PanGEO
INCORPORATED
Geotechnical & Earthquake
Engineering Consultants

October 12, 2018
PanGEO Project No. 18-282

David and Jaymee Lundin
4041 West Mercer Way
Mercer Island, WA 98040

Subject: **Geotechnical Engineering Report**
Proposed Residence
4041 West Mercer Way, Mercer Island, Washington 98040

Dear David and Jaymee,

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

In summary, the proposed house footprint is underlain by a surficial layer of loose sand, overlying very stiff to hard clay. The depth to the top of the clay generally increases from east (upslope) to west (downslope). In our opinion, the new structure may be supported by a conventional footings. Over-excavations, particularly along the downslope side of the house, may be needed to remove unsuitable soils from below the footings. A soldier pile wall represents a feasible excavation support system to allow for the construction of the proposed house basement while maintaining stability of the site.

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

Sincerely,

Bryce C. Townsend P.E.
Staff Geotechnical Engineer

3213 Eastlake Avenue East, Suite B
Seattle, WA 98102
Tel: (206) 262-0370

TABLE OF CONTENTS

GEOTECHNICAL ENGINEERING REPORT
PROPOSED RESIDENCE
4041 WEST MERCER WAY
MERCER ISLAND, WASHINGTON

1.0 GENERAL

PanGEO, Inc. is pleased to present the following geotechnical engineering report to assist the project team with the design of the proposed residence at 4041 West Mercer Way, in Mercer Island, Washington. This study was prepared in general accordance with our mutually agreed scope of services outlined in our proposal dated August 22, 2018, which was approved on August 24, 2018. Our scope of services included reviewing readily available geologic and geotechnical data, conducting a site reconnaissance, advancing test borings at the site, conducting engineering analyses, and preparing the following geotechnical report.

2.0 SITE AND PROJECT DESCRIPTION

The subject site is located at 4041 West Mercer Way, in Mercer Island, Washington, as shown on Figure 1, Vicinity Map. The site consists of an irregularly shaped parcel of waterfront property situated on Lake Washington. The property is bordered by single-family homes to the north and south, the lower portion of a joint-use paved driveway and several single-family homes to the east, and Lake Washington to the west. The property extends about 150 feet upslope along the north property line and about 225 feet upslope along the south property line.

The site is currently vacant and is accessed by a joint-use paved driveway that is accessed from West Mercer Way. The paved driveway is about 470 feet long from West Mercer Way to the east side of the proposed building development area with one switch-back for the west 250 feet of the driveway. The site property includes the sloped area between the switchback sections of the driveway which is generally uniform in slope angle. The downslope side of the lower section of the paved driveway switchback is supported by soldier pile wall with a concrete facing (see Plate 2 below). The wall is about 20 feet tall at its highest.

The majority of the site is covered by grass with few small trees. There is an existing 4- to 5-foot tall rockery bulkhead along the shoreline. There is a dirt access driveway that descends south to north form the paved access driveway which slopes down from the southeast proposed development area to the

West of the existing driveway soldier pile wall, the topography of the site slopes down moderately with slopes becoming more level towards the west rockery bulkhead along the shoreline. From the northeast corner of the property to the northwest corner of the property there is a total vertical relief of about 77 feet across about 150 feet. The west 50 feet of the property grades with slopes between 10 to 40 percent with the site slopes on the east side of the site generally at or above 40 percent, with the exception of the paved driveway and retaining structures supporting it. The sloped area between the paved driveway switchback has a generally uniform slope of about 50 percent with a vertical relief of about 70 feet at the maximum along the south property line.

Plate 1 below depicts current site conditions. Plate 2 below depicts the current side conditions for the existing soldier pile wall for the driveway approach.

Plate 1. *View of the site from the top of the dirt access driveway at the southeast corner of the proposed development, looking northwest.*

Plate 2. *View of the existing soldier pile wall for the driveway approach, looking northeast.*

We understand that the proposed project includes the construction of a two-level single-family residence with a basement in the west half of the property between the existing soldier pile wall and shoreline. A new private driveway will be constructed between the proposed house and the existing soldier pile wall. Based on our current understanding of the project, the basement finished floor elevation will be between 20½ and 22½ feet. The excavation necessary to construct the basement will extend up to about 20 feet below existing grades along the east side of the proposed development, becoming shallower as the site grades toward the shoreline. Figure 2 depicts the approximate location of the proposed house in relation to the property boundaries and existing site features.

We understand that a stairway is to be constructed generally near the south property line in the sloped area between the paved driveway switchback. Three additional parking spaces area also to be constructed on the upslope side of the bottom section of the paved access driveway.

The conclusions and recommendations outlined in this report are based on our understanding of the current development plans, which is in turn based on the project information provided. If the above project description is substantially different from your proposed improvements, or if the project scope changes, PanGEO should be consulted to review the recommendations contained in this study and make modifications, if needed.

3.0 SUBSURFACE EXPLORATIONS

3.1 PanGEO Borings

A subsurface exploration program was completed on September 20, 2018. The subsurface exploration program included drilling two test borings (PG-1 and PG-2) that were advanced near the shoreline. The approximate test boring locations were measured from existing site features and are indicated on the attached Site and Exploration Plan (Figure 2). The borings were drilled to depths of about 11½ and 16½ feet below the ground surface, respectively.

The test borings were drilled using a portable acker drill rig owned and operated by Boretec1 Inc. of Bellevue, Washington. The drill rig was equipped with a 4-inch outside diameter hollow stem auger, and soil samples were obtained from the borings at 2½ and 5-foot intervals in general accordance with Standard Penetration Test (SPT) sampling methods in accordance with ASTM D-1586, in which the samples are obtained using a 2-inch outside diameter split-spoon sampler. The sampler was driven into the soil a distance of 18 inches using a 140-pound weight falling a distance of 30 inches. The number of blows required for each 6-inch increment of sampler penetration was recorded. The number of blows required to achieve the last 12 inches of sample penetration is defined as the SPT N-value. The N-value provides an empirical measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils.

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

The soil samples retrieved from the borings were described using the system outlined on Figure A-1 of Appendix A and the summary boring logs are included as Figures A-2 through A-3.

3.2 Existing Subsurface Information

In addition to advancing two borings at the site, we reviewed the previous geotechnical report developed by Liu & Associates, Inc. (L&A), dated February 12, 2010. Two test borings (B-1 and B-2) were advanced by L&A to approximate depths of about 21½ and 20½ feet below the ground surface, respectfully. The approximate locations of the L&A borings are also shown on the attached Figure 2 and logs are included in Appendix B of this report for reference.

3.3 Laboratory Testing

Atterberg Limits and natural moisture contents tests were conducted on selected representative soil samples obtained from the borings. The test results from the moisture content tests are indicated at the appropriate depths on the boring logs. The Atterberg Limits test results are summarized on the logs and in Figure C-1 in Appendix C.

4.0 SUBSURFACE CONDITIONS

4.1 Site Geology

The Geologic Map of Mercer Island (Troost and Wisher, 2006) mapped the surficial geologic units on the western (downslope) portion of the subject as Pre-Olympia fine-grained deposits (map unit Qpof) and the eastern (upslope) portion of the subject site as Pre-Olympia coarse-grained deposits (map unit Qpoc). The area near the shoreline was mapped as lake deposits (Ql).

Pre-Olympia fine-grained deposits (Qpof) are described by Troost, et al. as hard, laminated to massive, silt and clay. Pre-Olympia coarse-grained deposits (Qpoc) are described as very dense, clean to silty sand and gravel with some silt layers. Lake deposits (Ql) are described as very soft to medium stiff, silt and clay with local sand layers, peat, and other organic sediments that were exposed due to the lowering of Lake Washington in 1916.

4.2 Soil Conditions

The subsurface explorations at the site generally encountered a sequence of fill over alluvium/colluvium and pre-Olympia fine-grained deposits. The pre-Olympia fine-grained deposits appeared to be consistent with the mapped geology described above. Alluvium and colluvium are often found in areas below steep slopes, which are present on the east side of the property.

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

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

> ***Fill:*** A surficial layer of loose, brown, silty, fine sand with gravel, organics, and wood and brick debris was encountered in borings PG-2, B-1, and B-2. Based on the loose and nonuniform consistency and presence of brick debris, we interpret this soil unit as undocumented fill. This soil unit extended to depths of about 4½, 2, and 1½ feet below the ground surface in borings PG-2, B-1, and B-2, respectively.
>
> ***Colluvium/Alluvium:*** Below the fill layer, all four borings encountered a soil unit generally consisting of very loose to medium dense, gray to gray-brown, sandy silt with some gravel. Based on the loose consistency, coloration, and generally disturbed consistency, we interpret this unit as colluvium and/or alluvium, likely associated with the steep slopes above the proposed development area. This soil unit extended to depths of 7, 11½, 5½, and 4½ feet in borings PG-1, PG-2, B-1, and B-2, respectively.
>
> ***Pre-Olympia Fine-Grained Deposits:*** Underlying the colluvium/alluvium deposits, all four test borings encountered a soil unit generally consisting of stiff to hard, blue-gray, massive, lean clay with silt. Based on the hard and massive consistency, we interpret this unit as the mapped pre-Olympia fine-grained deposits. This unit extended to the bottom of all test borings at 11½, 16½ , 21½,

and 20½ feet below the ground surface in borings PG-1, PG-2, B-1, and B-2, respectively.

4.3 Groundwater Conditions

Perched groundwater was encountered in all four test borings advanced at the site. Groundwater was observed at approximate elevations 20, 16, 21 and 30 feet in borings PG-1, PG-2, B-1, and B-2, respectively. The groundwater levels encountered in borings PG-1, PG-2, and B-1 generally correspond to the water level in Lake Washington.

In boring B-2, which was advanced on the upslope side of the proposed development, groundwater was noted at about 9 feet deep after drilling, however, shallower perched groundwater may also be present. It should be noted that groundwater levels may vary depending on the season, local subsurface conditions, and other factors, such as the level of Lake Washington.

5.0 Geologic Hazards Assessment

5.1 Potential Landslide Hazards

The subject site is mapped within a potential landslide hazard area according to the City of Mercer Island's Geologic Hazards Map. The map indicates that slopes of 15% or more and slopes between 40-79% are present at the site. The map indicates that landslide or mass wasting deposits exist at the site. The map also indicates the presence of a landslide scarp upslope from the subject site.

The approximately east half of the subject site (i.e., upslope from the driveway) contains a west facing steep slope that has a maximum relief of about 76 feet on the subject property from the northeast corner of the site to the shoreline, based on the survey information provided to us, and an approximate slope gradient between about 1.5H:1V to 1H:1V at the steepest. In general, the slope inclination is fairly uniform between the switchbacks of the driveway and the from the northeast corner of the property to about 80 feet from the shoreline where the slopes become more level.

Multiple site reconnaissances were performed at the site between August 28 and September 20, 2018. During our site visits, we did not observe any apparent evidence of slope instability or ground movement at the site. The soldier pile retaining walls for the switchback sections of the existing driveway appear near vertical with no signs of cracking in the concrete facing of the walls, and no signs of tension cracks in the driveway surface.

Based on our field observations, the general topography of the site and vicinity, and the results of our subsurface explorations, in our opinion the subject site is globally stable in its current configuration. Furthermore, it is our opinion that the proposed development as currently planned is feasible from a geotechnical engineering standpoint, and in our opinion will not adversely affect the overall stability of the site or adjacent properties, provided the recommendations outlined herein are followed and the proposed development is properly designed and constructed. Our recommendations include the use of a soldier pile wall to provide temporary support for the proposed basement excavation.

5.2 Seismic Hazards

Based on our review of the City of Mercer Island's Geologic Hazards Maps, the project site is not mapped as a seismic hazard area. The City of Mercer Island Code defines seismic hazard areas as those areas subject to risk of damage as a result of earthquake-induced ground shaking, slope failure, soil liquefaction or surface faulting.

Based on the hard clay soils underlying the proposed building area, in our opinion, the potential for soil liquefaction during an IBC-code level earthquake is considered negligible, and special foundation design considerations associated with soil liquefaction are not needed for this project.

It is also our opinion that the potential for seismic-induced landslidng is low at the site due to the hard clay underlying the site, and the presence of existing soldier pile walls for the existing paved driveway along the upslope side of the proposed development. Provided that the design of the excavation support and permanent basement walls are designed and constructed as recommended herein, it is our opinion that the stability of the site will not be compromised by the proposed development.

5.3 Erosion Hazards

The subject site is mapped within a potential erosion hazard area according to the City of Mercer Island's Geologic Hazards Map. Based on soil conditions encountered in the borings, the near-surface site soils are likely to exhibit moderate erosion potential. In our opinion, the erosion hazards at the site can be effectively mitigated with the best management practice during construction and with properly designed and implemented landscaping for permanent erosion control. During construction, the temporary erosion hazard can be effectively managed with an appropriate erosion and sediment control plan, including but not limited to installing silt fence at the construction perimeter, limiting removal of vegetation to the construction area, placing gravel or hay bales at the disturbed/traffic areas, covering stockpile soil or cut slopes with plastic sheets, constructing a temporary drainage pond to control surface runoff and sediment trap, placing quarry spalls at the construction entrance, etc. Permanent erosion control measures should include establishing vegetation, landscape plants, and hardscape established at the end of project, and reducing surface runoff to the minimum extent possible.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 SEISMIC DESIGN PARAMETERS

The 2015 International Building Code (IBC) seismic design section provides a basis for seismic design of structures. Table 1 below provides seismic design parameters for the site that are in conformance with the 2015 IBC, which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps.

Table 1 – Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec. [g] S_S	Spectral Acceleration at 1.0 sec. [g] S_1	Site Coefficients		Design Spectral Response Parameters	
			F_a	F_v	S_{DS}	S_{D1}
D	1.41	0.54	1.0	1.5	0.94	0.54

The spectral response accelerations were obtained from the USGS Earthquake Hazards Program website (2008 data) for the project latitude and longitude.

Liquefaction Potential: Liquefaction is a process that can occur when soils lose shear strength for short periods of time during a seismic event. Ground shaking of sufficient strength and duration results in the loss of grain-to-grain contact and an increase in pore water pressure, causing the soil to behave as a fluid. Soils with a potential for liquefaction are typically cohesionless, predominately silt and sand sized, loose to medium dense, and must be saturated.

The proposed building foundation should bear directly on the moist, hard, clay soils. As such, in our opinion the liquefaction potential below the proposed structure is low, and design considerations related to soil liquefaction are not necessary for this project. However, it should be noted that liquefaction is likely to occur along the shoreline during a seismic event.

6.2 Conventional Footings

Based on our understanding of the subsurface conditions at the site, in our opinion the proposed residence may be supported by conventional footings. Footings should be founded on the hard, clay soils anticipated to be present at the proposed foundation elevation.

Hard clay should be present at or near the footing elevations. Based on the subsurface conditions encountered in the test borings along the west side of the site (PG-1, PG-2, and B-1), the bearing soils may be at or below the proposed bottom of excavation along the west side of the proposed building. As such, depending on the actual footing elevations and local soil variations, localized over-excavation of the loose sand (colluvium/alluvium) may be required to reach the hard clay soils. We do not anticipate significant amounts of over-excavation will be required.

6.2.1 Allowable Bearing Pressure

We recommend a maximum allowable soil bearing pressure of 3,000 pounds per square foot (psf) be used to size the footings. The recommended allowable bearing pressure is for dead plus live loads. For allowable stress design, the recommended bearing pressure may be increased by one-third for transient loading, such as wind or seismic forces. Continuous and individual spread footings should have minimum widths of 18 and 24 inches, respectively.

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

6.2.2 Lateral Resistance

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

6.2.3 Perimeter Footing Drains

Footing drains should be installed around the perimeter of the residence, at or just below the invert of the footings. Under no circumstances should roof downspout drain lines be connected to the footing drain systems. Roof downspouts must be separately tightlined to appropriate discharge locations. Cleanouts should be installed at strategic locations to allow for periodic maintenance of the footing drain and downspout tightline systems.

We anticipate that basement walls will be constructed against the soldier pile wall. As such, drain mats should cover the entire face of the shoring wall prior to basement wall construction, to direct groundwater to the base of the excavation and interior perforated drain pipe for proper discharge.

6.2.4 Footing Subgrade Preparation

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

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

To mitigate subgrade soil disturbance, a rat-slab of lean-mix concrete should be considered as an option to protect the subgrade immediately after excavation, especially if groundwater seepage is present, or during wet weather conditions.

6.3 Floors Slabs

We anticipate that competent, native soil will be encountered at the slab level. Structural fill placed below the slab should be properly compacted in accordance with the structural fill recommendations presented in this report. The exposed subgrade should be compacted to a firm condition prior to placing the backfill or capillary break layer.

Interior concrete slab-on-grade floors should be underlain by a capillary break consisting of at least of 6 inches 5/8-inch, clean crushed rock (less than 3 percent fines). The capillary break material should meet the gradational requirements provided in Table 2, below.

Table 2 – Capillary Break Gradation

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

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

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

6.4 Below-Grade Wall Design Parameters

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

6.4.1 Lateral Earth Pressures

A temporary soldier pile wall will be used for shoring around the majority of the basement perimeter. For basement walls to be constructed against soldier pile walls, we recommend that the basement walls be designed using the same pressure as the shoring walls.

Where basement walls will be constructed and then conventionally backfilled such as the west side of the house which will need structural fill for the new driveway between the house and existing retaining wall, the basement walls should be designed for an earth pressure based upon an equivalent fluid weight of 50 pcf with level backslope.

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

A uniform pressure of 7H psf should be added to all basement walls to reflect the increase loading for seismic conditions, where H corresponds to the buried depth of the wall. A uniform pressure of 25 psf should also be added to the east basement wall to reflect the traffic surcharge assumed for lightweight vehicles

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

6.4.2 Lateral Resistance

Lateral forces from wind or seismic loading may be resisted by the combination of passive earth pressures acting against the embedded portions of the foundations and by friction acting on the base of the foundations. Passive resistance values may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf). This value includes a factor of safety of at least 1.5 assuming that a properly compacted structural fill will be placed adjacent to the sides of the footings. A coefficient friction of 0.3 may be used to determine the frictional resistance at the base of the footings. This coefficient includes a factor of safety of approximate 1.5.

6.4.3 Wall Backfill

Based on the results of our test borings, the onsite soils consist of silty sand overlying silty clay. As such, the onsite soils are not suitable to be re-used as wall backfill. For budgeting purpose, we recommend that wall backfill consist of imported free draining granular soils such as Seattle Mineral Aggregate Type 17 or Gravel Borrow as defined in Section 9-

03.14(1) of the WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT, 2016). In areas where the space is limited between the wall and the face of excavation, clean crushed 5/8-inch rock may be used as backfill without compaction.

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

6.4.4 Wall Drainage & Damp Proofing

Provisions for permanent control of subsurface water should be incorporated into the design and construction of the below-grade walls. As a minimum, 4-inch diameter perforated drainpipes should be installed behind and at the base of the wall footings, embedded in 12 to 18 inches of clean drainage gravel. The gravel should be wrapped in a geotextile filter fabric to prevent the migration of fines into the drain system. The drainpipe should be graded to direct water to a suitable outlet.

Where the below-grade wall will be constructed against a soldier pile wall, we recommend that prefabricated drainage mats, such as Mirafi 6000 or equivalent, be installed behind the walls (full face coverage) and the collected water should be directed through weep holes inside the building beneath the floor slab and tight-lined to an appropriate outlet.

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

6.5 ON-SITE INFILTRATION CONSIDERATIONS

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

7.0 EXCAVATION AND SHORING CONSIDERATIONS

Based on our current understanding of the planned basement elevations and assuming a 1½-foot thick perimeter footing, we anticipate excavations along the north, south, and east sides of the basement excavation with the maximum depths along the east side of the basement ranging from 13 feet at the northeast corner up to 23 feet deep at the southeast corner may be needed to construct the house foundation and basement walls, which we recommend be supported by a soldier pile wall with tieback anchors. We believe that a soldier pile walls with timber lagging represents the most appropriate method to support the excavation and maintain stability of the slope.

For the east parking spaces on the upslope side of the paved access driveway, we understand that excavation cuts up to 8 feet will be needed. To maintain an adequate stability of the slope, we recommend a permanent soldier pile and timber lagging wall. We do not anticipate tiebacks would be necessary for the permanent soldier pile walls that is less than 10 feet in height.

7.1 Temporary Unsupported Excavations

Temporary excavations should be constructed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. It is our opinion temporary excavations at the site parallel to the overall slope angle may be cut at a maximum 2H:1V inclination, to remain stable, and reduce the potential of destabilizing the site. Temporary excavations perpendicular to the overall slope angle (i.e. excavations that will not be surcharged by a backslope), may be cut at a maximum of 1H:1V.

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

No excavation activities should be allowed between the driveway wall and the east shoring wall for the house during the construction of the below grade walls.

7.2 Soldier Pile Wall

A soldier pile wall consists of vertical steel beams, typically spaced from 6 to 8 feet apart along the proposed excavation wall, spanned by timber lagging. Prior to the start of excavation, the steel beams are installed into holes drilled to a design depth and then backfilled with lean mix or structural concrete. As the excavation proceeds downward and the steel piles are subsequently exposed, timber lagging is installed between the piles to support the soils between piles. We offer the following geotechnical design recommendations for the proposed permanent and temporary soldier pile walls utilized for this project.

All soldier piles should be located at least 12 feet from the driveway wall.

7.2.1 Design Lateral Pressures

The attached Figure 4 should be used for design of the cantilevered temporary and permanent shoring walls at the site, or a wall with a single level of tiebacks. The design lateral earth pressures are as follows:

East Temporary Shoring Wall: 50 pcf

North and South Temporary Shoring Walls: 35 pcf

Permanent Shoring Wall: 50 pcf

The design lateral earth pressure for the temporary soldier pile walls consider only a level backslope condition with no changes made to the existing site grades. The design lateral earth pressure for the permanent soldier pile wall on the east side of the existing driveway considers the back slope surcharge pressure from the existing slope. Based on the current proposed basement elevation, we anticipate one row of tiebacks will be needed. If more than one row of tiebacks will be needed for wall stability, or to create a more cost-effective wall design, PanGEO will provide additional recommendations for tieback design upon request.

Above the bottom of excavation, the recommended active earth and surcharge pressures should be applied over the full width of pile spacing. Below the bottom of excavation, the active pressures should be applied over one pile spacing, and the passive resistance should be applied over two times the pile diameter.

We also recommend that the lagging be sized using an earth pressure equivalent to 50 percent of the design earth pressure shown in Figure 4 to account for the arching effects.

7.2.2 Vertical Capacity

We recommend the vertical capacity of the soldier piles be determined using an allowable skin friction value of 1 ksf for the portion of the pile below the bottom of the excavation, and an allowable end bearing value of 15 ksf.

7.2.3 Groundwater and Caving Soil Conditions

The drilling of soldier piles is anticipated to encountered loose fill, wet sand, and hard clay soils. Caving in fill and wet sand layers could occur during drilling. As a result, the drilling contractor should be prepared to use temporary casings and/or drilling fluids to stabilize the holes.

Groundwater is likely to accumulate at the bottom of drill holes. We recommend that the lean concrete or structural concrete backfill be placed with tremie pipes if more than one foot of water is present at the bottom of the holes.

When placing timber lagging, the height of each lift may need to be limited when the wet soils are encountered. The actual allowable vertical cut for timber lagging placement should be determined in the field, based on the actual conditions observed.

7.2.4 Temporary Surcharge

Depending on the contractor's site layout and construction methods, temporary excavation shoring may be subjected to surcharge loads from heavy construction equipment such as concrete pump trucks and cranes. As such, where appropriate, the shoring design should account for such surcharge loads. Input from the contractor will be needed to determine if such design considerations will be necessary.

7.2.5 Potential Conflicts with Existing Retaining Walls

The existing concrete retaining wall along the west side of the driveway may impact the design/installation of the excavation shoring. If the existing retaining wall is supported by piles, the locations of the basement soldier piles should be adjusted accordingly to avoid potential conflicts with tiebacks and the existing wall piles.

7.3 TEMPORARY TIEBACK ANCHORS

Tieback anchors, where needed, will extend underneath the existing paved driveway which is not part of the subject property. As a result, construction easements may be needed from the neighboring property owners, including the city. The easements should be obtained as early in the design process as possible because the project costs could be significantly impacted without the construction easements.

Excessive pile top deflection could occur before tiebacks are installed. To improve the performance of the tieback wall, it may be necessary to limit the first row of tiebacks to no more than 10 feet below pile top unless steel beams of sufficient size will be used to limit the magnitude of the cantilever deflection.

7.3.1 Anchor Load Transfer

The manner in which the tieback anchors carry load will depend on the type of anchor selected, the method of installation, and the soil conditions surrounding the anchor. Accordingly, we recommend use of a performance specification requiring the shoring contractor to install anchors capable of satisfactorily achieving the design structural loads, with a pullout resistance factor of safety of 2.0.

For planning purposes, however, the anchors may be sized for an assumed allowable skin friction value of 2.5 kips per lineal foot of anchor bond length, assuming that small diameter (about 6 inches) pressure-grouted tiebacks will be used. Pressurized grouting during installation and multiple post-grouting are likely needed in order to achieve the design capacity. We recommend that the allowable tieback loads be limited to about 120 kips per anchor.

The contractor, based on their intended installation method and their experience with similar soils conditions, may use a different value for sizing the anchors, subject to meeting the acceptance criteria outlined in this report.

7.3.2 No-Load Zone

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

7.3.3 Groundwater and Caving Soil Conditions

The drilling for tiebacks is expected to encounter wet sand layers and seams where caving of the drilled holes is likely to occur. As result, we recommend the use of temporary casing during installation to keep the drilled holes open, and to minimize the risk of potential ground loss and off-site settlement.

7.3.4 Verification Tests (200% Load Tests)

The actual capacity of the anchors should be confirmed with 200% verification tests. At least two 200% load tests should be performed prior to installing production anchors. The anchor testing should be conducted in accordance with the latest edition of the Post Tensioning Institute (PTI) "Recommendations for Prestressed Rock and Soil Anchors." Essentially elements of verification tests are as follows:

- Prior to installing production anchors, perform a minimum of two tests each on each anchor type, installation method and soil type with the tested anchors constructed to the same dimensions as production anchors;
- Test locations to be determined in conjunction and approved by the geotechnical engineer;
- Test anchors, which will be loaded to 200% of the design load, may require additional prestressing steel (steel load not to exceed 80% of the ultimate tensile strength) or reinforcing of the soldier pile;
- Load test anchors to 150% load in 25% load increments, holding each incremental load for at least 5 minutes and recording deflection of the anchor head at various times within each hold to the nearest 0.01 inch;
- At the 150% load, the holding period shall be at least 60 minutes;
- After completion of the 150% hold, load the anchor in 25% load increments to the 200% load, which shall be held for 10 minute, and
- A successful test shall provide a measured creep rate of 0.04 inches or less at the 150% load between 1 and 10 minutes, and 0.08 inches between 6 and 60 minutes, and both shall have a creep rate that is linear or decreasing with time. The applied load must remain constant during all holding periods (i.e. no more than 5% variation from the specified load).

Verification tested anchors or extended creep proof tested anchors not meeting the acceptance criteria will require a redesign by the contractor to achieve the acceptance criteria.

7.3.5 Proof Tests (130% load tests on all production anchors)

All production anchors should be proof tested to 130% of the design load. The anchor testing should be conducted in accordance with the latest edition of the Post Tensioning

Institute (PTI) "Recommendations for Prestressed Rock and Soil Anchors." Essentially elements of proof tests are summarized below:

- Load test all production anchors to 130% of the design load in 25% load increments, holding each incremental load until a stable deflection is achieved (record deflection of the anchor head at various times within each hold to the nearest 0.01inch);
- At the 130% load, the holding period shall be at least 10 minutes;
- A successful test shall provide a measured creep rate of 0.04 inches or less at the 150% load between 1 and 10 minutes with a creep rate that is linear or decreasing with time. The applied load must remain constant during the holding period (i.e. no more than 5% variation from the 130% load). Anchors failing this proof testing creep acceptance criteria may be held an additional 50 minutes for creep measurement. Acceptable performance would equate to a creep of 0.08 inches or less between 6 and 60 minutes with a linear or decreasing creep rate.

7.4 Baseline Survey and Monitoring

Ground movements will occur as a result of excavation activities. As such, ground surface elevations of the adjacent properties and driveway walls should be documented prior to commencing earthwork to provide baseline data. As a minimum, optical survey points should be established at the following locations:

- The top of every other soldier pile. These monitoring points should be monitored twice a week. The monitoring frequency may be reduced based on the monitoring results.
- The top of the east adjacent concrete driveway retaining wall and centerline of the east driveway. These monitoring points should be spaced no more than 20 feet apart. These monitoring points do not need to be regularly surveyed unless the top of wall deflections exceed about one inch.

The monitoring program should include changes in both the horizontal (x and y directions) and vertical deformations. The monitoring should be performed by the contractor or the project surveyor, and the results be promptly submitted to PanGEO for review. The results of the monitoring will allow the design team to confirm design parameters, and for the contractor to make adjustments if necessary.

We also recommend that the existing conditions long the driveway and the adjacent private properties be photo-documented prior to commencing on any earthwork at the site.

7.5 Groundwater Control

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

8.0 CONSTRUCTION CONSIDERATIONS

8.1 Material Reuse

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

8.2 Structural Fill and Compaction

In the context of this report, structural fill is defined as compacted fill placed under footings, concrete stairs, landings, slabs, or other load-bearing areas. The contractor should be aware that the onsite soils expected to be encountered during construction have a relatively high fines content and may be difficult to compact to the requirements of structural fill. As a result, the excavated site materials may not be suitable for use as structural backfill, particularly during periods of wet weather. If imported structural fill is needed, it should consist of a well-graded granular material, such as City of Seattle Type 17, or WSDOT Gravel Borrow (WSDOT 9-03.14(1)). Due to the presence of perched groundwater at the site, we do not recommend the use of crushed recycled concrete as structural fill.

Structural fill should be placed in 8- to 12-inch thick loose lifts and compacted to at least 95 percent maximum dry density, per ASTM D-1557 (Modified Proctor). In non-structural areas, the recommended compaction level may be reduced to 90 percent. Heavy compaction equipment should not operate directly over utilities until a minimum of 2 feet of backfill has been placed.

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

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

8.3 Wet Weather Construction

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

- Earthwork should be performed in small areas to minimize subgrade exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.
- During wet weather, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight based on the portion passing the 0.75-inch sieve. The fines should be non-plastic.
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water.
- Geotextile silt fences should be installed at strategic locations around the site to control erosion and the movement of soil.
- Excavation slopes and soils stockpiled on site should be covered with plastic sheeting.

8.4 EROSION CONSIDERATIONS

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

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

9.0 STATEMENT OF RISK

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

a. The geologic hazard area will be modified, or the development has been designed so that the risk to the lot and adjacent property is eliminated or mitigated such that the site is determined to be safe; or

b. An evaluation of site specific subsurface conditions demonstrates that the proposed development is not located in a geologic hazard area; or

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

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

It is our opinion that Criterion A and/or C can be met provided that the development is designed and constructed in accordance with the recommendations in this report. The proposed structures will be located at the toe of the steep slope, and will therefore not add a surcharge load to the existing slope. In addition, the design utilizes tieback soldier pile walls to support the cuts into the slope for the construction of the below-grade walls. As such, in our opinion the development will not negatively affect the stability of the slope, or the surrounding properties, but will likely increase the stability of the site.

In addition, in our opinion Criterion C can be met through best management practices during construction, including the proper use of a silt fence, minimize earthwork activities during periods heavy precipitation, minimize exposed areas in the wet season, etc. Permanent erosion control measures including landscape and hardscape installations will effectively mitigate the risk of erosion in the long term.

10.0 ADDITIONAL SERVICES

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

11.0 CLOSURE

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

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

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

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

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

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use this report.

Sincerely,

PanGEO, Inc.

BRYCE CAMERON TOWNSEND
STATE OF WASHINGTON
54333
REGISTERED
PROFESSIONAL ENGINEER
10/12/2018

Bryce C. Townsend, P.E.
Staff Geotechnical Engineer

SIEW LAN TAN
STATE OF WASHINGTON
30342
REGISTERED
PROFESSIONAL ENGINEER
10/12/2018

Siew L Tan, P.E.
Principal Geotechnical Engineer

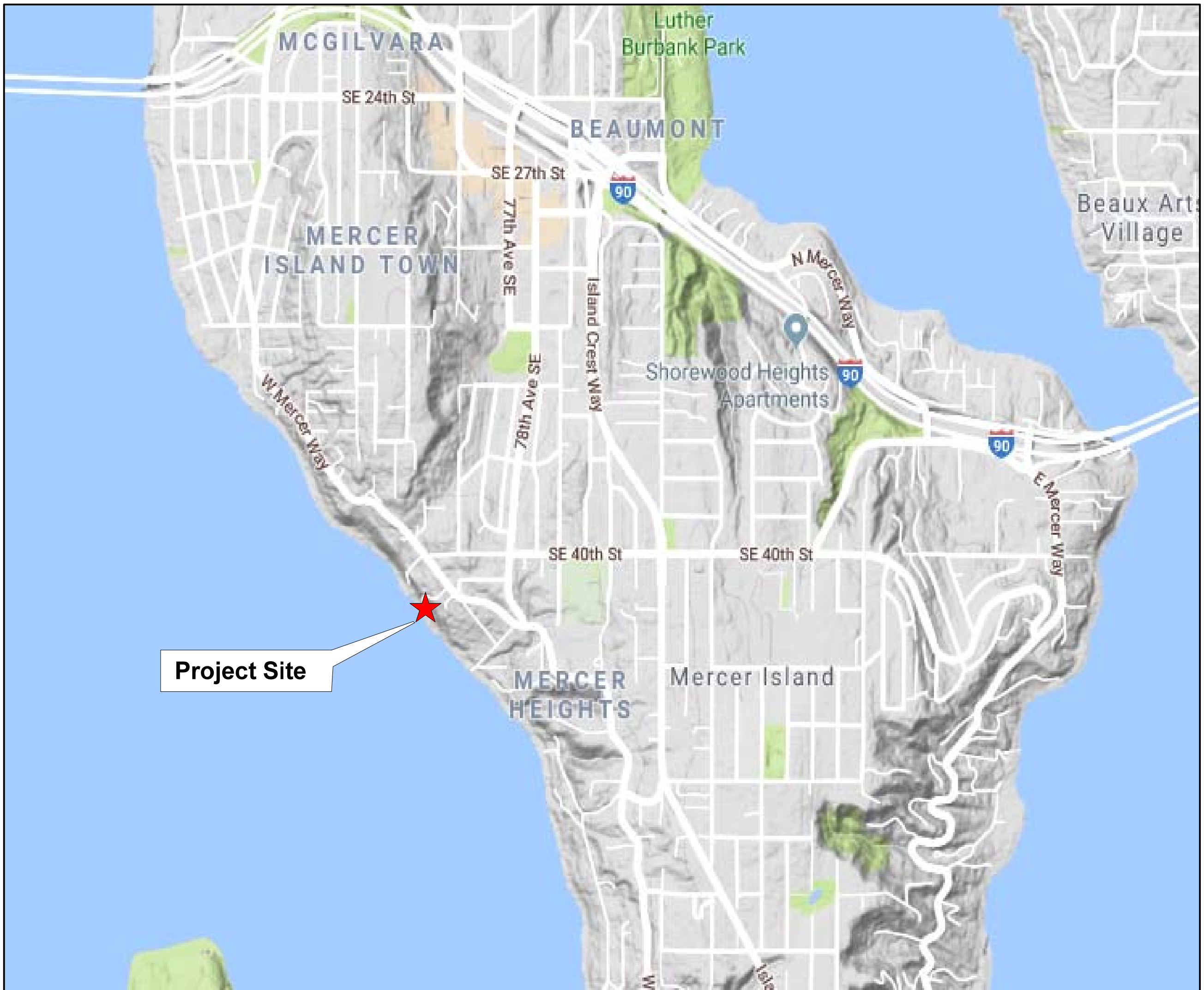
12.0 REFERENCES

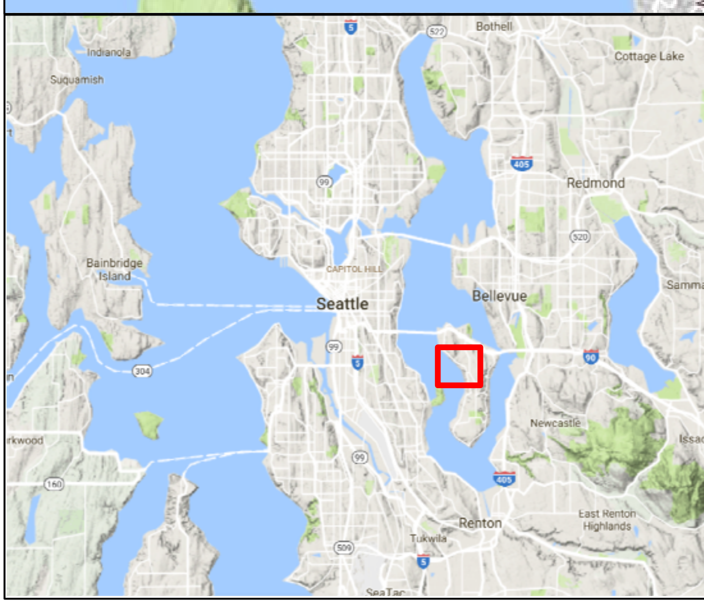
International Code Council, 2015, *International Building Code (IBC), 2015.*

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

United States Geological Survey, *Earthquake Hazards Program, 2008 Data,* accessed via: http://earthquake.usgs.gov/designmaps/us/application.php

WSDOT, 2016, *Standard Specifications for Road, Bridge and Municipal Construction, M 41-10.*





Reference: Google Terrain Map

N

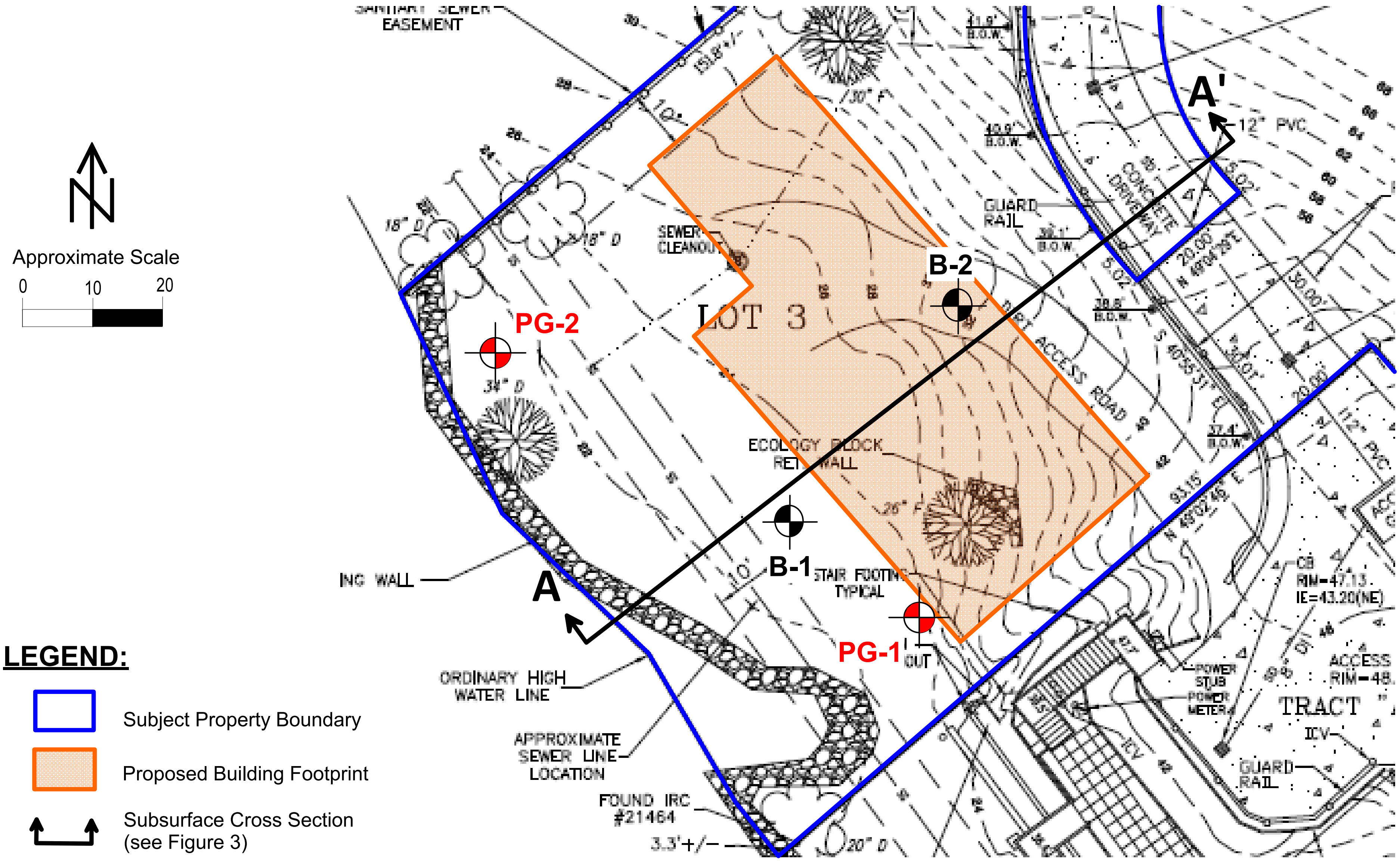
Not To Scale

PanGEO INCORPORATED

Proposed Development
4041 West Mercer Way
Seattle, WA 98040

VICINITY MAP

18-282	**1**

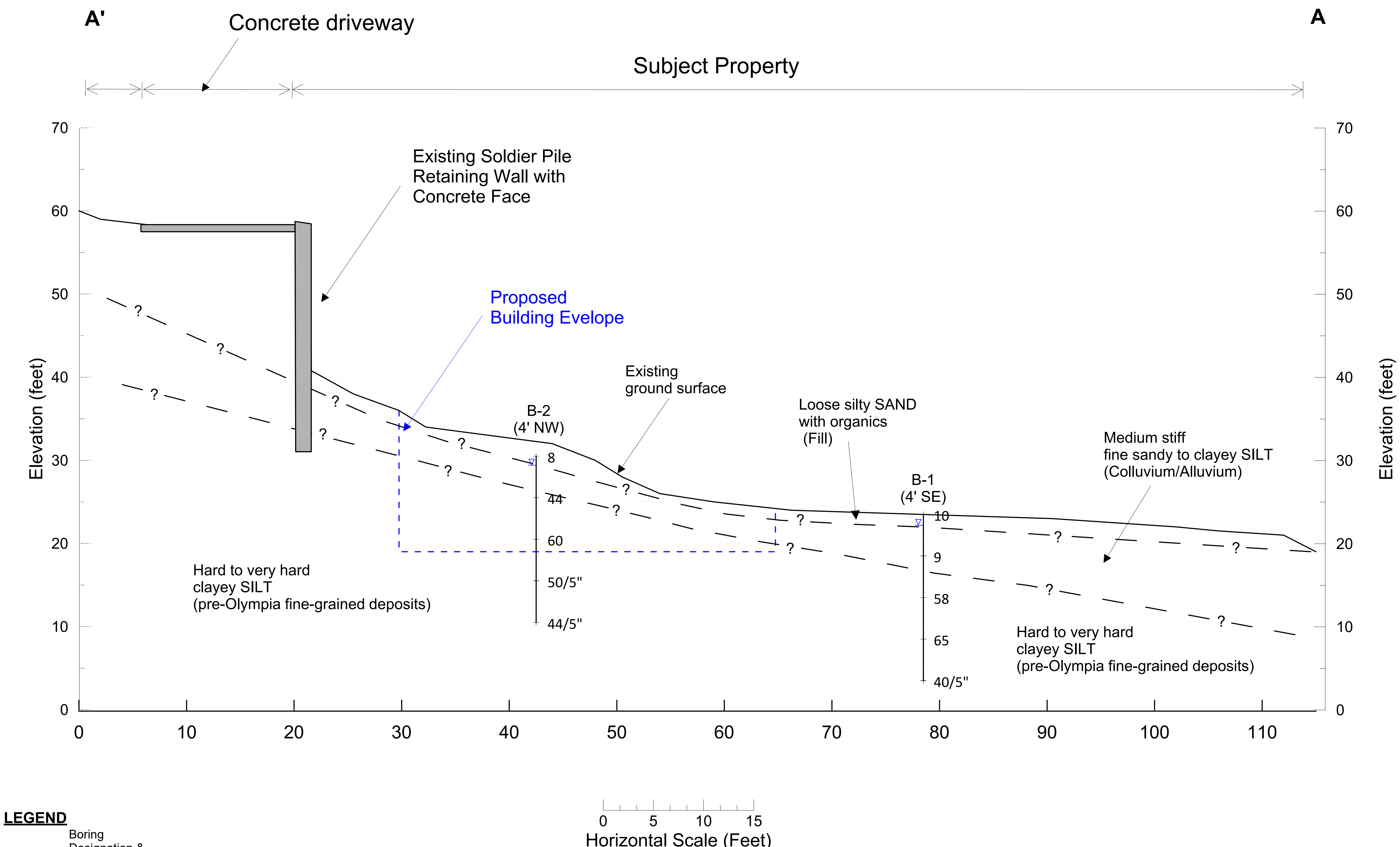


Note: Base map modified from Topographical Survey by Hansen Surveying dated October 21, 2009

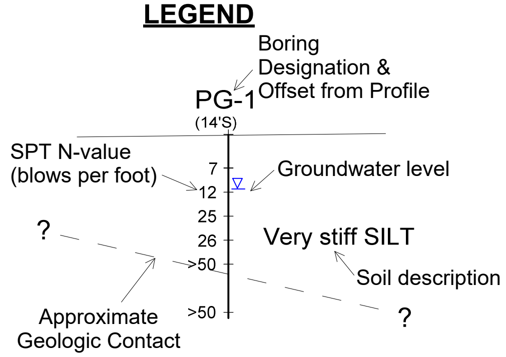
LEGEND:

- Subject Property Boundary
- Proposed Building Footprint
- Subsurface Cross Section (see Figure 3)
- Approx. Boring Location (Liu, 2010)
- Approx. Boring Location (PanGEO, 2018)

PanGEO INCORPORATED	Proposed Development 4041 West Mercer Way Seattle, WA 98040	SITE AND EXPLORATION PLAN	
		Project No. 18-282	Figure No. 2



LEGEND



Notes:
1. Existing ground profile based on the topographic survey of 4041 West Mercer Way prepared by Hansen Land Surveying & Cunsultants, 2018.
2. See Figure 2 for location of Section A.
3. See report text for a detailed explanation of the subsurface profile across the site.
4. The generalized soil profile is based on widely-spaced borings. Soil conditions may vary over a small distance, and the actual subsurface conditions may be different from the generalized soil profile depicted in this figure.

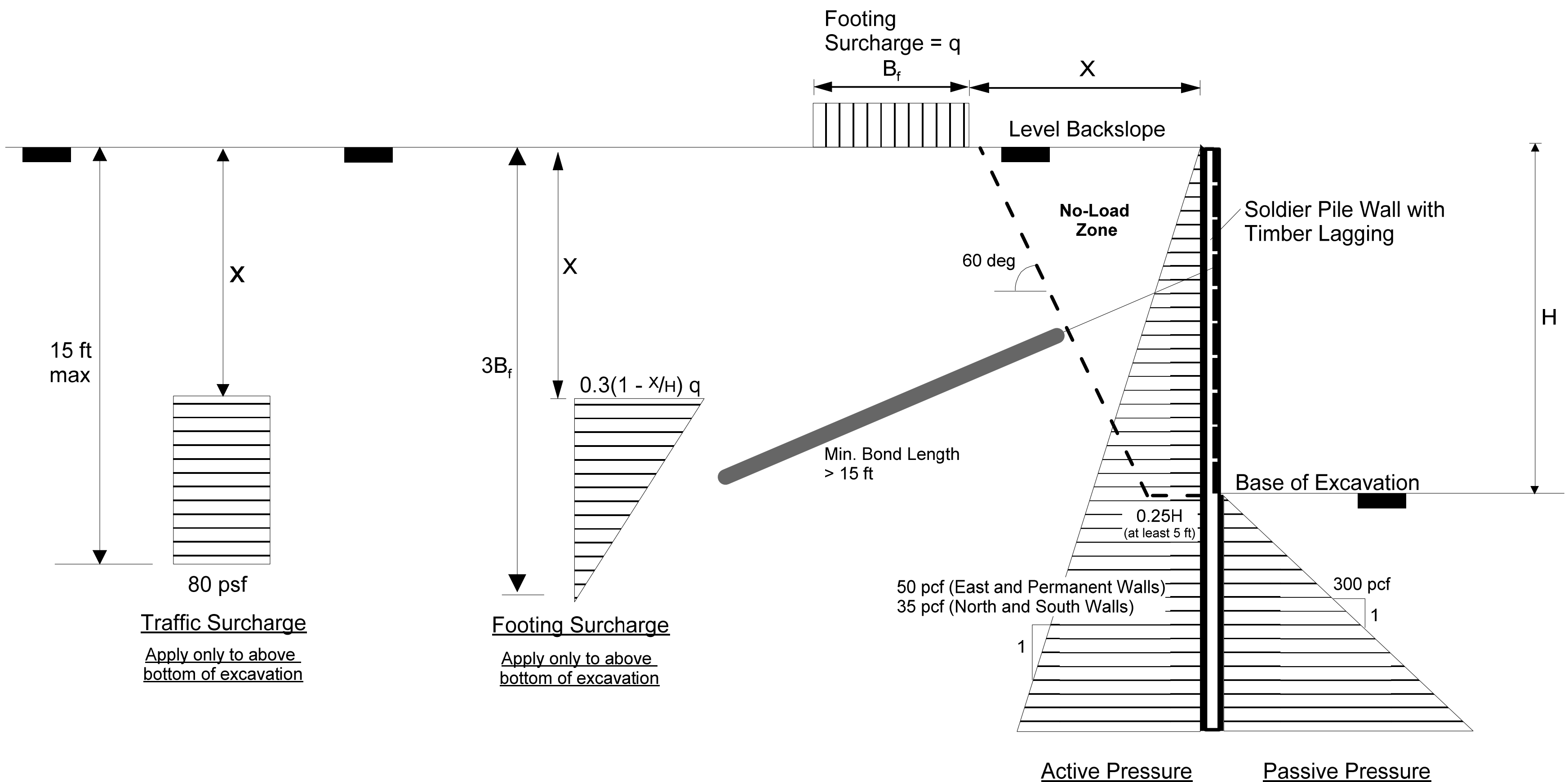
PanGEO INCORPORATED

Proposed Development
4041 East Mercer Way
Seattle, WA 98040

GENERALIZED SUBSURFACE PROFILE
SECTION A

Project No.	Figure No.
18-282	3

18-282 Xsection A.grf 10/11/18 (17:57) SMH



Notes:
1. Minumum embedment should be at least 10 feet below bottom of excavation.
2. A factor of safety of 1.5 has been applied to the recommended passive pressure values. No factor of safety has been applied to the recommended active earth pressure values.
3. Active pressures should be applied over the full width of the pile spacing above the base of the excavation, and over one pile diameter below the base of the excavation.
4. Surcharge pressures should be applied over the entire length of the loaded area.
5. Passive pressure should be applied to two times the diameter of the soldier piles.
6. Use 50% of the active and surcharge pressures for lagging design with soldier piles spaced at 8' or less.
7. Refer to report text for additional discussions.

PanGEO INCORPORATED

Proposed Development
4041 West Mercer Way
Seattle, WA 98040

DESIGN LATERAL PRESSURES
SOLDIER PILE WALL
CANTILEVERED OR WITH ONE TIEBACK

Project No.	Figure No.
18-282	**4**

APPENDIX A

PANGEO TEST BORING LOGS

RELATIVE DENSITY / CONSISTENCY

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

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Notes:

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

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

DESCRIPTIONS OF SOIL STRUCTURES

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

COMPONENT DEFINITIONS

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

TEST SYMBOLS

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

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

SYMBOLS

Sample/In Situ test types and intervals

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

MONITORING WELL

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

MOISTURE CONTENT

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

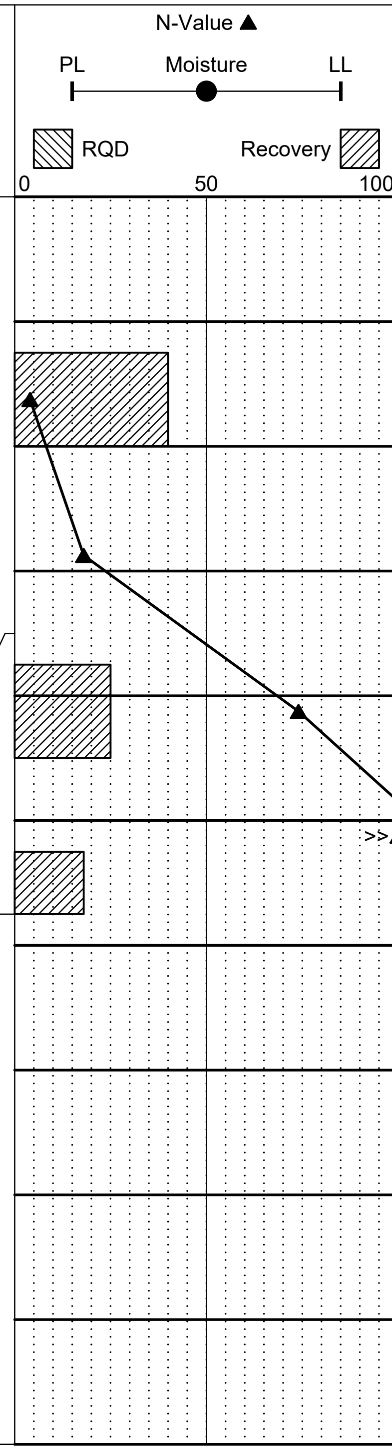
PanGEO INCORPORATED
Phone: 206.262.0370

Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	Lundin Residence	Surface Elevation:	Approx. 25 feet
Job Number:	18-282	Top of Casing Elev.:	
Location:	4041 West Mercer Way, Mercer Island, WA 98040	Drilling Method:	Hollow-Stem Auger
Coordinates:	Northing: , Easting:	Sampling Method:	STP

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						- Gray-brown, silty, fine SAND with some gravel and organcis at surface.
2–4	S-1		2 2 2			Very loose, very moist, gray-brown, silty SAND with gravel (SM); fine sand, some orgaincs present, discutrbed consistency. **[COLLUVIUM/ALLUVIUM DEPOSITS].**
5–6	S-2		28 8 10			- No sample recovery. Becomes medium dense, traces of wet sand and silt/clay in sample tip.
7						Driller's Note: Drilling action indicates transition into stiffer soils at about 7 feet deep.
8	S-3		29 32 42			Hard, wet, blue-gray, lean CLAY with silt (CL); some plasticity, some iron-oxide staining. **[PRE-OLYMPIA FINE-GRAINED DEPOSITS (Qpof)].**
10	S-4a S-4b		50/6 12 42			- No sample recovery in sample S-4a. A second sample S-4b driven with minimal recovery.
11.5						Bottom of boring at about 11.5 feet below the ground surface. Groundwater encountered at about 5 feet below ground surface.
12						
14						
16						
18						
20						



Completion Depth:	11.5ft
Date Borehole Started:	9/20/18
Date Borehole Completed:	9/20/18
Logged By:	B. Townsend
Drilling Company:	Boretec1 Inc.

Remarks: Standard Penetration Test (SPT) sampler driven with a 140 lb hammer. Hammer operated with a rope and cathead mechanism. Boring drilled by Boretec1, Inc using a limited access Acker Drill Rig. Ground surface elevation estimated from site topographical survey developed by Hansen Surveying.

PanGEO INCORPORATED

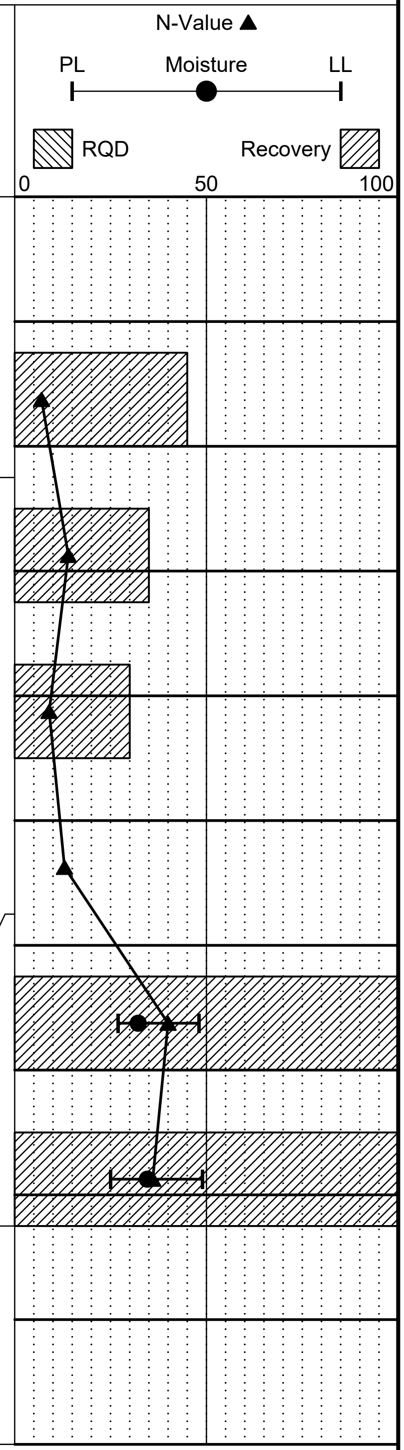
LOG OF TEST BORING PG-1

Figure A-2

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

Project:	Lundin Residence	Surface Elevation:	Approx. 21 feet
Job Number:	18-282	Top of Casing Elev.:	
Location:	4041 West Mercer Way, Mercer Island, WA 98040	Drilling Method:	Hollow-Stem Auger
Coordinates:	Northing: , Easting:	Sampling Method:	STP

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						- Brown, silty, fine SAND with some gravel and organcis at surface.
3	S-1		6 4 3			Loose, moist, brown, silty SAND with gravel (SM); fine sand. **[FILL].**
4.5						Medium dense, wet, dark blue-gray, very silty SAND with gravel (SM); fine sand, some organics present, disturbed consistency. **[COLLUVIUM/ALLUVIUM DEPOSITS].**
6	S-2		2 9 5			
8	S-3		4 4 5			- Becomes loose, very silty/clayey sand to sandy silt/clay.
10.5	S-4		10 8 5			- No sample recovery. Big gravel stuck in sample tube tip. Driller's Note: Drilling action indicates transition into stiffer soils at about 11.5 feet deep.
11.5						Hard, moist, blue-gray, lean CLAY with silt (CL); massive, some plasticity, some iron-oxide staining. **[PRE-OLYMPIA FINE-GRAINED DEPOSITS (Qpof)].**
13	S-5		12 18 22	ATT		S-5 ATT: MC=32.2%, LL=48, PI=27.
15.5	S-6		12 18 18	ATT		S-6 ATT: MC=34.6%, LL=49, PI=25.
16.5						Bottom of hole at about 16.5 feet below ground surface. Groundwater encountered at about 5 feet below ground surface.



Completion Depth: 16.5ft
Date Borehole Started: 9/20/18
Date Borehole Completed: 9/20/18
Logged By: B. Townsend
Drilling Company: Boretec1 Inc.

Remarks: Standard Penetration Test (SPT) sampler driven with a 140 lb hammer. Hammer operated with a rope and cathead mechanism. Boring drilled by Boretec1, Inc using a limited access Acker Drill Rig. Ground surface elevation estimated from site topographical survey developed by Hansen Surveying.

PanGEO INCORPORATED

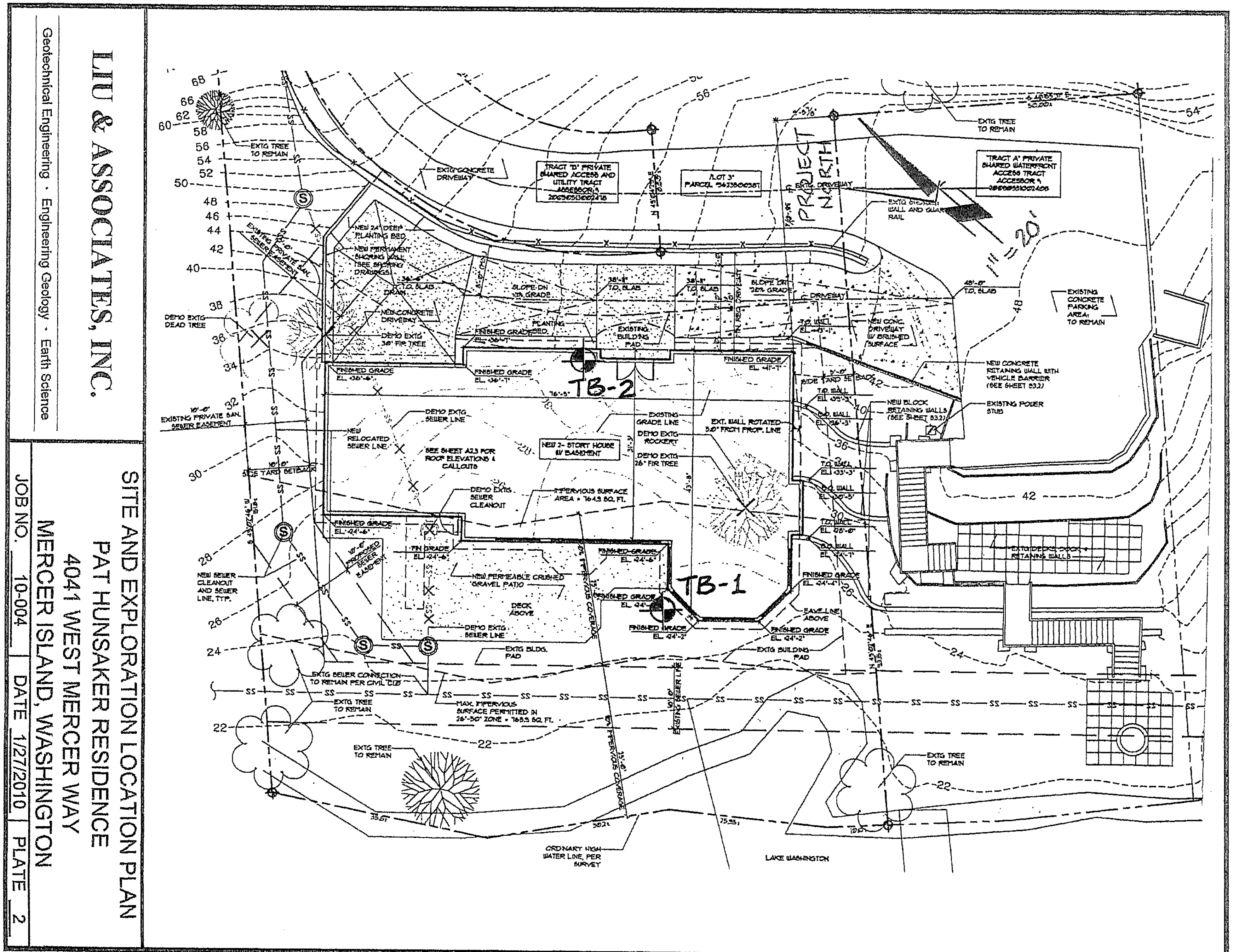
LOG OF TEST BORING PG-2

Figure A-3

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

APPENDIX B

PREVIOUS TEST BORINGS



PanGEO Figure B-1

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS MORE THAN 50% RETAINED ON THE NO. 200 SIEVE	GRAVEL MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE-GRAINED SOILS MORE THAN 50% PASSING ON THE NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50%	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILTY AND CLAY LIQUID LIMIT 50% OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC SILT, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

NOTES:

1. FIELD CLASSIFICATION IS BASED ON VISUAL EXAMINATION OF SOIL IN GENERAL ACCORDANCE WITH ASTM D2488-83.
2. SOIL CLASSIFICATION USING LABORATORY TESTS IS BASED ON ASTM D2487-83.
3. DESCRIPTIONS OF SOIL DENSITY OR CONSISTENCY ARE BASED ON INTERPRETATION OF BLOW-COUNT DATA, VISUAL APPEARANCE OF SOILS, AND/OR TEST DATA.

SOIL MOISTURE MODIFIERS:

DRY - ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH

SLIGHTLY MOIST - TRACE MOISTURE, NOT DUSTY

MOIST - DAMP, BUT NO VISIBLE WATER

VERY MOIST - VERY DAMP, MOISTURE FELT TO THE TOUCH

WET - VISIBLE FREE WATER OR SATURATED, USUALLY SOIL IS OBTAINED FROM BELOW WATER TABLE

LIU & ASSOCIATES, INC. Geotechnical Engineering · Engineering Geology · Earth Science	UNIFIED SOIL CLASSIFICATION SYSTEM
	PLATE 3

BORING NO. B-1

Logged By: JSL 2/2/2010 Ground Elev. ±

Depth ft.	USCS	Soil Description	Sample Type	Sample No.	(N) Blows/ ft.	W %	Other Test
	SM	Dark-brown to brown mixed, loose, silty fine SAND, trace gravel, with occasional wood debris and brick, fragments, moist (FILL or DISTURBED SOIL)	SS	1	10		
	SP/SM	Gray, loose to medium-dense, slightly silty, fine to medium SAND, very moist to wet (OLDER SAND)					
5	ML	Gray, medium-stiff, fine sandy to clayey SILT, in 1/2-to-1-in bedding, slighty moist (OLDER CLAY)	SS	2	9		
10	ML	Gray, hard to very-hard, clayey SILT, massive, slightly moist (OLDER CLAY)	SS	3	58		
15		- Same	SS	4	65		
20		- Same	SS	2	12/32/ 40/5"		
25		Test boring terminated at 21.5 ft, groundwater table measured 2.1 ft deep approximately 25 minutes after completion of drilling.					

LEGEND: SS - 2" O.D. Split-Spoon Sample
ST - 3" O.D. Shelby-Tube Sample
B - Bulk Sample

GROUNDWATER: Seal
Water Level
Observation Well Tip

LIU & ASSOCIATES, INC.

Geotechnical Engineering · Engineering Geology · Earth Science

BORING LOG
PAT HUNSAKER RESIDENCE
4017 WEST MERCER WAY
MERCER ISLAND, WASHINGTON

JOB NO. 10-004 DATE 2/2/2010 PLATE 4

BORING NO. B-2

Logged By: JSL 2/2/2010 Ground Elev. ±

Depth ft.	USCS	Soil Description	Sample Type	Sample No.	(N) Blows/ ft.	W %	Other Test
	SM	Dark-brown to dark-brown mixed, loose, silty fine SAND, with occasional wood debris and brick fragments, moist (FILL or DISTURBED SOIL)	SS	1	8		
	SP/SM	Gray, loose to medium-dense, slightly silty, fine to medium SAND, very moist to wet (OLDER SAND)					
5	ML	Light-gray, very-stiff to hard, fine sandy to clayey SILT, in about 1-in bedding, slighty moist (OLDER CLAY)	SS	2	44		
10	ML	Gray, hard to very-hard, clayey SILT, massive, slightly moist (OLDER CLAY)	SS	3	60		
15		- Same	SS	4	11/24/ 50/5"		
20		- Same	SS	2	24 & 44/5"		
25		Test boring terminated at 20.5 ft, groundwater table measured at 9.0 ft deep at completion of drilling.					

LEGEND: SS - 2" O.D. Split-Spoon Sample
ST - 3" O.D. Shelby-Tube Sample
B - Bulk Sample

GROUNDWATER: Seal
Water Level
Observation Well Tip

LIU & ASSOCIATES, INC.

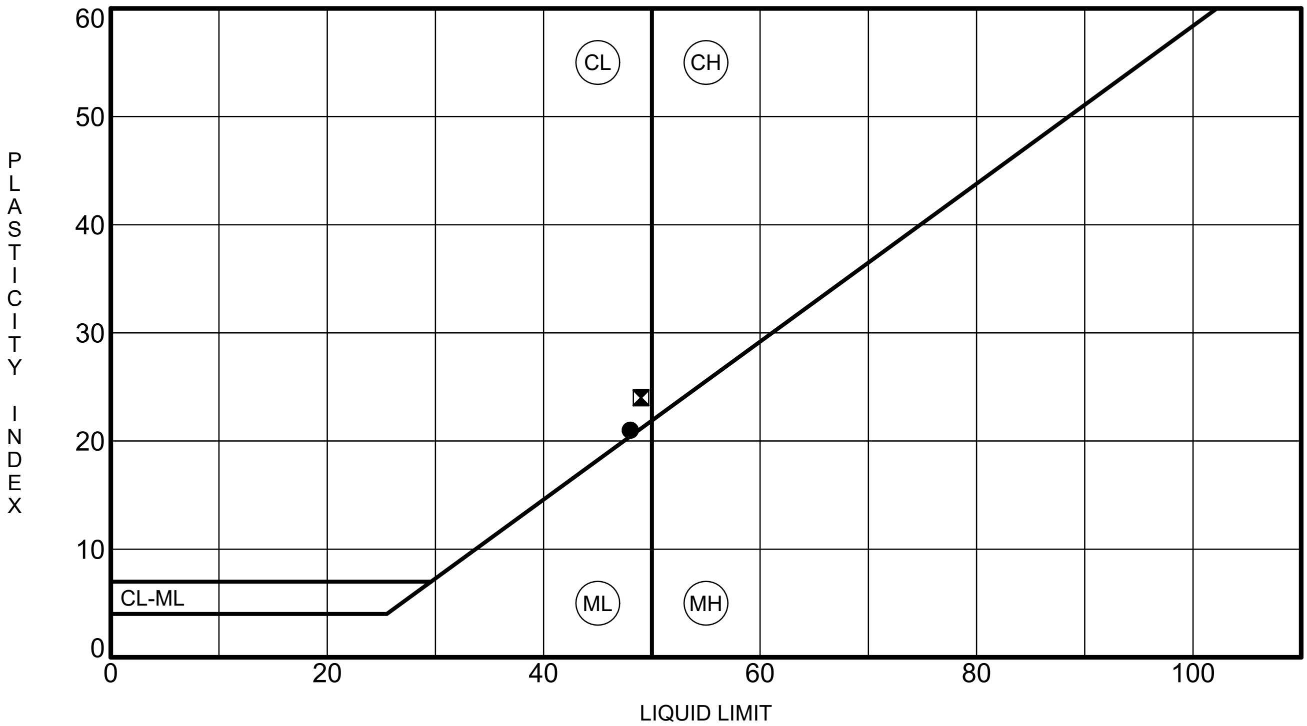
Geotechnical Engineering · Engineering Geology · Earth Science

BORING LOG
PAT HUNSAKER RESIDENCE
4017 WEST MERCER WAY
MERCER ISLAND, WASHINGTON

JOB NO. 10-004 DATE 2/2/2010 PLATE 5

APPENDIX C

LABORATORY TEST RESULTS



	Specimen Identification		M	LL	PL	PI	Fines	Classification
●	**PG-2**	**12.5**	**32**	**48**	**27**	**21**		**Lean CLAY (CL)**
⧗	**PG-2**	**15.0**	**35**	**49**	**25**	**24**		**Lean CLAY (CL)**

ATTERBERG LIMITS 18-282 4041 W MERCER WAY BORING LOGS.GPJ PANGEO.GDT 10/8/18

PanGEO INCORPORATED
Phone: 206.262.0370

ATTERBERG LIMITS

Project: Lundin Residence
Job Number: 18-282
Location: 4041 West Mercer Way, Mercer Island, WA 98040

Figure C-1