GEOTECHNICAL ENGINEERING REPORT (DRAFT)

PROPOSED DEVELOPMENT
2500 ELLIOTT AVENUE
SEATTLE, WASHINGTON

Project No. 15-166
October 7, 2015

Prepared for:

SummerHill Apartment Communities

PanGEO INCORPORATED

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October 7, 2015
File No. 15-166.200

SummerHill Apartment Communities
1191 Second Avenue, Suite 1570
Seattle, Washington 98101
Attention: Mr. Steve Orser

Re: Geotechnical Engineering Report - DRAFT
Proposed Development
2500 Elliott Avenue, Seattle, Washington

Dear Mr. Orser,

Please find attached our draft geotechnical engineering report to assist you and your project team with the design and construction of the proposed development. We will finalize this report once we receive review comments from the project team. In summary, based on the results of our study, we anticipate that the soil at the proposed foundation level consists of very dense, glacially consolidated silty sand with some gravel. In our opinion, conventional footings and slab-on-grade floors are suitable for the proposed project. The planned excavation will range from about 12 to 30 feet deep. It is our opinion that temporary shoring consisting of soil nail walls or soldier pile walls with timber lagging is feasible to support the proposed excavation. Based on the results of our test borings, relatively minor perched groundwater seepage may be encountered in the proposed excavation.

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

Sincerely,

Jon C. Rehkopf, P.E.
Senior Project Geotechnical Engineer

Encl.: Geotechnical Engineering Report - DRAFT

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2500 Elliott Avenue
Seattle, Washington

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering studies that were undertaken to support the design and construction of the proposed development at 2500 Elliott Avenue in Seattle, Washington. Our service scope included conducting a site reconnaissance, drilling three test borings, reviewing readily available geologic and geotechnical data in the vicinity of the site, and preparing this report summarizing our findings and presenting our geotechnical design recommendations.

2.0 PROJECT AND SITE DESCRIPTION

The subject site is located at 2500 Elliott Avenue, and is bounded on the southeast by Wall Street, on the southwest by Elliott Avenue, on the northwest by the Belltown Cottage Park, and on the northeast by a gravel-surfaced alleyway (see attached Figure 1). As depicted in Figure 2, the approximately 14,400 square-foot site is square in shape, and is currently developed with a two-story structure in the southwest corner of the site. The remainder of the site is currently developed as a gravel-surfaced parking area. An approximately 15-foot tall slope is present along the eastern margin of the site. The steep slope descends from the alleyway along the northeast property line down to the gravel parking area. The slope is also present along the east end of the southeast property line. In addition, an approximately 10-foot tall slope is located along the northwest margin of the site. The slopes are vegetated with dense blackberry bushes. Plate 1 to the right depicts current site conditions.

Plate 1. *Looking east at subject site. The site includes the gravel-surfaced parking lot on the left side of the photo, as well as the two-story masonry building.*

It is our understanding that the proposed development will include the demolition of the existing structure and the construction of a new residential building with eight above-grade levels, and one level of underground parking (below Elliott Avenue). We estimate the finished basement floor elevation of the new structure to be about elevation 25 feet (NAVD88). Due to the sloping topography across the site, the excavation for the underground parking garage is expected to vary from about 12 feet deep below Elliott Avenue and up to about 30 feet deep below the alleyway. The underground parking garage will most likely be constructed with zero set-backs from all but the eastern property line, which will be set-back about 2 feet from the property line.

2.1 Steep Slope ECA Considerations

According to the City of Seattle DPD mapping, the subject site contains a steep slope Environmental Critical Area (ECA) due to site grades of 40% or more along the east side of the property. However, because the site is within the downtown/high-rise zoning area, by definition the steep slope development restrictions do not apply.

The slope on the site may result in the ECA designation of a potential slide area. Based on our understanding of site conditions and the proposed development, provided that the project is designed and constructed in accordance with the recommendations in this report, in our opinion the proposed development will not negatively impact the stability of the site or surrounding properties. It may be noted that the project will likely be subjected to ECA reviews.

3.0 SUBSURFACE EXPLORATIONS

3.1 Current Subsurface Explorations

Three test borings (PG-1 through PG-3) were drilled at the project site on September 17, 2015. The approximate boring locations are indicated on the attached Figure 2. Boring PG-1 was drilled to a termination depth of about 40 feet below the existing ground surface, while PG-2 and PG-3 were drilled to about 26 feet deep, using an excavator-mounted drill rig owned and operated by Boretec, Inc. The drill rig was equipped with 6-inch outside diameter hollow stem augers. Soil samples were obtained from the borings at 2½- and 5-foot depth intervals. Standard penetration tests were performed in the borings 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 hammer falling a distance of 30 inches (cat-head mechanism). The number of blows required for each 6-inch increment of sampler penetration was recorded, and the blowcounts required for the last 12 inches of penetration is termed the SPT N-value. SPT N-value provides an empirical measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils. The approximate locations of test borings are indicated on the attached Figure 2, and the summary logs are included in the Appendix A for reference.

A geologist from our firm was present throughout the field exploration program to observe the drilling, assist in sampling, and to document the soil samples obtained from the borings. The completed borings were backfilled with drill cuttings and bentonite chips.

The soil samples retrieved from the borings were described using the system outlined on Figure A-1 of Appendix A and the summary boring logs are included as Figures A-2 through A-4.

3.2 Existing Subsurface Information

As part of our study, we reviewed the results of previous field exploration in the vicinity of the site. The information we reviewed includes three test borings that were previously advanced by PanGEO directly north of the subject site for the new building (Walton Lofts) constructed at the southwest corner of Western and Vine Street. In particular, Boring PG-3 (PanGEO, 2012), was completed near the NE corner of the subject site. The approximate location of the previous boring is indicated on the attached Figure 2, and the summary log is included in the Appendix B for reference.

3.3 City of Seattle Street Grading Profile

To gain a better understanding of previous grading that may have occurred at or adjacent the site, we reviewed City of Seattle street grading profiles adjacent to the project location along Elliott Ave and Wall Street. The profiles show original grades along the centerline and edge of right-of-way, as well as existing grade.

<u>Elliott Avenue:</u> The original street grading profile on Elliott Avenue indicated that original grades along the west property line of the subject site were generally 1 to 3 feet below existing grade, suggesting a few feet of fill was placed for the street construction.

<u>Wall Street:</u> The original street grading profile along Wall Street ended at Western Avenue, and therefore no information regarding original grade along Wall Street adjacent to our site could be obtained from the street grading profile.

<u>Summary:</u> In summary, the original street grading profile indicates that up to about 3 feet of fill may be present along the majority of the west property line. However, it should be noted that fill

may be present at the subject site from previous on-site developments and/or grading that would not be reflected in the street grading profiles.

4.0 SUBSURFACE CONDITIONS

4.1 SOIL

According to the geologic map of the Seattle area compiled by Troost and others (Troost, 2005), the project site is underlain by pre-Olympia deposits which typically consist of very dense and hard interbedded sand, gravel and silt. The exploratory borings advanced at the site generally encountered about five feet or less of loose to medium dense fill, over very dense, silty sand with some gravel (interpreted to be pre-Olympia deposits). In our opinion, the site subsurface conditions generally confirm the mapped geology.

A description of the generalized subsurface conditions encountered in the test borings is presented below. The summary boring logs are included in Appendix A.

Unit 1: Fill – Fill was encountered in PanGEO's borings PG-1, PG-2 and PG-3, as well as in the previous boring BH-3 (PanGEO, 2012) located to the north of the subject site. PG-1, which was advanced in the eastern corner of the site, along the alleyway at the top of the slope, encountered about 5 feet of fill, which generally consisted of loose, sandy gravel with some silt. Borings PG-2 and PG-3, which were located in the existing gravel parking lot area, encountered about 2½ feet of fill. The fill material encountered in PG-2 and PG-3 generally consisted of medium dense, sandy gravel and gravely sand with various amounts of silt. The previous boring, BH-3, which was advanced to north of the site across the alley, encountered about 8 feet of fill, which generally consisted of loose, silty sand with some gravel and brick debris.

Unit 2: Pre-Olympia Deposits – Below the fill, the three test borings advanced at the site (PG-1 through PG-3) and the existing test boring advanced to the north of the site (BH-3), encountered very dense, silty sand with gravel to at least 35 feet below surface grades. Layers of increased gravel or increased silt content were noted in this deposit. Boring PG-1 encountered a layer of fine to coarse gravel with sand and silt at a depth of 35 feet, which extended to the termination depth of the hole about 40 feet below surface grades.

4.2 Groundwater

Groundwater was encountered in borings PG-1, PG-2 and PG-3 at depths of 35 feet, 20 feet and 23 feet below surface grades, respectively, which corresponds to approximate groundwater elevations at each boring location of 17 feet, 16½ feet and 12 feet (NAVD88), respectively. The test borings previously drilled by others along Elliott Avenue adjacent to the site encountered groundwater at 18 and 19 feet below the ground surface, or at approximately elevation 16 to 17 feet (NAVD88). Zones of groundwater seepage or evidence of seepage were also encountered at various depths within clean sand and gravel layers, or at the bottom of the existing fill.

Based on the anticipated basement floor elevation of about 25 feet (NAVD88), we anticipate the groundwater seepage within the proposed excavation to be relatively minor. However, if elevator pits, jack shafts, or under-slab detention tanks will extend more than about 8 feet below the proposed basement floor slab, groundwater will likely be encountered. It should be noted that groundwater elevations may vary depending on the season, local subsurface conditions, and other factors. Groundwater levels are normally highest during the winter and early spring.

4.3 Laboratory Testing

Grain size distribution analyses and moisture content determinations were performed on representative samples obtained from the test borings. The grain size distribution tests were performed in general accordance with the procedure outlined in ASTM D422. The test results are included in the summary test boring logs, where appropriate, and in Appendix C of this report.

5.0 GEOTECHNICAL RECOMMENDATIONS

5.1 Seismic Design Parameters

The following provides seismic design parameters for the site that are in conformance with the 2012 and later 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:

Site Class	Spectral Acceleration at 0.2 sec. (g) S_S	Spectral Acceleration at 1.0 sec. (g) S_1	Site Coefficients		Design Spectral Response Parameters	
			F_a	F_v	S_{DS}	S_{D1}
C	1.36	0.53	1.0	1.3	0.91	0.46

5.2 Building Foundation

We understand the proposed basement floor slab will have a finished elevation of about 25 feet, or about 10 feet below the elevation of Elliott Avenue, and about 30 feet below the alley grade. Based on the results of our test borings and the planned basement depth, we anticipate that the soils at the proposed foundation level (about 2 feet below the finished slab) will likely consist of very dense glacial deposits consisting of silty sand with some gravel. In our opinion, conventional strip and spread footings are appropriate for the proposed project.

5.2.1 Allowable Bearing Pressure

We recommend that the footings be designed for a maximum allowable bearing pressure of 12,000 psf. This recommendation is applicable for footings founded on undisturbed, native glacial soil. If footings are supported on structural fill placed over very dense, undisturbed native soils, the structural fill should consist of lean-mix concrete (minimum 1½ sack). If compacted granular structural fill is placed below the footings, a reduced allowable bearing pressure of 4,000 psf should be used for design. If footings will be located within about 5 feet of the ground surface, limited over-excavation of the existing fill soils will likely be needed to reach the undisturbed native bearing soil. PanGEO can provide additional recommendations as requested.

For allowable stress design, the recommended allowable bearing pressure may be increased by 1/3 for transient conditions such as wind and seismic loadings.

All footings should have a minimum width of 24 inches. Exterior foundation elements should be placed at a minimum depth of 18 inches below final exterior grade. Interior spread foundations should be placed at a minimum depth of 12 inches below the top of slab.

Footing Subgrade Preparation – All footing subgrade should be carefully prepared. Any loose or softened soil should be removed from the footing excavation. The contractor should be aware that the site soils are moisture sensitive, and will become disturbed and soft when exposed to inclement weather conditions. As a result, depending on the groundwater and weather condition at the time of footing construction, it may be necessary to place 2 to 3 inches of lean-mix concrete on the exposed footing subgrade to protect against moisture.

Footing excavations should be observed by PanGEO to confirm that the exposed footing subgrade is consistent with the expected conditions and adequate to support the proposed building.

5.2.2 Foundation Performance

Total and differential settlements are anticipated to be within tolerable limits for footings designed and constructed as discussed above. Footing settlement under static loading conditions is estimated to be less than approximately ½ inch, and differential settlement between adjacent columns should be less than about ¼ inch. Most settlement will occur during construction as loads are applied.

5.2.3 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 350 pounds per cubic foot (pcf). This value includes a factor safety of at least 1.5 assuming that properly compacted structural fill will be placed adjacent to the sides of the footings. A friction coefficient of 0.4 may be used to determine the frictional resistance at the base of the footings. This coefficient includes a factor of safety of approximate 1.5.

5.3 BASEMENT WALLS

Presented below are our geotechnical recommendations for the design and construction of the proposed basement walls.

5.3.1 Lateral Earth Pressures

The basement walls may be designed for an earth pressure based upon an equivalent fluid weight of 35 pcf, assuming the basement walls will be built against shoring walls. For the basement wall that is built using conventional cut and fill approach, we recommend that an equivalent fluid weight of 45 pcf be applied for wall design. For the seismic condition, we recommend including an incremental uniform lateral earth pressure of 7H psf (where H is the height of the below grade portion of the wall) as an ultimate seismic load. The recommended lateral pressures assume that the backfill behind the wall consists of a free draining and properly compacted fill with adequate drainage provisions to prevent the development of hydrostatic pressure.

5.3.2 Wall Surcharge

Basement walls should be designed to accommodate traffic surcharge pressures if the traffic load is located within the height dimension of the wall. In addition, the base floor of the existing building to the northwest is higher than the proposed basement elevation of this project. As such, the footing surcharge from the adjacent building should be included in the design of the basement walls. The lateral pressure acting on the wall from surcharge loads may be determined by the surcharge diagram found on the attached Figures 3 and 4.

5.3.3 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 foundations and by friction acting on the base of the foundations. Passive resistance values may be determined using an equivalent fluid weight of 350 pounds per cubic foot (pcf). A friction coefficient of 0.4 may be used to determine the frictional resistance at the base of the footings. Both of these values include a safety factor of at least 1.5.

5.3.4 Wall Drainage/Damp Proofing

We recommend that provisions for permanent control of subsurface water be incorporated into the design and construction of the basement walls. Prefabricated drainage mats, such as Mirafi 6000 or equivalent, may be installed behind the basement walls and the collected water should be directed to a 4-inch diameter perforated collector pipe located along the inside perimeter of wall footing and discharged to an appropriate outlet.

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

5.3.5 Wall Backfill

Where wall backfill will be needed, free draining granular soils such as Seattle Mineral Aggregate Type 17 (City of Seattle Standard Specifications, 9-03.12(3)) are recommended. If encountered, on-site soils consisting of relatively clean sand may potentially be used as wall backfill, provided the backfill material is evaluated and approved by the project geotechnical engineer.

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.

5.4 CONCRETE SLAB ON GRADE

Conventional slab on grade construction may be used for the basement floor slab. The floor slab design may be accomplished using a modulus of subgrade reaction of 150 pci.

We recommend that the slab on grade be constructed on a minimum 4-inch thick capillary break placed on the undisturbed native soil or properly compacted structural fill over native soil. The capillary break should have no more than 10 percent passing the No. 40 sieve and less than 2 percent by weight of the material passing the U.S. Standard No. 200 sieve. If portions of the basement floor will house any equipment or facilities that are sensitive to moisture, we recommend that a minimum 10-mil polyethylene vapor barrier be placed below the subject portions of the slab.

5.5 TEMPORARY EXCAVATION AND SHORING

A temporary shoring system will be needed to support the proposed excavation. We understand the proposed building will be constructed with minimal setbacks from all property lines. In our opinion, either a soil nail wall or a soldier pile wall is feasible for the project. Based on the results of our test borings, up to about 5 feet of loose to medium dense, silty sand fill soil is anticipated along the property boundaries. For a soil nail wall, vertical elements may be needed to help maintain face stability within the upper loose to medium dense soils, and especially along the northwest property line adjacent to the existing neighboring structure.

If soldier pile walls are incorporated into the shoring design, a cantilevered wall may be used where excavation depths are less than about 12 to 15 feet deep. Single or multiple levels of tiebacks may be needed for deeper excavations.

Design recommendations for unsupported cuts, soil nail walls and soldier pile walls are provided in the following sections.

5.5.1 Unsupported Cuts

Where space is available, an unsupported slope cut may be incorporated into the shoring design. All temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes.

For planning purposes, temporary excavations could be sloped to as steep as 1H (Horizontal):1V (Vertical) within fill soils (i.e., the uppermost 5 feet), and as steep as 0.5(H):1(V) in the underlying very dense silty sand. If areas of seepage are encountered during construction, the slopes may need to be flattened. The stability of temporary excavation slopes should be evaluated in the field during construction based on actual observed soil conditions.

5.5.2 Surcharge Loads

Surcharge loads including but not limited to street traffic and construction equipment should be considered in the shoring design. Surcharge from typical street traffic may be considered equivalent to 2 feet of soils and could be designed for a uniform horizontal pressure of 100 psf.

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.

Existing Footing at Northwest Neighboring Property – It appears that the northwest adjacent building has a basement floor elevation higher than the proposed basement for this project. As such, the temporary shoring wall should be designed to support the adjacent footing surcharge. The surcharge pressures can be estimated using the diagrams included in Figures 3 and 4.

5.5.3 Temporary Construction Easements & Utilities

Soil nails or tiebacks will extend beyond the property boundaries. Temporary construction easements will be needed from the neighboring property owners, including the City of Seattle Department of Transportation. Depending on the lengths of the proposed soil nails and tiebacks, temporary construction easements may also be needed from the private property owner to the northeast, across the alleyway. The foundation depth of the structure across the alley should be verified, to insure it is not in conflict with the proposed soil nails or tiebacks.

We recommend that the easements be obtained as early in the design process as possible. Although soldier piles may be internally supported by braces or rakers, such construction methods will be significant more costly than tiebacks, and will impact the rate of construction.

Tiebacks and soil nails will also need to be designed to provide adequate clearance from utilities.

5.5.4 Performance and Monitoring

Ground movements will occur as a result of excavation activities. Shoring walls designed in accordance with the recommendations discussed above may be expected to deflect laterally about 1 inch or less. To confirm the performance of the excavation shoring, a monitoring program meeting the minimum requirements from DPD and SDOT should be performed. As a minimum, optical survey points should be established at:

- The top of soil nail wall, maximum 20 feet on center. These monitoring points should be monitored twice a week as required by the SDOT;
- The top of every other soldier pile. These monitoring points should be monitored twice a week as required by the SDOT; and
- The curbs and centerlines along the adjacent streets, and on adjacent structures. These monitoring points should be spaced no more than 20 feet apart, and baseline data should be collected before the start of excavations. These monitoring points do not need to be surveyed again unless the wall deflections exceed about ½ inch.

The monitoring program should include changes in both the horizontal (x and y directions) and vertical deformations. The monitoring should be performed by the contractor or the project surveyor to the nearest 0.01-foot, and the results be promptly submitted to PanGEO for review.

The results of the monitoring will allow the design team to confirm design parameters, and for the contractor to make adjustments if necessary.

We also recommend that the existing conditions along the public right-of-way and the adjacent private properties be photo-documented prior to commencing on any earthworks at the site.

5.5.5 Soil Nail Wall Design Considerations

In our opinion a soil nail wall is feasible to support the proposed excavation. For design purposes, we recommend the following soil parameters:

Soil Unit 1 – Loose to medium dense silty sand (upper 5 feet)

Friction Angle:	32 degrees
Apparent Cohesion:	50 psf
Soil Unit Weight:	125 pcf
Ultimate Load Transfer:	2.0 kips per foot along nail/soil interface

Soil Unit 2 – Very dense silty sand with gravel

Friction Angle:	38 degrees
Apparent Cohesion:	100 psf
Soil Unit Weight:	130 pcf
Ultimate Load Transfer:	6.0 kips per foot along nail/soil interface

The design load transfer value will be affected by many factors including soil conditions, drilling methods and grouting pressures. Therefore, the recommended design value must be verified in the field by conducting performance tests on at least two sacrificial nails in each soil type, and load testing up to 5 percent of production nails.

5.5.6 Soldier Pile Wall Design Considerations

We understand the proposed excavation will be about 12 to 30 feet deep. As such, a cantilevered or tie-back soldier pile walls or a combination of both is feasible to support the excavation. We recommend that the lateral earth pressures depicted on Figures 3 and 4 be used for design of soldier pile walls.

Vertical Capacity

If appropriate, the soldier piles may also be incorporated into the vertical support system for the building. 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 30 ksf.

Tiebacks

Excessive pile top deflection could occur before the tiebacks are installed. To improve the performance of the tieback wall, it may be necessary to limit the tiebacks to no more than 8 to 10 feet below the pile top unless steel beams of sufficient size will be used to limit the magnitude of the cantilever deflection.

The tieback bond lengths must be located behind a no-load zone (see Figures 3 and 4), and should have a minimum bond length of 15 feet beyond the no-load zone.

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

For temporary easement planning purposes, the anchors may be sized assuming an allowable skin friction value of 3 kips per lineal foot of anchor bond length within the very dense native soils (Unit 2), assuming that small diameter (about 6 inches) non-pressure grouted tiebacks will be used. However, if the contractor utilizes pressure grouted tiebacks, and one or multiple post-grouting, allowable skin friction values in the range of 8 to 10 kips per lineal foot of anchor bond length is reasonable, which would result in shorter tiebacks. We recommend that the shoring contractor review this report and collaborate with the shoring designer, owner, and PanGEO to determine the most cost effective tieback design, based on the planned method of tieback installation and grouting. We recommend that the allowable tieback loads be limited to about 120 kips per anchor.

The actual capacity of the anchors should be checked with 200 percent verification tests. At least two 200% tests should be performed. All production anchors should be proof tested to 150% of the design load. The anchor installations should be conducted in accordance with the latest edition of the Post Tensioning Institute (PTI) "*Recommendations for Prestressed Rock and Soil Anchors*." Elements of the testing are as follows:

Verification Tests (200% Tests)

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

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

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

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

In the tieback construction, a bond breaker shall be constructed in the no load zone when the installation procedures use single stage grouting.

Lagging

Lagging design recommendations are presented on Figures 3 and 4.

When placing timber lagging between soldier piles, the height of each lift may need to be limited to prevent the un-shored soil from sloughing below the timber boards. We recommend that the soil exposed for timber lagging be less than 4 to 6 feet, depending on the actual soil conditions encountered. The actual allowable vertical cut for timber lagging placement should be determined in the field, based on the actual conditions observed.

Surcharge loads including but not limited to street traffic and construction equipment should be considered in the lagging design. 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.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 DEMOLITION

Based on the results of our test borings, the site is underlain by up to 5 feet of fill. As such, there is the potential that unknown structures or debris, such as bricks, concrete or wood fragments, may be present within the fill.

6.2 SHORING WALL

6.2.1 Soil Nails and Tiebacks

The drilling for tiebacks or soil nails is anticipated to encounter loose to medium dense, silty sand over very dense silty sand with some gravel. The shoring contractor should be prepared to utilize temporary casings to maintain stability of the drilled holes in the loose to medium dense soils. In addition, layers of clean sand or gravel with water could be present within the very dense native soils, which could potentially require temporary casing to maintain hole stability.

In addition, all tiebacks and soil nails should be designed to provide adequate clearance from existing underground utilities.

6.2.2 Soldier Pile Wall

Soldier piles (or vertical elements) will be installed through loose fill (Unit 1) and into very dense silty sand with gravel (Unit 2). Layers of relatively clean gravel are also present within Unit 2. The contractor should be aware that temporary casing may be necessary to prevent caving of the soldier pile holes.

Although obstructions were not encountered in the test borings, they may exist within the fill (Unit 1). A backhoe should be used to remove any obstructions at shallow depths that preclude installation of the soldier pile. In addition, layers of gravel, such as what was encountered in boring PG-1, could contain cobbles or boulders, which can increase the duration of the soldier pile installation.

Perched groundwater will likely be present at the site, and water may accumulate at the bottom of drilled holes. Tremie methods shall be used for concrete placement in all holes having 6 or more inches of accumulated water. All drilled holes shall be filled with concrete on the same day.

6.2.3 Lagging

When placing timber lagging, the height of each lift may need to be limited to prevent excessive sloughing. We recommend that the soil exposed for timber lagging be less than 4 to 6 feet, depending on the actual soil conditions encountered. The actual allowable vertical cut for timber lagging placement should be determined in the field, based on the actual conditions observed. We recommend that voids behind lagging be backfilled with CDF.

6.3 TEMPORARY GROUNDWATER AND SURFACE WATER CONTROL

We do not anticipate a significant amount of groundwater to be present during excavation. However, some perched groundwater within sand and gravel seams of the native soils will likely be encountered. The contractor should be prepared to provide temporary groundwater control methods during excavation. If groundwater is encountered, a dewatering system consisting of trenches, sumps and pumps will likely be adequate to control perched groundwater or run-off from heavy precipitation in the excavation.

6.4 Structural Fill and Compaction

Fill placed next to proposed foundation and basement walls and underneath the slabs should consist of structural fill. For planning and budgeting purposes, we recommend granular import fill such as City of Seattle Type 17 be used as structural fill at the site. If encountered, on-site soils consisting of sand and gravel relatively free of silt and debris may potentially be used as structural fill at the site, provided the backfill material is evaluated and approved by the project geotechnical engineer.

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.

6.5 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 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 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 foundations should also be controlled. All collected runoff should be directed into conduits that carry the water away from the pavement or structure and into 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.

6.6 Wet Earthwork Recommendations

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

- 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 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.
- 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.

7.0 ADDITIONAL SERVICES

We anticipate the City of Seattle will require a plan review and geotechnical special inspections to confirm that our recommendations are properly incorporated into the design and construction of the proposed development. 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 excavation shoring system;
- Observe the stability of any open cut slopes;
- Review optical survey data provided by others to evaluate the performance of the shoring system;

- Verify adequacy of footing and 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.

8.0 LIMITATIONS

We have prepared this report for use by Summerhill Apartment Communities and the project 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,

PanGEO, Inc.

DRAFT

DRAFT

Jon C. Rehkopf, P.E.
Senior Project Geotechnical Engineer

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

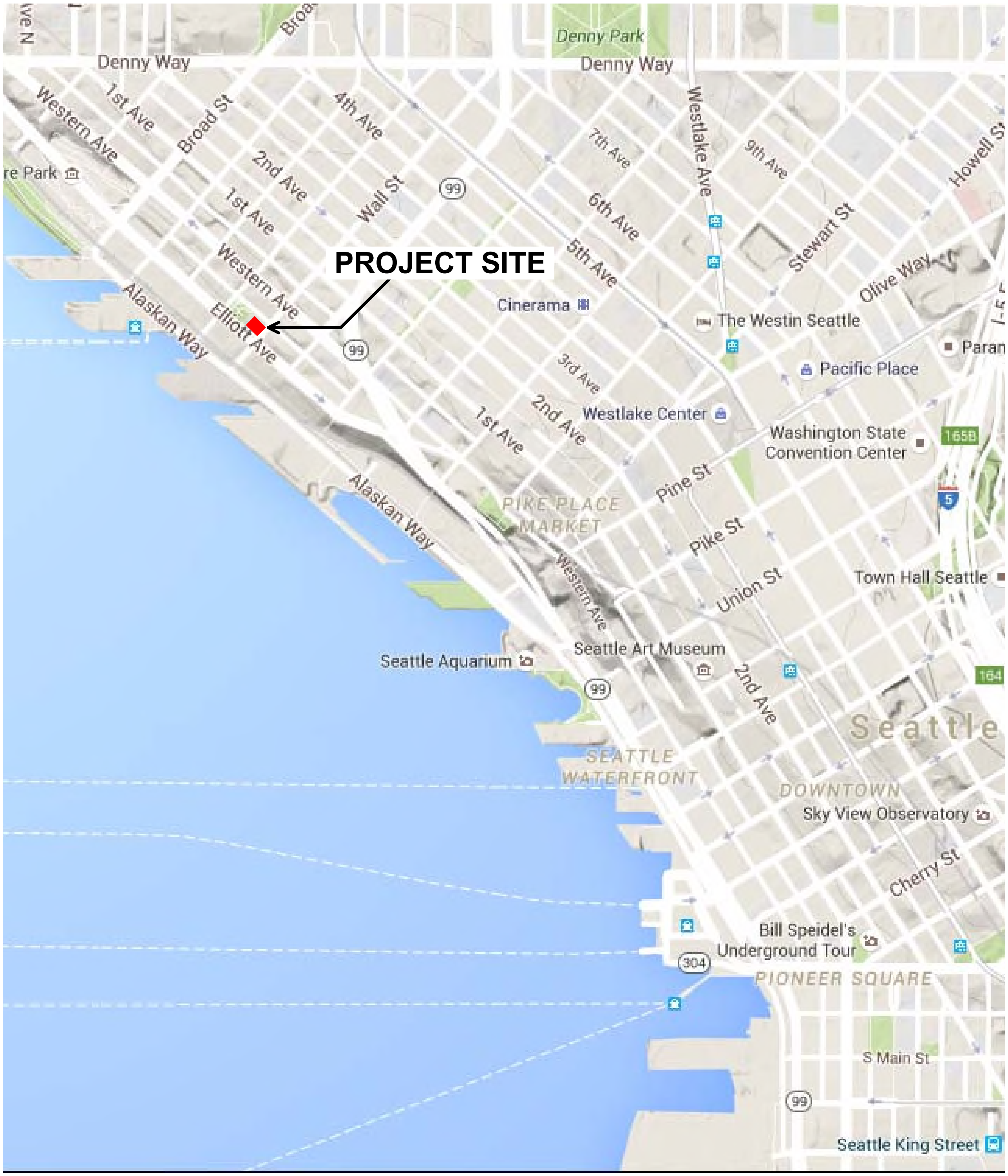
9.0 REFERENCES

International Building Code (IBC), 2012, International Code Council.

PanGEO, 2012. Geotechnical Boring PG-3 – Geotechnical Engineering Report, Western & Vine, Seattle WA. PanGEO Job No. 12-162.

Troost, Kathy Goetz, Booth, Derek B., Wisher, Aaron P., and Shimel, Scott A. *The Geologic Map of Seattle – a Progress Report*, USGS, Open-File Report 2005-1252, 2005.

Washington Administrative Code (WAC), 2013, Chapter 296-155 - Safety Standards for Construction Work, Part N - Excavation, Trenching, and Shoring, Olympia, Washington.



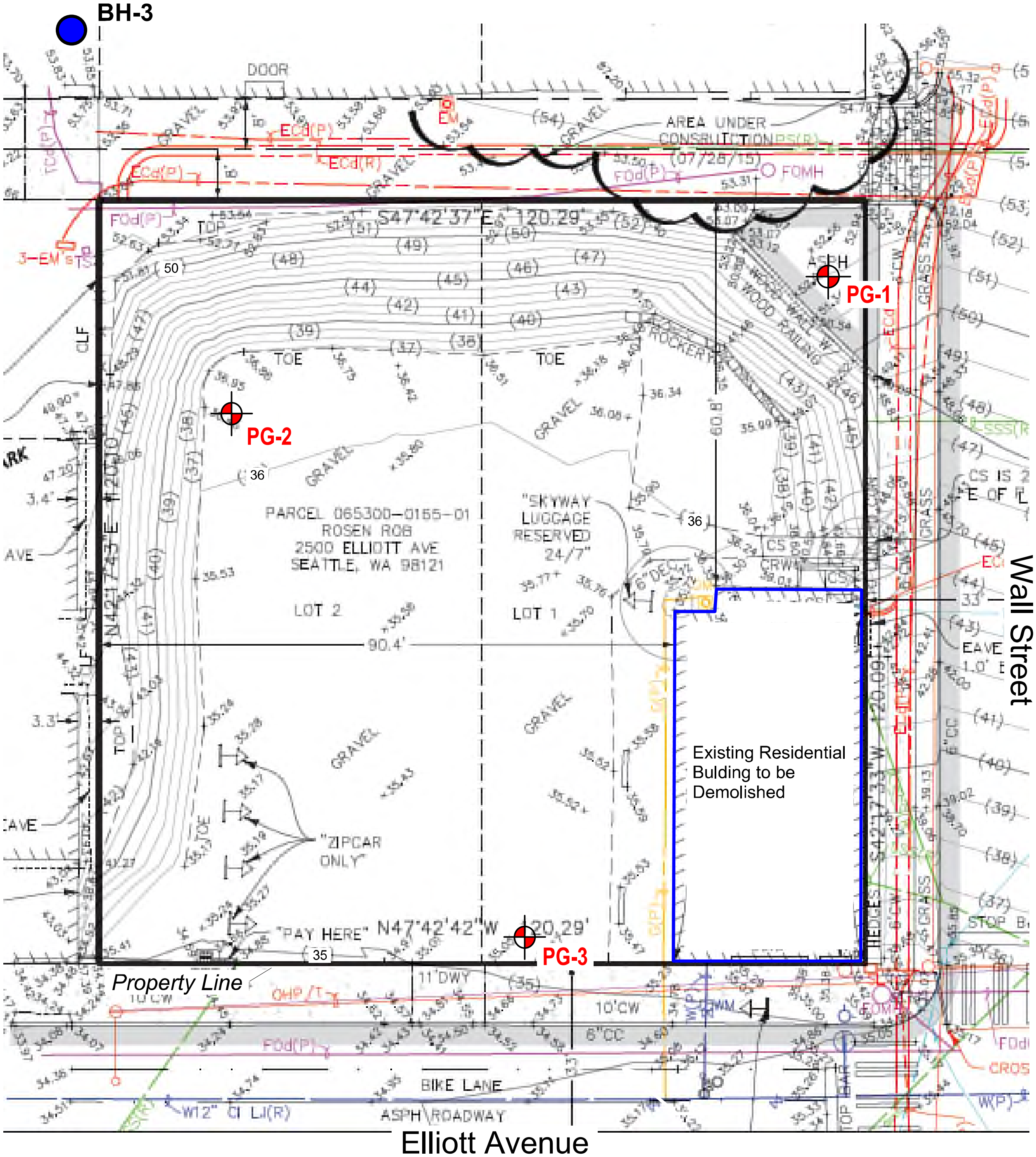
N

Not to Scale

Base Map: Google Maps

PanGEO INCORPORATED	**Proposed Development 2500 Elliott Avenue Seattle, WA**	**VICINITY MAP**
		Project No. **15-166** / Figure No. **1**

15-166.grf w/ file.dat 10/6/15 (12:18) NER



Legend:

PG-1 Approximate Boring Location

BH-3 Approximate Boring Location (PanGEO, 2012)

Approx. Scale: 1" = 25'

Reference: Base map modified from Topographic Survey Map by Bush, Roed & Hitchings, Inc.

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Proposed Development
2500 Elliott Avenue
Seattle, WA

SITE AND EXPLORATION MAP

Project No.	15-166	Figure No.	2

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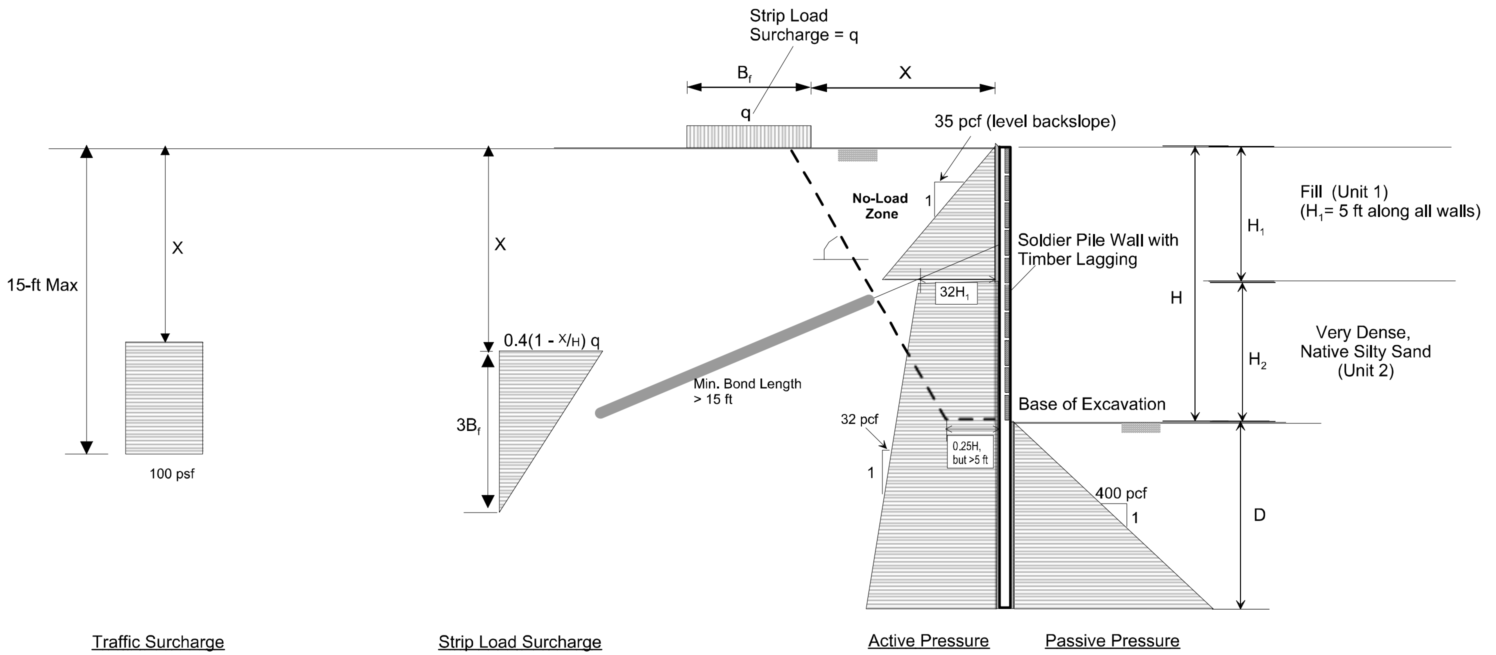
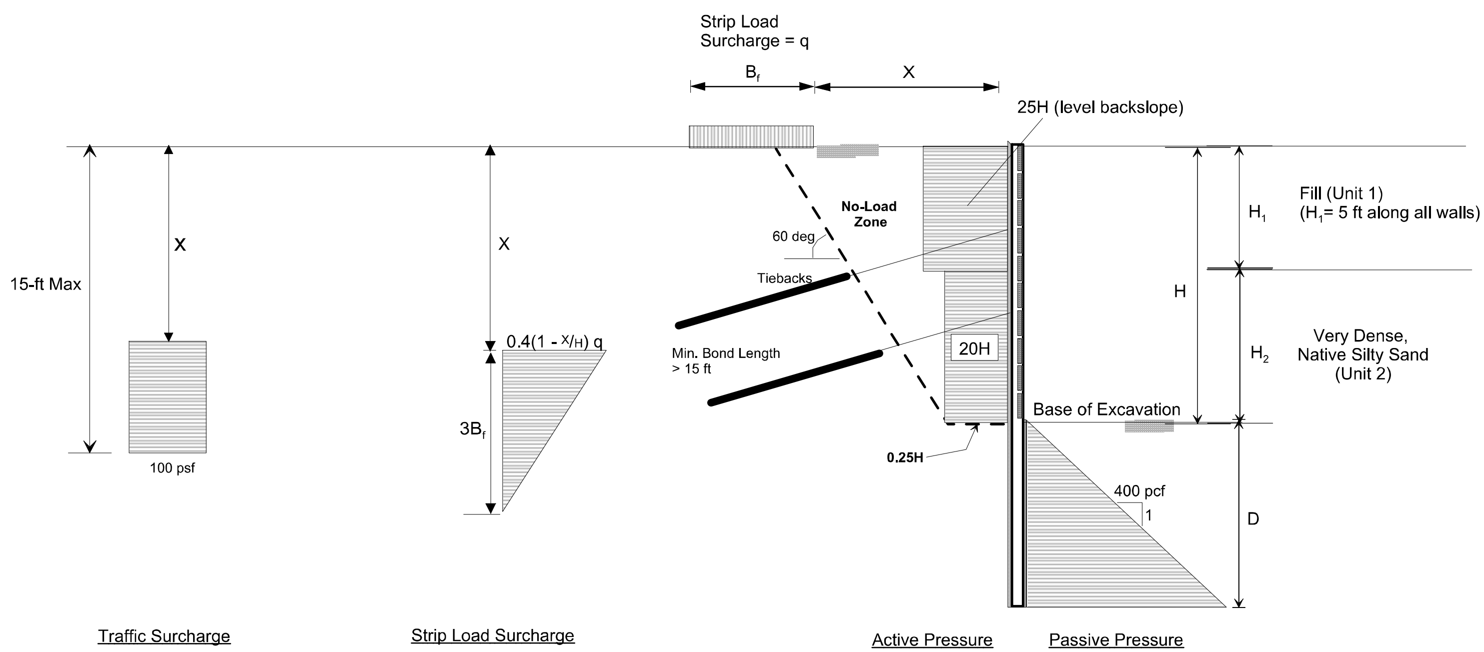


Notes:
1. Minumum embedment should be at least 10 feet below bottom of excavation.
2. A factor of safety of 1.5 has been applied to the recommended passive pressure values. No factor of safety has been applied to the recommended active earth pressure values.
3. Active pressures should be applied over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

PanGEO INCORPORATED	Proposed Development 2500 Elliott Avenue Seattle, WA	DESIGN LATERAL PRESSURES SOLDIER PILE WALL CANTILEVERED/ONE LEVEL OF TIEBACK
		Project No. 15-166
		Figure No. 3

15-166 Fig 3Soldier Pile EP - Cantl.grf 10/7/15 (22:01) JCR



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

PanGEO INCORPORATED	Proposed Development 2500 Elliott Avenue Seattle, WA	DESIGN LATERAL PRESSURES SOLDIER PILE WALL MULTIPLE LEVELS OF TIEBACK
		Project No. 15-166 / Figure No. 4

15-166 Fig 4 Soldier Pile EP - 2+ Tiebacks.grf 10/7/15 (22:08) JCR

APPENDIX A

SUMMARY BORING LOGS

RELATIVE DENSITY / CONSISTENCY

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

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

DESCRIPTIONS OF SOIL STRUCTURES

Layered: Units of material distinguished by color and/or composition from material units above and below

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	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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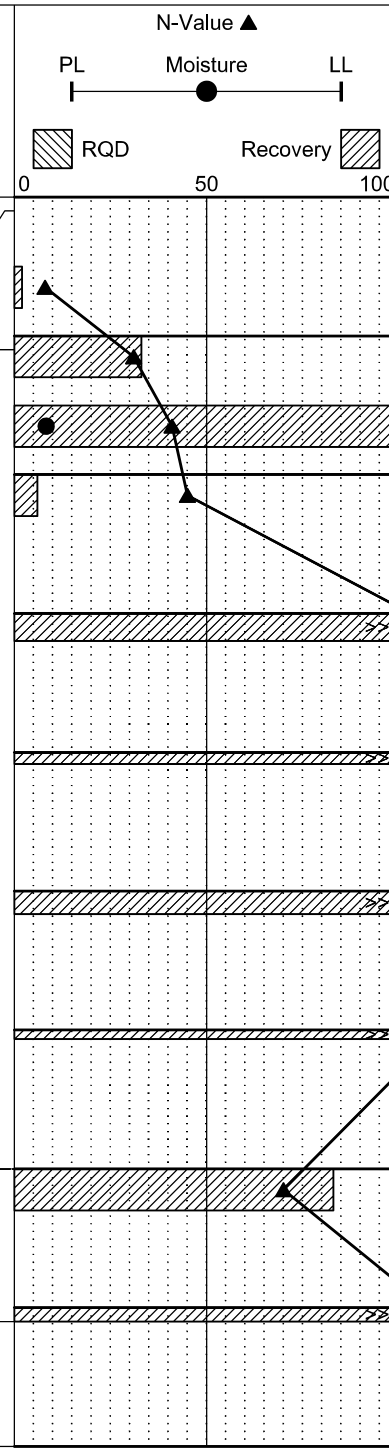
Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	Proposed Development, 2500 Elliot Street	Surface Elevation:	52.0ft
Job Number:	15-166	Top of Casing Elev.:	N/A
Location:	Seattle, Washington	Drilling Method:	Hollow Stem Auger
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Blows / 6 in.	Other Tests	MATERIAL DESCRIPTION
0				Approx. 2" asphalt over a thin layer of crushed rock base.
				Loose, moist, gray-brown, sandy GRAVEL with silt (GWS); (Fill).
	1	3 4 4		
5	2	9 12 19		Dense to very dense, moist, gray, silty SAND with gravel (SM); occasional cobbles, interbedded diamict sandy lenses (Pre-Olympia Deposits - Qpo).
	3	14 16 25	GS	Sample S-3: 33.8% fines, 8.3% moisture.
10	4	23 22 23		
15	5	21 50/6		Sample S-5: becomes very dense, occasional silt layers.
20	6	50/5		Sample S-6: increasing sand.
25	7	31 50/4		
30	8	50/4		
35	9	24 28 42		Dense, wet, gray-brown fine to coarse GRAVEL with sand (GW); trace to slightly silty (Pre-Olympia Deposits - Qpo).
				Sample S-10: silt and clay layers.
40	10	50/6		
				Bottom of the boring at 40.5 feet below ground surface. Groundwater was observed at around 35 feet at the time of drilling.
45				



Completion Depth:	40.5ft
Date Borehole Started:	9/17/15
Date Borehole Completed:	9/17/15
Logged By:	NER
Drilling Company:	Boretec

Remarks: Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism.

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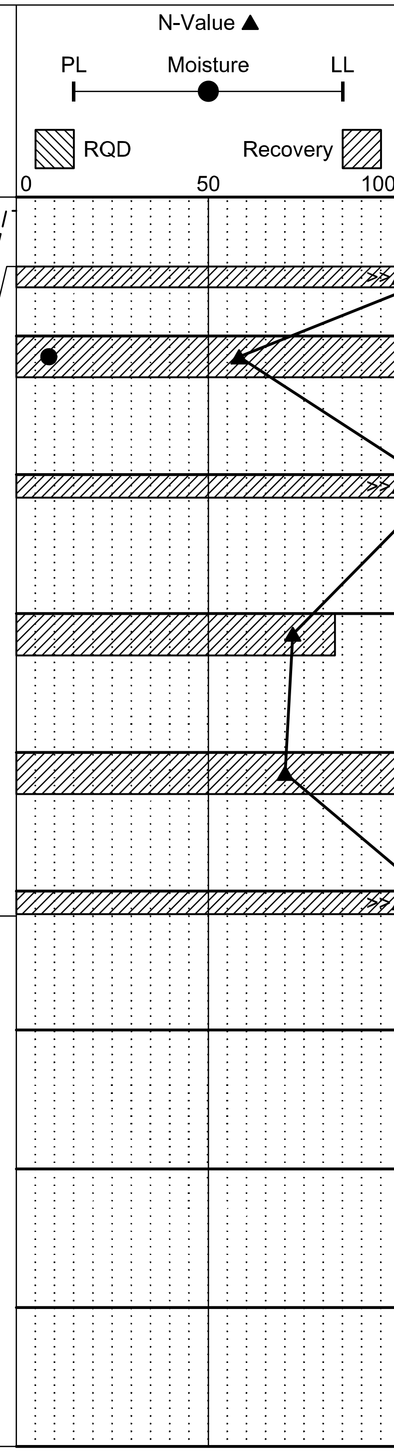
LOG OF TEST BORING PG-1

Figure A-2

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

Project:	Proposed Development, 2500 Elliot Street		Surface Elevation:	36.5ft
Job Number:	15-166		Top of Casing Elev.:	N/A
Location:	Seattle, Washington		Drilling Method:	Hollow Stem Auger
Coordinates:	Northing: , Easting:		Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						Medium dense, moist, gray-brown, sandy GRAVEL with silt (GWS); (Fill).
						Medium dense, moist, brown, gravelly, silty SAND (SM); based on drill cuttings (Fill).
	1		31 50/3			
5	2		16 27 31	GS		Very dense, moist to wet, brown to gray, silty SAND with gravel (SM); diamict texture, occasional sandy lenses and cobbles (Pre-Olympia Deposits - Qpo). Sample S-2: becomes iron oxide stained, 18% fines, 8.5% moisture.
10	3		26 50/4			
15	4		20 30 42			Sample S-4: increasing gravels.
20	5		21 30 40			Sample S-5: becomes wet.
25	6		41 50/4			
						Bottom of the boring at 25.9 feet below ground surface. Groundwater was observed at around 20 feet at the time of drilling.
30						
35						
40						
45						



Completion Depth:	25.9ft
Date Borehole Started:	9/17/15
Date Borehole Completed:	9/17/15
Logged By:	NER
Drilling Company:	Boretec

Remarks: Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism.

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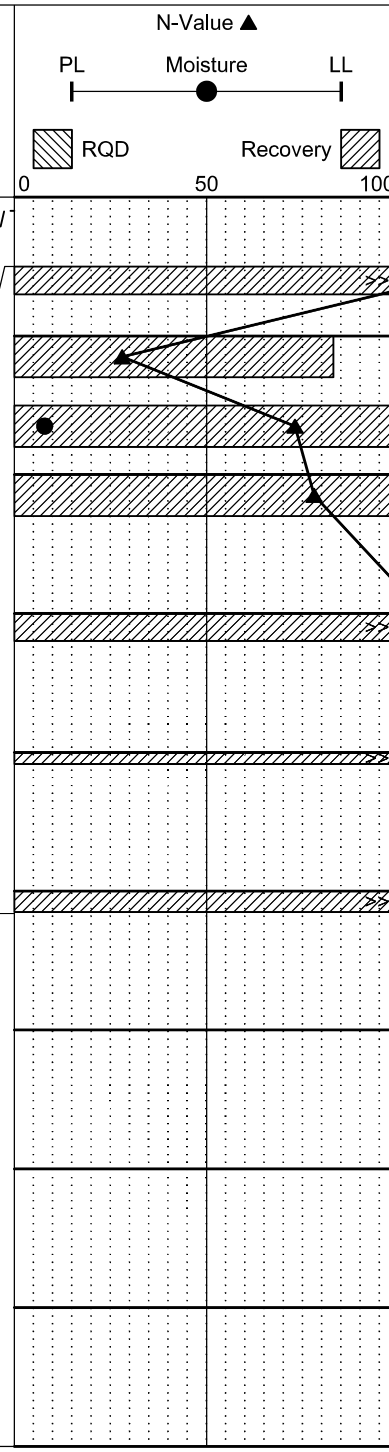
LOG OF TEST BORING PG-2

Figure A-3

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

Project:	Proposed Development, 2500 Elliot Street	Surface Elevation:	35.0ft
Job Number:	15-166	Top of Casing Elev.:	N/A
Location:	Seattle, Washington	Drilling Method:	Hollow Stem Auger
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						Medium dense, moist, gray-brown, sandy GRAVEL with silt (GWS); (Fill).
	1		12 50/6			Medium dense, moist, brown, gravelly, silty SAND (SM); based on drill cuttings (Fill).
5	2		18 17 11			Very dense to medium dense, moist to wet, gray-brown, silty SAND with gravel (SM); occasional cobbles, interbedded diamict sandy lenses (Pre-Olympia Deposits - Qpo). Sample S-2: medium dense, weathered.
	3		18 33 40	GS		Sample S-3: becomes very dense, gravelly, 19.3% fines, 7.9% moisture.
10	4		21 33 45			Sample S-4: iron oxide staining.
15	5		37 50/6			
20	6		50/5			
						- becomes wet.
25	7		43 50/3			
						Bottom of the boring at 25.8 feet below ground surface. Groundwater was observed at around 23 feet at the time of drilling.
30						
35						
40						
45						



Completion Depth:	25.8ft
Date Borehole Started:	9/17/15
Date Borehole Completed:	9/17/15
Logged By:	NER
Drilling Company:	Boretec

Remarks: Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism.

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LOG OF TEST BORING PG-3

Figure A-4

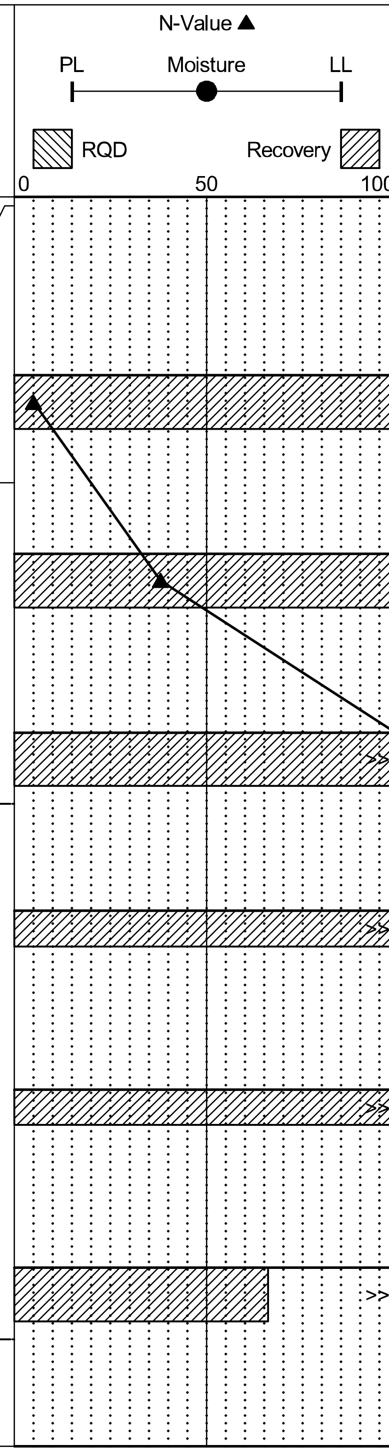
The stratification lines represent approximate boundaries. The transition may be gradual.

APPENDIX B

PREVIOUS EXPLORATIONS

Project:	Western & Vine	Surface Elevation:	n/a
Job Number:	12-162	Top of Casing Elev.:	
Location:	2521 Western Avenue, Seattle, WA	Drilling Method:	Hollow Stem Auger
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						3 inches thick asphalt pavement.
						Loose, damp, dark brown to brown, silty SAND with trace to some gravel, trace organics at upper 2.5 feet, some brick bebris at 5 feet (FILL).
5	S-1		4 3 2			
						Dense to very dense, damp, brown to gray, silty SAND with trace to some gravel, slight diamict.
10	S-2		16 18 20			
15	S-3		26 26 50/5			- 4 inches thick silt layer.
						Very dense, damp, gray, silty SAND with trace gravel to silty gravelly SAND, diamict structure.
20	S-4		26 50/5			
25	S-5		23 50/4			
30	S-6		21 22 50/5			
						Very dense, damp, brown, silty gravelly SAND with some iron oxide staining.
35						



Completion Depth:	61.5ft
Date Borehole Started:	10/13/12
Date Borehole Completed:	10/13/12
Logged By:	JC
Drilling Company:	Boretec, Inc.

Remarks: Boring was drilled with a trailer mounted drill rig. Standard Penetration Test (SPT) sampler driven with a 140 lb hammer using a rope and cathead dropping 30 inches per stroke.

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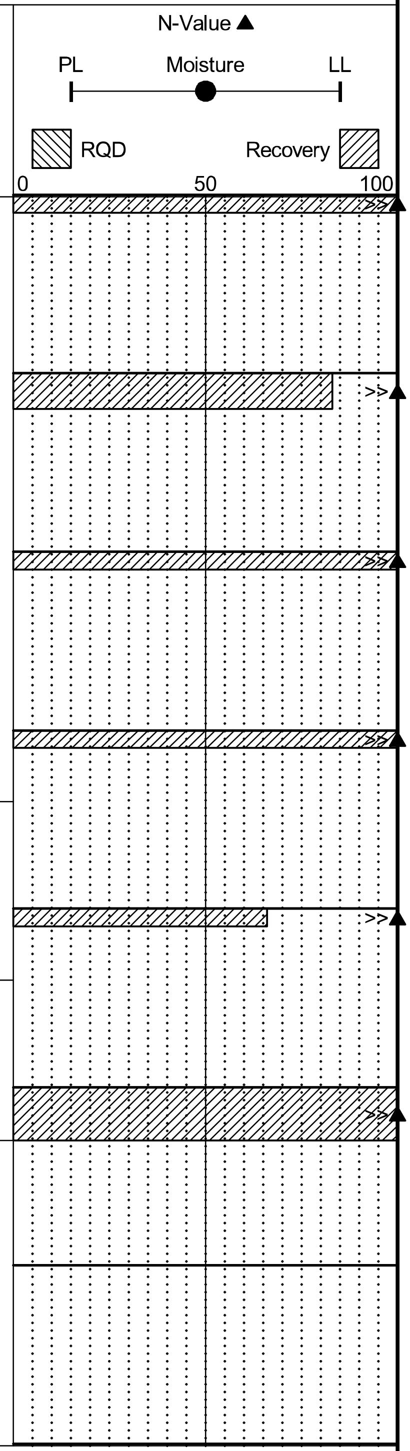
LOG OF TEST BORING BH-3

Figure A-4

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

Project:	Western & Vine	Surface Elevation:	n/a
Job Number:	12-162	Top of Casing Elev.:	
Location:	2521 Western Avenue, Seattle, WA	Drilling Method:	Hollow Stem Auger
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
35	S-7		50/4			Very dense, damp, brown, silty gravelly SAND with some iron oxide staining. *(Continued)*
40	S-8		28 50/3			
45	S-9		50/6			
50	S-10		50/2			- silty gravel layer.
						Very dense, moist, blue gray to gray, clayey gravelly SAND.
55	S-11		50/6			
						Very dense, wet (saturated), gray, silty finr SAND to fine sandy SILT.
60	S-12		24 27 50/4			
						Bottom of boring at 61.5 feet below ground surface. Groundwater was encountered at 56.5 feet during drilling.
65						
70						



Completion Depth:	61.5ft	Remarks: Boring was drilled with a trailer mounted drill rig. Standard Penetration Test (SPT) sampler driven with a 140 lb hammer using a rope and cathead dropping 30 inches per stroke.
Date Borehole Started:	10/13/12	
Date Borehole Completed:	10/13/12	
Logged By:	JC	
Drilling Company:	Boretec, Inc.	

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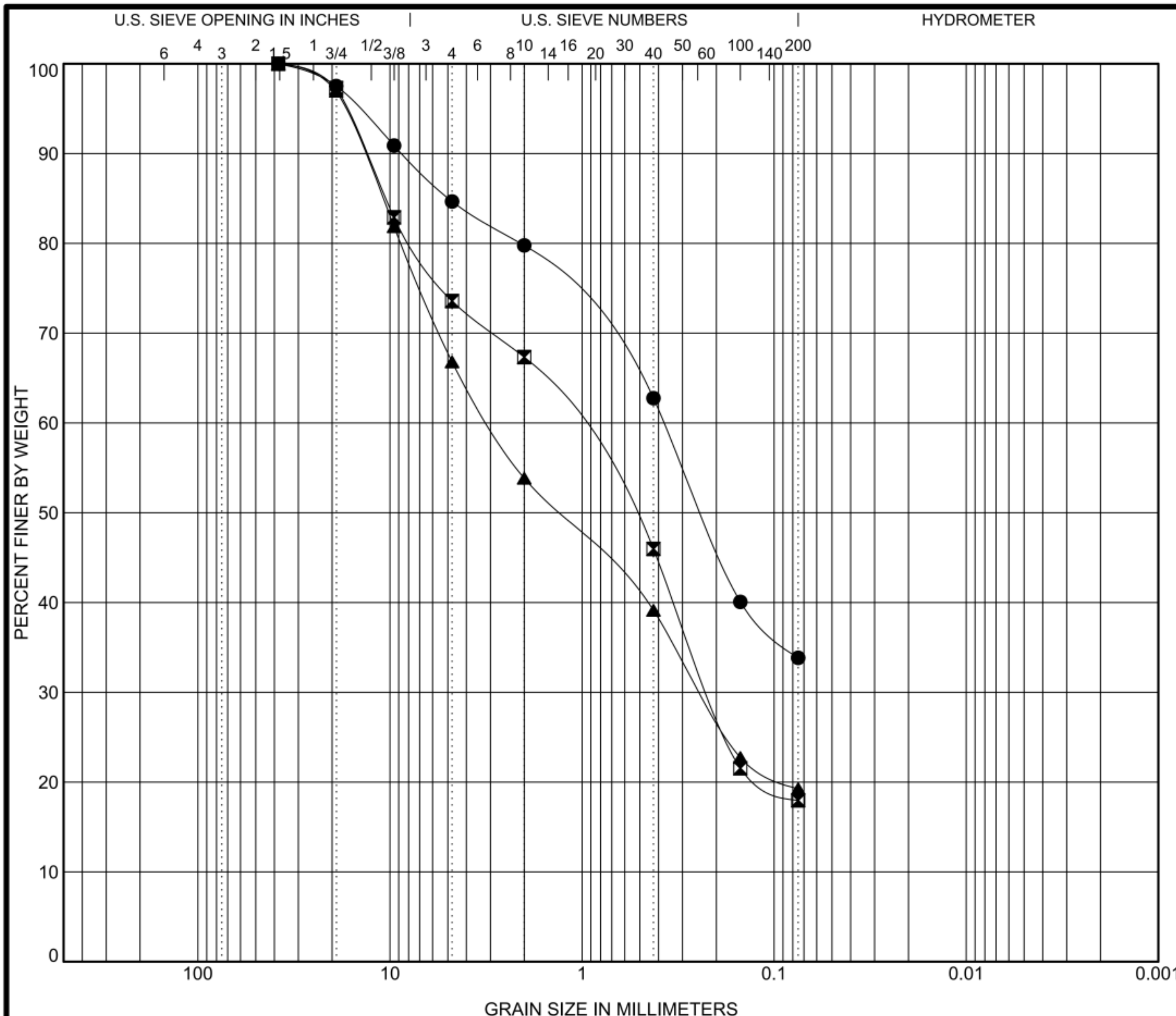
LOG OF TEST BORING BH-3

Figure A-4

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

APPENDIX C

LABORATORY TEST RESULT



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	**PG-1**	**@ 7.5 ft.**	**Silty SAND with gravel**					
⧗	**PG-2**	**@ 5.0 ft.**	**Silty SAND with gravel**					
▲	**PG-3**	**@ 7.5 ft.**	**Gravelly silty SAND**					

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**PG-1**	**7.5**	**38.1**	**0.374**			**15.3**	**50.8**	**33.8**	
⧗	**PG-2**	**5.0**	**38.1**	**1.175**	**0.215**		**26.4**	**55.6**	**18.0**	
▲	**PG-3**	**7.5**	**38.1**	**3.017**	**0.238**		**33.2**	**47.5**	**19.3**	

GRAIN SIZE 15-166 BORING LOGS.GPJ PANGEO.GDT 10/6/15

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GRAIN SIZE DISTRIBUTION

Project: Proposed Development, 2500 Elliot Street
Job Number: 15-166
Location: Seattle, Washington

Figure C-1