Exhibit 2

GEOTECHNICAL REPORT

PROPOSED DEVELOPMENT
3410-3420 23rd AVENUE WEST
SEATTLE, WASHINGTON

Project No. 18-243
September 2018

Prepared for:

Mirra Homes

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September 28, 2018
File No. 18-243

Mirra Homes
11624 SE 5th St, Ste 210
Seattle, WA 98005
Attn: Brooke Friedlander

Subject: **Geotechnical Report**
Proposed Development
3410-3420 23rd Ave W, Seattle, WA

Dear Ms. Friedlander,

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

In summary, based on the borings drilled, the project site is generally underlain by fill and colluvium overlying Lawton Clay and Pre-Fraser Deposits. In our opinion, permanent soldier pile walls will be needed near the east property line to provide the long-term stability of the site for the post-construction condition. Temporary soldier pile walls may also be needed to support the excavations for foundation construction in other areas. Based on the soil conditions and anticipated basement finish floor elevations, in our opinion, a deep foundation system, such as pin piles, should be used to support the proposed buildings.

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

Sincerely,

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

Encl.: Geotechnical Report

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GEOTECHNICAL REPORT
PROPOSED DEVELOPMENT
3410-3420 23RD AVENUE WEST
SEATTLE, WASHINGTON

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering study that was undertaken to support the design and construction of the proposed development in Seattle, Washington. We completed our engineering study in accordance with our proposal dated July 19, 2018, which was approved by you on July 20, 2018. Our service scope included reviewing available geologic and geotechnical data in the site vicinity, drilling three test borings at the site, performing engineering analyses, and developing the geotechnical design recommendations presented in this report.

2.0 PROJECT AND SITE DESCRIPTION

The subject site consists of three adjoining parcels with addresses of 3410, 3416, and 3420 – 23rd Avenue West in Seattle, Washington (see Vicinity Map, Figure 1). The combined site is rectangular in shape, and is bordered to the west by 23rd Avenue West, to the east by an alley, and to the north and south by existing single-family houses (see Figure 2). The site is currently occupied by 3 single-family residences. Based on review of the topographic survey map, the site generally slopes down from west to east with an average gradient of about 25 percent with a total vertical relief of about 30 feet. However, steep slopes exist in the eastern portion of the site and the total steep slope height is about 10 to 12 feet.

We understand that you plan to remove the existing buildings and to construct a total of six (6) townhome buildings (see Figure 2). Based on review of the preliminary design plans, the proposed townhome buildings will be 3-story wood frame structures with concrete slab floors (see Plate 1 on page 2). We anticipate that temporary excavations up to 7 to 8 feet will be needed for the foundation construction.

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

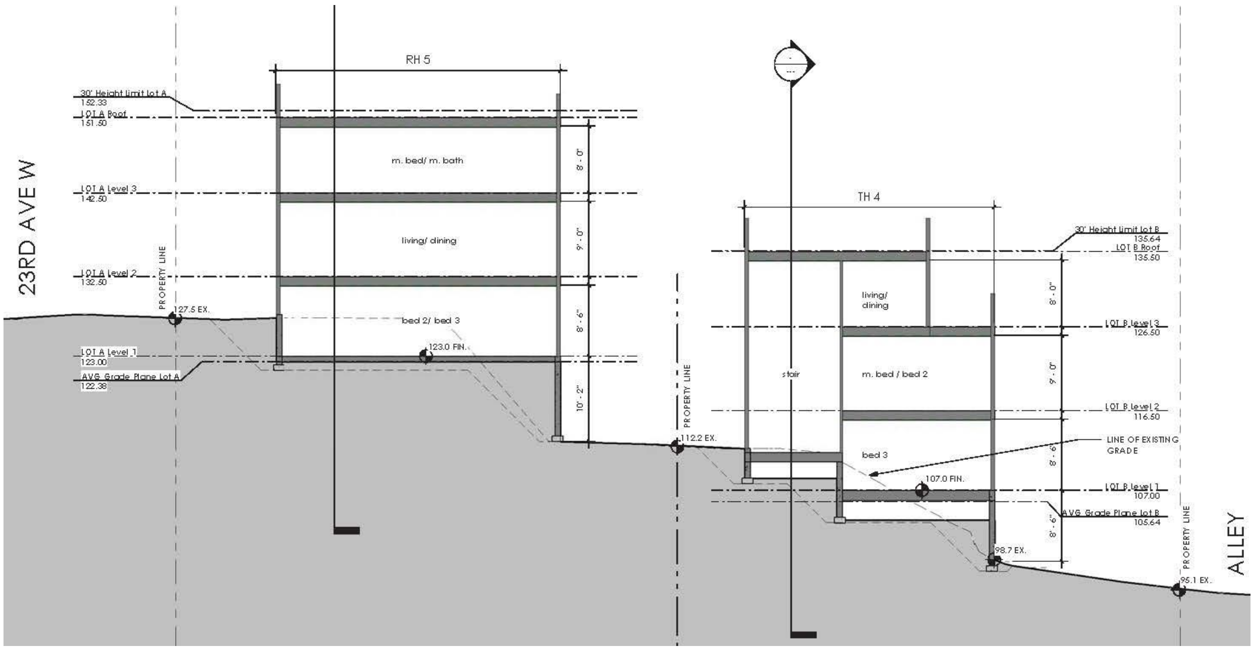


Plate 1. Typical east-west building section, looking north.

3.0 SUBSURFACE EXPLORATIONS

PanGEO completed three test borings (PG-1 through PG-3) at the subject site on August 3, 2018. The approximate locations of the borings are indicated on the attached Figure 2. The borings were drilled to depths of about 26.5 feet in PG-1, 21.5 feet in PG-2, and 16.5 feet in PG-3, using a hand-operated portable drill rig owned and operated by CN Drilling of Seattle, Washington.

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

A geologist from PanGEO was present during the field exploration to observe the drilling, assist in sampling, and to describe and document the soil samples obtained from the borings. The soil samples were described and field classified in general accordance with the symbols and terms outlined in Figure A-1, and the summary boring logs are included as Figures A-2 through A-4.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Site Geology

Based on our review of *The Geologic Map of Seattle – A Progress Report* (Troost, et al., 2005) the surficial geology in the vicinity of the site is mapped as Lawton Clay (Map Unit: Qvlc) and Pre-Fraser Deposits (Map Unit: Qpf), with Vashon Advance Outwash (Geologic Map Unit: Qva) mapped approximate one half of a block to the west.

Troost, et al. describes Lawton Clay (Qvlc) as laminated to massive silt, clayey silt, and silty clay deposited in lowland proglacial lakes that was subsequently overridden by glacial ice and is typically very stiff to hard. Pre-Freser Deposits (Qpf) typically consists of very dense and hard, interbedded sand, gravel, silt, and diamicts of indeterminate age and origin. Advance Outwash (Qva) typically consists of moderately to well sorted, slightly oxidized sand and gravel that has been overridden by glacial ice and is typically dense.

4.2 Original Street Grading Profiles

Based on our review of the historic street grading profiles obtained from the City of Seattle archives, original grades along the centerline of 23th Avenue West were raised up to about 16 feet in the past street grading, and original grades along the west property line (33' east of the 23rd Avenue West centerline) were raised about 6 to 21 feet in the past street grading, which is generally consistent with the existing fill thickness observed in PG-1. Although the street grading profile did not contain any grading information at the site, it is our opinion that it is likely fill was also placed at the site from previous on-site developments and/or street grading based on the thickness of the fill along the west property line.

4.3 Soil

The soil conditions encountered in the test borings generally consisted of fill and colluvium overlying Lawton Clay and pre-Fraser deposits. A summary description of the generalized soil units encountered in the test borings is presented below. Please refer to the summary boring logs in Appendix A for more details.

UNIT 1: Fill – Fill was encountered below the topsoil to a depth of about 7.5, 10, and 5.5 feet below the surface at PG-1, PG-2, and PG-3, respectively. Fill encountered generally consisted of loose, multicolored, silty sand with trace gravel and organics.

UNIT 2: Colluvium Deposits **–** Below the fill at PG-1 and PG-2 the borings encountered soft to medium stiff silt and very loose sand and silty sand to about 23 and 18 feet below the existing grade in PG-1 and PG-2, respectively. We interpret this unit as colluvium deposits. This unit was not encountered in boring PG-3.

UNIT 3: Lawton Clay Deposits **–** Below Unit 2, PG-1 and PG-2 encountered stiff to hard, light brown to gray silt. We interpret this unit to be Lawton Clay deposits mapped at the site. This unit extended to the termination depth in PG-1 and PG-2 at 26.5 and 21.5 feet, respectively. This unit was not encountered in PG-3.

UNIT 4: Pre-Fraser Deposits **–** Below Unit 1, PG-3 encountered medium dense to dense, sand and silty sand to the termination depth of about 16.5 feet below the surface. We interpret this unit as mapped Pre-Fraser deposits. This unit was not encountered in borings PG-1 and PG-2.

4.4 GROUNDWATER

Perched groundwater was encountered in borings PG-1 and PG-2 at depths of about 17.5 and 15 feet below existing grades, respectively. The groundwater was perched atop the Lawton Clay deposits. Groundwater was not encountered within the drilling depth in PG-3. 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 SITE STABILITY AND ECA CONSIDERATIONS

5.1 HISTORICAL LANDSLIDES

According to the City of Seattle GIS maps and site topographic survey map, the eastern portion of the site are a steep slope (40% or greater) ECA. The entire site is also mapped as a potential landslide ECA due to its geologic conditions but is not as a known slide ECA.

As part of our study, we reviewed records of historical landslides in the Seattle Landslide Study commissioned by the Seattle Public Utilities (SPU) to gain a general understanding of the past landslide activities in the project vicinity. Our review of the Seattle Landslide Study indicated that there were four past known slides in the immediate vicinity of the subject site:

1. 3212 – 23rd Avenue W – Located approximately one block south of the site. Occurred in January 1997;
2. 3232 – 23rd Avenue W – Located about block south of the site. Occurred in January 1997.
3. 3253 – 23rd Avenue W – Located approximately one-half block south of the site. Occurred in January 1986; and
4. 3616 - 24th Avenue W – Located approximately one block northwest of the site. Occurred in February 1983.

Detailed information for these known slides was not available. However, according to the City's records, these known slides are generally small in size, and are located in the steeper slope areas or associated with the retaining wall failure.

5.2 SLOPE STABILITY ANALYSIS

We conducted a site reconnaissance of the subject lot on August 3, 2018. No obvious evidence of surface slope instability was observed at the site during our visit. To evaluate the factor of safety against potential future slope instability, we performed slope stability analysis using the computer program *Slide v6.0* (Rocscience, 2010) based on the post-construction conditions, as currently planned.

The cross section for the post-construction condition used for slope stability analysis is shown on Figure 3. The approximate location of the cross-section A-A' is shown in the Figure 2. Search routines were used to identify the surface that has the lowest factor of safety. The seismic stability was analyzed using pseudo-static procedures, where the effect of earthquake ground shaking is represented by the use of a "seismic coefficient" in the stability calculations. In our pseudo-static stability analysis, one-half of the expected peak ground acceleration, or 0.2636g (one half of 0.5272g), was used. The soil parameters for the soil units were assigned based on empirical correlations using SPT blowcount values measured in the borings, and our experience with similar soil conditions and published literatures. The profiles and soil parameters used in our slope stability analysis are shown in Figures 5 through 10.

Based on our analyses, without improvements/mitigations, the factors of safety against future slope instability for the seismic conditions are not adequate for the post-construction conditions. To improve the site slope stability for the post-construction condition, we recommended that permanent soldier pile walls be installed along the east of the proposed east buildings. Based on our analysis, the permanent soldier pile wall system may consist of driven soldier piles, such as W8x35, combined with helical anchors. The driven soldier piles should be spaced 6 feet or less and should extend at least 10 feet into Units 3 and 4 (i.e. soldier pile tip elevation of 78 feet or lower). The soldier pile wall should also be determined based on the structural design of the walls. The design recommendations for the soldier piles are presented in the Section 6.2 of this report.

The results of our post-construction slope stability analyses with the permanent soldier pile walls are summarized on Figures 5 and 10, for static and pseudo-static conditions. Based on the results of the analysis, it is our opinion that, with the recommended site stabilization, the post-construction site slope has adequate factors of safety against failures under static and seismic conditions.

Qualifications – Based on the results of our study, it is our opinion that the proposed development as planned will have adequate factors of safety against potential future slope instability and will not have adverse impacts on the subject and surrounding properties, provided that the recommendations presented in this report are properly incorporated into the design of the project, and the project is properly designed and constructed. However, it should be noted that any development on or near a steep slope or a potential landslide area always involves some level of risk. In addition, future activities on and off the site could also affect the stability of the subject site.

5.3 Relief from Steep Slope Development Standards

As described above, the eastern portion of the site is a steep slope ECA. While development within a steep slope ECA and their buffers are restricted, SDCI allows for relief from the prohibition on steep slope development if one of the following applies:

a) The proposed development is located where existing development is located;

b) The development is located on steep slope areas that have been created through previous legal grading activities;

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

d) When the Director determines, based on geotechnical expertise, that application of the steep slope regulations would prevent necessary stabilization of landslide-prone areas.

Based on the the SDCI GIS Maps, the topographic survey of the site (see Figure 2), and our field observations, the vertical rise in the mapped steep slope ECA areas near the site are less than 20 feet, and is 30 feet from other steep slope areas, and thereby qualifies for relief under provision (C). However, it should be noted that the City of Seattle Department of Construction and Inspections (SDCI) will make the final decision on applicability of the above criteria.

In summary, it is our opinion the subject site meets criteria (C) to receive relief from prohibition from steep slope development. In addition, provided that the recommendations presented in this report are followed during the design and construction of this project, it is our opinion the proposed project will not adversely impact the site stability for the subject and surrounding properties.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 SEISMIC DESIGN PARAMETERS

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

Table 1 – Summary Seismic Design Parameters per 2015 IBC

Site Class	Spectral Acceleration at 0.2 sec. (g) 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}
E	1.318	0.512	1.0	1.50	0.791	0.819

Soil Liquefaction Potential: Perched groundwater was present above Lawton Clay in PG-1 and PG-2. The saturated loose silty sand below the perched groundwater table may potentially liquefy during ab IBC-event earthquake. If occurs, the seismic settlement is estimated to be on the order of one inch, due to the thin thickness of the saturated soils, and will be effectively mitigated by pin pile foundations recommended for the proposed project. Our recommendations for the pin pile design are presented in the "Geotechnical Recommendations" sections.

6.2 SOLDIER PILE WALLS

As discussed in the Section 5.2 of this report, permanent soldier pile walls will be needed near the east property line to stabilize the site as part of the proposed development. Additionally, temporary soldier pile walls may also be needed in other locations to support the temporary excavations. We recommend that the lateral earth pressures depicted on Figure 11 be used for design of the soldier pile walls.

The active earth pressure should be applied over the full width of pile spacing above the base of excavation, and over one pile diameter below the base of excavation. The passive resistance should be applied over two pile diameter or one pile spacing, whichever is less. The minimum soldier pile embedment should be determined by the shoring wall designer. The stabilization wall on the east of the east building should extend to elevation 192 feet or lower. Permanent solder piles should have corrosion protection coating.

The recommended passive earth pressure assumes level ground surface at the bottom of the wall, and the level bench extends at least 10 feet in front of the wall. If the ground surface in front of the wall needs to be sloped to accommodate the difference in finish floor elevation, the passive resistance in the sloped portion of the ground should be ignored or reduced for design calculations.

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 and an allowable end bearing of 10 ksf in Soil Units 3 and 4. We recommend that any voids behind the timber lagging be backfilled with Controlled Density Fill (CDF).

***Lagging*:** Lagging design recommendations are presented on Figure 11.

Helical Anchors: Based on our slope stability analyses, one row of earth anchors will be needed to provide additional lateral resistance for the soldier pile wall (see Figures 5 through 10). The anchors should be installed along the soldier pile wall and have a maximum horizontal spacing of 12 feet and a minimum length of 25 feet. The anchors should be embedded into the dense glacial soils (Units 3 & 4) and have an allowable axial tension capacity of 20 kips per anchor, at an inclination of 25 degrees below horizontal.

6.3 BUILDING FOUNDATIONS

6.3.1 General

Based on the soil conditions and anticipated basement finish floor elevations, in our opinion, the proposed buildings should be supported by deep foundations, such as small diameter steep pipes, to avoid excessive long-term building settlement. The soldier piles may also be incorporated in the building design to support the buildings. The following sections present our recommendations for the pin pile foundations. Please refer to the Section 6.2 for soldier pile design.

6.3.2 Pin Pile Foundations

***Pin Pile Sizes* -** In our opinion, 3-, 4-, or 6-inch diameter, Schedule 40, galvanized, steel pipes (pin piles) may be used to support the new structure. Three, four, and six-inch diameter pin piles are typically installed using small hammers mounted on a small excavator.

***Pin Pile Capacity* -** The number of piles required depends on the magnitude of the design load. Allowable axial compression capacities of 6, 10, and 15 tons may be used for the 3-, 4-, and 6-inch diameter pin piles, respectively, with an approximate factor of safety of 2. Penetration resistance required to achieve the capacities will be determined based on the hammer used to install the pile. Tensile capacity of pin piles should be ignored in design calculations.

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

Pile splices may be made with compression fitted sleeve pipe couplers (see Typical Splicing Detail on page 11). Splicing using welding of pipe joints should not be used, as welds will typically be broken during driving.

Three-, four-, and six-inch diameter piles are typically installed using small (approximately 1,100 to 3,000 pound) hammers mounted to a small excavator. The criterion for driving refusal is defined as the minimum amount of time (in seconds) required to achieve one inch of penetration, and it varies with the size of hammer used for pile driving. For 3-, 4-, and 6-inch pin piles, the Table 4 below is a summary of driving refusal criteria for different hammer sizes that are commonly used:

Table 2 - Summary of Commonly-Accepted Driving Criteria for 3-, 4-, and 6-inch Pin Pile with a 6, 10, and 15-ton Allowable Axial Compression Load

Hammer Model	Hammer Weight (lb) / Blows per minute	3" Pile Refusal Criteria (seconds per inch of penetration)	4" Pile Refusal Criteria (seconds per inch of penetration)	6" Pile Refusal Criteria (seconds per inch of penetration)
Hydraulic TB 425	1,100 / 900	6	10	20
Hydraulic TB 725X	2,000 / 600	3	4	10
Hydraulic TB 830X	3,000 / 500	N/A	N/A	6

Please note that these refusal criteria were established empirically based on previous load tests on 3-, 4-, and 6-inch pin piles. Contractors may select a different hammer for driving these piles, and propose a different driving criterion. In this case, it is the contractor's responsibility to demonstrate to the Engineer's satisfaction that the design load can be achieved based on their selected equipment and driving criteria.

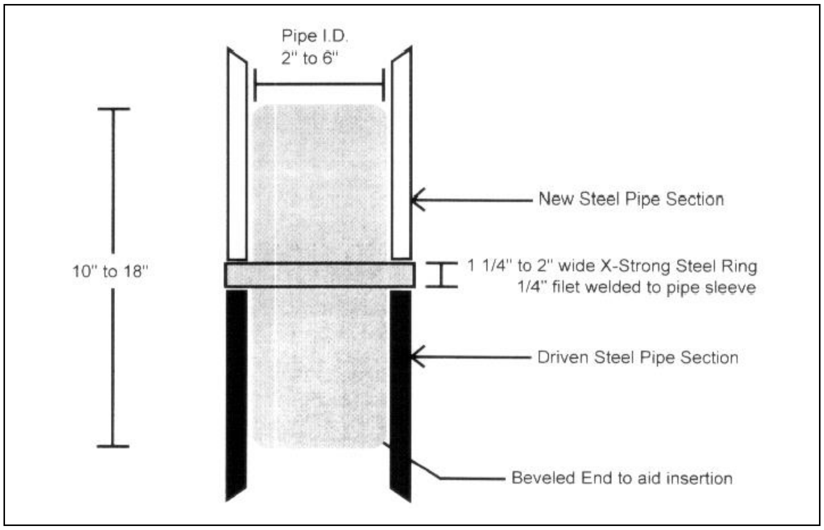
Pin Pile Specifications - We recommend that the following specifications be included on the foundation plan:

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

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

3. Piles shall be driven in nominal sections and connected with compression fitted sleeve couplers (see 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.

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



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

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

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

Estimated Pile Length – The subsurface conditions at the site will likely vary substantially across the site. Based on the soil conditions at the site and our experience in the project area, for planning and cost estimating purposes, we estimate that pile length may range from about 25 to 35 feet.

6.4 CONCRETE RETAINING/BASEMENT WALLS

Retaining walls, if needed, should be properly designed to resist the lateral earth pressures exerted by the soils behind the wall. Proper drainage provisions should also be provided behind the walls to intercept and remove groundwater that may be present behind the wall. Our geotechnical recommendations for the design and construction of the retaining/basement walls are presented below.

6.4.1 Lateral Earth Pressures

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

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

6.4.2 Wall Surcharge

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

6.4.3 Lateral Resistance

Lateral forces from wind or seismic loading and unbalanced lateral earth pressures may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundation and by friction acting on the base of the foundation. Passive resistance values may be determined using an equivalent fluid weight of 250 pounds per cubic foot (pcf). A friction coefficient of 0.35 may be used to determine the frictional resistance at the base of the foundation. Both of these values include a safety factor of at least 1.5.

6.4.4 Wall Drainage

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

We recommend that a building envelope specialist be consulted for damp-proofing and waterproofing recommendations.

6.4.5 Wall Backfill

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

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

6.5 CONCRETE SLAB-ON-GRADE

It is our opinion that conventional slab-on-grade concrete floors may be used for the proposed building. We recommend that on-grade floor slabs be supported on competent a minimum of 12 inches of structural fill placed on recompacted on-site soils.

The concrete slab-on-grade floors should be underlain by at least 4 inches of capillary break, which consists 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 20 percent passing the No. 4 sieve and less than 5 percent by weight of the material passing the U.S.

Standard No. 100 sieve. We also recommend that a minimum 10-mil polyethylene vapor barrier be placed below the proposed basement slab.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 SITE PREPARATION

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

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

7.2 TEMPORARY EXCAVATION

As currently planned, temporary excavations for the proposed project will be up to 7 to 8 feet for the proposed buildings. We anticipate the excavations to encounter mostly fill and loose to medium dense silty sand. All temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring.

It is our opinion that temporary excavations for the proposed construction, where appropriate, should be sloped 1H:1V or flatter. Based on our current understanding of the building design, unsupported open cuts are feasible in most areas except some areas along the north and south site boundaries. Where space is not sufficient for unsupported open cuts, temporary shoring will be needed to support the temporary excavations. In our opinion, cantilever soldier pile walls may be used as the temporary shoring wall, and the design of the the soldier pile walls are presented in the Section 6.2 of this report. Where space may be limited, the use of L-shaped footings may be required to conserve space for the temporary cuts.

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

7.3 Material Reuse

In the context of this report, structural fill is defined as compacted fill placed under footings, concrete stairs and landings, and slabs, or other load-bearing areas. In our opinion, the on-site soils are not suitable to be reused as structural fill. Structural fill should consist of imported, well-grade, granular material, such as City of Seattle Type 2 and 17 or WSDOT Gravel Borrow. Well-graded recycled concrete may also be considered as a source of structural fill. Use of recycled concrete as structural fill should be approved by the geotechnical engineer. The on-site soil can be used as general fill in the non-structural and landscaping areas. If use of the on-site soil is planned, the excavated soil should be stockpiled and protected with plastic sheeting to prevent softening from rainfall in the wet season.

7.4 Structural Fill Placement and Compaction

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

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

7.5 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 City of Seattle storm sewer.

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

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

8.0 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 the stability of open cut slopes;
- Observe the installation of soldier piles and helical anchors;
- 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 Mirra Homes LLC and the project design team. Recommendations contained in this report are based on a site reconnaissance, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of work.

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

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

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

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

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

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

Sincerely,

HONGBO MICHAEL XUE
STATE OF WASHINGTON
35016
REGISTERED
PROFESSIONAL ENGINEER

9/28/2018

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

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

10.0 REFERENCES

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

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

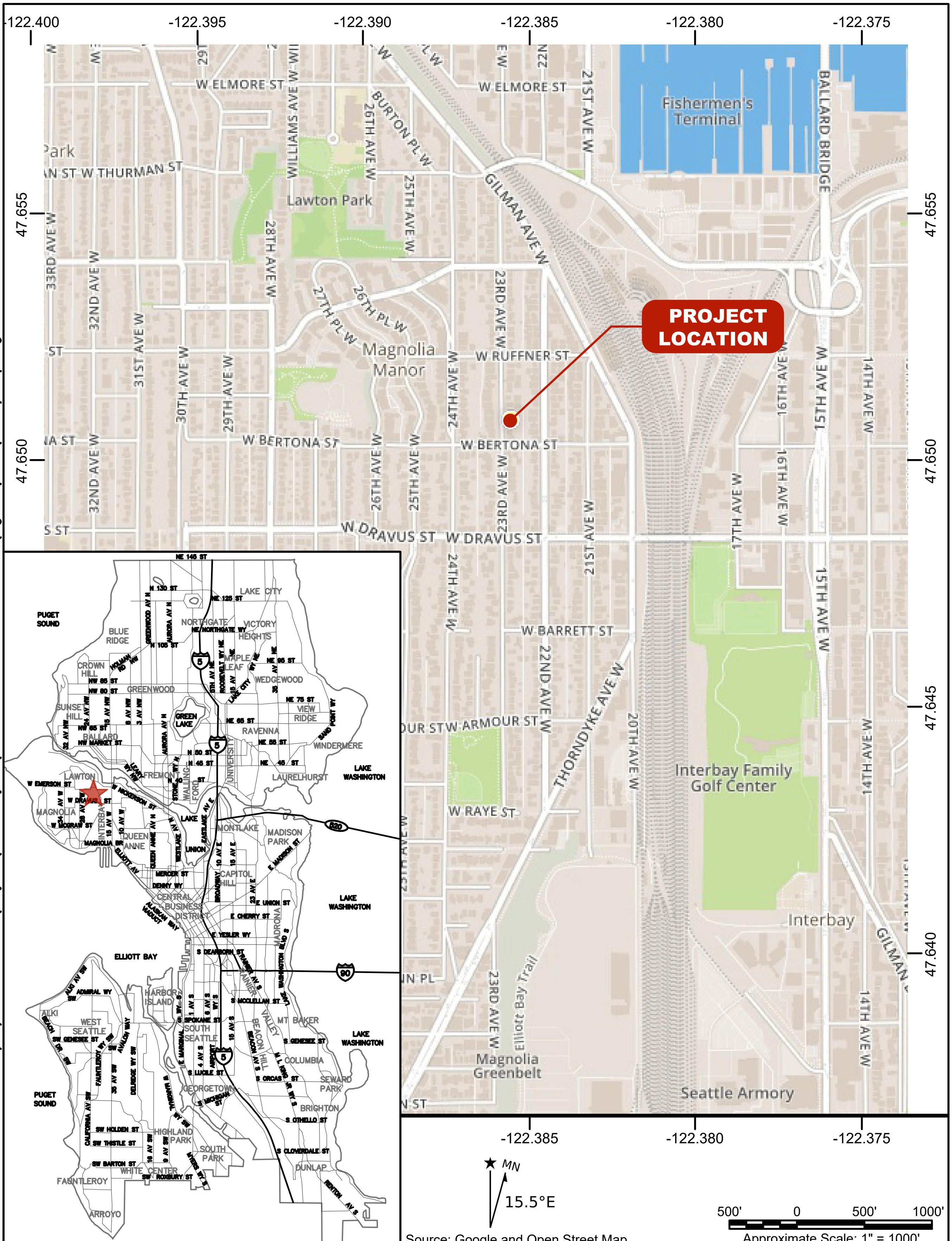
City of Seattle, 2017, *Standard Specifications for Road, Bridges, and Municipal Construction*.

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

Shannon & Wilson, Inc., 2003, *Seattle landslide study update, addendum to the Seattle landslide study, stability improvement areas: Unpublished consultant report no. 21-1-08913-016 for Seattle Public Utilities, Seattle, Wash., 12 p.*

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.

WSDOT, 2018, *Standard Specifications for Road, Bridges, and Municipal Construction*.



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DATE: 8/17/2018

CHECKED BY: HMX

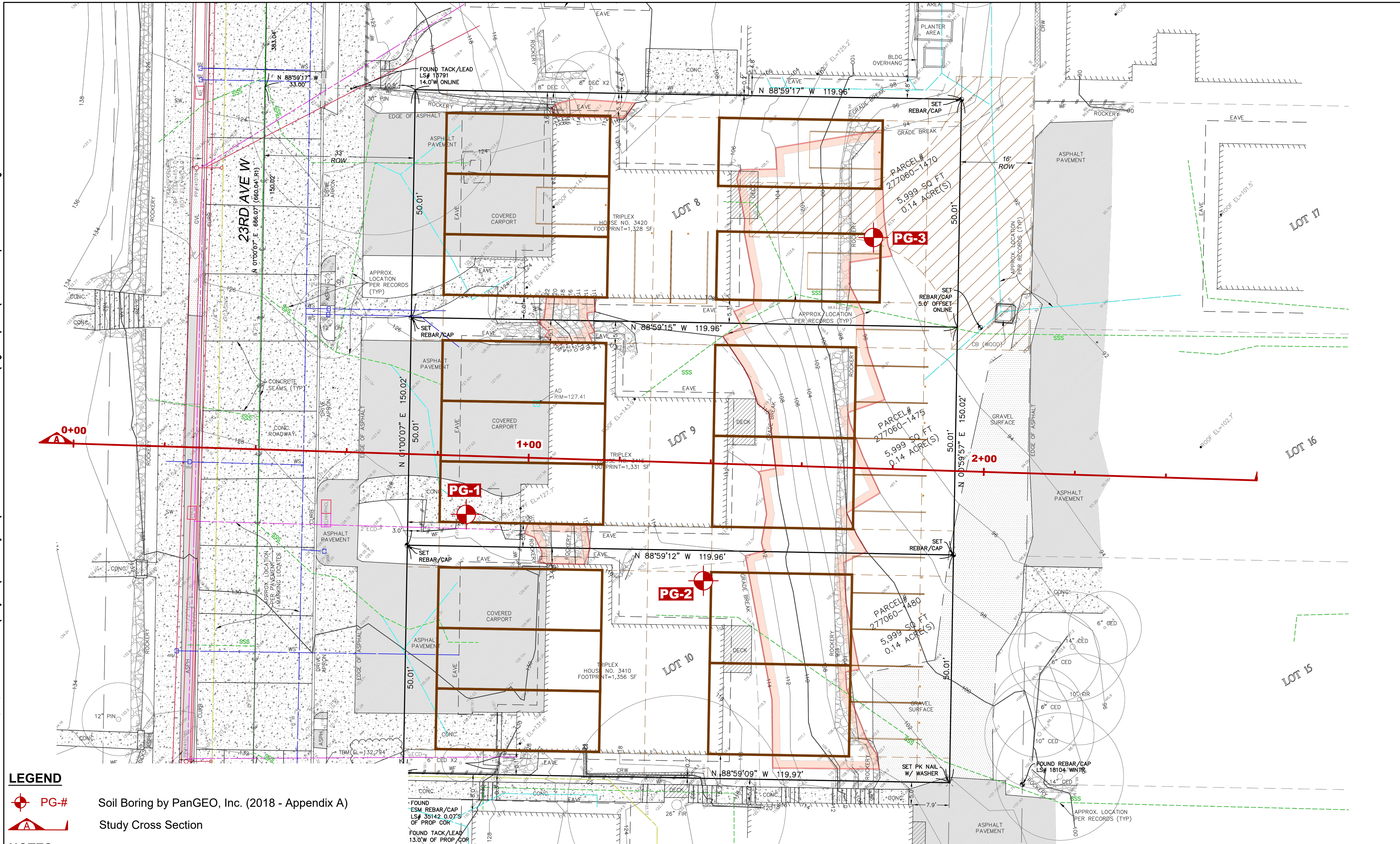
DRAWN BY: NTW

PanGEO INCORPORATED

Proposed Development
3410 - 3420 23rd Avenue W
Seattle, Washington

VICINITY MAP

PROJECT NO.	FIGURE NO.
18-243	1



LEGEND

PG-#	Soil Boring by PanGEO, Inc. (2018 - Appendix A)
A	Study Cross Section

NOTES

1. Base map derived from Topographic and Boundary Survey dated 6/27/2018 by Terrane and Preliminary Site Plan provided by Mirra Homes on 7/12/2018
2. Location of borings are approximate and based on the relative locations of known site features and are provided for relative information only and are not substitution for field survey.
3. Vertical Datum: NAVD '88

10' 0 10' 20'
scale in feet
Scale: 1" = 20'

PanGEO INCORPORATED

Proposed Development
3410 - 3420 23rd Avenue W
Seattle, Washington

SITE AND EXPLORATION PLAN

PROJECT NO. 18-243

FIGURE NO. 2

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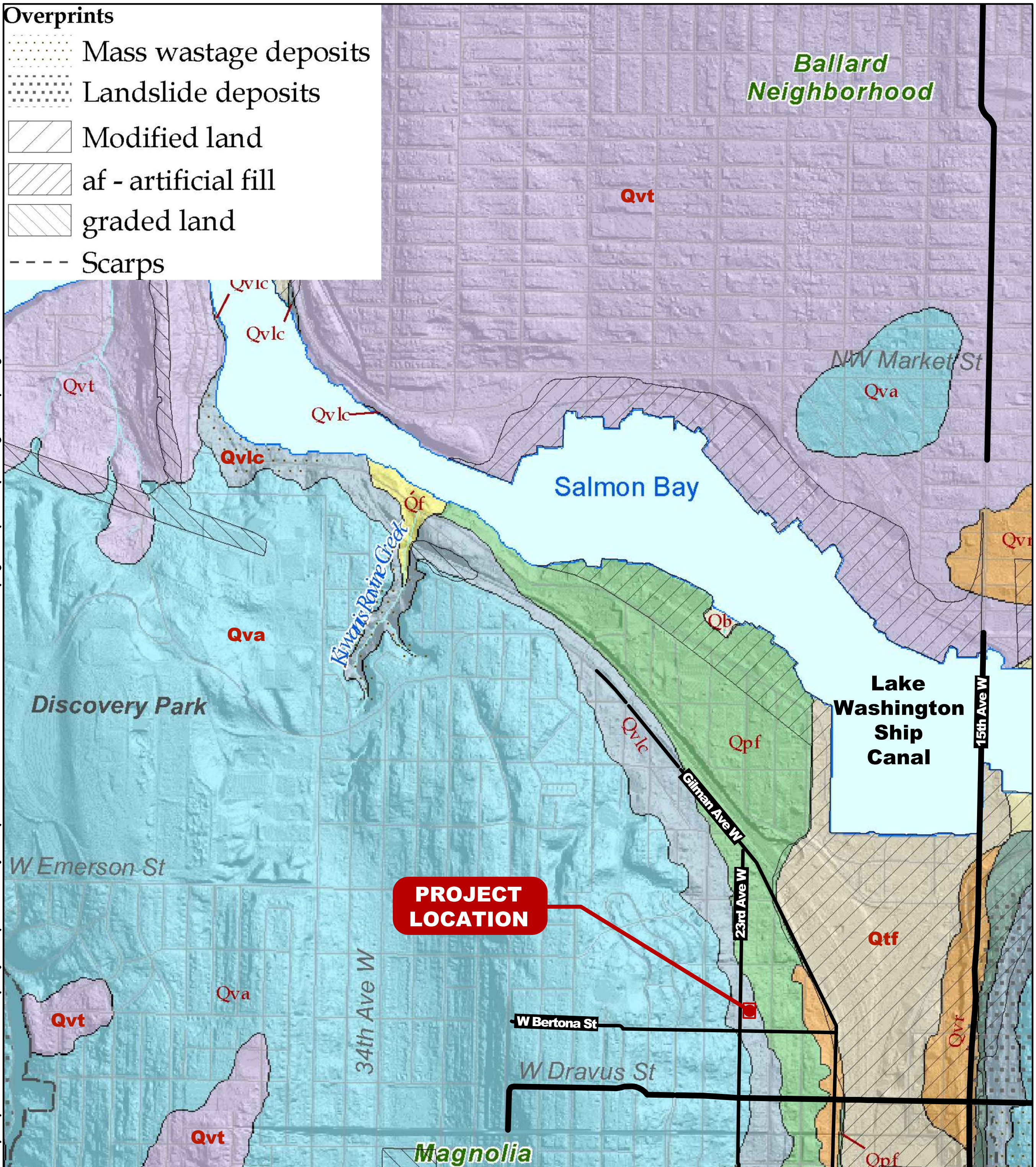
DATE: 8/16/2018

CHECKED BY: HMX

DRAWN BY: NTW



GEOLOGIC UNITS

Qb	Beach deposits
Qtf	Tideflat deposits
Qf	Fan deposits
Qvr	Vashon recessional outwash deposits
Qvt	Vashon glacial till
Qva	Vashon advance outwash deposits
Qvlc	Vashon lawton clay deposits
Qpf	Pre-Fraser glaciation age deposits

NOTES

1. Derived from the Geologic Map of Seattle (Troost, et. al. 2005)
2. Detailed descriptions of the geologic units can be found in the text of the report.
3. Only the applicable geologic units are listed.

750' 0 750' 1500'

Approximate Scale: 1" = 1500'

PanGEO INCORPORATED

Proposed Development
3410 - 3420 23rd Avenue W
Seattle, Washington

GEOLOGIC MAP

PROJECT NO. 18-243

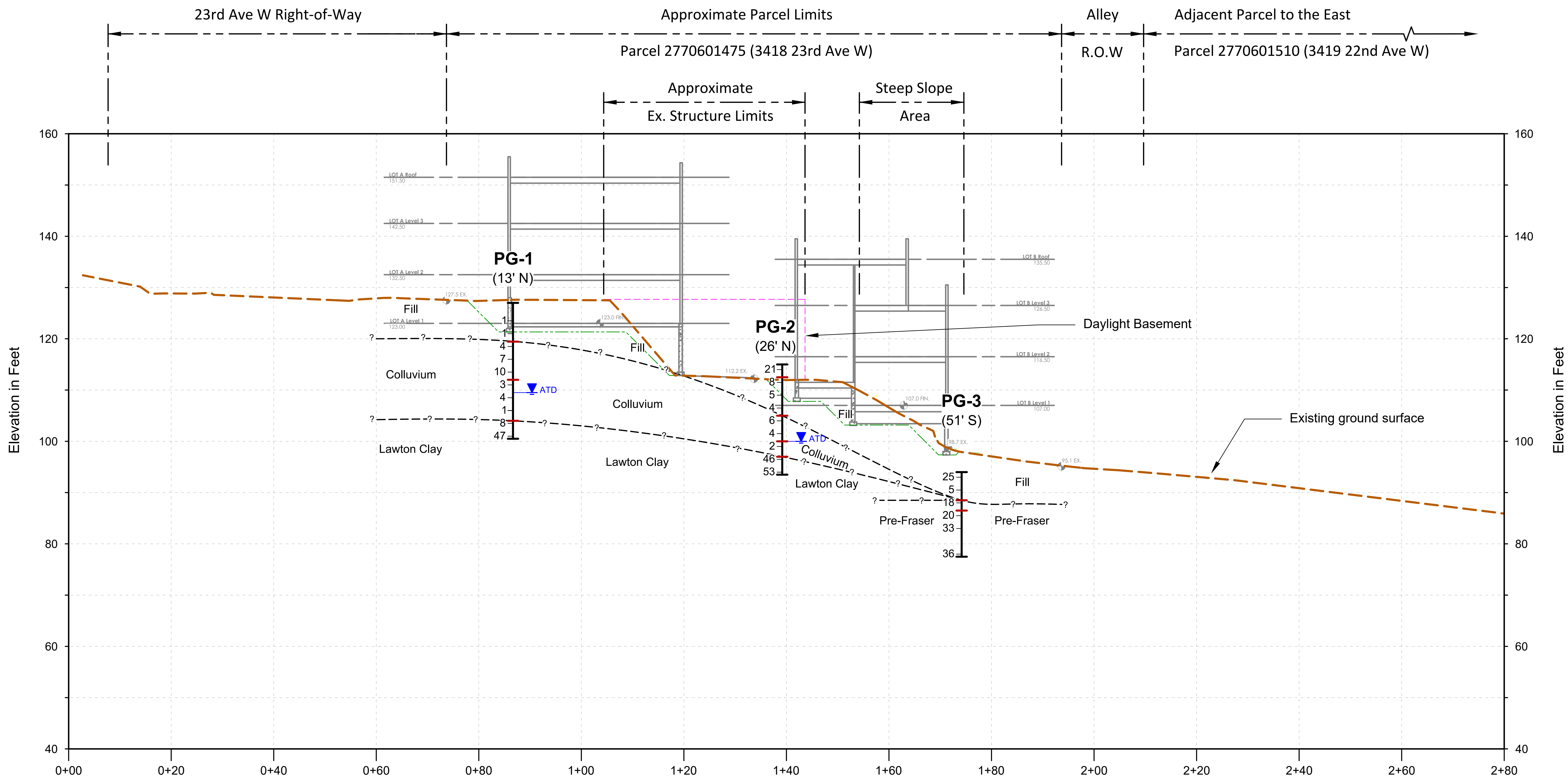
FIGURE NO. 3

Z:\Projects\2018 Projects\18-243 3410-3420 23rd Ave W\Figures\CADD\Geologic Map.dwg

DATE: 8/17/2018

CHECKED BY: HMX

DRAWN BY: NTW



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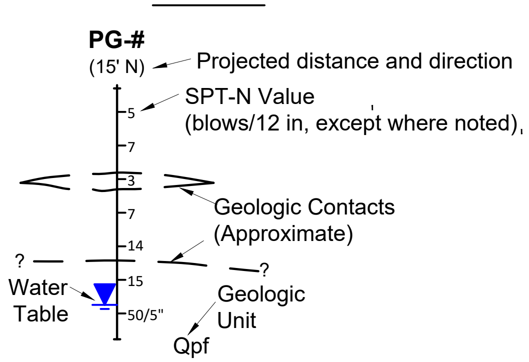
DATE: 9/11/2018

CHECKED BY: HMX

DRAWN BY: NTW

LEGEND



TEST BORING

GEOLOGIC UNIT

See text of report for soil unit descriptions.

Hf	Fill
Qmw	Colluvium deposits
Qvlc	Lawton Clay member of Vashon Drift
Qpf	Pre-Fraser glaciation age deposits

NOTES

1. Location of borings are approximate and based on the relative locations of known site features.
2. Existing ground surface based Topographic and Boundary Survey dated 6/27/2018 by Terrane.
3. Geologic contacts between borings are inferred
4. Vertical Datum: NAVD '88

H. Scale: 1" = 20'

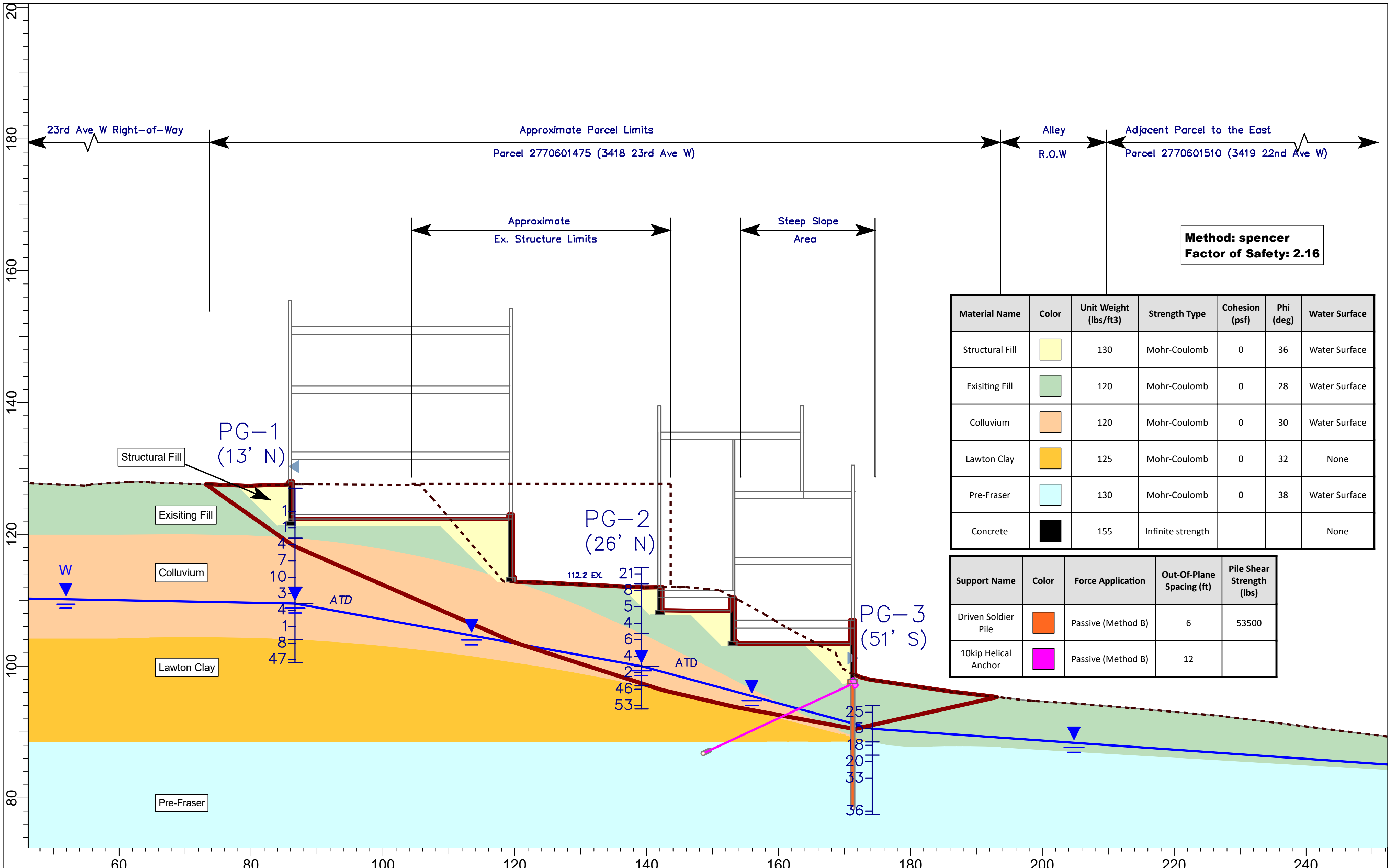
V. Scale: 1" = 20'

PanGEO INCORPORATED

Proposed Development
3410 - 3420 23rd Avenue W
Seattle, Washington

GENERALIZED
SUBSURFACE PROFILE
CROSS-SECTION A

PROJECT NO.	FIGURE NO.
18-243	4



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Structural Fill		130	Mohr-Coulomb	0	36	Water Surface
Exisiting Fill		120	Mohr-Coulomb	0	28	Water Surface
Colluvium		120	Mohr-Coulomb	0	30	Water Surface
Lawton Clay		125	Mohr-Coulomb	0	32	None
Pre-Fraser		130	Mohr-Coulomb	0	38	Water Surface
Concrete		155	Infinite strength			None

Support Name	Color	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)
Driven Soldier Pile		Passive (Method B)	6	53500
10kip Helical Anchor		Passive (Method B)	12	

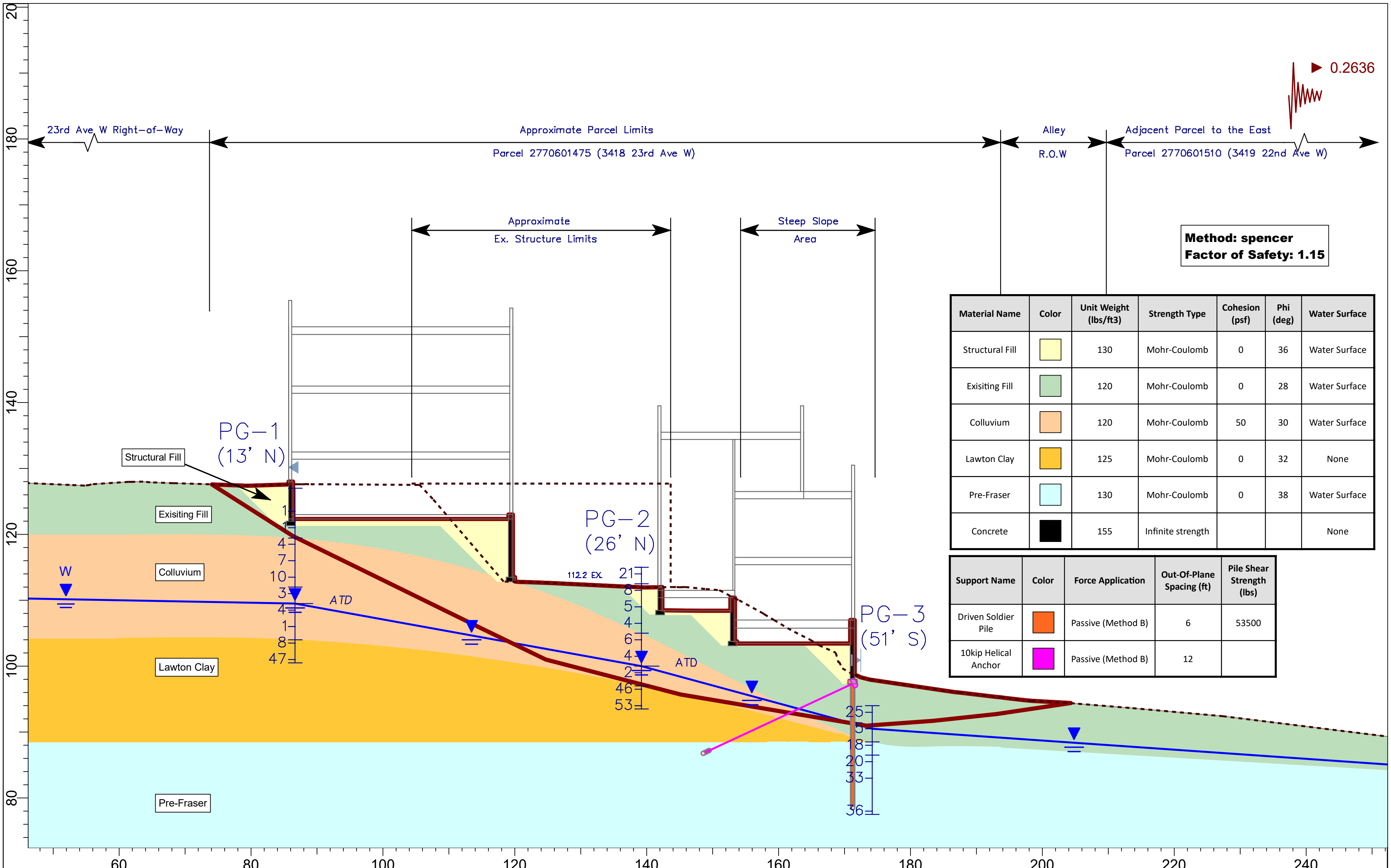
PanGEO INCORPORATED

Proposed Development
3410 - 3420 23rd Avenue W
Seattle, Washington

Static Global Stability Analysis

Proposed Condition - Entire Site

Project No.	Figure No.
18-243	**5**



Method: spencer
Factor of Safety: 1.15

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Structural Fill		130	Mohr-Coulomb	0	36	Water Surface
Exisiting Fill		120	Mohr-Coulomb	0	28	Water Surface
Colluvium		120	Mohr-Coulomb	50	30	Water Surface
Lawton Clay		125	Mohr-Coulomb	0	32	None
Pre-Fraser		130	Mohr-Coulomb	0	38	Water Surface
Concrete		155	Infinite strength			None

Support Name	Color	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)
Driven Soldier Pile		Passive (Method B)	6	53500
10kip Helical Anchor		Passive (Method B)	12	

PanGEO INCORPORATED

SLIDEINTERPRET 6.039

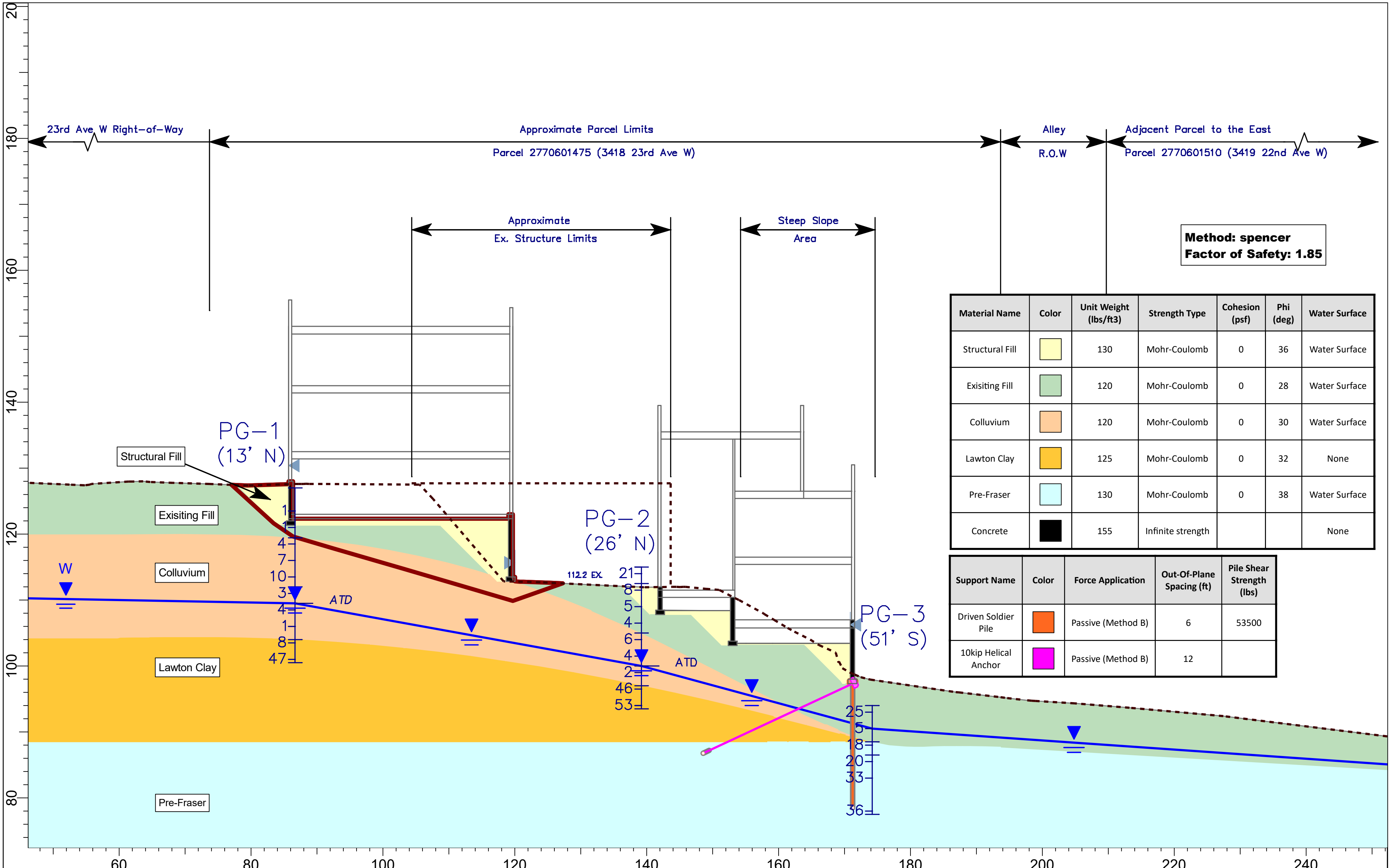
Proposed Development
3410 - 3420 23rd Avenue W
Seattle, Washington

Pseudo-Static (Seismic) Global Stability Analysis

Proposed Condition - Entire Site

Project No. **18-243**

Figure No. **6**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Structural Fill		130	Mohr-Coulomb	0	36	Water Surface
Exisiting Fill		120	Mohr-Coulomb	0	28	Water Surface
Colluvium		120	Mohr-Coulomb	0	30	Water Surface
Lawton Clay		125	Mohr-Coulomb	0	32	None
Pre-Fraser		130	Mohr-Coulomb	0	38	Water Surface
Concrete		155	Infinite strength			None

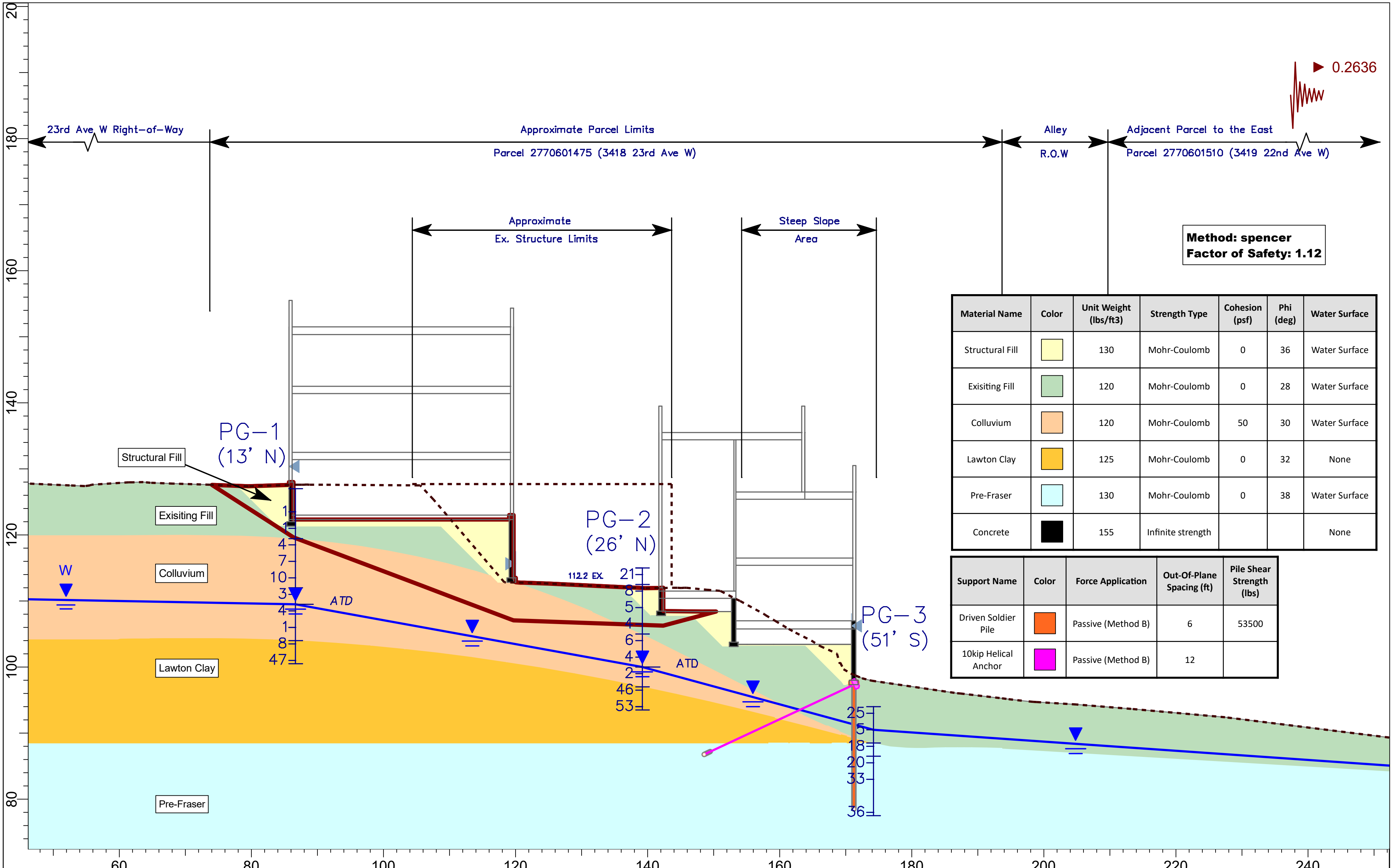
Support Name	Color	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)
Driven Soldier Pile		Passive (Method B)	6	53500
10kip Helical Anchor		Passive (Method B)	12	

PanGEO INCORPORATED

Proposed Development
3410 - 3420 23rd Avenue W
Seattle, Washington

Static Global Stability Analysis
Proposed Condition - Upper Half

Project No.	Figure No.
18-243	7



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Structural Fill		130	Mohr-Coulomb	0	36	Water Surface
Exisiting Fill		120	Mohr-Coulomb	0	28	Water Surface
Colluvium		120	Mohr-Coulomb	50	30	Water Surface
Lawton Clay		125	Mohr-Coulomb	0	32	None
Pre-Fraser		130	Mohr-Coulomb	0	38	Water Surface
Concrete		155	Infinite strength			None

Support Name	Color	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)
Driven Soldier Pile		Passive (Method B)	6	53500
10kip Helical Anchor		Passive (Method B)	12	

PanGEO INCORPORATED

SLIDEINTERPRET 6.039

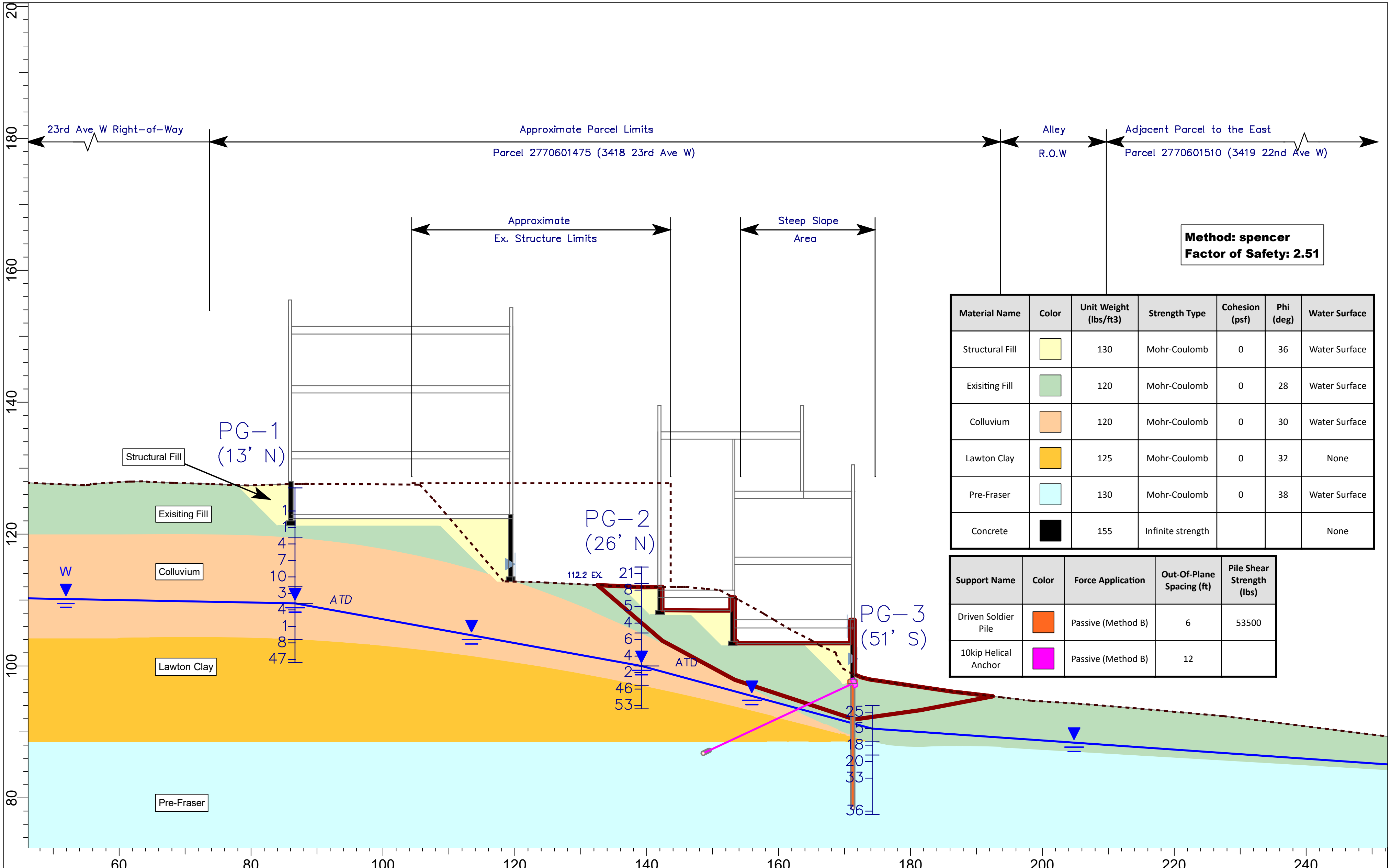
Proposed Development
3410 - 3420 23rd Avenue W
Seattle, Washington

Pseudo-Static (Seismic) Global Stability Analysis

Proposed Condition - Upper Half

Project No. **18-243**

Figure No. **8**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Structural Fill		130	Mohr-Coulomb	0	36	Water Surface
Exisiting Fill		120	Mohr-Coulomb	0	28	Water Surface
Colluvium		120	Mohr-Coulomb	0	30	Water Surface
Lawton Clay		125	Mohr-Coulomb	0	32	None
Pre-Fraser		130	Mohr-Coulomb	0	38	Water Surface
Concrete		155	Infinite strength			None

Support Name	Color	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)
Driven Soldier Pile		Passive (Method B)	6	53500
10kip Helical Anchor		Passive (Method B)	12	

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SLIDEINTERPRET 6.039

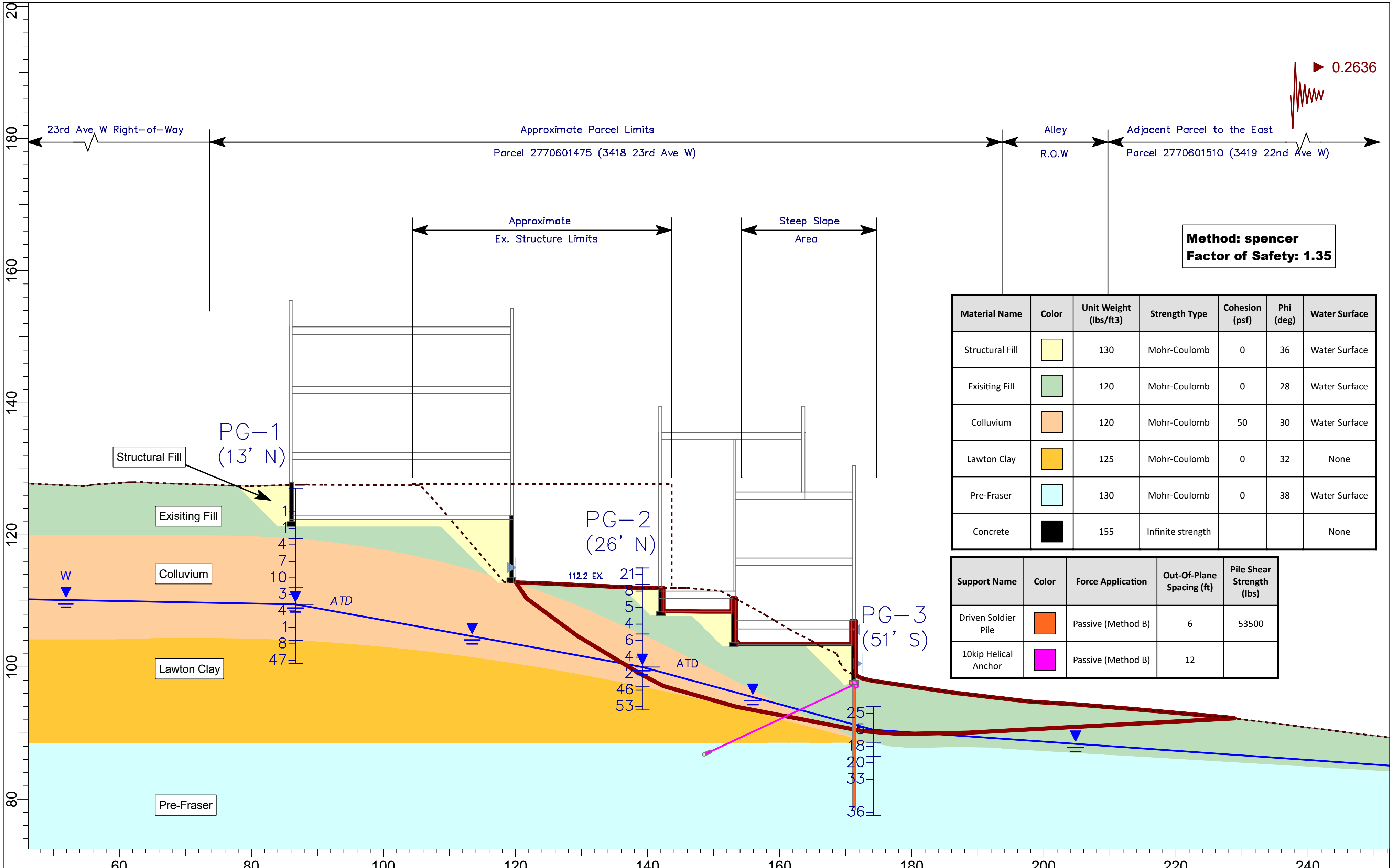
Proposed Development
3410 - 3420 23rd Avenue W
Seattle, Washington

Static Global Stability Analysis

Proposed Condition - Lower Half

Project No. **18-243**

Figure No. **9**



Method: spencer
Factor of Safety: 1.35

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Structural Fill		130	Mohr-Coulomb	0	36	Water Surface
Exisiting Fill		120	Mohr-Coulomb	0	28	Water Surface
Colluvium		120	Mohr-Coulomb	50	30	Water Surface
Lawton Clay		125	Mohr-Coulomb	0	32	None
Pre-Fraser		130	Mohr-Coulomb	0	38	Water Surface
Concrete		155	Infinite strength			None

Support Name	Color	Force Application	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)
Driven Soldier Pile		Passive (Method B)	6	53500
10kip Helical Anchor		Passive (Method B)	12	

PanGEO INCORPORATED

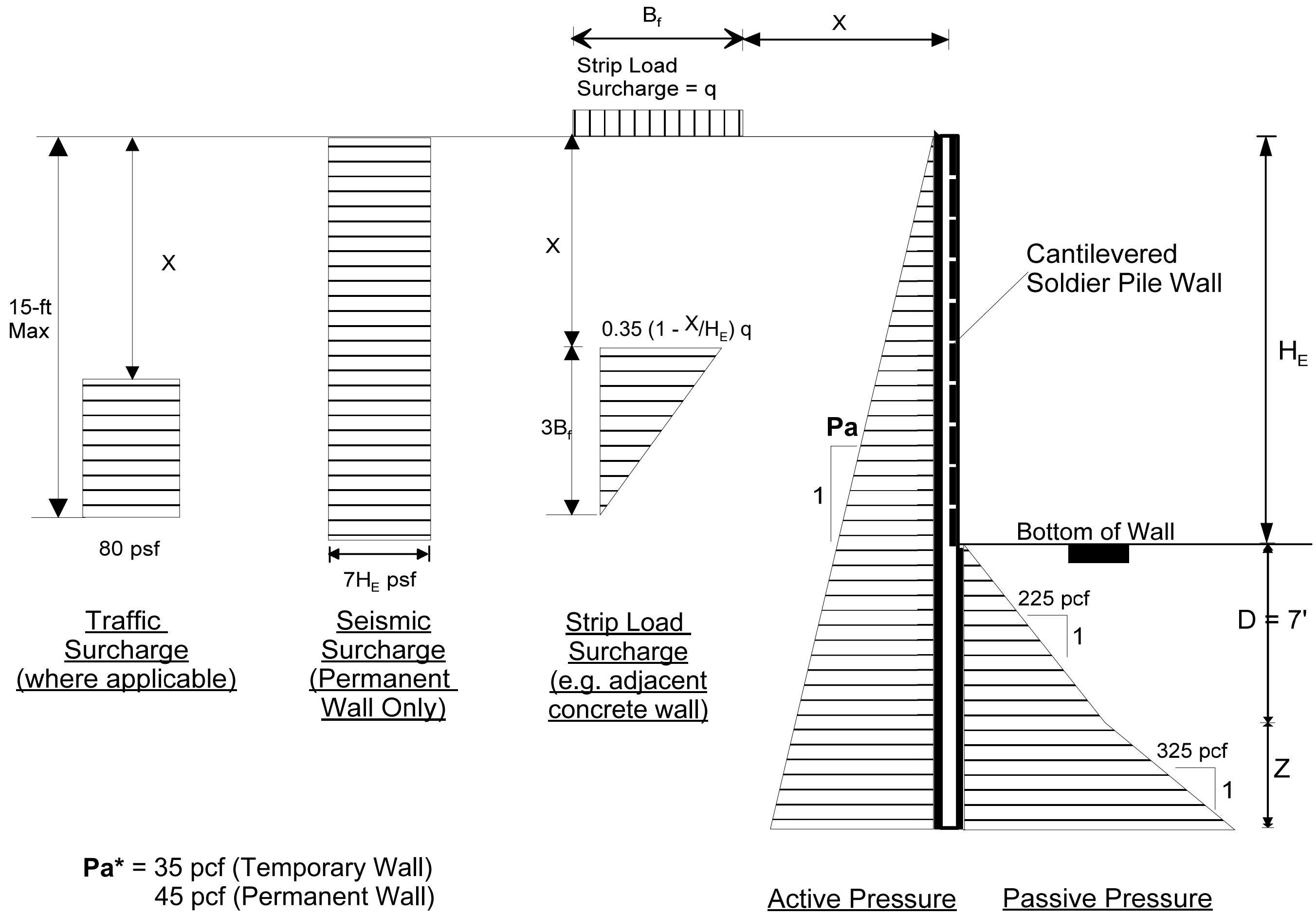
SLIDEINTERPRET 6.039

Proposed Development
3410 - 3420 23rd Avenue W
Seattle, Washington

Pseudo-Static (Seismic) Global Stability Analysis

Proposed Condition - Lower Half

Project No.	Figure No.
18-243	10



LEGEND

H_E = Height of Excavation (ft)

Z = Embedment Depth (min. 10 ft)

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 earth pressure value. No factor of safety has been applied to the recommended active or surcharge earth pressure values.
3. Active and surcharge pressures should be applied over the full width of the pile spacing above the base of the excavation, and over one pile diameter below the base of the excavation.
4. Seismic pressures should be applied over the full width of the pile spacing for permanent walls.
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.

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Proposed Development
3410-3420 23rd Ave W
Seattle, Washington

DESIGN LATERAL EARTH PRESSURES
CANTILEVERED SOLDIER PILE WALL
ONE ROW HELICAL ANCHOR

Project No. **18-243** Figure No. **11**

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

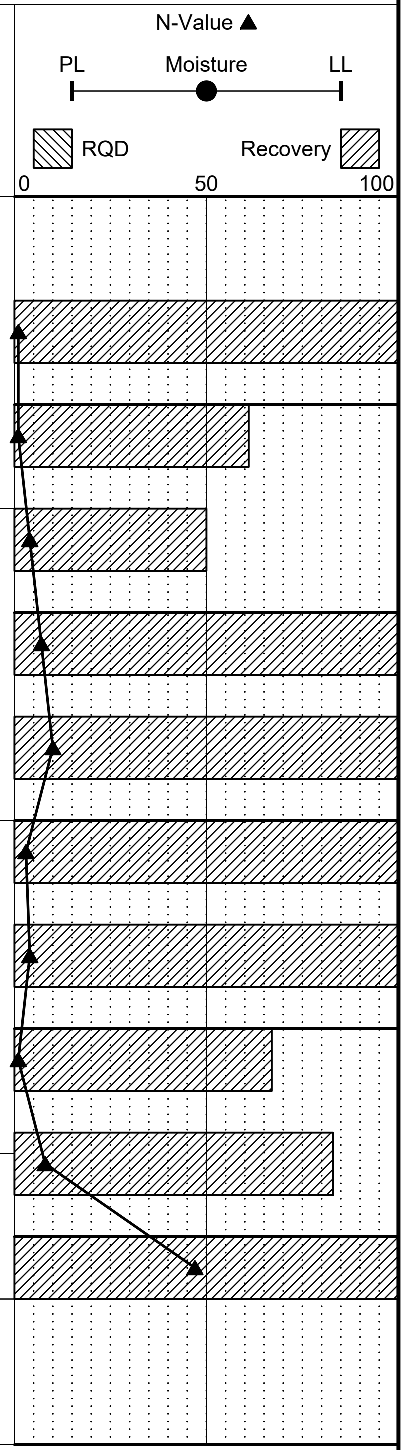
PanGEO INCORPORATED
Phone: 206.262.0370

Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	3410-3420 23rd Ave W	Surface Elevation:	127.0ft
Job Number:	18-243	Top of Casing Elev.:	N/A
Location:	3410-3420 23rd Ave W, Seattle, WA	Drilling Method:	HSA
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						Very loose, moist, light brown, poorly graded, medium SAND. **(Fill).** Top 2.5 feet hand dug.
	S-1		1 1 0			Occasional minor iron oxide staining.
5	S-2		1 0 1			
	S-3		1 2 2			Soft, moist, gray SILT with sand; occasional wood fragments. **(Colluvium).**
10	S-4		1 3 4			Becomes medium stiff.
	S-5		7 5 5			Layer of organic silt: 3 inches thick.
15	S-6		1 1 2			Very loose, very moist, gray, silty, medium SAND; pockets of clean sand. **(Colluvium).**
	S-7		1 1 3			Becomes wet.
20	S-8		0 1 0			
	S-9		1 2 6			Stiff, moist, light brown SILT; subhorizontally laminated. **(Lawton Clay).**
25	S-10		10 17 30			Becomes hard and gray.
						Boring terminated at 26.5 feet below ground surface. Perched groundwater was encountered from 17.5 to 23 feet during drilling.
30						



Completion Depth:	26.5ft
Date Borehole Started:	8/3/18
Date Borehole Completed:	8/3/18
Logged By:	J. Manke
Drilling Company:	CN Drilling

Remarks: Borings drilled using an acker hand portable drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Surface elevation estimated based on the City of Seattle GIS map.

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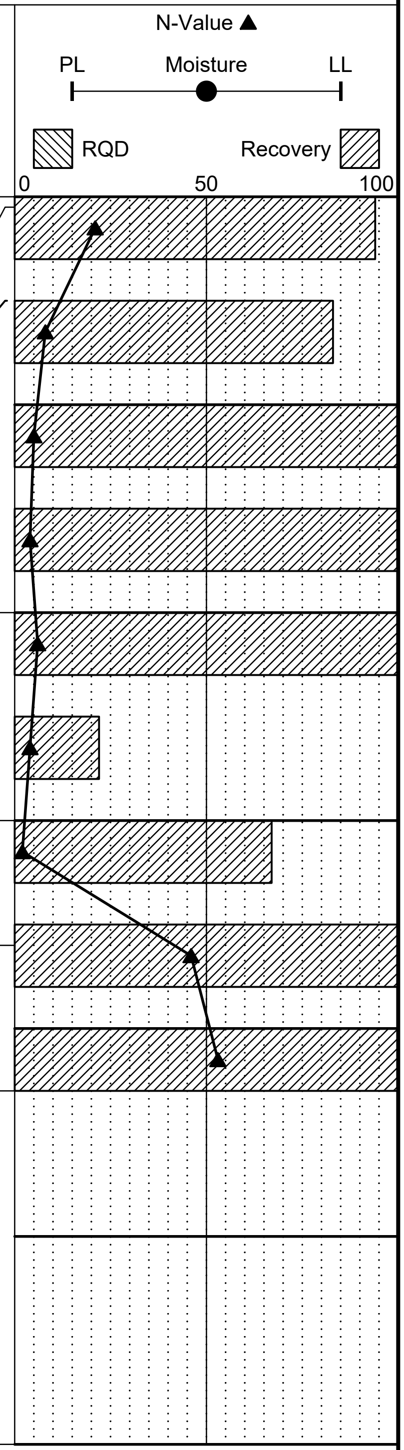
LOG OF TEST BORING PG-1

Figure A-2

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

Project:	3410-3420 23rd Ave W	Surface Elevation:	115.0ft
Job Number:	18-243	Top of Casing Elev.:	N/A
Location:	3410-3420 23rd Ave W, Seattle, WA	Drilling Method:	HSA
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0	S-1		8 10 11			Grass and topsoil: 3 inches thick. Medium dense, moist, brown, silty SAND with gravel. **(Fill).**
	S-2		3 4 4			Water added at 1 foot. Loose, moist, gray-brown, silty fine SAND; minor iron oxide staining. **(Fill).**
5	S-3		1 2 3			
	S-4		1 1 3			
10	S-5		5 3 3			Soft, moist, gray SILT with sand; moderately oxidized zone, distupted texture. **(Colluvium).**
	S-6		1 2 2			Wood in sampler tip.
15	S-7		1 1 1			Very loose, wet, gray, poorly graded, medium SAND. **(Colluvium).**
	S-8		10 17 29			Hard, moist, light brown SILT with sand; subhorizontally laminated. **(Lawton Clay).**
20	S-9		11 24 29			Becomes gray.
						Boring terminated at 21.5 feet below ground surface. Perched groundwater was encountered from about 15 to 18 feet during drilling.
25						
30						



Completion Depth:	21.5ft
Date Borehole Started:	8/3/18
Date Borehole Completed:	8/3/18
Logged By:	J. Manke
Drilling Company:	CN Drilling

Remarks: Borings drilled using an acker hand portable drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Surface elevation estimated based on the City of Seattle GIS map.

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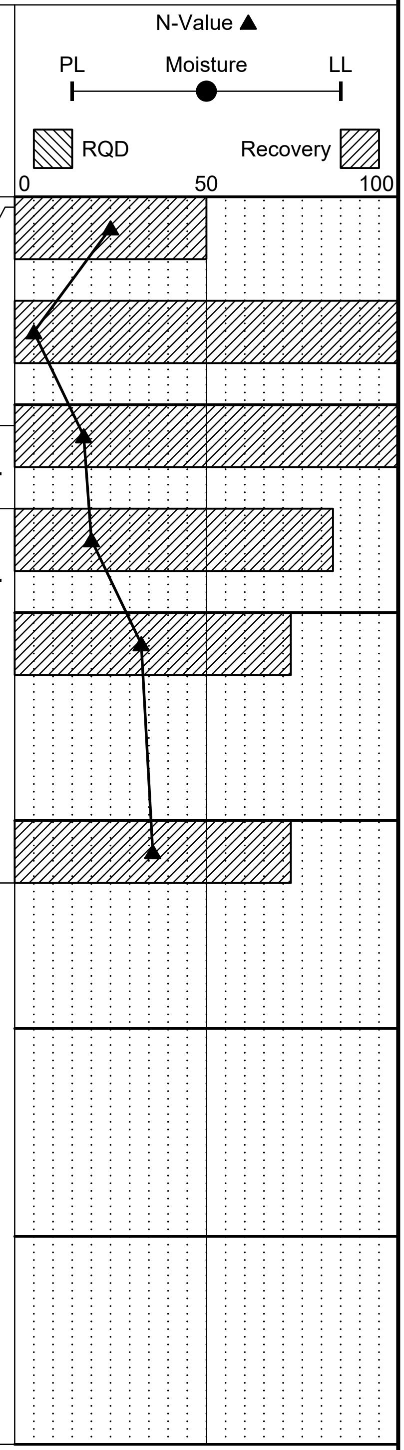
LOG OF TEST BORING PG-2

Figure A-3

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

Project:	3410-3420 23rd Ave W	Surface Elevation:	94.0ft
Job Number:	18-243	Top of Casing Elev.:	N/A
Location:	3410-3420 23rd Ave W, Seattle, WA	Drilling Method:	HSA
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0	S-1	SPT	9, 10, 15			Grass and topsoil: 3 inches thick. Medium dense, dry, light brown, silty SAND with gravel. **(Fill).**
	S-2	SPT	1, 2, 3			Becomes loose, moist, and dark brown, with occasional organics and minor iron oxide staining.
5	S-3	SPT	5, 8, 10			Medium dense, moist, brown, silty SAND with trace gravel. **(Pre-Fraser Deposits).**
	S-4	SPT	6, 9, 11			Medium dense, moist, gray, poorly graded, medium SAND; occasional layers of iron oxide staining. **(Pre-Fraser Deposits).**
10	S-5	SPT	7, 15, 18			Becomes dense.
15	S-6	SPT	8, 14, 22			
						Boring terminated at 16.5 feet below ground surface. Groundwater was not observed during drilling.
20						
25						
30						



Completion Depth:	16.5ft
Date Borehole Started:	8/3/18
Date Borehole Completed:	8/3/18
Logged By:	J. Manke
Drilling Company:	CN Drilling

Remarks: Borings drilled using an acker hand portable drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Surface elevation estimated based on the City of Seattle GIS map.

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LOG OF TEST BORING PG-3

Figure A-4

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