary Excavations

As we understand from the project plan, the proposed residence will have basement floor at approximately 10 feet below the grades. Therefore, an excavation depth of approximately 10 feet will be required to house the basement floor in the site.

As a general rule, all temporary soil cuts greater than 4 feet in height associated with site regarding or excavations should be adequately sloped back or properly shored to prevent sloughing and collapse. As for the current estimation purposes, in our opinion, for temporary excavations, the side slopes should be laid back at a minimum slope inclination of 1H:1V (Horizontal:Vertical) in the existing fills above the till. A minimum slope inclination of 3/4H:1V is to be maintained in the very dense till deposit (OSHA soil Type A) encountered at approximately 4 to 6 feet below the grades. However, estimation for the proper inclination of excavation side slopes should be made on-site after inspecting the soil and groundwater conditions, which will be revealed during the actual excavation in the site.

All temporary soil cuts greater than 4 feet in height, if cannot be sloped back because of the limited horizontal distance between the top of the excavation line and the property line, a properly shoring system is to be considered to prevent sloughing and collapse of the excavation slope. The decision of requirement of a shoring system is to be made on-site when the actual excavation will take place.

Any excavation side inclinations will assume that the ground surface behind the cut slopes is level, that surface loads from equipment and materials are kept a sufficient distance away from the top of the slope. If these assumptions are not valid, we should be contacted for additional recommendations. Flatter slopes may be required if soils are loose or caving and/or water, are encountered along the slope faces. If such conditions occur and the excavation cannot stand by itself, or the excavation slope cannot be flattened because of the space limitations between the excavation line and the boundary of the property, temporary shoring may be considered. The shoring will assist in preventing slopes from failure and provide protection to field personnel during excavation. Because of the diversity available of shoring stems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor engaged to complete the installation. We can assist in designing the shoring system by providing with detailed shoring design parameters including earth-retaining parameters, if required.

Where sloped embankments are used, the top of the slopes should be barricaded to prevent vehicles and storage loads within 10 feet of the top of the slopes. Greater setbacks may be necessary when considering heavy vehicles, such as concrete trucks and cranes. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the top of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. All temporary slopes should be protected from surface water runoff.

The owner and the contractor should be aware that in no case should the excavation slopes be greater than the limits specified in local, state, and federal safety regulations, particularly, the Occupational Safety and Health Administration (OSHA) regulations in the "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P, dated October 31, 1989" of the Federal Register, Volume 54, the United States Department of Labor. As mentioned above, we also recommend that the owner and the contractor should follow the local and state regulations such as WSDOT section 2-09.3(3) B, Washington Industrial Safety and Health Act (WISHA), Chapter 49.17RCW, and Washington Administrative Code (WAC) Chapter 296-115, Part N. These documents are to better insure the safety of construction worker entering trenches or excavation. It is mandated by these regulations that excavations, whether they are for utility trenches or footings, be constructed in accordance with the guidelines provided in the above documents. We understand that these regulations are being strictly enforced and, if they are not closely followed, both the owner and the contractor could be liable for substantial penalties.

Stability of temporary excavations is a function of many factors including the presence of, and abundance of groundwater and seepage, the type and density of the various soil strata, the depth of excavation, surcharge loadings adjacent to the excavation, and the length of time and weather conditions while the excavation remains open. It is exceedingly difficult under these unknown and variable circumstances to pre-establish a safe and maintenance-free temporary excavation slope angle at this time of the study. We therefore, strongly recommend that all temporary, as well as permanent, cuts and excavations in excess of 4 feet be examined by a representative of PGE during the actual construction to verify that the recommended slope inclinations are appropriate for the actual soil and groundwater seepage conditions exposed in the cuts. If the conditions observed during the actual construction are different than anticipated during this study then, the proper inclination of the excavation and cut slopes or requirements of temporary shoring should be determined depending on the condition of the excavations and the slopes.

The above information is provided solely for the benefit of the owner and other design consultants, and under no circumstances should be construed to imply that PGE assumes responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred. Therefore, the contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures.

We expect that the excavation can be completed using conventional equipments such as bulldozers or backhoes.

9.3 Construction Monitoring

Problems associated with earthwork and construction can be avoided or corrected during the progress of the construction if proper inspection and testing services are provided. Since this project involves so many aspects of geotechnical engineering related construction activities such as the stripping of vegetations, removals of tree stumps and root balls, removals of existing fills, final native subgrade preparation, cut and filling, overexcavation, fill placement and compaction of fills, retaining wall construction, and foundation bearing capacity, we recommend that an experienced geotechnical inspector should inspect all the above activities. It is recommended that the above construction activities be monitored by a representative from our firm since we have the prior knowledge, familiarity, and better understanding with our recommendations.

9.4 Building Foundation Recommendations

The conventional shallow spread footings can be used to support the isolated interior column footings of the basement level and the basement retaining walls can be supported by continuous strip footings.

The footings must supported on the 'competent' native subgrade described as the native soil unit i.e., glacial till that is to be compacted and proofrolled adequately (following the procedures described earlier in Section 9.2.2, 'Subgrade Preparation' of this report) to firm and unyielding conditions prior to placing footings. We recommend that a maximum net allowable bearing capacity of 3000 pounds per square foot (psf) can be used for the design of the footings to be placed above the 'competent' native subgrade consisted of glacial till.

For short-term loads, such as wind and seismic, a 1/3 increase in the above allowable capacity can be used. We recommend that continuous footings have a minimum width of 18 inches and individual column footings a minimum width of 24 inches. All exterior footings should bear at least 18 inches below the final adjacent finish grade to provide adequate confinement of the bearing materials and frost protection.

<u>Settlement</u>

Based on our settlement potential evaluation of the above shallow foundation options, we anticipate that properly designed and constructed foundations supported on the recommended materials should experience total and differential settlements of less than 1 inch and 1/2 inch, respectively. The majority of these settlements are expected to occur during construction. This estimation was done without the aid of any laboratory consolidation test data, but on the basis of our experience with similar types of structures and subsoil conditions.

Lateral Load Resistance

Lateral foundation loads can be resisted by friction between the foundation base and the supporting soil, and by passive earth pressure acting on the face of the embedded portion of the foundation. For frictional resistance, a coefficient of 0.35 can be used. For passive earth pressure, the available resistance can be computed using an equivalent fluid pressure of 300 pcf, which includes a factor of safety of 1.5. This value assumes the foundation must be poured "neat" against the undisturbed native soils or structural fill placed and compacted as described earlier in Section 9.2.6, 'Fill Placement and Compaction Requirements' of this report.

Footing Subgrade Inspection

Variations in the quality and strength of the potential bearing soils can occur with depth and distance away from the test pit. Therefore, careful evaluation of these bearing materials is recommended at the time of construction to verify their suitability to support the above recommended bearing pressure. We recommend that PGE representative examine the bearing materials prior to placing forms or rebar.

9.5 Basement Retaining Wall Waterproofing

If the building is to be designed as water-tight then waterproofing of the building is to be considered. A structural building envelope consultant or an architect with experience in this regard can make recommendations regarding waterproofing design specifications. Generally, waterproof barriers should be used between buried wall and the retained earth.

9.6 Building Design Seismic Parameters

The seismic site classifications are to be based on the Table 20.3.1 of ASCE 7- Chapter 20. Based on our evaluations of the subsurface conditions and the above table above, we interpret the underlying bearing soils to correspond to 'C', which refers to very dense soils. The seismic design parameters for the structural design of the building should follow ASCE 7-16 code standards, which are provided in Appendix D of this report. According to the ASCE 7-16 code standards, the mapped spectral response accelerations $S_s = 1.406$ and $S_1 = 0.485$, and corresponding site co-efficient values $F_a = 1.2$ and $F_v = 1.5$, respectively, should be used for the design of the building. Structural design of the buildings at the project site should follow 2018 International Building Code (IBC) standards.

9.7 Slab-on-grade Floor for Building

If slab-on-grade option is chosen for the proposed basement floor, then the slab-on-grade floor may be placed on an adequately compacted and proofrolled, firm and unyielding, and stable native subgrades ('competent' native subgrade), described as the native soil unit glacial till. The final native subgrades must be compacted and proofrolled adequately following the procedures described earlier in Section 9.2.2, 'Subgrade Preparation' of this report to firm and unyielding conditions prior to placing the slab-on-grade floor.

The slab-on-grade floor can also be placed on new structural fill pad, to be bearing on the 'competent' native subgrade described as the native soil unit glacial till that is to be compacted and proofrolled adequately following the procedures described earlier in Section 9.2.2, 'Subgrade Preparation' of this report to firm and unyielding conditions. The fills in the new fill pad must be compacted adequately following the procedures described earlier in Section 9.2.6, 'Fill Placement and Compaction Requirements' of this report. The new, imported structural fills should be consisted as per the recommendations provided later on in Section 9.2.5 ‘Structural Fill’ of this report.

After subgrade preparation is completed, the slab should be provided with a capillary break to retard the upward wicking of ground moisture beneath the floor slab. The capillary break would consist of a minimum of 4 to 6-inch thick clean, free-draining sand or pea gravel. The structural fill requirements specified in Section 9.2.5, Structural Fills, could be used as capillary break materials except that there should be no more than 2 percent of fines passing the no. 200 sieve. Alternatively, ‘Gravel Backfill for Drains’ per 2014 WSDOT Standard Specifications 9-03.12(4) can be used as capillary break materials. Where moisture by vapor transmission is undesirable, we recommend the use of a vapor barrier such as a 6-mil. or 0.006 inch thick durable plastic sheeting (such as Crossstuff, Moistop, or Visqueen) between the capillary break layer and the floor slab to prevent the upward migration of ground moisture vapors through the slab. During the casting of the slab, care should be taken to avoid puncturing the vapor barrier. At owner’s or architecture’s discretion, the membrane may be covered with 2 inches of clean, moist sand as a ‘curing course’ to guard against damage during construction and to facilitate uniform curing of the overlying concrete slab. The addition of 2 inches of sand over the vapor barrier is a non-structural recommendation. Styrofoam, as an additional layer can be placed between the concrete floor slab and the capillary break layer where heated area for provision of better insulation is to be required. A typical slab-on-grade section with the above features is provided in Figure 3 of this report.

Based on the subgrade preparation as described earlier in Section 9.2.2 of this report, a modulus of subgrade reaction value of about 150 pounds per cubic inch (pci) can be used to estimate slab deflections, which could arise due to elastic compression of the subgrades.

9.8 Basement Concrete Retaining Wall

Design Considerations for Concrete Wall

The basement floor will have permanent concrete retaining walls to support the excavation depths below the current grades. Based on the soil conditions, and the wall heights and the lateral loading to be exerted on the walls, the design parameters for the concrete retaining walls are developed, which are presented on Figure 2. The at-rest earth pressure condition for a fixed at top wall is provided in Figure 2. A typical detail of the basement concrete retaining wall restrained at the top is provided in Figure 3. A general wall construction notes and inspection details are provided in Figure 4.

The details of the walls with reinforcement, size and thickness of the walls, and the footing sizes and thickness are to be determined by the project structural engineer, which are to be provided in a separate wall design report.

Lateral Earth Pressure: Our lateral earth pressure recommendations are based on the assumption that the basement wall will have fixed at top and therefore the lateral earth pressures to be acting on the wall will be of at-rest (K_0) condition.

Soil Design Parameters: It is assumed that the void areas between the excavation cut face and the backside of the drainage layer of the retaining wall will be backfilled with adequately compacted structural fills or similar quality native soils if found to be suitable for backfilling purpose. Based on this assumption, the following geotechnical parameters are used for the design of the gravity walls.

Retained total soil unit weight, γ:	125 pcf
Water unit weight, γ_w:	62.5 pcf
Retained soil friction angle, ϕ:	34 deg.
Cohesion, c:	0 psf
At-rest Earth pressure Co-efficient, K_0:	0.44

At-rest condition (restrained at top) earth pressure:	55 pcf
Passive Earth pressure Co-efficient, K_p:	3.6
Allowable static condition passive earth pressure:	300 pcf (includes a FS value of 1.5)
Allowable dynamic condition passive earth pressure:	225 pcf (static value is reduced by 25%)
Allowable friction co-efficient, μ:	0.35
Allowable static bearing capacity:	3000 psf
Allowable dynamic condition bearing capacity:	4000 psf (static bearing capacity is increased by 1/3rd)
Traffic Surcharge Loading, if there is any from garage or parking or driveway	250 psf
Backhill Slope Surcharge Loading:	(Slope height/2)* γ *K_a psf
Seismic Loading for at-rest condition & level backhill:	8H psf

Surcharge Loading: The basement retaining walls are considered to be with a final grade having level ground above behind the wall, which will be remained in unaltered condition throughout the life of the wall. Therefore, no surcharge loading needs to be added due to the sloped backhill above the wall.

Traffic Loading

The basement retaining walls may be subjected to traffic surcharge loading because of the adjacent driveway, parking, or garage, which must be accounted for the wall loading.

Based on the final building plan and the lot grading plan, ultimately the project structural engineer will be deciding whether the basement retaining wall will be subjected to any type of surcharge loading, and what type surcharge loading on the backhill side of the wall top; for example, surcharge loading due to backhill slope, or traffic, or any other type of additional surcharge loading i.e., stockpile, or other structure, or heavy equipment operation.

Hydrostatic Pressure: The wall is to be designed considering a drainage layer behind the wall, therefore, no hydrostatic pressure is considered to be acting behind the wall.

Seismic Loading: The walls must be designed for the seismic loading condition. Based on a peak horizontal acceleration of 0.3g from the 500-year return period earthquake, as recommended for design by the UBC, the seismic loadings are provided in earlier in sub-section 'Soil Design Parameters'. The seismic loading or the lateral pressures due to the seismic loading should be considered to be acting uniformly over the full height of the wall. The seismic pressures will have a rectangular pressure distribution on the wall.

During an earthquake event the static active earth pressure and the static allowable bearing capacity will temporarily increase and the passive resistance at the wall footing will temporarily decrease, as described by Seed and Whitman (1970) and Sheriff (1983). The static active and the static allowable bearing capacity can be increased by 1/3rd or 30 percent and the passive lateral earth resistance can be reduced by 25 percent. These increment and the decrement amounts are applied to determine the transient bearing capacity and the passive resistance values for the seismic condition, provided later on in this section in the respective sub-sections.

Bearing Capacity: The wall is designed based on the assumption that the final native subgrades to be consisted of very dense glacial till, described earlier as 'competent native subgrade', which would be able to provide a maximum net allowable bearing capacity of approximately 3000 pounds per square foot (psf) to support the wall footing. The wall footing must be at least 2 feet above the ground water table, if there is any.

The recommendations for the verifications of the allowable bearing capacity value and the quality and strength of the potential bearing soils, and the settlement design criteria are to be followed as per the Section 9.4 of this report.

Construction Recommendations for Concrete Wall

The proposed concrete retaining walls must be built following the recommendations, specifications, and the guidelines provided in the following sub-sections, and as per the notes and the details provided in Figure 3 and 4. In addition to these figures, the contractor must also follow the instruction, specification, and the wall details to be provided by the project structural engineer in a separate report and structural plan set.

Wall Footing Subgrade Preparation

Prior to placing the base rock layer, the Section 9.2.1, 'Clearing and Grubbing' and Section 9.2.2, 'Subgrade Preparation' should be followed to prepare the final native subgrades. The geotechnical engineer on-site must examine the final subgrade for the wall foundations to verify that the conditions are suitable to support the wall.

Wall Footing

The wall footing must be placed on a properly prepared subgrade as it is recommended above. It should be noted that the actual subgrade conditions at the bottom of wall footings will be revealed during the actual construction of the walls. Therefore, the competency of the subgrades to support the wall footings to be determined only after the footing excavations are to be completed up to the final bottom elevation of the footings. We recommend that the on-site geotechnical engineer should verify the competency of the final footing subgrades for being able to provide the recommended allowable bearing capacity value of 3000 psf.

Keyway

A keyway for the wall foundation below the bottom of the footing consisting of a shallow trench shall be constructed along the full length of the walls. The keyway will provide lateral passive resistance for the walls against their sliding. The lateral passive resistances could be achieved from the passive earth pressures acting on the face of the embedded portion of the keyway. The actual cross-sectional size of the keyway requirement should be determined by the project structural engineer.

Embedment Depth:

In addition to the keyway, a minimum embedment depth of 2 feet or more below the final floor elevation grade at the toe of the wall must be provided to achieve the lateral passive resistance against the sliding. The lateral passive resistances could be achieved from the passive earth pressures acting on the face of the embedded portion of the wall and the footing. The actual embedment depth requirement should be determined by the project structural engineer based on the requirements of the wall design.

Drainage Layer

To control seepage from behind the walls and to prevent any hydrostatic pressures build up behind the walls a drainage mat consisted of Mirafi G100N or equivalent as shown in the wall details in Figure 3 should be installed.

Footing Drainpipe

A perforated or slotted, 6 inch diameter flexible, Schedule 40 PVC drainpipe should be installed at the base of wall foundation benches. This will prevent any water accumulation at the toe of the walls. The pipe shall be bedded on and surrounded by “Gravel Backfill for Drains” (WSDOT/APWA 9-03.12(4)) to a minimum thickness of 3 inches at the bottom and 12 inches above the drainpipe. The drainpipe and the drain rock should be encapsulated in a geotextile filter fabric such as Mirafi 140N, or equivalent. The perforated pipe should be laid with a longitudinal slope towards its discharging ends and finally be connected to a tightline or directed to approved discharge points. The drainpipe must not discharge at the toe of the wall. The details of the drain pipe is shown in Figure 3.

Surface Seal

A 12 inch thick surface seal should be placed above the Mirafi drainage mat to prevent migration of fines into the drainage mat. The surface seal may be consisted of topsoil layer.

Wall Toe Backfilling

The backfills to be placed in front of the wall footing embedment area should be compacted adequately to firm and unyielding conditions. The backfill placement and compaction should be as per the recommendations provided in Section 9.2.6, 'Fill Placement and Compaction Requirements' of this report.

Backfilling Behind Walls

The void areas in between the drainage layer behind the walls and the cut faces to be created during the excavation for housing the walls must be filled as per the guidelines provided in Section 9.2.6, 'Fill Placement and Compaction Requirements' of this report. The backfills must be used as per the recommendations provided earlier in Section 6.2.5, ‘Structural Fills’ of this report.

10.0 Geologic Hazards

10.1 Erosion Hazard

Uncontrolled surface water with runoff over unprotected site surfaces during construction activities is considered the single most important factor that impacts the erosion potential of a site. The erosion process may be accelerated significantly when factors such as soils with high fines, sloped surface, and wet weather combines together. Taking into consideration of the factor in this site such as the higher fines content (approximately 23%) in the near surface soils the project site is likely to experience some impact due to the erosion during the wet winter months.

The erosion hazard can be mitigated if the following measurements are implemented.

- Mass grading activities and the earthwork should be completed within the dry summer period.
- Measurements such as the control of surface water must be maintained during construction.
- Vegetation clearing must be kept very limited in this site to reduce the exposed surface areas. It is recommended that following the clearing of the vegetations, grading the open exposed areas should be covered with mulch or hydro seed.
- No disturbance or removal of the existing vegetations, tress, and undergrowths should be made beyond the vegetation clearing limit.
- Temporary erosion and sedimentary control (TESC) plan, as a part of the Best Management Practices (BMP) must be developed and implemented as well. The TESC plan should include the use of geotextile barriers (silt fences) along any down-slope, straw bales to de-energize downward flow, controlled surface grading, limited work areas, equipment washing, storm drain inlet protection, and sediment traps. The TESC plan may need to be reviewed and modified periodically to address the changing site conditions during ongoing progress of the construction and the weather.
- A permanent erosion control plan is to be implemented following the completion of the construction. Permanent erosion control measurements such as establishment of landscaping, replantation of trees and groundcover vegetations as soon as feasible in areas that are necessarily disturbed by earthwork activities, control of downspouts and surface drains, control of sheet flow, prevention of discharging water from the construction areas to the adjacent properties and streets are to be implemented following the completion of the construction.
- Install temporary or permanent tightline pipes, where necessary and practical, to convey

stormwater to approved discharge points.

10.2 Seismically Induced Geotechnical Hazard - Liquefaction

As part of the seismic evaluation of the site, the liquefaction potential of the site was also evaluated. Liquefaction is a phenomenon, which takes place due to the reduction or complete loss of soil strength due to increased pore water pressure during a major earthquake event. Liquefaction primarily affects geologically recent deposits of fine-grained sands that are below the groundwater table. Based on the soil and groundwater conditions, it is our opinion that the on-site soils are not prone to liquefaction; therefore, potential for widespread liquefaction and its associated hazards over the site during a seismic event is none. Therefore, subsurface conditions do not warrant additional mitigation techniques relating to liquefaction hazards.

10.3 Landslide Hazard

Based on the level flat ground condition in the proposed residence area, with minor downgrade slopes with gradients of approximately 11%, and the presence of very dense till, the landslide hazard potential in this site is considered as nil.

11.0 Additional Services

Additional services described below can be performed by PGE in the event the project requires such services. These services will be performed upon written authorization of the client or the civil engineer, and with additional cost to perform such services, under a separate contract between PGE and the client.

11.1 Design Phase Engineering Services

As the geotechnical engineer of record for the proposed development, at owner’s option, PGE can perform a review of the final project plans and specifications to verify that the geotechnical recommendations of this report have been properly interpreted and incorporated into the project final design and specifications, and that the impact of the final site grades, the proposed building and its footing, and any other structure.

11.2 Construction-time Testing and Inspection Services

As the geotechnical engineer of record for the proposed development, at owner's option, PGE can provide geotechnical consultation, material testing, and construction monitoring services during the construction of the project described earlier in Section 9.3, Construction Monitoring of this report. These services are important for the project to confirm that the earthwork and the general site development are in compliance with the general intent of design concepts, specifications, and the geotechnical recommendations presented in this report. Also, participation of PGE during the construction will help PGE engineers to make on-site engineering decisions in the event that any variations in subsurface conditions are encountered or any revisions in design and plan are made.

PGE can assist the owner before construction begins to develop an appropriate monitoring and testing plan to aid in accomplishing a fast and cost-effective construction process.

12.0 Geotechnical Special Inspections

The construction of the proposed development in this site involves several aspects of the geotechnical engineering that are considered to be critical for the successful completion of the project and continue that throughout the project life. Therefore, PGE recommends that the following geotechnical special inspection services to be performed during the construction of the proposed development. According to PGE, the following items should be considered as a minimum but not limited to.

- A professional geotechnical engineer should be retained to provide geotechnical consultation, material testing, and construction monitoring services during the construction of the project.
- A pre-construction meeting should be held on-site to discuss the geotechnical aspects of the development and the special inspection services to be performed during the construction.
- The site preparation activities including but not limited to stripping, cut and filling, final subgrade preparation for foundation, floor slab, paved driveway, and retaining wall be monitored by a geotechnical engineer or his representative under the engineer's supervision.
- A list of the possible items that require special geotechnical inspection and approval by the geotechnical engineer is as follows:

- Stripping of topsoils.
- Removal of unsuitable, existing fills.
- Compaction and proofrolling of any exposed native subgrades that are intended to provide direct support for any load bearing structure such as new fill pad, slab-on-grade floor, footing, retaining wall, and paved driveway.
- Any structural fills to be used in this site, and structural fills placement and its compaction.
- Temporary or permanent excavation inclinations and excavation stability.
- Backfilling and its compaction, and drainage behind retaining walls.
- The footing bearing materials, bearing capacity value, and the embedment depth of the footings prior to placing forms and rebars.
- Subgrade preparation for soil supported slab-on-grade floor.
- Subgrade preparation for paved driveways.
- Compaction of CSBC, CSTC, and laying of concrete pavement in driveway.
- Site drainage.
- Installation of drainage systems such as footing excavation drain and footing drain, and daylighting of such drains and downspout or roof drains.
- Bedding and the backfilling materials, and backfilling of utility lines.
- Buffer distances from the vegetation clearing limit and the vegetation clearing limit.
- Any other items specified in the approved project plans to be prepared by other consultants relevant to the geotechnical aspect of the project.

13.0 Report Limitations

The conclusions and recommendations presented in this report are based on our soil investigation, the laboratory test results, geological literature review, and our engineering evaluation. The study was performed using a mutually agreed-upon scope of work between PGE and the client.

It should be noted that PGE cannot take the responsibility regarding the accuracy of the information provided in the project plan prepared by other consultants. If any of the information considered during this study is not correct or if there are any revisions to the plans for this project, PGE should be notified immediately of such information and the revisions so that necessary amendment of our geotechnical recommendations can be made. If such information and revisions are not notified to PGE, no responsibility should be implied on PGE for the impact of such information and the revisions on the project.

Variations in subsurface (soil and groundwater) conditions may reveal during the construction of the proposed below grade infiltration system. The nature and the extent of the subsurface variations may not be evident until construction occurs. If any subsurface 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 if there are any changes in the project scope.

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 others 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. PGE should be notified if the project is delayed by more than 24 months from the date of this report so that we may review to determine that the conclusions and recommendations of this report remain applicable to the changed conditions.

The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' method, 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 including its evaluation, conclusions, specifications, recommendations, or professional advice 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 is the property of our client Hien Phan, and has been prepared for the exclusive use of our client and its authorized representatives for the specific application to the proposed development at the subject site in Mercer Island, Washington.

It is the client's responsibility to see that all parties to this project, including the civil engineer, designer, contractor, subcontractor, future homeowner, 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 PGE of such intended use and for permission to copy this report. Based on the intended use of the report, PGE may require that additional work be performed and that and updated report be reissued. Noncompliance with any of these requirements will release PGE from any liability resulting from the use this report.

14.0 Closure

We trust the information presented in this report is sufficient for your current needs. We appreciate the opportunity to provide the geotechnical services at this phase of the project and look forward to continued participation during the design and construction phase of this project. Should you have any questions or concerns, which have not been addressed, or if we may be of additional assistance, please do not hesitate to call us at 425-218-9316 or 425-643-2616.

Respectfully submitted,

Santanu Mowar, P.E.

SANTANU MOWAR
STATE OF WASHINGTON
37403
REGISTERED PROFESSIONAL ENGINEER
EXPIRES 01-01-2024

Santanu Mowar.

PGE Pacific Geo Engineering
Geotechnical Engineering, Consulting & Inspection

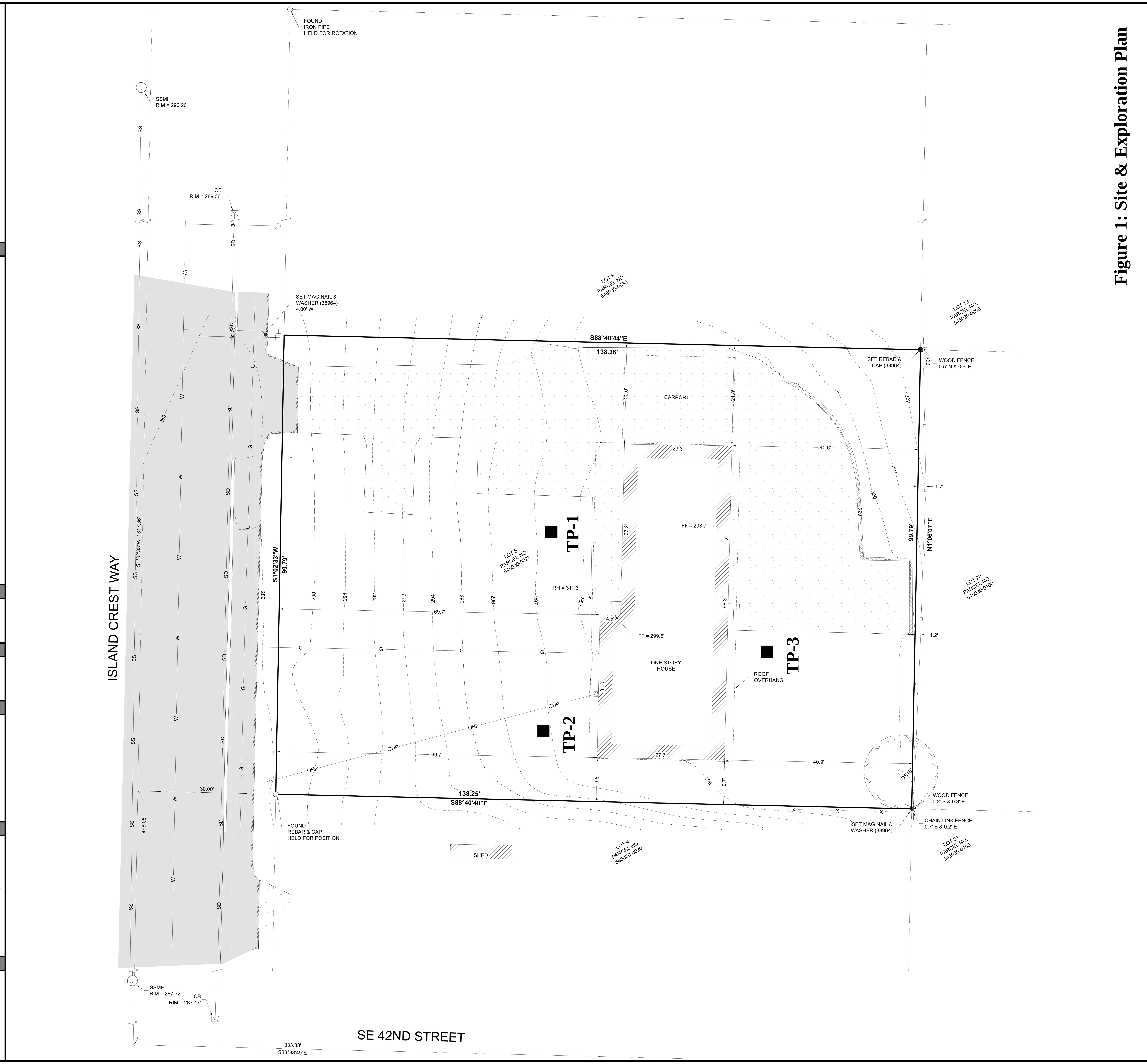
D:\Geotechnical\2022-proj\22-631

Attachments:

Figure 1	Site & Exploration Plan
Figure 2	Lateral Earth Pressure Diagrams
Figure 3	Basement Retaining Wall Details
Figure 4	Basement Retaining Wall Construction Notes
Appendix A	Soil Test Pit Logs
Appendix B	Laboratory Test Report – B-1

Figure 1: Site & Exploration Plan

NW 1/4, NW 1/4, SEC 18, TWP 24N, RNG 5E, W.M.



GRAPHIC SCALE

5 0 5 10 20

1 INCH = 10 FT.

LEGEND

○	FOUND REBAR AS DESCRIBED	— OHP —	OVERHEAD POWER
	SET MAG NAIL AS DESCRIBED		LIGHT POLE
●	SET 5/8" X 24" IRON ROD W/1" YELLOW PLASTIC CAP	— X —	CHAINLINK FENCE
	POWER METER	— □ —	WOOD FENCE
	GAS METER		CONCRETE WALL
	UTILITY POLE		ASPHALT SURFACE
	CATCH BASIN		CONCRETE SURFACE
	SANITARY SEWER MANHOLE	DS	DECIDUOUS
	FIRE HYDRANT		* INDICATES MULTI-TRUNK
	WATER METER		
— SS —	APPROXIMATE LOCATION SANITARY SEWER LINE		
— SD —	APPROXIMATE LOCATION STORM DRAIN LINE		
— G —	APPROXIMATE LOCATION UNDERGROUND GAS LINE		
— W —	APPROXIMATE LOCATION UNDERGROUND WATER LINE		

LEGAL DESCRIPTION

LOT 5, BLOCK A, MERCER CREST, ACCORDING TO THE PLAT THEREOF RECORDED IN VOLUME 42 OF PLATS, PAGE 26, RECORDS OF KING COUNTY, WASHINGTON;

SITUATE IN THE CITY OF MERCER ISLAND, COUNTY OF KING, STATE OF WASHINGTON.

BASIS OF BEARINGS

ACCEPTED A BEARING OF N01°02'33"E FOR THE CENTERLINE OF ISLAND CREST WAY BASED ON FOUND MONUMENTS.
RECORD OF SURVEY BY TERRANE FOR JULIE ROSS, RECORDED ON JULY 21, 2022 IN VOLUME 469 OF SURVEYS, PAGES 220 AND 221, UNDER RECORDING NO. 20220721900022, RECORDS OF KING COUNTY, WASHINGTON.

PROJECT INFORMATION

PROPERTY OWNER:	THANH-HIEN THI PHAN 4102 ISLAND CREST WAY MERCER ISLAND, WA 98040
TAX PARCEL NUMBER:	545030-0025
PROJECT ADDRESS:	4102 ISLAND CREST WAY MERCER ISLAND, WA 98040
ZONING:	R-9.6
JURISDICTION:	CITY OF MERCER ISLAND
PARCEL ACREAGE:	13,801 S.F. (0.317 ACRES) AS SURVEYED

GENERAL NOTES

1. THIS SURVEY WAS COMPLETED WITHOUT BENEFIT OF A CURRENT TITLE REPORT. EASEMENTS AND OTHER ENCUMBRANCES MAY EXIST ON THIS PROPERTY THAT ARE NOT SHOWN HEREON.
2. INSTRUMENTATION FOR THIS SURVEY WAS A 3-SECOND SPECTRAPRECISION FOCUS 35 TOTAL STATION AND AN EMLID REACH RS2 GPS RECEIVER. PROCEDURES USED IN THIS SURVEY MEET OR EXCEED STANDARDS SET BY WAC 332-130-090.
3. THE INFORMATION ON THIS MAP REPRESENTS THE RESULTS OF A SURVEY MADE IN AUGUST 2022 AND CAN ONLY BE CONSIDERED AS INDICATING THE GENERAL CONDITIONS EXISTING AT THAT TIME.
4. UTILITIES SHOWN ON THIS SURVEY ARE BASED UPON ABOVE GROUND OBSERVATIONS AND AS-BUILT PLANS WHERE AVAILABLE. ACTUAL LOCATIONS OF UNDERGROUND UTILITIES MAY VARY AND UTILITIES NOT SHOWN ON THIS SURVEY MAY EXIST ON THIS SITE.
5. ALL MONUMENTS WERE LOCATED DURING THIS SURVEY UNLESS OTHERWISE NOTED.

VERTICAL DATUM & CONTOUR INTERVAL

DATUM

ELEVATIONS SHOWN ON THIS DRAWING WERE DERIVED FROM GPS OBSERVATION USING THE WSRN.

DATUM - NAVD 88

1.0' CONTOUR INTERVAL - THE EXPECTED VERTICAL ACCURACY IS EQUAL TO 1/2 THE CONTOUR INTERVAL OR PLUS / MINUS 0.5' FOR THIS PROJECT.

Site Surveying, Inc.
21923 NE 11th Street Sammamish, WA 98074
Phone: 425.298.4412
www.sitesurveymapping.com

DRN | REVISION | DATE

TOPOGRAPHIC SURVEY

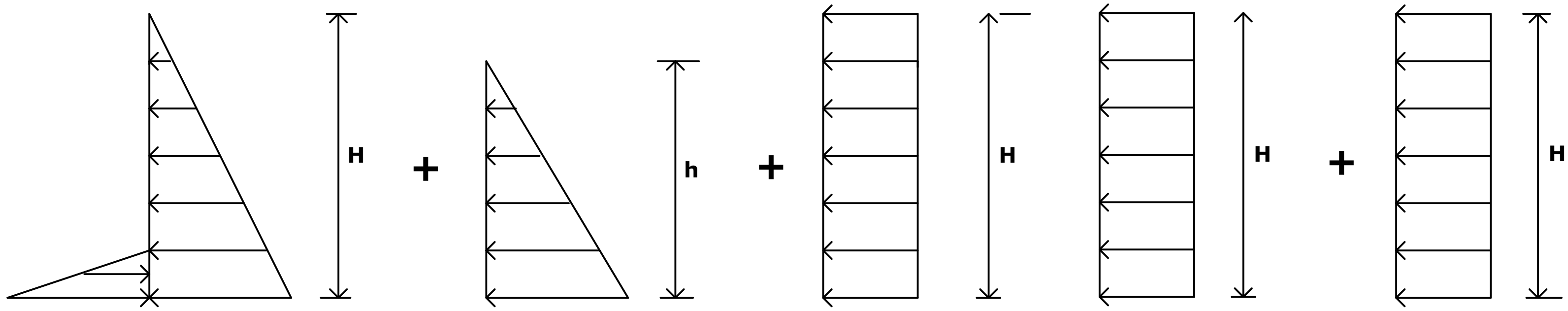
HIEN PHAN - SOUTHERN BIRCH MORTGAGE
4102 ISLAND CREST WAY
MERCER ISLAND, WA 98040

PROJECT NO. 22-392

DRAWN BY: MTS
CHECKED BY: TNW
DATE: 8/18/2022

SHEET 1 OF 1

AT-REST CONDITION DESIGN PARAMETERS & LATERAL PRESSURE DIAGRAMS - FIXED TOP CONCRETE RETAINING WALL – Basement Level



Lateral passive earth pressure = 300 pcf for static condition and 225 pcf for seismic condition. A FS of 1.5 is included in the above values. Passive pressure is to be neglected for upper 1 ft of footing embedment depth

'At-rest' condition lateral earth pressure = 55 pcf for level backhill slope above wall

Hydrostatic pressure = 62.5h psf, only if no drainage layer

Seismic loading = 8H psf for level bckhill slope above wall

Surcharge loading due to backhill slope above wall = (Slope height/2)* $\gamma * K_a$ psf, if only applicable

Lateral earth pressure for traffic surcharge = 0.44q psf, if only applicable

Note:-

1. The appropriate combination of the various loading components to be exerted on the wall are to be decided by the project structural engineer for the design of the retaining wall; factors such as the wall's condition (cantilever or at-rest), structural loading system, external loading to be exerted on the wall (surcharge loading and traffic loading), wall location and height, and drainage layer behind the wall should be considered.

2. The seismic loading is to be considered for the wall design.

Design Considerations:-

1. Wall is at-rest condition, i.e., the wall is restrained against rotation at top or fixed at top.

2. The seismic pressures are based on a design acceleration coefficient of 0.30g.

3. The static passive pressure is reduced by 25% for seismic condition.

4. The static bearing capacity is increased by`⅓ for the seismic condition.

Wall Design Parameters:-

'H' is the soil retained height of the wall
'h' is the height of the water table above the bottom of 'H'
Retained soil unit wt., γtotal = 125 pcf
Unit wt. of water, γw = 62.5 pcf
Retained soil friction angle, ø = 34º
Co-eff. of at-rest earth pressure, Ko = 0.44
Co-eff. of passive earth pressure, Kp = 3.6
Base friction co-eff. µ = 0.35
Static bearing capacity = 3000 psf (competent glacial till)
Seismic bearing capacity = 4000 psf (static value is increased by by 1/3rd)
Traffic surcharge, q = 250 psf

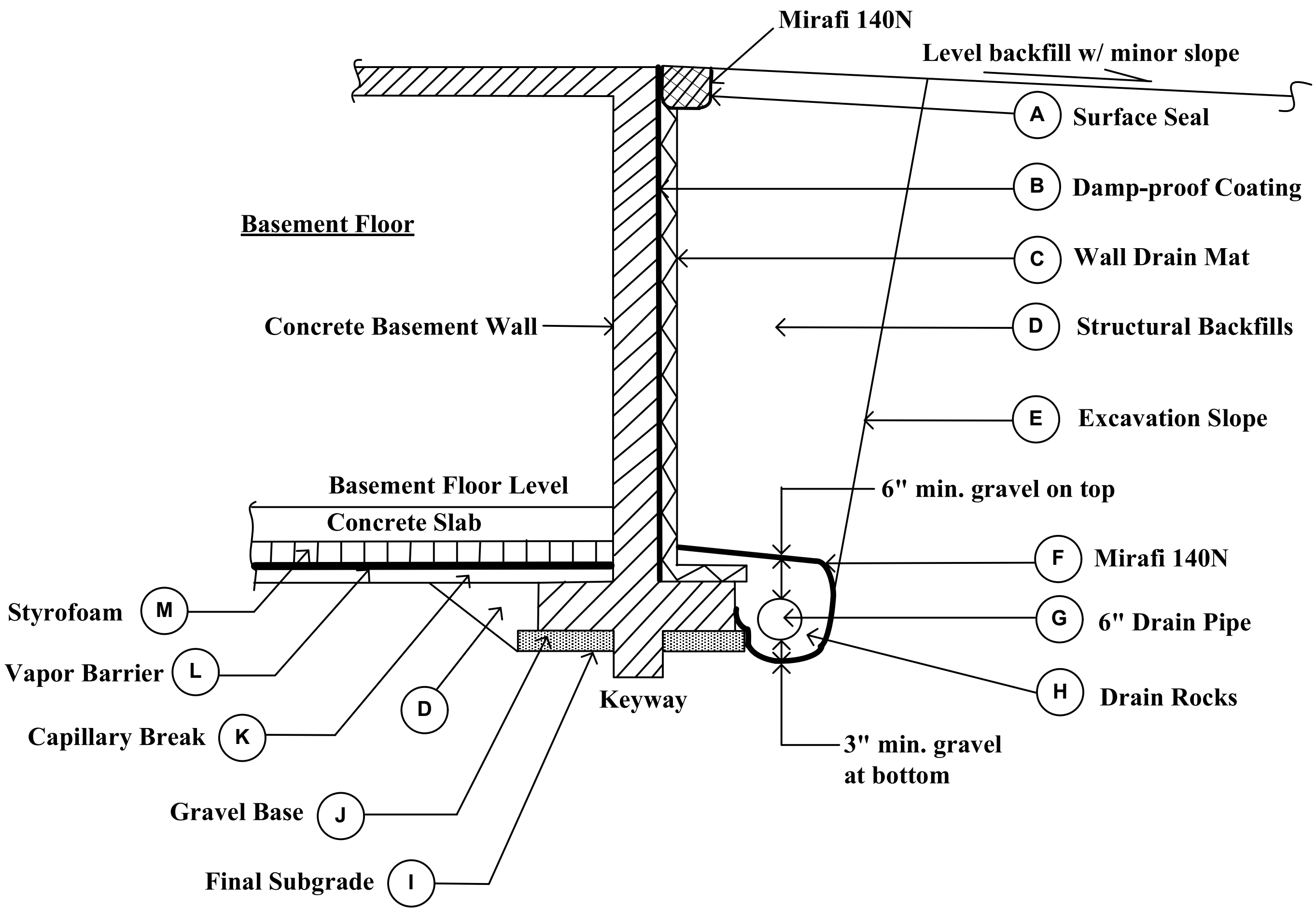
Figure 2 Not to Scale

Project – Hien Phan Residence
4102, Island Crest Way; Mercer Island; WA

Project No. – 22-631

PGE Pacific Geo Engineering

BASEMENT WALL FOOTING DRAIN
(Schematic Only)



NOTES:-

(A) 12" thk. surface seal of low permeability soil or top soil, to be wrapped with Mirafi 140 N

(B) Bituminous or Polymer Damp-proof Coating

(C) Wall Drain Mat Mirafi G100N or Equiv. When the below-grade foundation wall 4 ft below grade

(D) Backfills in the void areas between the excavation face & wall drain, & at wall toe must be compacted to 95% of fills' Mod. Proc. Max. dry density value as per ASTM D 1557. Fills must not contain any organic & other deleterious materials. Fills must be placed in 12 inch thk. individually compacted lifts

(E) Possible excavation face slope to be determined based on actual soil and groundwater conditions to be revealed during the construction

(F) Mirafi 140 N must wrap around drain rocks

(G) 6" dia. rigid PVC pipe w/perforations (1/4" max. dia.) to be in the lower half of pipe, & lower quadrant segment un-perforated to facilitate flow of water. The pipe must be placed as low as possible and the pipe should be sloped towards approved discharge point so that no backflow should occur into the pipe.

(H) The pipe must be enveloped by drain rocks consisted of ¾" minus washed gravel (free draining)

(I) Final subgrade must be adequately proofrolled to firm & unyielding conditions; the allowable bearing capacity of 2500 psf to be verified on-site by geotechnical engineer prior to the placing of rebars and forms

(J) Min. 6" thk compacted gravel base, which must be extended 6" beyond both sides of the footing as shown

(K) Capillary Break layer – min. 4" thk, of free-draining 5/8-inch crushed rocks containing no more than 2% fines. Slab-on-grade floor should be placed directly on a capillary break layer in unheated areas e.g., garage, storage rooms

(L) Vapor Barrier – a durable 6-mil. (0.006 inch) plastic membrane be placed over capillary break layer as a vapor retarder

(M) Styrofoam, as an additional layer can be placed between the slab and the capillary break layer where heated areas for provision of better insulation

Figure 3 Not to Scale

Project – Hien Phan Residence
4102, Island Crest Way, Mercer Island, WA

Project No. – 22-631

PGE Pacific Geo Engineering
Geotechnical Engineering, Consulting & Inspection

CONCRETE RETAINING WALL CONSTRUCTION NOTES

NOTES:-

1. A min. footing embedment depth of 2 ft. is required below the final grade at the toe of the wall. However, greater embedment depth may be necessary to achieve adequate passive resistance against sliding, based on the structural engineer's wall design requirement.

2. If any void area behind the wall is created between the drainage layer and the cut face and at the toe of the wall during the excavation of the wall construction then such void area must be backfilled with approved structural fills, to be compacted to 95% of fills' max. dry density value, which should be determined as per the laboratory Mod. Proctor Test ASTM D1557.

3. The void area backfilling behind the retaining wall should be compacted with care within the horizontal distance equal to the height of the retained soil height to avoid over compaction and hence overstressing the wall. No heavy compaction equipment such as vibratory roller or hoe-pac be used to compact the fills because of these equipment will impose excess surcharge loading on the wall, which may cause a lateral instability to the wall. A walk-behind big vibratory plate compactor should be used to compact the fills behind the wall. See section 9.2.6 of PGE's geotechnical report 21-631.

4. For actual dimensions of the wall such as the wall height & thickness, footing size & thickness, footing embedment depth, keyway size & depth, reinforcement details, concrete grade and strength, concrete mix design, and any other wall details should be designed by the project structural engineer.

5. The wall must be built as per the recommendations provided in Figure 3, and section 9.8 of PGE's geotechnical report 22-631, and the wall design details to be prepared by the project structural engineer.

Construction Inspection

The proposed concrete wall require engineering supervision by a special inspector as per the building code. Items such as footing subgrades preparation and proofrolling, crushed rock base layer laying and compaction, footing embedment depth, allowable bearing capacity value, fill placement and compaction at the toe and back of the wall, drainage layer, geosynthetic filter fabric & its laying, footing drainpipe, encapsuling of footing drainpipe, keyway preparation, swale, size, thickness, and height of the wall and their reinforcement details, concrete mix design and compressive strength, and any other components of the wall should be verified on-site by a special inspector.

Figure 4 Not to Scale

Project – Hien Phan Residence
4102, Island Crest Way; Mercer Island; WA

Project No. – 22-631

PGE Pacific Geo Engineering
Geotechnical Engineering, Consulting & Inspection

Appendix A

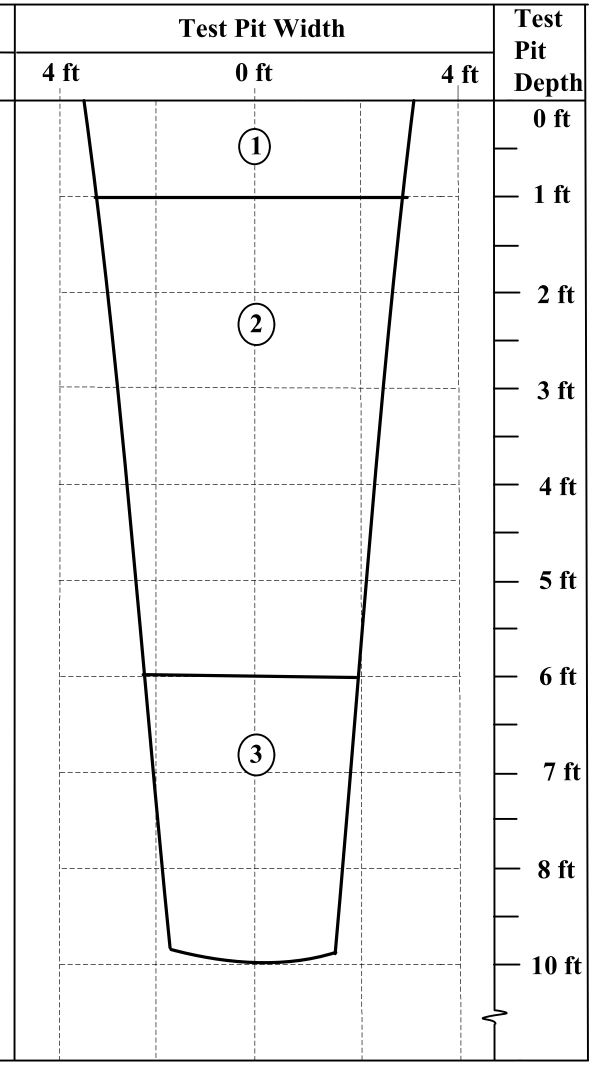
Soil Test Pit Log

TEST PIT - 1

Date of Excavation	08/25/2022

Surface Elev. Ft.	

Soil Layer Depth	Soil Layer Descriptions	USCS Soil Class	Sample Nos.	Sample Depth	Laboratory Test Results: Moist. Content	Laboratory Test Results: - #200 Sieve
0 – 1 ft	1 Top Soil - Brn., Silt w/ Roots & Organics, Sl. Moist, Loose					
1 ft – 6 ft	2 Native Fills: Lt. Brn., Silty Sand w/ Gravel, & brick bats & decayed wood debris; Moist, Loose at shallow depth & Med. Dense at grater depth; Minor, scattered signs of iron-oxidized stains					
6 ft – 10 ft	3 Lt. Gray, Silty Sand w/ sub-rounded to well rounded Gravel, & Occasional Cobble (Glacial Till); Moist; V. Dense; Weakly cemented chunks; Difficult digging through till	SM	S-1	@ 8 ft	13.2 %	23.3 % (Graph B-1)



Notes -			
Test Pit Location	See site plan	Mottling Depth	Minor, scattered signs in fills
Ground Cover	Grass	Water/Seepage Depth	None
Test Pit Depth	10 ft	Cave in Depth	None
Permeability			

Figure A-1 Not to Scale

Project – Hien Phan Residence
4102, Island Crest Way, Mercer Island, WA
Project No. – 22-631

PGE Pacific Geo Engineering

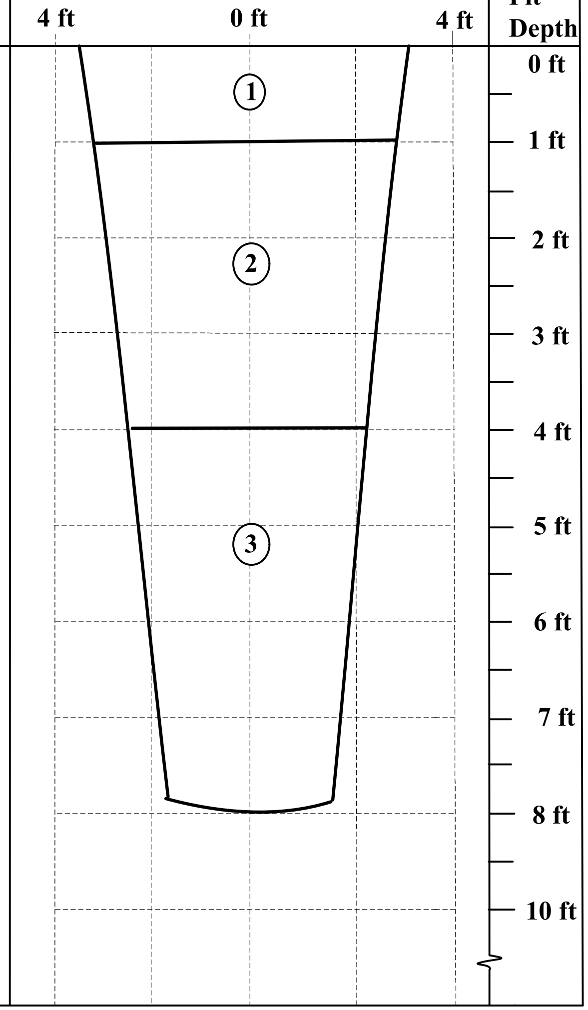
TEST PIT - 2

Date of Excavation	08/25/2022

Surface Elev. Ft.	

Soil Layer Depth	Soil Layer Descriptions	USCS Soil Class	Sample Nos.	Sample Depth	Laboratory Test Results: Moist. Content	Laboratory Test Results: - #200 Sieve
0 – 1 ft	1 Top Soil - Brn., Silt w/ Roots & Organics, Sl. Moist, Loose					
1 ft – 4 ft	2 Native Fills: Lt. Brn., Silty Sand w/ Gravel; Moist, Loose at shallow depth & Med. Dense at grater depth; Minor, scattered signs of iron-oxidized stains					
4 ft – 8 ft	3 Lt. Gray, Silty Sand w/ sub-rounded to well rounded Gravel, & Occasional Cobble (Glacial Till); Moist; V. Dense; Weakly cemented chunks; Difficult digging through till	SM	S-1	@ 6 ft	12.8 %	

Test Pit Width



Notes -			
Test Pit Location	See site plan	**Mottling Depth**	Minor, scattered signs in fills
Ground Cover	Grass	**Water/Seepage Depth**	None
Test Pit Depth	8 ft	**Cave in Depth**	None
Permeability			

Figure A-2 Not to Scale

Project – Hien Phan Residence
4102, Island Crest Way, Mercer Island, WA

Project No. – 22-631

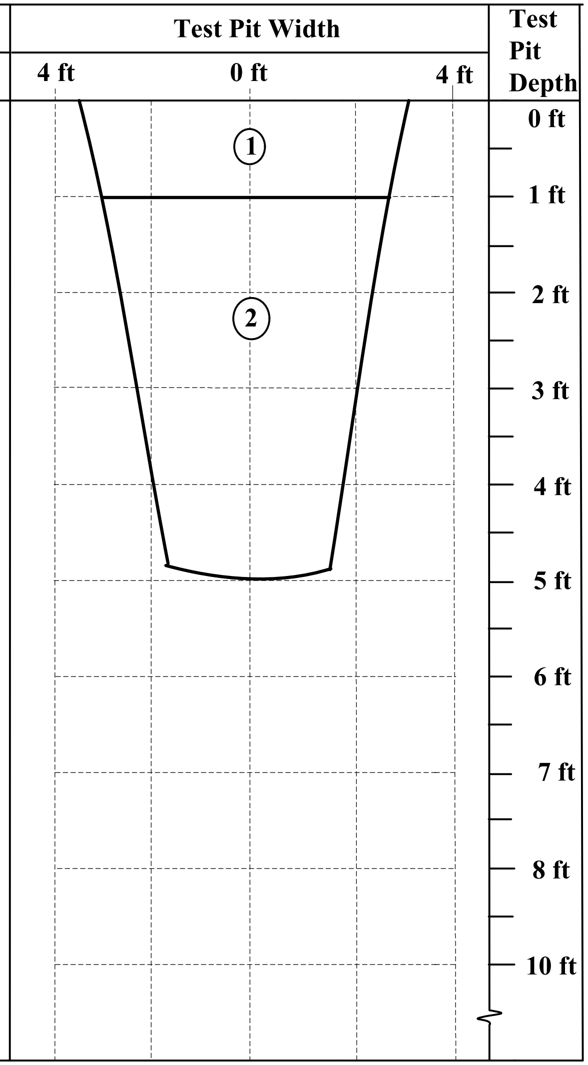
PGE Pacific Geo Engineering
Geotechnical Engineering, Consulting & Inspection

TEST PIT - 3

Date of Excavation	08/25/2022

Surface Elev. Ft.	

Soil Layer Depth	Soil Layer Descriptions	USCS Soil Class	Sample Nos.	Sample Depth	Laboratory Test Results: Moist. Content	Laboratory Test Results: - #200 Sieve
0 – 1 ft	1 Top Soil - Brn., Silt w/ Roots & Organics, Sl. Moist, Loose					
1 ft – 5 ft	2 Lt. Gray, Silty Sand w/ sub-rounded to well rounded Gravel, & Occasional Cobble (Glacial Till); Sl. Moist; Med. Dense upper 1 feet & V. Dense below 2 feet; Weakly cemented chunks; Difficult digging through till	SM	S-1	@ 4 ft	5.6 %	



Notes -			
Test Pit Location	See site plan	Mottling Depth	None
Ground Cover	Grass	Water/Seepage Depth	None
Test Pit Depth	5 ft	Cave in Depth	None
Permeability			

Figure A-3 Not to Scale

Project – Hien Phan Residence
4102, Island Crest Way, Mercer Island, WA
Project No. – 22-631

PGE Pacific Geo Engineering
Geotechnical Engineering, Consulting & Inspection

KEY TO EXPLORATION LOG

Sample Descriptions:

Classification of soils in this report is based on visual field and laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates, and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual classification methods in accordance with ASTM D-2488-17 were used as an identification guide. Where laboratory data available, soil classifications are in general accordance with ASTM D2487-17. Soil density/consistency in borings is related primarily to the Standard Penetration Resistance values. Soil density/consistency in test pits is estimated based on visual observations of excavations. Undrained shear strength = ½ unconfined compression strength.

RELATIVE DENSITY OR CONSITENCY VS. SPT N-VALUE					
COARSE GRAINED SOILS: SAND OR GRAVEL			**FINE GRAINED SOILS: SILT OR CLAY**		
Density	N (Blows/ft.)	Approx. Relative Density (%)	Consistency	N (Blows/ft.)	Approx. Undrained Shear Strength (psf)
Very Loose	0 – 4	0- 15	Very Soft	0 – 2	<250
Loose	4 – 10	15 – 35	Soft	2 – 4	250 –500
Medium Dense	10 – 30	35 – 65	Medium Stiff	4 – 8	500 – 1000
Dense	30 – 50	65 – 85	Stiff	8 – 15	1000 – 2000
Very Dense	>50	85 – 100	Very Stiff Hard	15 – 30 > 50	2000 – 4000 > 4000

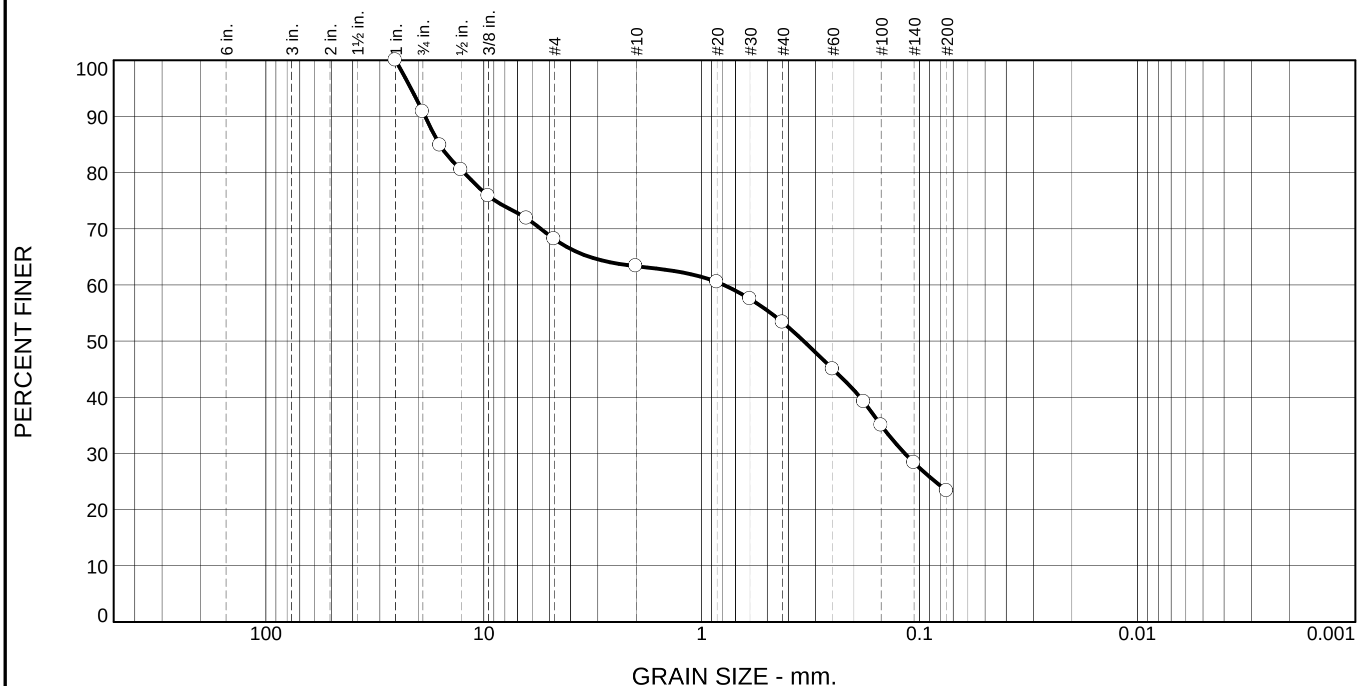
MOISTURE CONTENT DEFINITIONS	
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

DESCRIPTIONS FOR SOIL STRATA AND STRUCTURE					
General Thickness or Spacing		**Structure**		**General Attitude**	
Parting	< 1/16 in	Pocket	Erratic, discontinuous deposit of limited extent	Near Horizontal	0 - 10 deg
Seam	1/16 - 1/2 in	Lens	Lenticular deposit	Low Angle	10 - 45 deg
Layer	½ - 12 in	Varved	Alternating seams of silt and clay	High Angle	45 - 80 deg
Stratum	> 12 in	Laminated	Alternating seams	Near Vertical	80 - 90 deg
Scattered	< 1 per ft	Interbedded	Alternating Layers		
Numerous	> 1 per ft	Fractured	Breaks easily along definite fractured planes		
		Slickensided	Polished, glossy, fractured planes		
		Blocky, Diced	Breaks easily into small angular lumps		
		Sheared	Disturbed texture, mix of strengths		
		Homogeneous	Same color and appearance throughout		

Appendix B

Laboratory Test Reports

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	9.2	22.6	4.9	10.0	30.0	23.3	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1 in	100.0		
3/4 in	90.8		
5/8 in	84.8		
1/2 in	80.5		
3/8 in	75.8		
1/4 in	71.8		
#4	68.2		
#10	63.3		
# 20	60.5		
#30	57.5		
#40	53.3		
#60	45.0		
#80	39.2		
#100	35.0		
#140	28.3		
#200	23.3		

* (no specification provided)

Material Description

Lt. Gray Silty Sand w/ Gravel

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-2-4(0)

Coefficients

D_{90}= 18.5996 D_{85}= 15.9720 D_{60}= 0.7920

D_{50}= 0.3403 D_{30}= 0.1169 D_{15}=

D_{10}= C_u= C_c=

Remarks

Date Received: 08-25-22 **Date Tested:** 08-29-22

Tested By: Chris Moore

Checked By: Santanu Mowar, PE

Title: Pricipal

Location: TP-1

Sample Number: S-1 **Depth:** @ 8 ft

Date Sampled: 08-25-22

Pacific Geo Engineering, LLC

Geotechnical Engineering, Consultation, Testing, & Inspection

Client: Southern Birch Mortgage

Project: Hien Phan Residence

Project No: 22-631 **Figure** B-1

PGE Pacific Geo Engineering LLC

Geotechnical Engineering, Consulting & Inspection

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification	
				Group Symbol	Group Name[B]
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[E]	GW	Well-graded gravel[F]
			$Cu < 4$ and/or $1 > Cc > 3$[E]	GP	Poorly graded gravel[F]
		Gravels with Fines More than 12% fines[C]	Fines classify as ML or MH	GM	Silty gravel[F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel[F, G, H]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[E]	$Cu \geq 6$ and $1 \leq Cc \leq 3$[E]	SW	Well-graded sand[I]
			$Cu < 6$ and/or $1 > Cc > 3$[E]	SP	Poorly graded sand[I]
		Sands with Fines More than 12% fines[D]	Fines classify as ML or MH	SM	Silty sand[G, H, I]
			Fines classify as CL or CH	SC	Clayey sand[G, H, I]
Fine-Grained Soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line[J]	CL	Lean clay[K, L, M]
			PI < 4 or plots below "A" line[J]	ML	Silt[K, L, M]
		organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$	OL	Organic clay[K, L, M, N] / Organic silt[K, L, M, O]
	Silts and Clays Liquid limit 50 or more	inorganic	PI plots on or above "A" line	CH	Fat clay[K, L, M]
			PI plots below "A" line	MH	Elastic silt[K, L, M]
		organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$	OH	Organic clay[K, L, M, P] / Organic silt[K, L, M, Q]
Highly organic soils		Primarily organic matter, dark in color, and organic odor		PT	Peat

[A]Based on the material passing the 3-in. (75-mm) sieve.

[B]If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C]Gravels with 5 to 12% fines require dual symbols:
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay

[D]Sands with 5 to 12% fines require dual symbols:
SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay

[E]$Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F]If soil contains ≥ 15% sand, add "with sand" to group name.

[G]If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H]If fines are organic, add "with organic fines" to group name.

[I]If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J]If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K]If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel", whichever is predominant.

[L]If soil contains ≥ 30% plus. No. 200 predominantly sand, add "sandy" to group name.

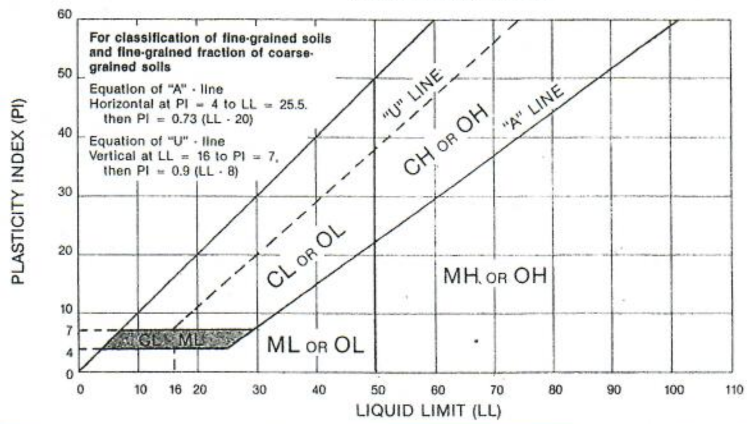
[M]If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

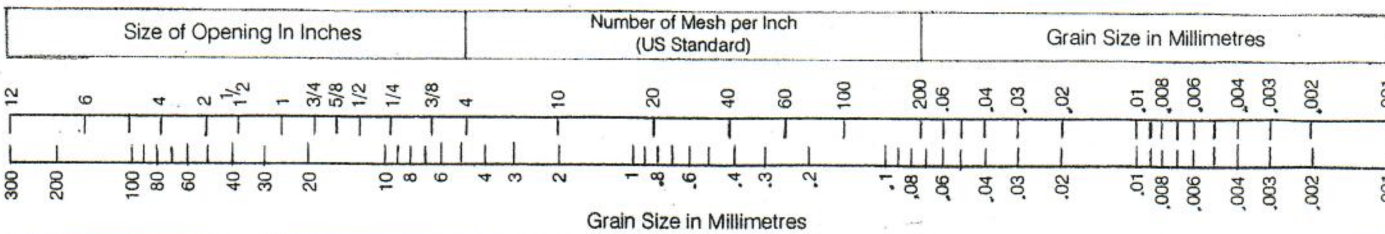
[N]PI ≥ 4 and plots on or above "A" line.

[O]PI < 4 or plots below "A" line.

[P]PI plots on or above "A" line.

[Q]PI plots below "A" line.





Cobble	Gravel		Sand			Silt and/or Clay
	Coarse	Fine	Coarse	Medium	Fine	