GEOTECHNICAL REPORT

PROPOSED ADU ADDITION
4602 EAST MERCER WAY
MERCER ISLAND, WASHINGTON

Project No. 20-365
October 2020

Prepared for:

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October 30, 2020
File No. 20-365

Shakiba Ahmadi
SUZANNE ZAHR, INC.
2441 76th Ave SE, Suite 160
Mercer Island, WA 98040

Subject: **Geotechnical Report**
Proposed ADU Addition
4602 East Mercer Way, Mercer Island, WA

Dear Shakiba:

Attached please find our geotechnical report for the proposed ADU Addition project in Mercer island, Washington. This report documents the subsurface conditions at the site and presents our geotechnical engineering recommendations for the proposed development.

In summary, based on the borings drilled, the project site is generally underlain by approximately 4 to 8 feet of fill overlying native medium dense to very dense, glacially-overridden sand. Based on the soil conditions and anticipated finish floor elevation, in our opinion, the proposed ADU may be supported by a combination of shallow footings and small diameter driven pipe piles (pin piles).

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

Sincerely,

H. Michael Xue, P.E.
Principal Geotechnical Engineer

Encl.: Geotechnical Report

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MERCER ISLAND, WASHINGTON

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering study that was undertaken to support the design and construction of the proposed ADU addition at the above site in Mercer Island, Washington. We completed our study in accordance with our proposal dated September 23, 2020, which was subsequently approved on September 24, 2020. Our service scope included reviewing available geologic and geotechnical data in the site vicinity, drilling two test borings and advance two hand borings at the site, performing engineering analyses, and developing the geotechnical design recommendations presented in this report.

2.0 PROJECT AND SITE DESCRIPTION

The project site is approximately 34,041 square foot waterfront lot located at 4602 East Mercer Way in the City of Mercer Island, Washington (see Vicinity Map, Figure 1). The site is approximately rectangular in shape, and is bordered to the west by East Mercer Way, to the east by Lake Washington, and to the north and south by existing single-family residences. A single-family house with a detached carport/storage occupies the eastern portion of the site. Based on review of the GIS maps, the site generally slopes down from west to east with an average gradient of about 22 percent with a total vertical relief of about 125 feet.

We understand that the proposed project will consist of removing the existing detached carport/storage structure located to the west of the single-family house, and to construct a new detached accessary dwelling unit (ADU) at the approximate same location. Based on review of the preliminary design plans, we also understand that the proposed detached ADU will be a 2-story wood frame structure. Additionally, the existing driveway will be widened toward south and a retaining wall up to 10 feet high will be needed to retain the slope on the south side of the driveway. We anticipate that temporary excavations up to about 10 feet will likely be needed for the foundation and retaining wall construction.

The conclusions and recommendations in this report are based on our understanding of the proposed development, which is in turn based on the project information provided. If the above project description is incorrect, or the project information changes, we should be consulted to review the recommendations contained in this study and make modifications, if needed.

3.0 SUBSURFACE EXPLORATIONS

PanGEO completed two test borings (PG-1 and PG-2) at the subject site on October 15, 2020. The approximate locations of the borings are indicated on the attached Figure 2. The borings were drilled to depths of about 16½ feet in PG-1 and 21½ feet in PG-2, using a hand-operated portable drill rig owned and operated by CN Drilling of Seattle, Washington.

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

A geologist from PanGEO was present during the field exploration to observe the drilling, assist in sampling, and to describe and document the soil samples obtained from the borings. The soil samples were described and field classified in general accordance with the symbols and terms outlined in Figure A-1, and the summary boring logs are included as Figures A-2 and A-3.

In addition to PG-1 and PG-2, two hand borings (HB-1 and HB-2) were excavated around the proposed ADU structure to further evaluate the near-surface soil conditions on October 27, 2020. The approximate hand boring locations are also plotted on Figure 2. The hand borings were excavated to depths of about 6 to 7½ feet, the maximum depths for hand auger refusal and feasible depth for hand tools, below the existing grade. The hand borings were excavated using hand auger and tools, and the relative density and consistency of the underlying soil was estimated based on probing the soils inside the hand borings and the difficulty of completing the excavation. The summary hand boring logs are included as Figures A-4 and A-5.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Site Geology

The Geologic Map of Mercer Island (Troost and Wisher, 2006) mapped the surficial geologic unit at the subject site as Pre-Olympia Nonglacial Deposits (Qpon), with Lake Deposits (Ql) mapped along the lakeshore.

> Lake Deposit (Ql) typically consists of very loose to loose sand to very soft to medium stiff silt and clay with peat and other organic sediments deposited adjacent to Lake Washington.
>
> Pre-Olympia Non-Glacial deposits (Qpon) are described by Troost, et al. as dense and hard, sand, gravel, silt, clay, and organic deposits of nonglacial origin that had been overridden by Olympia Interglaciation.

4.2 Soil

The soil conditions encountered in the test borings generally consisted of fill overlying Pre-Olympia Non-Glacial Deposit. A summary description of the generalized soil units encountered in the test borings is presented below. Please refer to the summary boring logs in Appendix A for more details.

> ***UNIT 1: Fill*** – Fill was encountered from surface to depths of about 4 and 8 feet in PG-1 and PG-2, respectively. Fill encountered generally consisted of very loose to loose, sand to silty sand with occasional rootlets and organics.
>
> ***UNIT 2: Pre-Olympia Non-Glacial Deposits*** – Below fill, the borings generally encountered medium dese to very dense, fine to medium sand. This unit extended to the termination depths in of 21½ and 26½ feet in PG-1 and PG-2, respectively. We interpret this unit as the Pre-Olympia Non-Glacial deposit mapped at the site.

4.3 Groundwater

Groundwater was not encountered within the drilling depths during drilling. It should be noted that groundwater levels will vary depending on the season, local subsurface conditions, and other factors. Groundwater levels are normally highest during the winter and early spring.

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. A site reconnaissance of the subject property was conducted on October 15, 2020. The site is well developed with structures. During our site reconnaissance, we did not observe obvious evidence of slope instability or ground movement at the site. Based on our field observations and the results of our subsurface explorations, in our opinion, the subject site appears to be globally stable in its current configuration. Furthermore, it is our opinion that the proposed ADU 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 design and constructed.

5.2 EROSION HAZARDS EVALUATION

The majority of the site is mapped as a potential erosion hazard area in accordance with the City of Mercer Island's Geologic Hazards Map. Based on the USDA Soil Survey data and our test borings, the site soils are anticipated to exhibit moderate erosion potential when disturbed and left unprotected. However, 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 also be effectively managed with an appropriate erosion and sediment control plan, including but not limited to installing a silt fence at the construction perimeter, placing quarry spalls or hay bales at the disturbed and traffic areas, covering stockpiled soil or cut slopes with plastic sheets, constructing a temporary drainage pond to control surface runoff and sediment trap, placing rocks at the construction entrance, etc.

Permanent erosion control measures should be applied to the disturbed areas as soon as feasible. These measures may include but not limited to planting and hydroseeding. The use of permanent erosion control mat may also be considered in conjunction with planting/hydroseeding to protect the soils from erosion.

5.3 Seismic Hazards

Based on our review of the City of Mercer Island's Geologic Hazards Maps, the subject site is mapped within 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, and soil liquefaction or surface faulting.

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, must be loose, and be below the groundwater table.

Based on the dense soil conditions below fill and lack of groundwater table within the drilling depths in the borings, in our opinion, the potential for soil liquefaction during an IBC-code level earthquake at the site is considered negligible, and therefore, special design considerations associated with soil liquefaction is not needed for this project.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 Seismic Design Parameters

The Table 1 provides seismic design parameters for the site that are in conformance with the 2015 editions of the International Building Code (IBC), which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps. The spectral response accelerations were obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2008 data) for the project latitude and longitude.

Table 1 – Summary Seismic Design Parameters per 2015 IBC

Site Class	Spectral Acceleration at 0.2 sec. (g)	Spectral Acceleration at 1.0 sec. (g)	Site Coefficients		Design Spectral Response Parameters	
	S_S	S_1	F_a	F_v	S_{DS}	S_{D1}
D	1.418	0.544	1.0	1.50	0.945	0.544

6.2 BUILDING FOUNDATIONS

6.2.1 General

Based on the soil conditions and anticipated building finish floor elevations, in our opinion, the proposed ADU may be supported on a combination of small diameter pipe piles (pin piles) and conventional shallow footings. The northern half of the ADU should be supported on pin piles and the southern half of the ADU may be supported on conventional footings. The following sections present our recommendations for the pin pile foundations and shallow footings.

6.2.2 Pin Pile Foundations

***Pin Pile Sizes* -** In our opinion, 3-, 4-inch diameter, Schedule 40, steel pipes (pin piles) may be used to support the northern half of the ADU. Three, four-inch diameter pin piles are typically installed using small hammers mounted on a small excavator.

Pin Pile Capacity - The number of piles required depends on the magnitude of the design load. Allowable axial compression capacities of 6 and 10 tons may be used for the 3-, 4-inch diameter pin piles, respectively, with an approximate factor of safety of 2. Penetration resistance required to achieve the capacities will be determined based on the hammer used to install the pile. Tensile capacity of pin piles should be ignored in design calculations.

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

Pile splices may be made with compression fitted sleeve pipe couplers (see Typical Splicing Detail on page 8). Splicing using welding of pipe joints should not be used, as welds will typically be broken during driving.

Three-, four-, and six-inch diameter piles are typically installed using small (approximately 850 to 3,000 pound) hammers mounted to a small excavator. The criterion for driving refusal is defined as the minimum amount of time (in seconds) required to achieve one inch of penetration, and it varies with the size of hammer used for pile driving. For 3-, 4-inch pin piles, the Table 2 is a summary of driving refusal criteria for different hammer sizes that are commonly used:

Table 2 - Summary of Commonly-Accepted Driving Criteria for 3-, 4-inch Pin Pile with a 6, 10-ton Allowable Axial Compression Load

Hammer Model	Hammer Weight (lb) / Blows per minute	3" Pile Refusal Criteria (seconds per inch of penetration)	4" Pile Refusal Criteria (seconds per inch of penetration)
Hydraulic TB 325	850 / 900	10	16
Hydraulic TB 425	1,100 / 900	6	10
Hydraulic TB 725X	2,000 / 600	3	4

Please note that these refusal criteria were established empirically based on previous load tests on 3-, 4-inch pin piles. Contractors may select a different hammer for driving these piles, and propose a different driving criterion. In this case, it is the contractor's responsibility to demonstrate to the Engineer's satisfaction that the design load can be achieved based on their selected equipment and driving criteria.

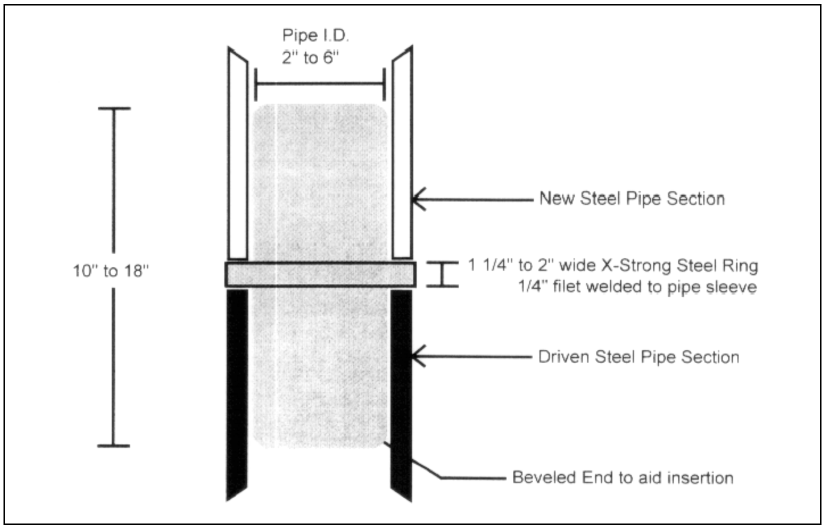
Pin Pile Specifications - We recommend that the following specifications be included on the foundation plan:

1. Three-, four-, or six-inch diameter piles should consist of Schedule-40, ASTM A-53 Grade "A" pipe.

2. The piles shall be driven to refusal as shown in Table 2 above.

3. Piles shall be driven in nominal sections and connected with compression fitted sleeve couplers (see typical detail on below) We discourage welding of pipe joints, particularly when galvanized pipe is used, as we have frequently observed welds broken during driving.

4. The geotechnical engineer of record or his/her representative shall observe pin pile installation.



Typical Splicing Detail

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

Lateral Forces - The capacity of pin pipes to resist lateral loads is very limited and should not be used in design. Therefore, lateral forces from wind or seismic loading should be resisted by the passive earth pressures acting against the pile caps and below-grade walls or from battered piles (batter no steeper than 3(H):12(V)). ***Friction at the base of pile-supported concrete grade beam should be ignored in the design calculations.*** Passive resistance values may be determined using an equivalent fluid weight of 200 pounds per cubic foot (pcf). This value includes a safety factor of about 1.5 assuming that properly compacted granular fill will be placed adjacent to and surrounding the pile caps and grade beams.

Grade Beam/Pile Cap Embedment - We recommend that the grade beams and pile caps located around the perimeter of the structure be embedded such that the bottom of the grade beam is at least 16 inches below the adjacent ground surface.

Estimated Pile Length – The subsurface conditions at the site will likely vary substantially across the site. Based on the soil conditions at the site and our experience in the project area, for planning and cost estimating purposes, we estimate that pin pile lengths of about 20 to 30 feet.

Obstructions – Obstructions may be encountered during pile driving. Where possible, the obstructions should be removed to facilitate the pile driving. If obstructions cannot be removed, the structural engineer of record should be notified to revise the pile layout to accommodate the adjustment.

6.2.3 Conventional Footings

As previously indicated, the southern half of the ADU may be supported on the conventional shallow footings. The following sections present our recommendations for the shallow footing design. In designing the footings, the shape of footings will need to be considered regarding the available space for temporary excavations. Where space may be limited for an unsupported open cut, it may be necessary to use L-shaped perimeter footings in order to conserve space and to allow the temporary excavations to be made within the property limits.

Allowable Bearing Pressure – We recommend that an allowable soil bearing pressure of 2,000 pounds per square feet (psf) be used to size the footings, bearing on the native competent soils or compacted structural fill/lean-mix concrete placed on the native dense soils. The recommended allowable bearing pressure is for dead plus live loads. For allowable stress design, the recommended bearing pressure may be increased by one-third for transient loading, such as wind or seismic forces. Continuous and individual spread footings should have minimum widths of 18 and 24 inches, respectively. Footings should be placed at least 18 inches below final exterior grade. Interior footings should be placed at least 12 inches below the top of slab.

Foundation Performance – Total and differential settlements are anticipated to be within tolerable limits for foundation designed and constructed as discussed above. For the proposed building supported by conventional footings bearing on competent native soils and structural fill/lean-mix concrete, the building settlement under static loading conditions is estimated to be approximately one inch, and differential settlement should be on the order of about ½ inch. Most settlement should occur during construction as loads are applied.

Lateral Resistance – Lateral forces from wind or seismic loading may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundations and walls, and by friction acting on the base of the foundations. Passive resistance values may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf). This value includes a factor safety of at least 1.5 assuming that densely compacted structural fill will be placed adjacent to the sides of the foundation. A friction coefficient of 0.35 may be used to determine the frictional resistance at the base of the foundation. This coefficient includes a factor of safety of approximately 1.5. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches of soil should be neglected.

Footing Subgrade Preparation – All footing subgrades should be carefully prepared. The adequacy of footing subgrades should be verified by a representative of PanGEO, prior to placing forms or rebar. The footing subgrades should be in a dense condition prior to concrete pour. Any over-excavations in the footing areas should be backfilled with compacted CSBC/Gravel Borrow or lean-mix concrete/CDF. Footing excavations should be observed by PanGEO to confirm that the exposed footing subgrades are consistent with the expected conditions and adequate to support the design bearing pressure.

6.3 Concrete Retaining Walls

Retaining 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 that may be present behind the wall. Our geotechnical recommendations for the design and construction of the retaining/basement walls are presented below.

6.3.1 Lateral Earth Pressures

Concrete cantilever walls should be designed for an equivalent fluid pressure of 35 pcf for level backfills behind the walls assuming the walls are free to rotate. If walls are to be restrained at the top from free movement, such as basement walls, equivalent fluid pressures of 45 pcf should be used for level backfills behind the walls. Walls with a maximum 2H:1V backslope should be designed for an active and at rest earth pressure of 50 and 60 pcf, respectively.

Permanent walls should be designed for an additional uniform lateral pressure of 8H psf for seismic loading, where H corresponds to the buried depth of the wall. The recommended lateral pressures assume that the backfill behind the wall consists of a free draining and properly compacted fill with adequate drainage provisions.

6.3.2 Wall Surcharge

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

6.3.3 Lateral Resistance

Lateral forces from wind or seismic loading and unbalanced lateral earth pressures may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundation and by friction acting on the base of the foundation. Passive resistance values may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf). A friction coefficient of 0.35 may be used to determine the frictional resistance at the base of the foundation. Both of these values include a safety factor of at least 1.5.

6.3.4 Wall Drainage

Provisions for wall drainage should consist of a 4-inch diameter perforated drainpipe placed behind and at the base of the wall footings, embedded in 12 to 18 inches of clean crushed rock or pea gravel wrapped with a layer of filter fabric. A minimum 18-inch wide zone of free draining granular soils (i.e. clean washed or crushed rock) is recommended to be placed adjacent to the wall for the full height of the wall. Alternatively, a composite drainage material, such as Miradrain 6000, may be used in lieu of the clean crushed rock or pea gravel. This alternative may be preferable if a soldier pile system is used for temporary shoring. The drainpipe at the base of the wall should be graded to direct water to a suitable outlet.

6.3.5 Wall Backfill

In our opinion, the on-site excavated soils are not suitable for use as wall backfill. We recommended that wall backfill consist of free draining granular soils, such as Seattle Mineral Aggregate Type 17 (2014 City of Seattle Standard Specifications, 9-03.12(2)) or approved equivalent.

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

6.4 Concrete Slab-On-Grade

In our opinion, conventional concrete slab-on-grade floors may be used. The floor slabs should be supported on recompacted on-site sand or compacted structural fill. Any on-site soils at the slab subgrade that cannot be recompacted to a dense condition should be over-excavated 12 inches and over-excavation should be replaced with compacted structural fill.

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

6.5 Temporary Excavation and Shoring

As currently planned, excavations up to about 10 feet are needed for the foundation and retaining wall construction. We anticipate the excavations to mainly encounter fill over medium dense to very dense native sand. All temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring.

6.5.1 Unsupported Open Cuts

Based on the subsurface conditions at the site, for planning purposes, it is our opinion that temporary excavations for the proposed construction may be sloped 1H:1V or flatter. Based on our current understanding of the building layout and anticipated finished floor elevations, it appears that sufficient space is available for unsupported open cuts except along the retaining wall on the south side of the driveway. Where sufficient space is not available for unsupported open cuts, temporary shoring will need to be used to support the temporary excavations. Where space may be limited, the use of L-shaped footings may be required to conserve space for the temporary cuts.

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

6.5.2 Temporary Shoring Wall

Where sufficient space is not available for unsupported open cuts, temporary shoring will be needed to support the excavations. In our opinion, a soldier pile wall is considered appropriate to support the temporary excavations at the retaining wall location. The following sections present our recommendations for the design of cantilevered soldier pile wall:

Design Earth Pressure – We recommend that the design parameters outlined in Figure 3 be used for the soldier pile wall design. Above the bottom of excavation, the recommended active earth pressure should be applied over the full width of the pile spacing. Below the bottom of excavation, the passive resistance should be applied over two times the pile diameter and the active and at-rest pressure applied over one single pile diameter. The recommended passive earth pressure assumes level ground surface at the bottom of the excavation. The soldier piles should have a minimum embedment of 10 feet.

If the soldier piles will be incorporated into the design of permanent walls, a uniform seismic earth pressure indicated in Figure 3 should be included in design calculations. The seismic pressure should be applied to the portion located above the bottom of the excavation.

Surcharge Loads – The shoring walls should be designed to accommodate surcharge pressures if surcharge loads are located within the height dimension of the wall. Depending on the shoring wall location, potential surcharge from the existing buildings to the west and north will need to be considered in the shoring wall design and can be estimated using Figure 3.

It should be noted that heavy point loads located close to the top of the walls, such as outriggers of heavy cranes or pump trucks, should be individually analyzed and incorporated into the wall design.

Vertical Capacity – If soldier piles will be used as a permanent foundation system, the soldier piles should have a minimum embedment of 10 feet into the underlying native soil to achieve adequate axial capacities. Soldier piles incorporated into the permanent load bearing system may be designed using an allowable skin friction value of 1.0 ksf for the portion of the pile below the bottom of the excavation, and an allowable end bearing value of 15 ksf.

Design Top of Pile Deflection – In general, the top of shoring piles should be designed with one-inch deflection or less. However, where shoring walls are located next to the existing structures, the shoring walls should be designed to have smaller deflection so that the shoring wall movement will not impact the existing structures.

Permanent Walls – Permanent soldier piles should be properly protected against corrosion. This may include proper coatings or upsizing of piles. In addition, it should be noted that timber lagging between soldier piles have limited design life, and the installation of a permanent concrete facing in front of the timber lagging may be considered.

Lagging – Lagging design recommendations for general conditions are presented on Figure 3. Lagging located within 10 feet of the top of the shoring which may be subjected to surcharge loads from construction equipment of material storage should be individually analyzed.

The voids behind lagging may be backfilled with clean crushed rocks if no severe sloughing occurs during excavations. If severe sloughing occurs during lagging installation, we recommend Control Density Fill (CDF) be used as backfill behind the lagging.

Construction Considerations – We recommend that the following should be incorporated into the project plans and specifications:

- The geotechnical engineer shall verify the suitability of all soldier pile holes before concrete placement;
- Tremie methods shall be used for concrete placement in all holes having 6 or more inches of accumulated water if perched ground water or heavy precipitation is encountered during construction.
- All soldier pile holes drilled shall be filled with lean concrete mix or structural concrete on the same day.
- If soldier piles are designed as a permanent foundation system, any soft/disturbed soils encountered at the bottom of the hole during drilling will need to be removed using a cleanout bucket prior to placing the steel beam and concrete.

When placing timber lagging, the height of each lift may need to be limited to prevent the un-shored soil from escaping through the base of the timber boards. We recommend that the soil exposed for timber lagging be no more than 4 to 5 feet deep. The actual allowable vertical cut for timber lagging placement should be determined in the field, based on the actual conditions observed.

Performance Monitoring – Because ground deformations will occur due to the excavation (open cut or shored), we recommend that existing conditions on the adjacent private properties be photo-documented prior to the start of the project. We also recommend that survey points be installed on every other soldier pile and on adjacent structures.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 SITE PREPARATION

Site preparation for the proposed project includes removing the existing structure, clearing and excavations to the design subgrade. All stripped surface materials should be properly disposed off-site or be "wasted" on site in non-structural landscaping areas.

Following site excavations, the adequacy of the subgrade where structural fill, foundations, slabs, or pavements are to be placed should be verified by a representative of PanGEO. The subgrade soil in the improvement areas, if recompacted and still yielding, should also be over-excavated and replaced with compacted structural fill or CDF/lean-mix concrete.

7.2 MATERIAL REUSE

In the context of this report, structural fill is defined as compacted fill placed under footings, concrete stairs and landings, and slabs, or other load-bearing areas. In our opinion, the on-site soils are not suitable to be reused as structural fill. Structural fill should consist of imported, well-grade, granular material, such as WSDOT CSBC or Gravel Borrow, or approved equivalent. Well-graded recycled concrete may also be considered as a source of structural fill. Use of recycled concrete as structural fill should be approved by the geotechnical engineer. The on-site soil can be used as general fill in the non-structural and landscaping areas. If use of the on-site soil is planned, the excavated soil should be stockpiled and protected with plastic sheeting to prevent softening from rainfall in the wet season.

7.3 STRUCTURAL FILL PLACEMENT AND COMPACTION

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

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

7.4 Erosion and Drainage 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 to collect runoff and prevent water from entering the excavation. All collected water should be directed to a positive and permanent discharge system such as a City of Seattle storm sewer.

It should be noted that the site soils are prone to surficial erosion. Special care should be taken to avoid surface water on open cut excavations. We recommend that the exposed temporary slopes be covered with plastic sheeting.

Permanent control of surface water and roof runoff should be incorporated in the final grading design. In addition to these sources, irrigation and rain water infiltrating into landscape and planter areas adjacent to paved areas or building walls should also be controlled. All collected runoff should be directed into conduits that carry the water away from the pavement or structure and into City of Seattle storm drain systems or other appropriate outlets. Adequate surface gradients should be incorporated into the grading design such that surface runoff is directed away from structures.

7.5 Wet Earthwork Recommendations

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below:

- Earthwork should be performed in small areas to minimize subgrade exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.

- During wet weather, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight based on the portion passing ¾-inch sieve. The fines should be non-plastic.

- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water.

- Geotextile silt fences should be strategically located to control erosion and the movement of soil. Erosion control measures should be installed along all the property boundaries.

- Excavation slopes and soils stockpiled on site should also be covered with plastic sheets.

8.0 ADDITIONAL SERVICES

To confirm that our recommendations are properly incorporated into the design and construction of the proposed residence, 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, will also require geotechnical construction inspection services. PanGEO can provide you a cost estimate for construction monitoring services at a later date.

9.0 LIMITATIONS

We have prepared this report for use by Suzanne Zahr, Inc. and the project design team. Recommendations contained in this report are based on a site reconnaissance, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of work.

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

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

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

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

Within the limitation of scope, schedule and budget, PanGEO engages in the practice of geotechnical engineering and endeavors to perform its services in accordance with generally accepted professional principles and practices at the time the Report or its contents were prepared. No warranty, express or implied, is made.

We appreciate the opportunity to be of service to you on this project. Please feel free to contact our office with any questions you have regarding our study, this report, or any geotechnical engineering related project issues.

Sincerely,

John Manke

HONGBO MICHAEL XUE
STATE OF WASHINGTON
35016
REGISTERED
PROFESSIONAL ENGINEER
10/30/2020

John A. Manke, G.I.T.
Staff Geologist

H. Michael Xue, P.E.
Principal Geotechnical Engineer

10.0 REFERENCES

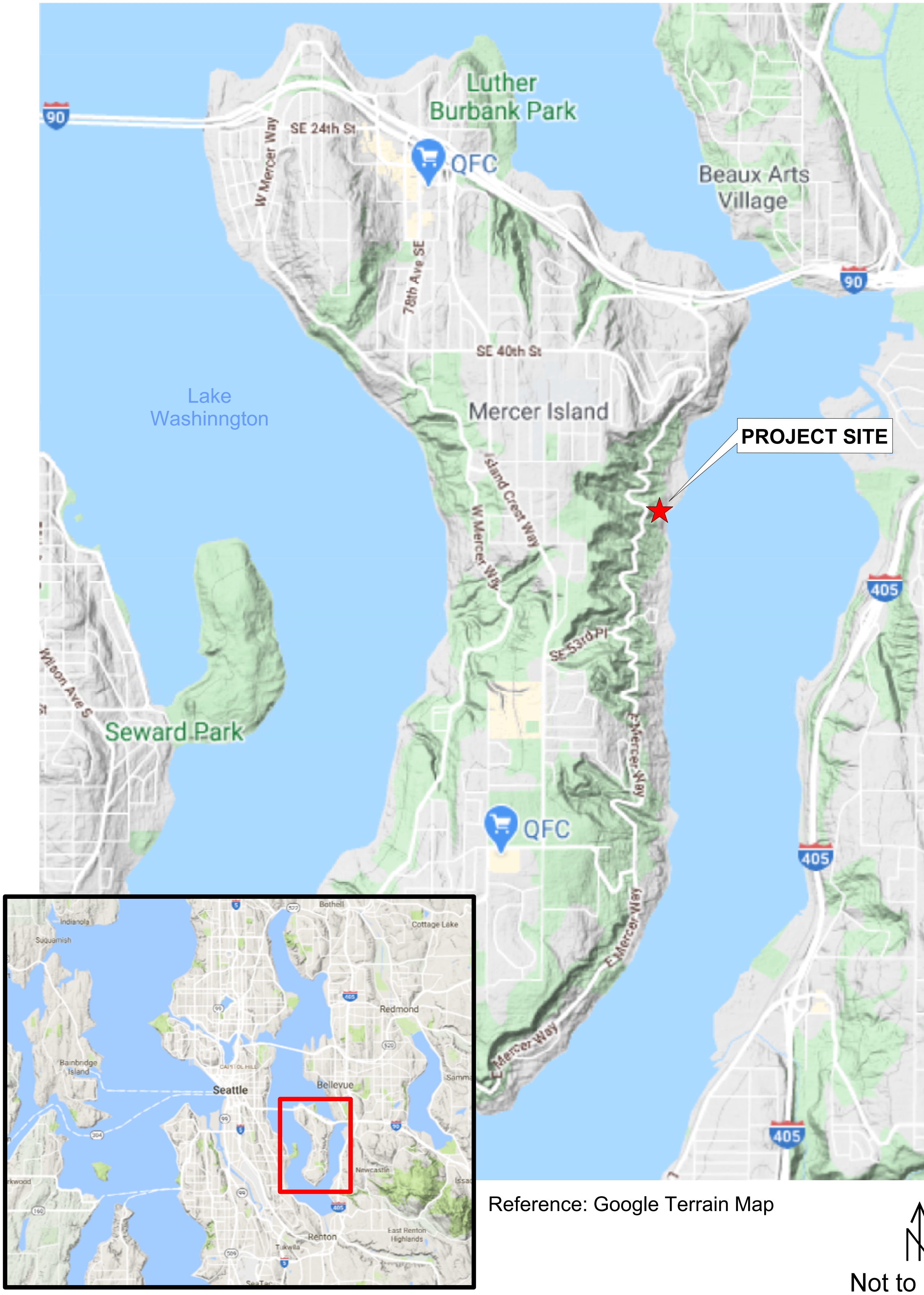
ASTM D1586-11, *Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*, ASTM International, West Conshohocken, PA, 2011, www.astm.org.

ASTM D2488-17, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedures)*, ASTM International, West Conshohocken, PA, 2017, www.astm.org.

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

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

WSDOT, 2020, *Standard Specifications for Road, Bridges, and Municipal Construction*.

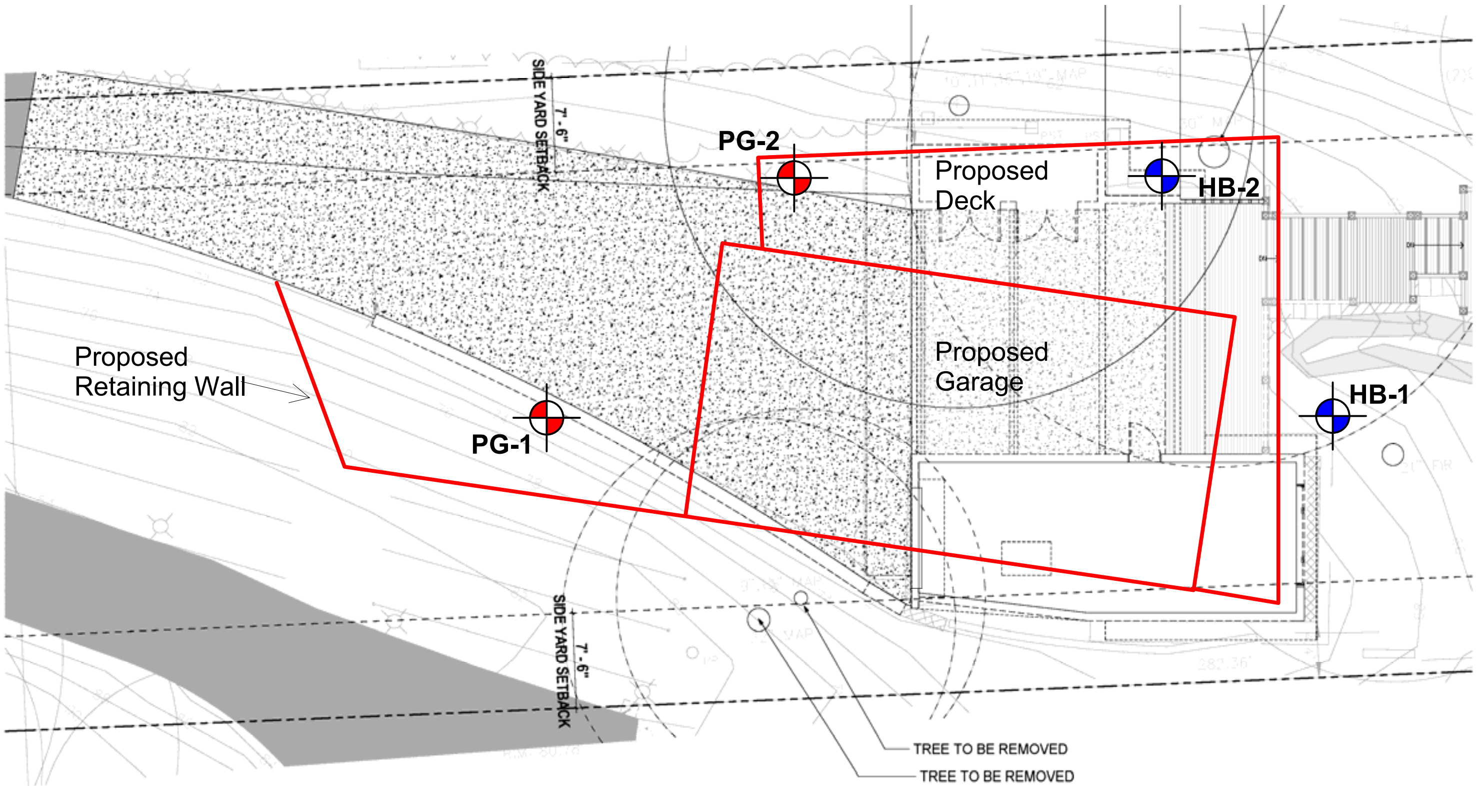


Reference: Google Terrain Map

N

Not to Scale

PanGEO INCORPORATED	**Proposed ADU Addition** **4602 East Mercer Way** **Mercer Island, Washington**	**VICINITY MAP**	
		Project No. **20-365**	Figure No. **1**



Approx. Scale
1" = 15'

Legend:

Approx. Test Boring Locations

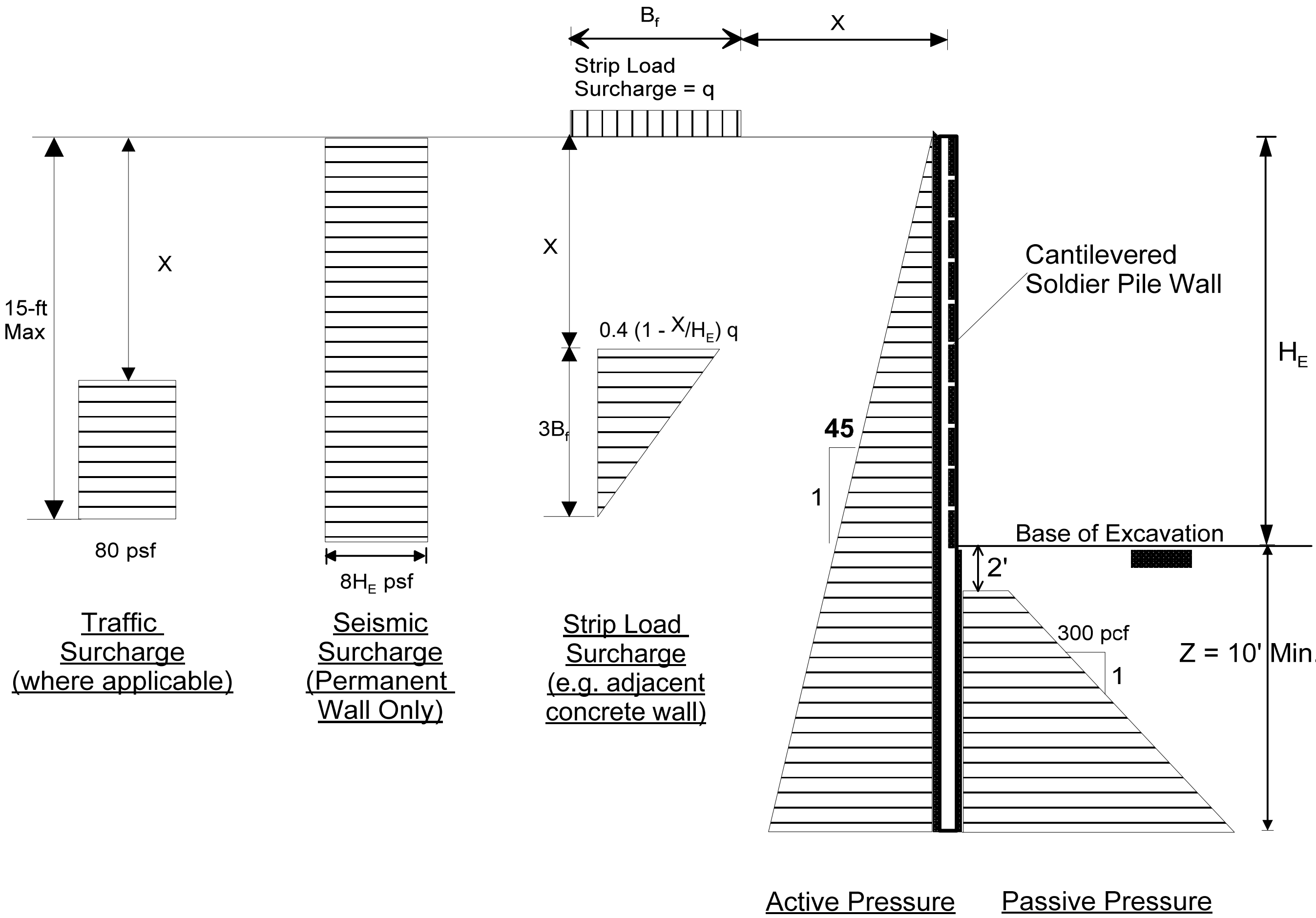
Approx. Hand Boring Locations

PanGEO INCORPORATED

Proposed ADU Addition
4602 East Mercer Way
Mercer Island, Washington

SITE AND EXPLORATION PLAN

Project No.	Figure No.
20-365	**2**



LEGEND

H_E = Height of Excavation (ft)

Z = Embedment Depth (min. 10 ft)

Notes:
1. Embedment (Z) should be determined by summation of moments at the bottom of the soldier piles. Minimum embedment should be at least 10 feet.
2. A factor of safety of 1.5 has been applied to the recommended passive earth pressure value. No factor of safety has been applied to the recommended active earth pressure values.
3. Active and surcharge 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. Passive pressure should be applied to two times the diameter of the soldier piles.
5. Use uniform earth pressure of 200 psf and 250 psf for lagging design with soldier piles spaced at less than 8 feet and greater than 8 feet, respectively.
6. If the soldier pile wall will be used as part of the permanet foundation system, a uniform lateral earth pressure of 8H psf (where H is the wall height) should be added to the static pressure for evaluating the seismic condition.
7. Shoring Walls located next to the existing structures should be designed to have smaller deflection so that the shoring wall movement will not impact the existing structures.
8. Refer to report text for additional discussions.

PanGEO INCORPORATED	**Proposed ADU Addition** **4602 E Mercer Way** **Mercer Island, Washington**	**DESIGN LATERAL EARTH PRESSURES** **CANTILEVERED SOLDIER PILE WALL** Project No. **20-365** Figure No. **3**

APPENDIX A

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

COMPONENT DEFINITIONS

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

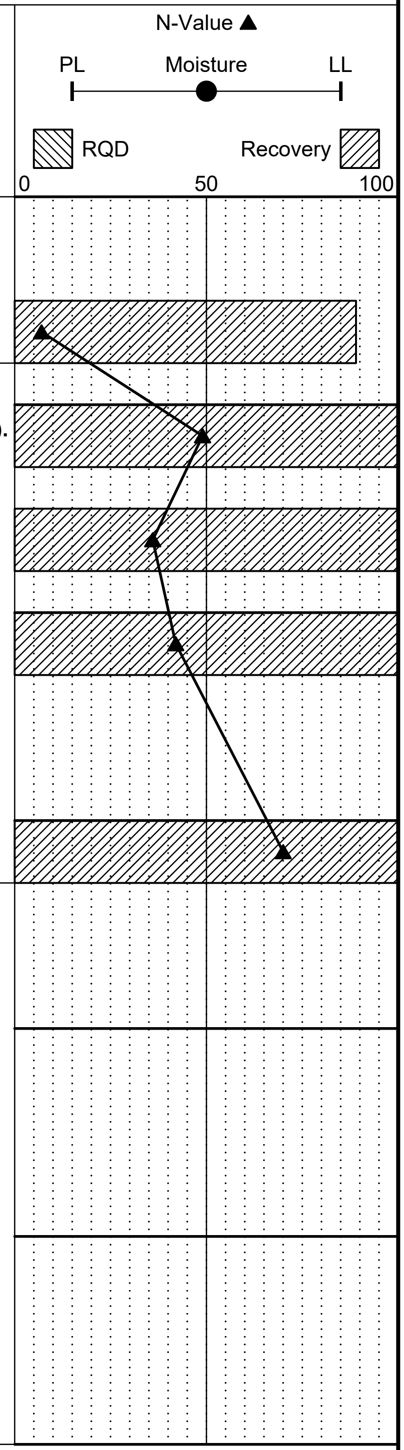
- Groundwater Level at time of drilling (ATD)
- Static Groundwater Level
- Cement / Concrete Seal
- Bentonite grout / seal
- Silica sand backfill
- Slotted tip
- Slough
- Bottom of Boring

MOISTURE CONTENT

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

Project:	4602 E Mercer Way	Surface Elevation:	72.0ft
Job Number:	20-365	Top of Casing Elev.:	N/A
Location:	4602 E Mercer Way, MI	Drilling Method:	HSA
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						Loose, moist, brown, medium SAND with trace gravel and rootlets. **(Fill).**
	S-1		1 3 4			
5	S-2		12 21 28			Dense, moist, light brown, clean, fine to medium SAND; occasional thin layers of iron oxide staining. **(Pre-Olympia Non-Glacial Deposits).**
	S-3		9 16 20			
10	S-4		15 18 24			Becomes fine sand.
15	S-5		17 30 40			Becomes very dense.
						Boring terminated at about 16.5 feet below ground surface. Groundwater was not observed during drilling.
20						
25						
30						



Completion Depth:	16.5ft
Date Borehole Started:	10/15/20
Date Borehole Completed:	10/15/20
Logged By:	J. Manke
Drilling Company:	CN Drilling

Remarks: Borings drilled using an acker hand portable drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Surface elevation estimated based on Site Plan provided by the client

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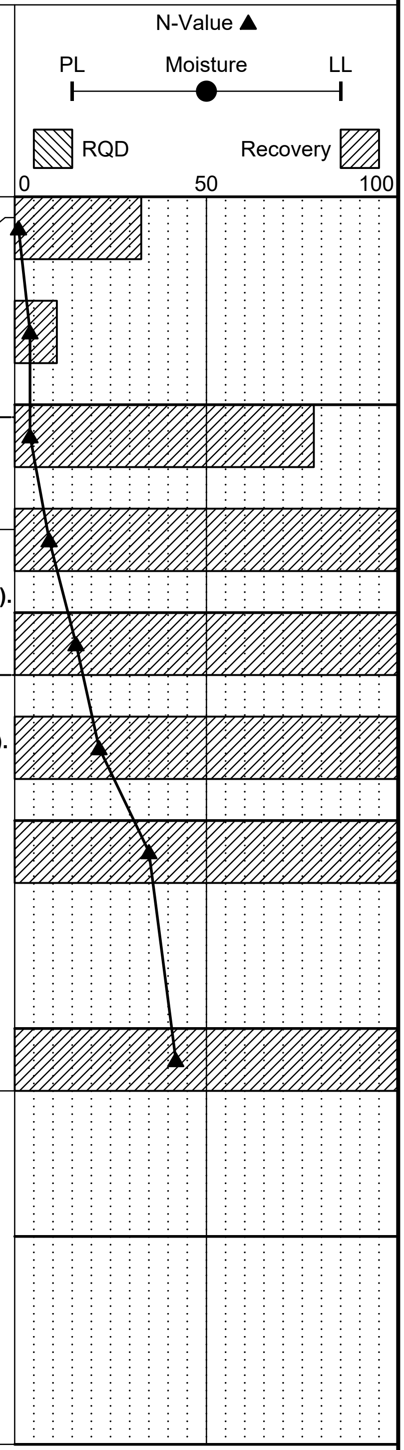
LOG OF TEST BORING PG-1

Figure A-2

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

Project:	4602 E Mercer Way	Surface Elevation:	68.0ft
Job Number:	20-365	Top of Casing Elev.:	N/A
Location:	4602 E Mercer Way, MI	Drilling Method:	HSA
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0	S-1		1 0 1			Landscaping mulch and topsoil: 6 inches thick.
						Very loose, moist, light brown to brown, slightly silty SAND; with rootlets and organics. **(Fill).**
	S-2		1 2 2			
5	S-3		1 2 2			Very loose, moist, light brown, clean, fine to medium SAND with trace fine gravel.
	S-4		7 4 5			Loose to medium dense, moist, light brown, clean, fine to medium SAND with gravel. **(Weathered Pre-Olympia Non-Glacial Deposits).**
10	S-5		5 7 9			
						Medium dense, moist, light brown, clean, fine to medium SAND; occasional thin layers of iron oxide staining. **(Pre-Olympia Non-Glacial Deposits).**
	S-6		4 8 14			
15	S-7		7 15 20			Becomes dense.
20	S-8		8 16 26			Becomes fine sand.
						Boring terminated at about 21.5 feet below ground surface. Groundwater was not observed during drilling.
25						
30						



Completion Depth:	21.5ft
Date Borehole Started:	10/15/20
Date Borehole Completed:	10/15/20
Logged By:	J. Manke
Drilling Company:	CN Drilling

Remarks: Borings drilled using an acker hand portable drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Surface elevation estimated based on Site Plan provided by the client

PanGEO INCORPORATED

LOG OF TEST BORING PG-2

Figure A-3

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

Figure A-4: Summary Log of Hand Boring HB-1

Approximate ground surface elevation: 67 feet

Depth (ft)	Material Description
0 – 2	**Unit 1:** 4 inches of mulch over loose, moist, brown, slightly silty SAND with trace gravel (SP-SM); numerous roots [Fill]
2 – 6	**Unit 3:** Medium dense, moist, light-brown, fine to medium SAND with gravel (SP); [Pre-Olympia Non-Glacial Deposits] - Becomes dense at 4½ feet

Notes:

1. HB-1 was terminated at 6 feet below ground surface.
2. Groundwater was not observed in the boring.

Plate 1: Cuttings from about 4 foot below surface in HB-1.

Figure A-5: Summary Log of Hand Boring HB-2

Approximate ground surface elevation: 66 feet

Depth (ft)	Material Description
0 – 5½	**Unit 1:** Loose, moist, brown, slightly silty SAND with trace gravel (SP-SM); numerous roots, angular gravel, and concrete fragments [Fill]
5½ – 7½	**Unit 3:** Medium dense, moist, light-brown, fine to medium SAND with gravel (SP); [Pre-Olympia Non-Glacial Deposits]

Notes:

3. HB-2 was terminated at 7½ feet below ground surface.
4. Groundwater was not observed in the boring.

Plate 2: Cuttings from about 6 feet below surface in HB-2.