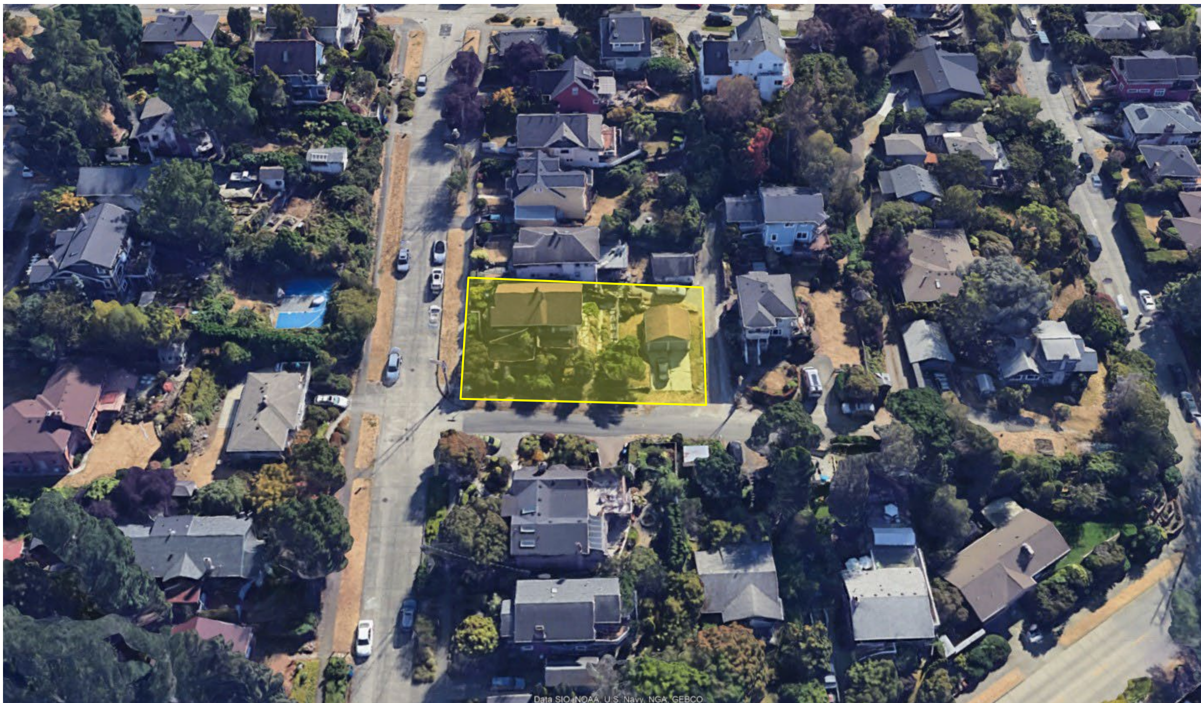
GEOTECHNICAL ENGINEERING REPORT
PROPOSED SINGLE-FAMILY RESIDENCE & DADU
5303 46TH AVENUE SW
SEATTLE, WASHINGTON

Project No. 22-238
June 14, 2023



Credit: Google Earth

Prepared for:
Mr. Bryan Bentrott

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June 14, 2023
PanGEO Project No. 22-238

Bryan Bentrott
5303 46th Avenue Southwest
Seattle, Washington

Re: **Geotechnical Engineering Report**
Proposed Single-Family Residence & DADU
5303 46th Avenue SW, Seattle, Washington

Dear Mr. Bentrott:

Please find attached our geotechnical engineering report to support the design and construction of the proposed single-family residence and detached accessory dwelling unit (DADU) located at the above address, in Seattle, Washington. This report documents the subsurface conditions at the site and our geotechnical engineering recommendations for the proposed project.

In summary, based on the results of our subsurface explorations advanced at the site, the eastern portion of the property is underlain to approximately 15 to 17½ feet of very loose to medium dense sand, over medium dense to dense native sand, while the lower, eastern portion of the site is underlain by about 11 feet of loose sand, over medium dense to hard native sand and silt. In our opinion the proposed residence may be supported by small diameter driven pipe piles (also known as pin piles) bearing in the competent soils below the loose surficial soils. The proposed single-story DADU in the western portion of the site may be supported by a mat foundation, or by driven pipe piles if the DADU will be two-levels. Groundwater is not anticipated within the depth of the proposed excavations.

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

Sincerely,

Jon C. Rehkopf, P.E.
Principal Geotechnical Engineer

Encl.: Geotechnical Engineering Report

TABLE OF CONTENTS

LIST OF FIGURES

GEOTECHNICAL ENGINEERING REPORT
PROPOSED SINGLE-FAMILY RESIDENCE & DADU
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SEATTLE, WASHINGTON

1.0 INTRODUCTION

This report presents the results of geotechnical engineering studies that were undertaken to support the design and construction of the proposed single-family residence and DADU at 5303 46th Avenue Southwest in Seattle, Washington. Our study was performed in general accordance with our mutually agreed scope of work as outlined in our proposal dated May 17, 2022, which was subsequently authorized on that same date. Our service scope included reviewing existing geologic and geotechnical data in the vicinity of the site, performing a site reconnaissance, advancing four test borings at the site, and developing the conclusions and recommendations presented in this report.

2.0 SITE AND PROJECT DESCRIPTION

The project site is located at 5303 46th Avenue Southwest, in the West Seattle neighborhood of Seattle, Washington, as shown on the attached Figure 1, Vicinity Map. The site is bounded by single-family residences to the west and south, by 46th Avenue Southwest on the east, and by Southwest Englewood Street on the north. The subject site is rectangular in shape and has an area of approximately 6,300 square-feet. The site is currently occupied by a single-family residence with a daylight basement, that is located in the southeastern portion of the site. In addition, a detached garage is located in the western portion of the site.

The topography in the vicinity of the site generally slopes down at steep to moderate angles from east to west and south to north. At the site, west of the existing single-family residence, there are two 4- to 5-foot-tall parallel rockeries with a path in between. North of the residence is a 2 to 3-foot-tall rockery that runs parallel to the north property line.

Plates 1 through 3 on the following page depict current site conditions.

Environmental Critical Areas (ECA) - Based on our review of the City of Seattle Department of Construction and Inspections (SDCI) mapping, steep slope ECAs are mapped in the central portion of the property. Our evaluation of the Steep Slope ECAs at the site is discussed in Section 5.0 of this report.

Plate 1. Existing residence, looking west from 46th Avenue Southwest (June, 2022).

Plate 2. View of mapped steep slope ECA in the center of the property along the west side of the existing house, looking east (June, 2022).

Plate 3. View of existing garage where DADU is proposed, looking south (June, 2022).

We understand that the proposed project consists of removing the existing house, detached garage, and site hardscaping, and constructing a new two-story single-family residence with daylight basement. In addition, a new one-story DADU will be constructed over the footprint of the existing garage. Site retaining walls may also be incorporated into the design to provide level yard areas. Depending on the proximity of the daylight basement walls to the southern and eastern property lines, temporary shoring may be necessary to support the adjacent property during construction. We anticipate that excavations up to about 12 feet deep may be needed to construct the basement of the proposed home.

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

3.0 SUBSURFACE INFORMATION

3.1 TEST BORINGS

Four test borings (PG-1 through PG-4) were drilled at the site on June 17, 2022, to explore subsurface conditions. The approximate boring locations are indicated on Figure 2. Test borings were drilled to approximate depths between 16½ to 26½ feet below grade. The test borings were drilled using a portable drilling machine operated by CN Drilling of Seattle, Washington.

Soil samples were obtained from the borings at 2½- and 5-foot intervals in conjunction with Standard Penetration Test (SPT) sampling methods in general accordance with ASTM test method ASTM D-1586, *Standard Test Method for Penetration Test and Split Barrel Sampling of Soils*, 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 sampler 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.

Aa engineer from PanGEO was present during the field explorations to observe the drilling, assist in sampling, and describe and document the soil samples obtained from the borings. The summary boring logs are included in Appendix A as Figures A-2 through A-5. The soil samples were described using the system outlined on Figure A-1.

3.2 HISTORIC STREET GRADING

The historical street grading profile along 46th Avenue Southwest was reviewed to gain an understanding of previous grading activities adjacent to the subject site. Based on the street grading profile, it appears that fill, on the order of about 3 to 4 feet thick, was placed along the west side of 46th Avenue Southwest during original grading of the roadway. The information on the original street grading profile is consistent with our observations of a level yard facing 46th Avenue SW along the eastern property line, which appears to be filled. However, as described below, based on the results of our test borings, we anticipate that more than 4 feet of fill was placed on the subject property.

4.0 SUBSURFACE CONDITIONS

4.1 GEOLOGY

Based on our review of the Geologic Map of Seattle – a Progress Report (Troost et al., 2005), the subject property is underlain by Vashon advance outwash deposits typically consisting of moderately to well sorted, slightly oxidized sand and gravel that has been overridden by glacial ice, and are typically dense.

4.2 SOIL CONDITIONS

Based on the results of our test borings, the east side of the site is generally underlain by loose, sandy fill, over medium dense to very dense advance outwash, which is consistent with the mapped geology. On the west side of the site in boring PG-4, below a layer of loose sandy fill and loose to medium dense advance outwash, hard fine-grained deposits of silt were encountered at 14 feet below surface grades to the termination of the hole. A description of the soil units encountered in our test borings is presented below. Detailed descriptions of the soils encountered in our test borings can be seen in our boring logs included in Appendix A.

> **Fill/Modified Land:** About 6 to 17½ feet of very loose to medium dense, fine to medium sand and silty sand with varying amounts of gravel, was encountered in all four test borings. We interpreted this soil to likely be fill or modified land associated with previous developments and/or grading at the site. Scattered organics and burned wood debris was observed in this unit.
>
> **Advance Outwash:** Underlying the fill/modified land, the borings generally encountered a loose to dense, fine to medium sand with trace silt to the termination depth of each boring, except in PG-4, as described below. This unit increased in density with depth, and contained layers of gravel. We interpret this soil unit as advance outwash, which is consistent with the geologic mapping of the area.
>
> **Fine Grained Olympia Age Deposits:** Underlying the advanced outwash in PG-4 at 14 feet, was a layer of hard, laminated silt. This is consistent with fine grained Olympia age deposits mapped near the site.

4.3 GROUNDWATER CONDITIONS

Groundwater was not encountered within the termination depth of our test borings at the time of drilling. 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 STEEP SLOPE ECA CONSIDERATIONS

Steep Slope ECAs are defined as slopes with an incline of 40 percent or greater within a vertical elevation change of at least 10 feet. Steep slope ECAs are mapped in the central portion of the property. The approximate limits of the Steep Slope ECA as mapped by the City of Seattle are shown on the attached Figure 2. It is important to note that the actual steep slope areas will be defined by the City of Seattle based on the topographic survey of the site.

The City prohibits disturbance to steep slope areas and their 15-foot buffers unless a *Relief from Prohibition on Steep Slope Development* (Relief) is approved by the City. A *Relief from Steep Slope Development* is typically granted by the SDCI if one of the following criteria are met:

a) Development is located within the footprint of existing, lawfully constructed, structures or paved areas, not including landscaped areas or areas that have been graded;

b) Development is located on a steep slope erosion hazard area that has been created through previous legal grading activities, including but not limited to rockeries or retaining walls resulting from right-of-way improvements;

c) Development is located on a steep slope erosion hazard area that is less than 20 feet in vertical rise and that is 30 feet or more from other steep slope erosion hazard areas; or

d) Development is a necessary stabilization measure to mitigate an active landslide hazard on the applicant's lot or from an abutting lot, and such development meets the following requirements:

    i. The applicant demonstrates that the stabilization is the minimum necessary to mitigate the landslide hazard; and

    ii. The applicant uses the least intrusive option available to mitigate the landslide hazard.

Based on a review of the project topographic survey, the vertical relief of the steep slope in the center of the property, which is comprised of a rockery and retaining structures, is less than 20 feet, and is located more than 30 feet from another steep slope ECA. Additionally, the area has been developed with a rockery (see Plate 2, page 2). As such, in our opinion, the Steep slope ECA along the middle and norther side of the property would meet criteria (b) and (c), for a *Relief from Steep Slope Development*.

Approval of *Relief*: We understand that SDCI approved *Relief from Prohibition on Steep Slope Development* in a decision letter dated 8/22/2022, based on criteria (b) above, which is consistent with our opinion. As such, development will not be restricted at the site in the steep slope areas or their buffers. It may be noted, however, that the project will still be required to go through the ECA review process during permitting.

It is our opinion that the site can be redeveloped without adversely affecting the stability of the subject site, neighboring properties or Steep Slope ECAs, provided that the project utilizes proper engineering design and construction practices as recommended in this report.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 Seismic Design Considerations

Site Class: We anticipate that the project will be designed in accordance with the 2018/2021 edition of the International Building Code (IBC). We recommend a seismic site class D (Stiff Soil) be used for design of the structure.

Liquefaction Potential: Based on the lack of groundwater encountered in the explorations, it is our opinion that the potential for earthquake-induced soil liquefaction is negligible. As such, in our opinion, special design considerations associated with soil liquefaction are not necessary for this project.

6.2 Foundation Design

Proposed Residence: Based on the subsurface conditions encountered at our test boring locations, about 15 to 18 feet of very loose to loose sandy material is anticipated at the location of the proposed residence. In our opinion, a conventional shallow foundation system would experience undesirable settlements due to the relatively thick layer of loose soils present below the structure. It is our opinion that utilizing small-diameter, driven steel pipe piles (i.e. pin piles) that would transfer building loads to the underlying competent advance outwash soil below the loose soils would represent a cost-effective foundation support system for the residence.

Proposed DADU: Similarly, the subsurface conditions encountered in the boring location near the area of the site that will contain the proposed DADU consisted of about 11 feet of loose sand over medium dense to dense native soils. As such, if the DADU will be a two-story structure, it is our opinion that utilizing small-diameter driven steel pipe piles (i.e. pin piles) would represent a cost-effective foundation support option for the DADU, as conventional shallow foundations are not recommended due to the potential risk of excessive foundation settlement. If the DADU will be a one-story structure, similar to the existing garage, in our opinion a mat foundation, or structural slab with thickened edges, would represent a feasible foundation type to support the lightweight one-story structure with relatively minimal risk of significant future settlement.

Our recommendations for pin pile supported foundations or a mat foundation for the one-level DADU, are provided in the following sections.

6.2.1 Driven Small Diameter Steel Pipe Piles – SFR and 2-story DADU

The following driven pipe pile recommendations pertain to design and construction of the single-family residence and 2-story DADU.

Pin Pile Size: Small diameter driven pipe piles are utilized to transfer the structure loads through the loose soil to the underlying dense native soils. Pipe piles of 2- to 4- inches in diameter are typically utilized for this purpose. 3-inch and 4-inch diameter pin piles are typically installed using small to large hammers (600 to 2,000 lbs) mounted on a small to medium-sized excavator. 2-inch diameter pin piles are typically installed using portable, handheld equipment and are well suited for areas where limited site access exists, or in low headroom areas (i.e. inside a basement). For this project, 2-inch diameter pipe piles could be used, but 2-inch piles may not be as economical due to their lower capacity. In addition, 2-inch piles are limited to 30 feet in length, as required by SDCI code, and due to the thick layers of loose sands at the site, there is a risk that 2-inch piles could exceed 30 feet in length. We anticipate that 3- to 4-inch diameter pin piles would be well-suited for this project. If larger diameter piles are desired, such as 6- or 8-inch diameter pipes, PanGEO can provide additional recommendations upon request.

Pin Pile Capacity: The number of piles required depends on the magnitude of the design load. An allowable axial compression capacity of 3 tons (6 kips) per 2-inch diameter pile, 6 tons (12 kips) per 3-inch diameter pile, and 10 tons (20 kips) per 4-inch diameter piles may be used, with an approximate factor of safety of at least 2.0. Penetration resistance required to achieve the capacities will be determined based on the hammer used to install the pile. The tensile capacity of pin piles should be ignored in design calculations.

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

Pin Pile Specifications: We recommend that the following specifications applicable to the pile diameter selected for design be included on the foundation plan:

1. 2-inch diameter piles should consist of Schedule-80, ASTM A-53 Grade “A” pipe.

2. 3-inch and 4-inch diameter piles should consist of Schedule-40, ASTM A-53 Grade "A" pipe.

3. 2-inch piles shall be driven to refusal with a minimum 90-lb jackhammer. Refusal is defined as no more than 1 inch of penetration for 1 minute of continuous driving with the operator leaning heavily on the hammer. Alternatively, 2-inch piles may also be driven with a 140-lb hammer without the weight of the operator on the hammer.

4. 3-inch piles shall be driven to refusal with a minimum 600-lb hydraulic hammer. We recommend the following refusal criteria based on the size of hammer utilized:

Hammer Size	Blow per Minute	Refusal Criteria (3-inch pile)
600 lbs	1000	12 seconds per inch
850 lbs	900	10 seconds per inch
1100 lbs	900	6 seconds per inch

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

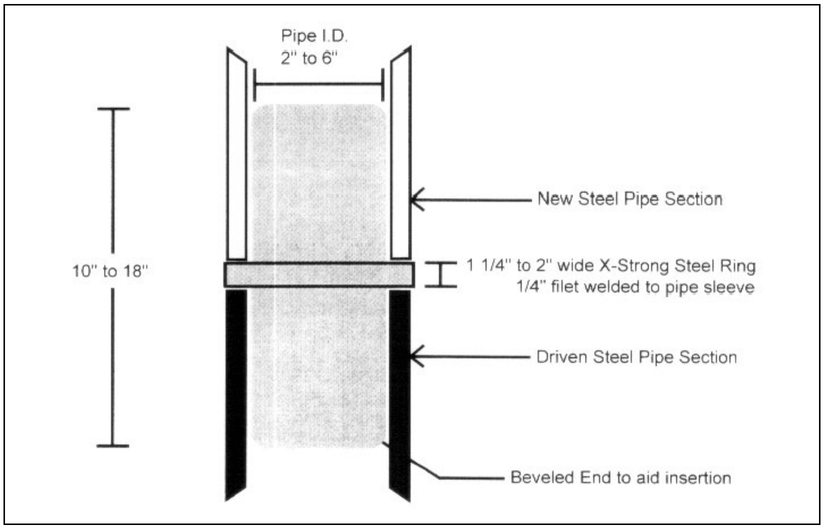
5. 4-inch piles shall be driven to refusal with a minimum 850-lb hydraulic hammer. We recommend the following refusal criteria based on the size of hammer utilized:

Hammer Size	Blow per Minute	Refusal Criteria (4-inch pile)
850 lbs	900	16 seconds per inch
1100 lbs	900	10 seconds per inch
2000 lbs	600	4 seconds per inch

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

6. Piles shall be driven in nominal sections and connected with compression fitted sleeve couplers (see detail below – Courtesy of McDowell Pile King, Kent, WA). We discourage

welding of pipe joints, particularly when galvanized pipe is used, as we have frequently observed welds broken during driving.



7. At least 3% (but no more than 5) of the 3-inch and 4-inch diameter pin piles should be load tested. All load tests shall be performed in accordance with the procedure outlined in ASTM D1143. The maximum test load shall be 2 times the design load. The objective of the testing program is to verify the adequacy of the driving criteria, and the efficiency of the hammer used for the project.
8. As required by the SDCI, the geotechnical engineer of record or his/her representative shall provide full time observation of pile installation and testing.

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

Lateral Resistance: Lateral capacity of vertical pin piles should be ignored in design calculations. Some resistance to lateral loads may be accomplished by battering the piles to a slope of 1(H):4(V), or steeper. Passive soil resistance values for embedded pile caps and grade beams may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf). This value includes a factor of safety of at least 1.5 assuming that properly compacted structural fill will be placed adjacent to the sides of the pile caps and grade beams. For the seismic condition, the recommended passive pressure may be increased by one-third. Friction at the base of pile-supported concrete grade beam should be ignored in the design calculations.

Estimated Pile Length: The required pile length in order to develop the recommended pile capacity is expected to vary, depending on the depth of loose soil and the actual driving conditions encountered. For planning and cost estimating purposes, we anticipate the piles to penetrate from 10 to 15 feet into the dense native sandy soils, and therefore lengths may be about 25 to 35 feet long as measured from existing grade. We recommend that a minimum pile length of 10 feet be specified on the project plans.

Obstructions: Obstructions may be encountered within the fill. Where possible, the obstructions should be removed to facilitate the pile driving. If obstructions cannot be removed, the structural engineer of record should be notified to revise the pile layout to accommodate moving the piles.

6.2.2 Mat Foundation – Proposed 1-Story DADU

If the DADU will be a one-story structure, based on the subsurface conditions anticipated at the proposed DADU location, it is our opinion that a mat foundation or structural slab with thickened edges is acceptable to support the proposed building. The foundation should be designed so that it is sufficiently stiff to spread the concentrated loads from the structure out over a wide area, thus reducing the bearing pressure on the underlying soils to a low level. The mat foundation will also mitigate the effects of potential differential settlement.

Subgrade Preparation: The mat foundation/structural slab should be founded on soil compacted to a firm condition. After the demolition of the existing garage, we recommend that the footprint of the DADU be compacted with a jumping-jack type compactor, or hoe-pac, to a firm and unyielding condition. If loose/soft soils are present that cannot be adequately recompacted, an over excavation of two feet and replacement of two feet of properly compacted granular structural fill is recommended.

Mat Slab Design: The foundation should be thickened to a minimum depth of 18 inches below the adjacent finish grade around the perimeter of the mat. The thickened edges of the structural slab should have a minimum width of 18 inches. For design of the mat foundation/structural slab with thickened edges bearing on the prepared subgrade as discussed above, a modulus of subgrade reaction of 50 pounds per cubic inch (pci) may be used. The mat foundation/structural slab foundations should be designed with a maximum allowable bearing pressure of 1,000 psf.

We anticipate that the average pressure on the soils below the mat slab/structural slab will be less than 300 psf. Provided the mat slab subgrade is prepared as described above, mat foundation/structural slab settlement is estimated to be approximately one inch with differential settlement on the order of ½-inch during the static loading condition.

Lateral loads acting on the foundations may be resisted by passive earth pressure developed against the embedded portion of the foundation system and by frictional resistance at the bottom of the foundation. For foundations bearing on the compacted subgrade, a frictional coefficient of 0.40 may be used to evaluate sliding resistance. Passive soil resistance may be calculated using an equivalent fluid unit weight of 300 pcf, assuming level ground conditions adjacent to the foundation, and properly compacted structural fill will be placed against the foundation. 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.

6.3 Below-Grade Walls

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

6.3.1 Wall Foundations

The recommendations outlined in the *Foundation Design* Section 6.2 of this report remain applicable for wall design and construction.

6.3.2 Lateral Earth Pressures

The below grade portions of the walls should be designed for an earth pressure based upon an equivalent fluid weight of 35 pcf for a wall that is allowed to yield (active condition), and 55 pcf for a wall that is restrained (at-rest condition). For the seismic condition, we recommend a uniform lateral earth pressure of at least 11H psf (where H is the height of the below grade portion of the wall) be added to the static pressure for sizing the walls for the ultimate condition. The recommended lateral pressures assume that adequate wall drainage will be incorporated into the design and construction of the walls to prevent the development of hydrostatic pressure.

6.3.3 Wall Drainage

Provisions for permanent control of subsurface water should be incorporated into the design and construction of below-grade walls. For walls constructed with conventional free-draining backfill, a footing drain consisting of a 4-inch diameter perforated pipe embedded in at least 12 inches of washed gravel wrapped with a geotextile fabric should be placed at the base of the wall footings. We recommend that prefabricated drainage mats, such as Mirafi 6000 or equivalent, be installed behind the walls to promote wall drainage.

6.3.4 Wall Backfill

Wall backfill should consist of free draining granular soils. It is our opinion that the fines content of the majority of the on-site soils it too high to be considered for use as wall backfill. Imported wall backfill should consist of granular soils such as City of Seattle Type 17 mineral aggregate or a PanGEO 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 (Modified Proctor). Within 5 feet of the wall, the backfill should be compacted to 90 percent of the maximum dry density.

6.4 CONCRETE SLAB

6.4.1 Slab-on-Grade

A conventional slab-on-grade may be used for the floor of the proposed residence or DADU, however, due to the existing loose soils anticipated at both locations, floor slabs may experience settlement and associated distress such as cracking or tilting. To reduce the potential of slab settlement and distress, we recommend over-excavating and re-compacting a minimum of 2 feet of the existing sandy soils below the floor slab with a jumping-jack type compactor or hoe-pac. In addition, the reinforcement of the floor slab could be increased to control cracking. If a high-performing slab is desired, a structural slab, as described below, should be utilized.

6.4.2 Structural Slab

If a high level of performance is desired with a low risk of settlement or distress/cracking, floor slabs should be designed as a structural slab that spans between the pile-supported foundation of the residence or DADU.

6.4.3 Capillary Break

We recommend that the floor slab be constructed on a minimum 4-inch thick capillary break. The capillary break should consist of free-draining, clean crushed rock or well-graded gravel compacted to a firm and unyielding condition. The capillary break material should have no more than 10 percent passing the No. 4 sieve and less than 5 percent by weight of the material passing the U.S. Standard No. 100 sieve. COS Type 22 clean (i.e. clean 5/8-inch crushed rock) typically meets our gradation recommendation for capillary break. We also recommend that a 10-mil polyethylene vapor barrier be placed below the slab.

6.5 Site Retaining Walls

We understand that retaining walls may be incorporated into the design of the project to create level yard areas. The heights of the walls are not known at this time. We understand that the walls will most likely be cast-in-place concrete walls. The site retaining walls may be designed using the same recommendations presented above in *Section 6.3 Below-Grade Walls*, and the foundations should be supported by small diameter driven pin piles, and described above in Section *6.2.2,* to avoid excessive wall settlements.

6.6 Temporary Excavations and Shoring

We anticipate that excavations will be needed for the construction of the proposed basement. The excavation is anticipated to encounter a variable amount of loose to medium dense fill soils. Where space is available, an unsupported slope cut will be the most cost-effective means of excavation support. If a 1.5H:1V projection from the bottom of the excavation daylights outside the property line, either temporary shoring is required by the City or a temporary easement is needed from the neighboring property owner. If temporary shoring is needed a soldier pile wall represents a feasible temporary shoring system. Due to the loose to very loose sandy soils anticipated at the site, we do not anticipate that a concrete block temporary shoring wall will be feasible, due to the potential of sloughing of the soils prior to block wall construction.

Temporary excavations greater than 4 feet deep should be properly sloped or shored, however, vertical excavations 4 feet deep or less will not be allowed within 4 feet of property lines unless a temporary easement from the neighboring property owner is obtained. 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.

6.6.1 Temporary Open Cuts

For planning purposes, we recommend that temporary excavations up to 12 feet deep for the basement, as well as for other site features, be sloped no steeper than 1.5H:1V (horizontal:vertical) due to the anticipated loose, sandy soils at the site. All cuts must be re-evaluated in the field by PanGEO during construction based on actual observed soil conditions and the presence of groundwater seepage. If groundwater seepage is encountered, however, the temporary slope will likely need to be cut to shallower angles to maintain stability. During wet weather, runoff water should be prevented from entering excavations. We also recommend that heavy construction equipment, building materials and excavated soil should not be allowed within a distance equal to 1/3 the slope height from the top of any excavation.

6.6.2 Temporary Shoring – Cantilevered Soldier Pile Wall

A temporary soldier pile wall will be needed if there is not enough space for temporary open cuts around the proposed basement excavation. If desired, soldier pile walls could also be used for permanent site retaining walls.

A cantilevered soldier pile wall consists of vertical steel beams, typically spaced from 6 to 8 feet apart along the proposed wall alignment, spanned by timber lagging to support the adjacent soil. Prior to the start of excavation, the steel beams are installed into holes drilled to a design depth and then backfilled with structural concrete and/or lean mix concrete per the shoring design. Because of the potential for loose soils, it may be necessary to use temporary casings to maintain the stability of the drilled hole. As the excavation proceeds downward and the steel piles are subsequently exposed, timber lagging is installed between the piles and any voids backfilled with free-draining material or controlled density fill (CDF).

The soldier pile wall system should be designed to provide adequate protection for the workers, adjacent structures, utilities, and other facilities. Excavations should be performed in accordance with the current requirements of WISHA. Construction should proceed as rapidly as feasible, to limit the time temporary excavations are open/exposed.

Design Lateral Pressures – For a cantilevered soldier pile wall the earth pressures depicted on Figure 3 should be used for design. The lateral earth pressures shown on Figure 3 should be increased for any surcharge loads resulting from traffic, construction equipment, building loads or backslopes if they are located within the height dimension of the wall. Above the bottom of the excavation, or base of wall, the recommended active earth and surcharge pressures should be applied over the full width of pile spacing. Below the bottom of the excavation or base of wall, the active and surcharge pressures should be applied over one pile diameter or width, and the passive resistance should be applied over two times the pile diameter or width.

If the soldier pile wall will be permanent, such as for site retaining walls, we recommended a uniform seismic pressure of 11H (psf) should be included in the pile design. For the seismic condition, the recommended passive pressure may be increased by one third.

Lagging - Lagging design recommendations for the anticipated conditions are presented on Figure 3. Lagging for temporary walls typically consists of timber boards. Lagging for permanent walls may also consist of treated timber boards, as well as precast or cast-in-place concrete, or steel sheets. For the permanent condition, if timber lagging is utilized, treated timber should be specified, and the saw cut ends of the lagging should be treated on-site prior to lagging installation. It should be noted that even treated timber lagging will eventually deteriorate, and would need to be replaced. The lifespan of treated timber lagging may range from 15 to 25 years. The advantage of concrete or steel lagging is that they would be permanent.

Performance – Retaining walls designed in accordance with the recommendations discussed above may be expected to deflect laterally about 1 inch or less.

Drainage – For permanent soldier pile walls with concrete facing, we recommend weep holes be provided at the base of the wall spaced 8 feet on center. For temporary walls with timber lagging, no additional drainage provisions are required, as the gaps in the timber boards will allow water to seep through.

Construction Considerations – Due to the loose fill soils, caving of the drilled holes could occur, and the contractor should be prepared to use temporary casing to maintain hole stability during soldier pile installations. If more than 6 inches of water accumulates at the bottom of the drilled hole prior to concrete placement, tremie methods of concrete placement will be required.

Driven Soldier Piles – Depending on the actual excavation depth, driven soldier piles may be considered. However, driven soldier piles typically have a smaller steel section than drilled soldier piles, and therefore have limited retention heights of less than about 8 feet. In addition, driven piles may have difficulty penetrating gravelly layers of soil, or dense soils, and vibration affects to adjacent properties must be considered.

Survey Monitoring – Ground movements will occur resulting from excavation activities. As a result, conditions of the adjacent structures and ground surface elevations should be documented prior to commencing earthwork to provide baseline data. As a minimum, we recommend that the existing adjacent building to the south be monitored during construction. This may include monitoring any existing cracks, and photo-documenting conditions. Optical survey points should also be established on the corners of the existing building to the south, as well as on the tops of every other soldier pile. Both vertical and horizontal deformations should be measured at least weekly during the excavation process. The monitoring frequency may be reduced based on the results of the monitoring. We recommend that the monitoring be performed by a licensed surveyor, and the results 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.

6.7 Permanent Drainage And Infiltration Considerations

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 directed away from structures and walls, adequately collected, and discharged to a suitable outlet. Under no circumstances should collected surface water or downspout drains be allowed to discharge behind retaining walls. Furthermore, roof downspouts should be tightlined to a suitable outlet, and not discharged into the wall or perimeter footing drain system.

Based on the SDCI mapping, infiltration testing is not required at the site. If infiltration of on-site stormwater is desired, a field infiltration test will need to be conducted to estimate an infiltration rate of the on-site soils to allow for proper sizing of the infiltration facility. In addition, the impact of infiltration on the site soils, and on the adjacent steep slope ECAs must be evaluated by PanGEO. Due to the thick deposits of loose sandy fill at the site, we anticipate that infiltration in the loose soils will result in settlement of the fill. As such, the location of the proposed infiltration facility, if desired, should be carefully considered, and coordinated with PanGEO.

6.8 Permanent Erosion Control Considerations

Permanent erosion control measures such as covering exposed ground surfaces with topsoil or mulch, and installing landscaping, should be performed as soon as possible after construction to limit the time the exposed surfaces are susceptible to erosion.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Material Reuse

The contractor should be aware that some of the site soils are moisture sensitive (fill soils consisting of silty sand), and will become disturbed and soft when exposed to inclement weather conditions. As such, we do not recommend the on-site silty sand soils be re-used as structural fill or wall backfill for this project. If deeper excavations expose the relatively clean outwash sands, we anticipate that these soils could be re-used as wall backfill.

7.2 Structural Fill and Compaction

We anticipate that structural fill will be needed to backfill the existing basement, and may also be needed to backfill footing over-excavations. Structural fill should consist of imported, free-draining granular material such as COS Seattle Type 17 material, crushed rock, or a PanGEO approved equivalent. The 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 (Modified Proctor).

7.3 Surface Drainage and Temporary 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 geotextile silt fences to collect runoff and prevent water from entering excavations or to prevent runoff from the construction area from 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. Potential problems associated with erosion around the development may be reduced by establishing vegetation within disturbed areas immediately following grading operations.

7.4 Wet Weather Earthwork Recommendations

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

- Because some of the on-site soils are moisture sensitive, all footing surface should be protected against inclement weather. It is the contractor's responsibility to protect the footing subgrade from disturbance. One option is to place a 2- to 3-inch thick layer of lean-mix concrete or a 4-inch thick (min.) layer of 2-inch crushed rock on the footing subgrade as soon as the subgrade is exposed.
- 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. Due to the sensitivity of the silty

soils to moisture, a rocked construction pad consisting of 2-inch crushed rock should be considered over the footprint of the proposed building.

- Bales of straw and/or 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 project, 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 Seattle DCI, as part of the permitting process, will also require geotechnical construction inspection services. Specifically, we anticipate that the following construction support services may be needed:

- Review final project plans and specifications;
- Verify implementation of erosion control measures;
- Observe installation of pin piles and pin pile load testing;
- Verify footing subgrades;
- Observe the stability of open cut slopes;
- Monitor temporary shoring installations;
- Verify adequacy of slab subgrades;
- Confirm the adequacy of the compaction of structural backfill;
- Observe installation of subsurface drainage provisions, and;
- Other consultation as may be required during construction.

Modifications to our recommendations presented in this report may be necessary, based on the actual conditions encountered during construction.

9.0 LIMITATIONS

We have prepared this report for use by Bryan Bentrott and the project team. Recommendations contained in this report are based on a site reconnaissance, the results of our subsurface exploration program, 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,

PanGEO, Inc.

JON C. REHKOPF
STATE OF WASHINGTON
40412
REGISTERED
PROFESSIONAL ENGINEER
6/14/23

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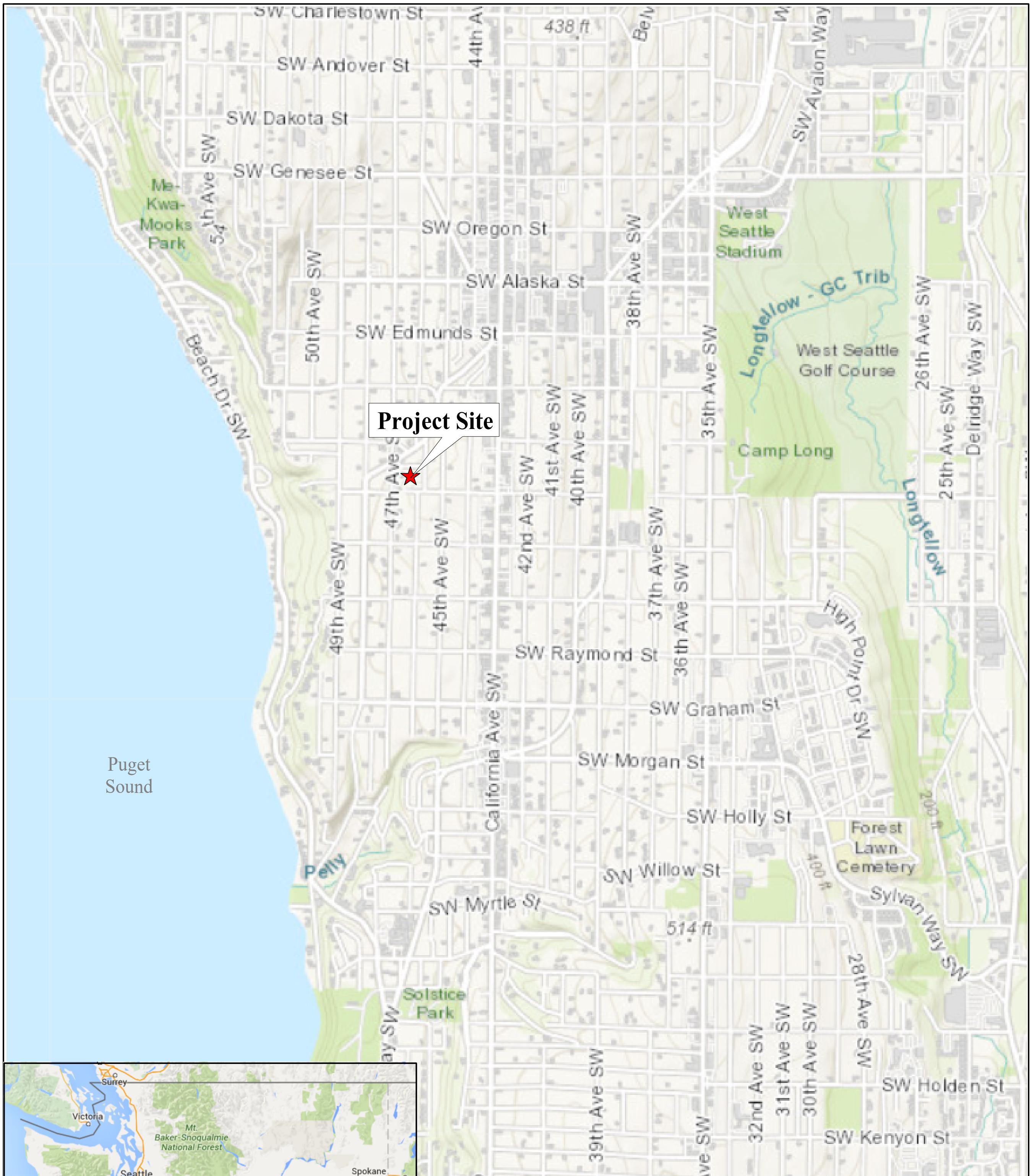
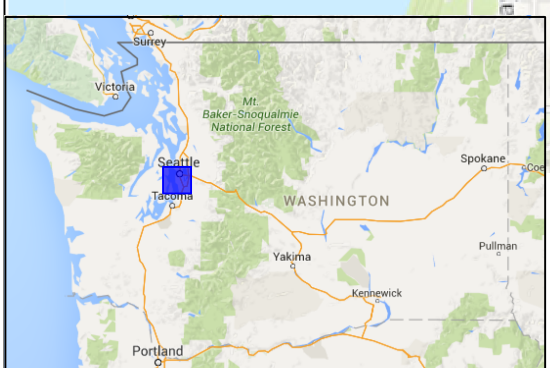
10.0 REFERENCES

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

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

Troost, K.G., Booth, D. B., Wisher, A. P., Shimmel, S. A., 2005, *The Geologic Map of Seattle-A Progress Report, Seattle, Washington – U. S. Geological Survey Open File Report 2005-1252, scale 1:24,000.*





Base Map: ESRI Topographic

NORTH

Not to Scale

22-238 Fig 1.grf 6/14/23 (10:10:39) LD

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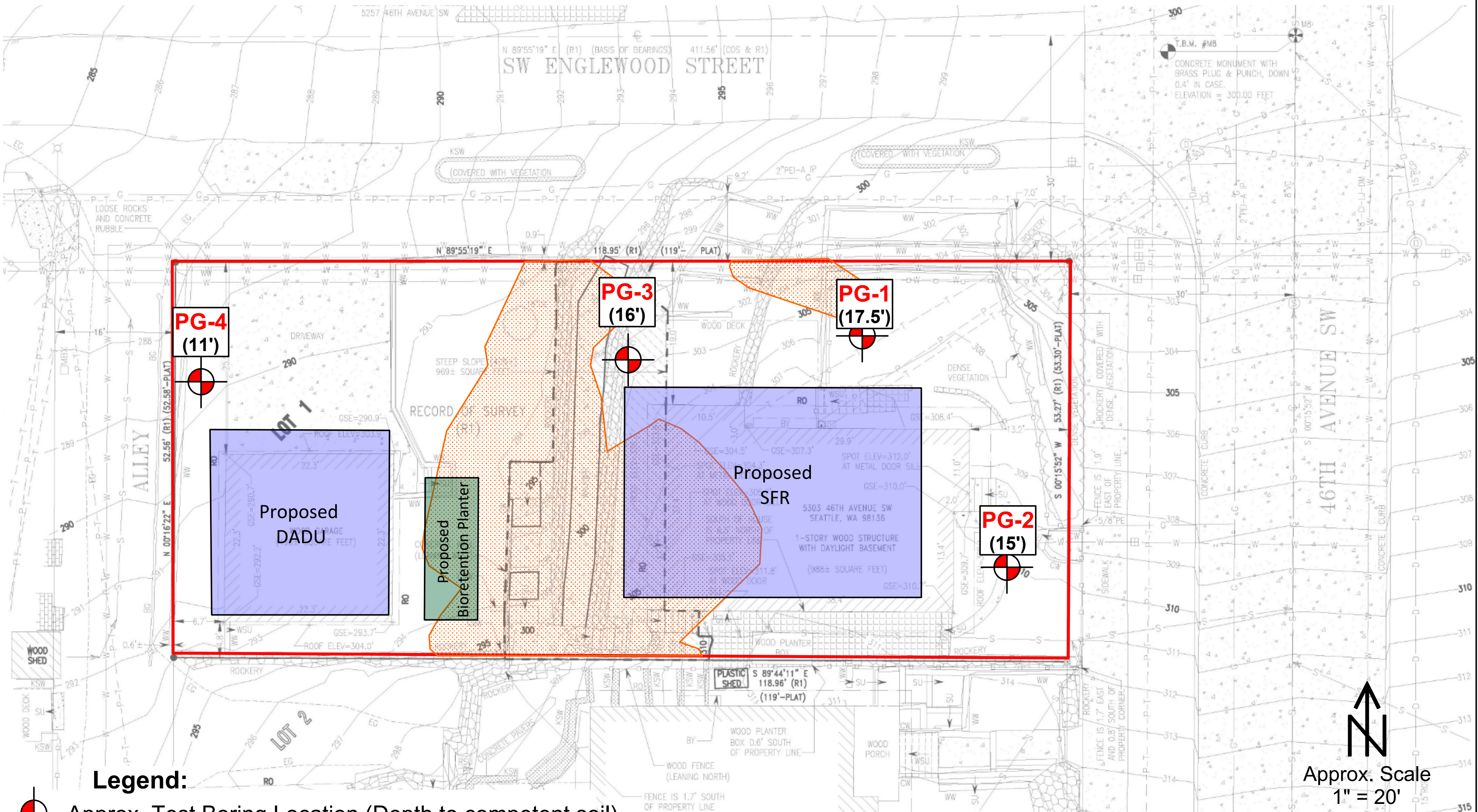
Proposed SFR and DADU
5303 46th Avenue SW
Seattle, Washington

VICINITY MAP

Project No.	22-238	Figure No.	1



Legend:

- Approx. Test Boring Location (Depth to competent soil)
- Proposed Structures
- Proposed Bioretention Planter
- Property Line
- Steep Slope ECA (per SDCI GIS map)

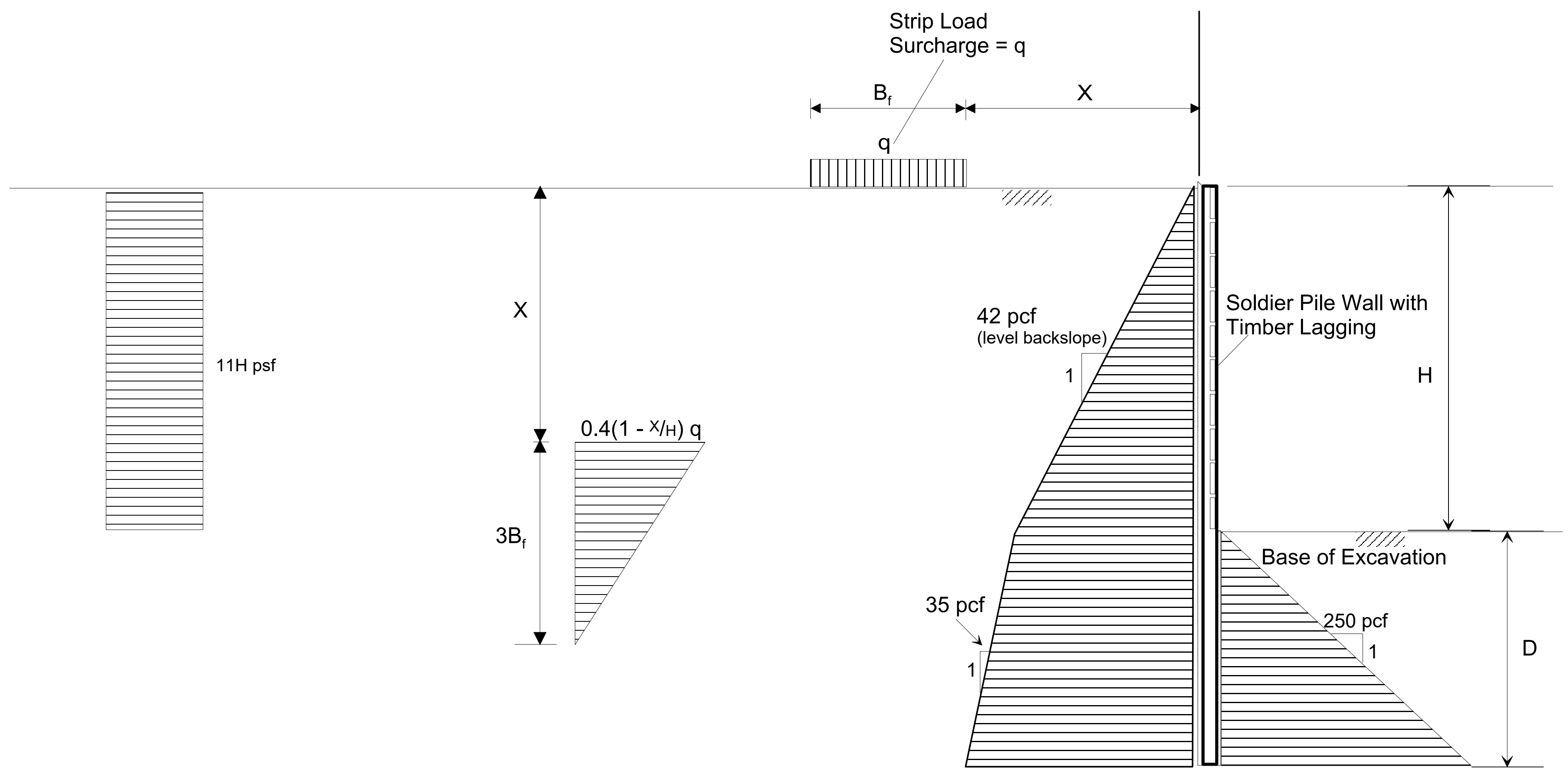
Note: Base map modified Boundary and Topgraphic Survey Drawing By Escarez and Assoicates Dated 6/10/2022

PanGEO INCORPORATED

Proposed SFR and DADU
5303 46th Avenue SW
Seattle, Washington

SITE AND EXPLORATION PLAN

Project No.	22-238	Figure No.	2



Seismic Surcharge
(Permanent Wall Only)

Strip Load Surcharge
(If Present)

Active Pressure

Passive Pressure

Notes:
1. Minumum embedment (D) should be at least 10 feet below finished grade at the base of the wall.
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 SFR and DADU
5303 46th Avenue SW
Seattle, Washington

DESIGN LATERAL PRESSURES
CANTILEVERED SOLDIER PILE WALL

Project No.	Figure No.
22-238	3

21-238 Fig 3 Soldier Pile EP - Cantl.grf 6/14/23 (10:26:52) LD

APPENDIX A

SUMMARY 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

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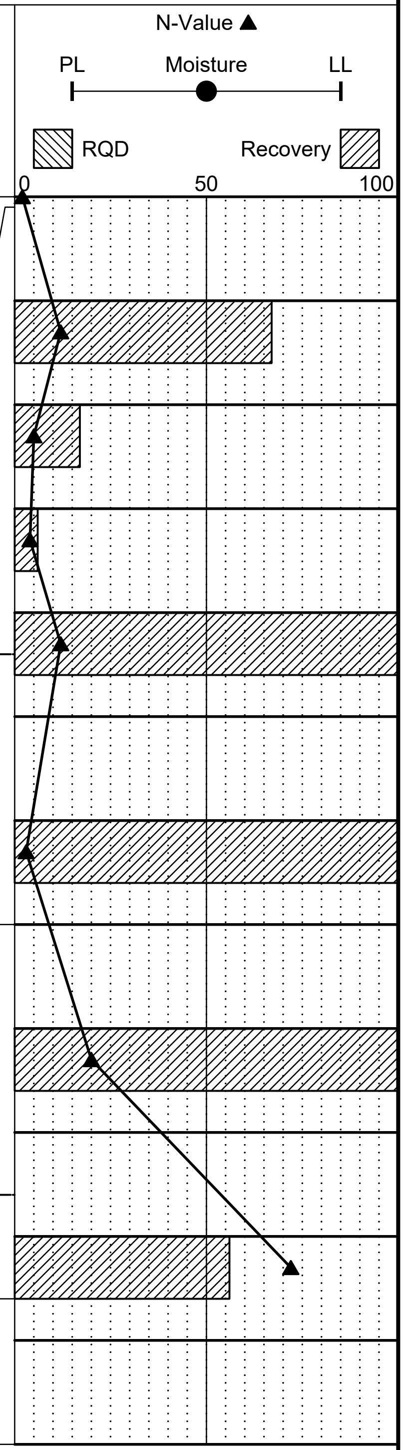
Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	Proposed SFR and DADU	Surface Elevation:	308.0ft
Job Number:	22-238	Top of Casing Elev.:	N/A
Location:	5303 46th Avenue SW, Seattle	Drilling Method:	Portable Acker, hollow stem auger
Coordinates:	Northing: 47.554726, Easting: -122.390897	Sampling Method:	SPT w/rope & cathead

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0	S-1		1 1 1			**TOPSOIL [Hf]** Very loose, brown, silty gravelly SAND, moist; trace organics, moist.
						MODIFIED LAND / FILL - [Hf] Very loose, brown, silty SAND, trace to some gravel, moist.
2.5	S-2		2 5 7			*--Becomes medium dense.* *--Difficult drilling, possible gravel layer.*
5.0	S-3		1 2 3			*--Becomes loose.*
7.5	S-4		2 1 3			
10.0	S-5		3 7 5			Medium dense, brown orange, SAND trace silt, moist, scattered organics.
12.5						*--Driller added water.*
15.0	S-6		1 1 2			Becomes very loose, gray, fine to medium SAND with trace silt, moist.
17.5						**ADVANCED OUTWASH - [Qva]** Medium dense, gray/brown, SAND trace silt, moist; minor bands of iron oxide staining.
20.0	S-7		6 8 12			
22.5						Very dense, brown sand with multicolored gravel, sandy GRAVEL, moist; gravel rounded.
25.0	S-8		14 27 45			Boring terminated at about 26.5 feet below grade. Groundwater was not observed during drilling.
27.5						
30.0						



Completion Depth:	26.5ft
Date Borehole Started:	6/17/22
Date Borehole Completed:	6/17/22
Logged By:	L Dunham
Drilling Company:	CN Drilling

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is provided for relative information only and is not a substitution for field survey. **Datum: WGS84/NAVD88**

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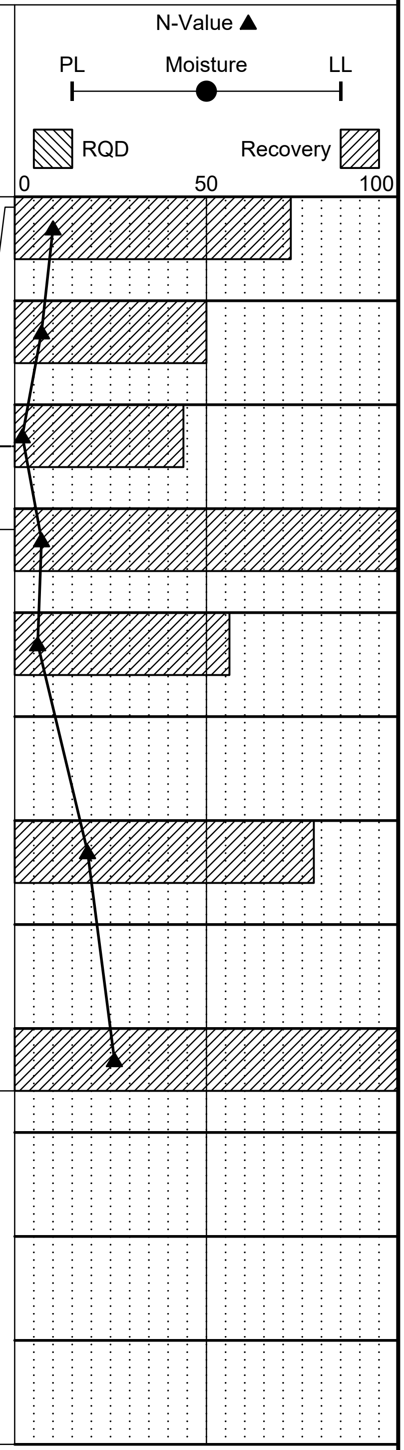
LOG OF TEST BORING PG-1

Figure A-2

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

Project:	Proposed SFR and DADU	Surface Elevation:	310.0ft
Job Number:	22-238	Top of Casing Elev.:	N/A
Location:	5303 46th Avenue SW, Seattle	Drilling Method:	Portable Acker, hollow stem auger
Coordinates:	Northing: 47.554643, Easting: -122.390832	Sampling Method:	SPT w/rope & cathead

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0	S-1		2 5 5			**TOPSOIL [Hf]** Loose to medium dense, brown, silty gravelly SAND, moist; prevalant organics, moist.
2.5	S-2		2 3 4			**MODIFIED LAND / FILL - [Hf]** Loose to medium dense, brown, silty gravelly SAND, moist; minor iron oxide staining, scattered charcoal bits.
5.0	S-3		1 1 1			Very loose, brown orange, SAND trace silt, moist.
7.5	S-4		2 3 4			**ADVANCED OUTWASH - [Qva]** Loose to medium dense, speckled gray/brown, fine to medium SAND trace silt, moist.
10.0	S-5		2 3 3			*--Sand becomes medium to coarse.* *--Minor iron oxide staining.*
12.5						
15.0	S-6		7 9 10			*--Sand becomes fine to medium, trace fine gravel.*
17.5						
20.0	S-7		6 11 15			Boring terminated at about 21.5 feet below grade. Groundwater was not observed during drilling.
22.5						
25.0						
27.5						
30.0						



Completion Depth:	21.5ft
Date Borehole Started:	6/17/22
Date Borehole Completed:	6/17/22
Logged By:	L Dunham
Drilling Company:	CN Drilling

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is provided for relative information only and is not a substitution for field survey. **Datum: WGS84/NAVD88**

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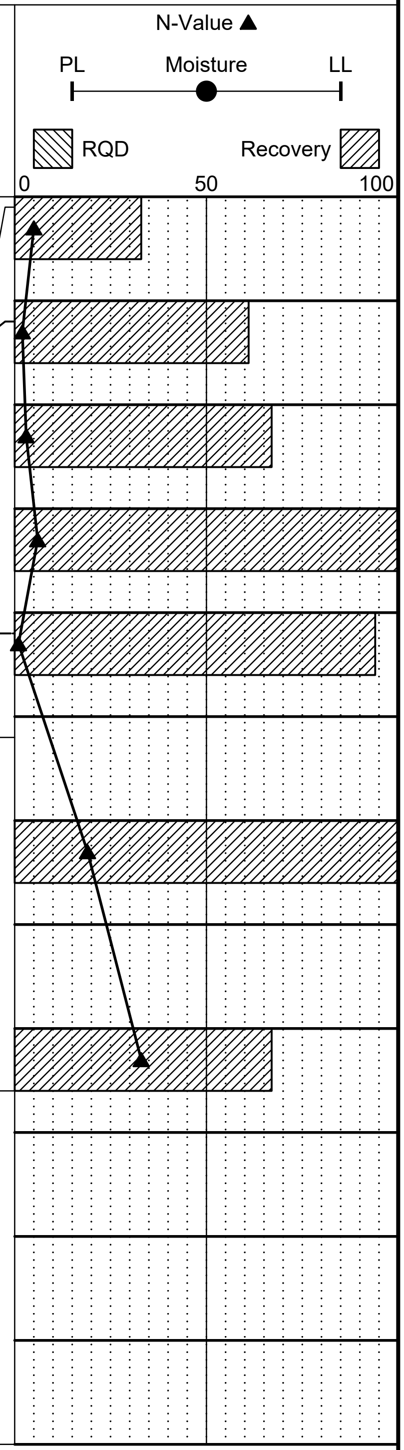
LOG OF TEST BORING PG-2

Figure A-3

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

Project:	Proposed SFR and DADU	Surface Elevation:	303.0ft
Job Number:	22-238	Top of Casing Elev.:	N/A
Location:	5303 46th Avenue SW, Seattle	Drilling Method:	Portable Acker, hollow stem auger
Coordinates:	Northing: 47.554723, Easting: -122.391031	Sampling Method:	SPT w/rope & cathead

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0	S-1		1 2 3			**TOPSOIL [Hf]** Loose, brown, silty gravelly SAND, moist; prevalent organics, moist.
						MODIFIED LAND / FILL - [Hf] Loose, dark brown, silty SAND trace gravel, moist.
2.5	S-2		1 0 2			Very loose to loose, gray brown, silty gravelly SAND, moist.
5.0	S-3		1 1 2			
7.5	S-4		2 2 4			
10.0	S-5		1 1 0			Very loose, speckled brown orange, SAND trace silt, moist.
12.5						**ADVANCED OUTWASH - [Qva]** Loose to medium dense, gray/brown, fine to medium SAND trace silt, moist.
15.0	S-6		2 7 12			
17.5						
20.0	S-7		7 14 19			*--Becomes dense.*
22.5						Boring terminated at about 21.5 feet below grade. Groundwater was not observed during drilling.
25.0						
27.5						
30.0						



Completion Depth:	21.5ft
Date Borehole Started:	6/17/22
Date Borehole Completed:	6/17/22
Logged By:	L Dunham
Drilling Company:	CN Drilling

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is provided for relative information only and is not a substitution for field survey. **Datum: WGS84/NAVD88**

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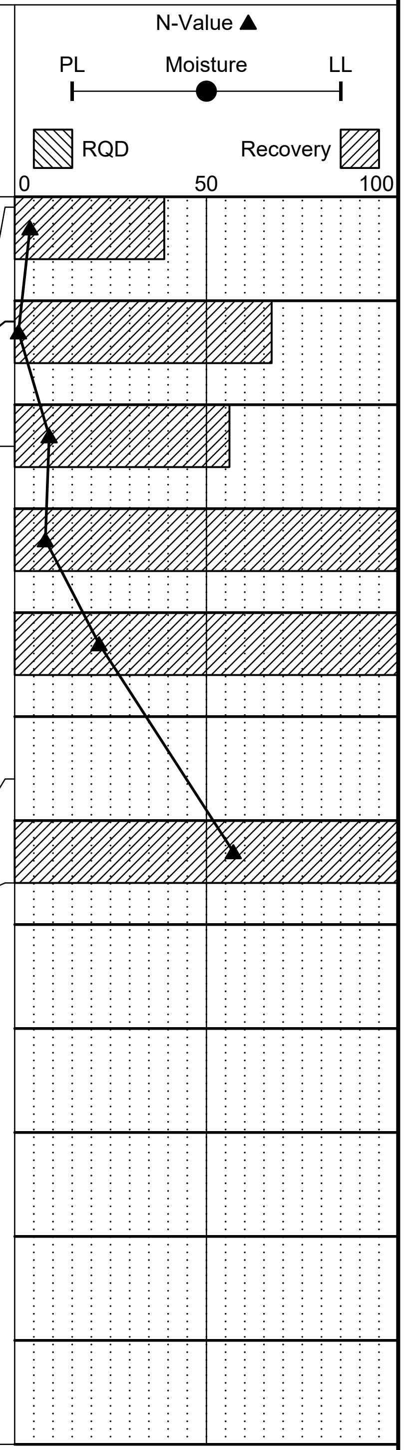
LOG OF TEST BORING PG-3

Figure A-4

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

Project:	Proposed SFR and DADU	Surface Elevation:	290.0ft
Job Number:	22-238	Top of Casing Elev.:	N/A
Location:	5303 46th Avenue SW, Seattle	Drilling Method:	Portable Acker, hollow stem auger
Coordinates:	Northing: 47.554699, Easting: -122.391267	Sampling Method:	SPT w/rope & cathead

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0	S-1		1 1 3			**TOPSOIL [Hf]** Loose, brown, silty gravelly SAND, moist; prevalent organics, moist.
						MODIFIED LAND / FILL - [Hf] Loose, brown, silty SAND trace gravel, moist.
2.5	S-2		1 0 1			Very loose, brown orange, SAND some silt, moist.
5.0	S-3		1 3 6			*--Trace gravel.*
						ADVANCED OUTWASH - [Qva] Loose to medium dense, speckled gray/brown, fine to medium SAND trace to some silt, moist.
7.5	S-4		4 4 4			*--Driller added water.* *--Bands of minor iron oxide staining.*
10.0	S-5		5 9 13			
12.5						*--Driller added water.* *--Driller indicated gravel from 13 to 14 feet.*
15.0	S-6		10 22 35			**FINE GRAINED OLYMPIA AGE DEPOSIT [Qpof]** Hard, gray, SILT some sand non-plastic, moist; laminated in 1/8" layers, easily break appart at layers.
						Boring terminated at about 16.5 feet below grade. Groundwater was not observed during drilling.
17.5						
20.0						
22.5						
25.0						
27.5						
30.0						



Completion Depth:	16.5ft
Date Borehole Started:	6/17/22
Date Borehole Completed:	6/17/22
Logged By:	L Dunham
Drilling Company:	CN Drilling

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is provided for relative information only and is not a substitution for field survey. **Datum: WGS84/NAVD88**

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LOG OF TEST BORING PG-4

Figure A-5

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