GEOTECHNICAL REPORT

PROPOSED APARTMENTS
300 WEST REPUBLICAN
SEATTLE, WASHINGTON

Project No. 19-061.200
March 2023

Prepared for:

kōz Development

PanGEO INCORPORATED
3213 Eastlake Avenue East
Seattle, Washington 98102
Tel: 206.262.0370

Geotechnical & Earthquake Engineering Consultants

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March 6, 2023
PanGEO Project No. 19-061.200

Mr. Jason Anderson
kōz Development
1830 Bickford Avenue, Suite 201
Snohomish, Washington 98290

Subject: **Geotechnical Engineering Report**
Proposed Apartments
300 West Republican Street, Seattle, Washington

Dear Mr. Anderson:

As requested, PanGEO, Inc. is pleased to present this geotechnical report to assist the project team with the proposed development at 300 West Republican Street in Seattle, Washington. This report documents the subsurface conditions at the site and our geotechnical recommendations for the proposed project.

In summary, the site is generally underlain by about 4 to 8½ feet of loose fill overlying medium dense to very dense silty sand and very stiff to hard silt/clay. Based on the soil conditions anticipated at the plan foundation level, it is our opinion that conventional footings may be used to support the proposed building. The temporary excavation for the below-grade wall construction may be accomplished with a combination of unsupported cuts, where there is sufficient room and temporary cantilevered soldier pile walls where a zero-lot line excavation is planned.

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

Sincerely,

Scott D. Dinkelman, LEG
Principal Engineering Geologist

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

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SEATTLE, WASHINGTON

1.0 GENERAL

This report presents the results of a geotechnical engineering study that was undertaken to support the design and construction of the proposed apartments at 300 West Republican Street in Seattle, Washington. This study was conducted in general accordance with our mutually agreed scope of services outlined in our proposal dated January 26, 2023. Our scope of services included reviewing readily available geologic and geotechnical data, reviewing the logs of four borings previously drilled at the site, performing laboratory tests and engineering analyses, and developing the geotechnical design recommendations presented in this report.

2.0 SITE AND PROJECT DESCRIPTION

The subject site is located at 300 West Republican Street in the Lower Queen Anne neighborhood of Seattle, Washington (see Figure 1, Vicinity Map). The subject property consists of two adjoining parcels with a combined area of about 14,400 square feet. The combined site is rectangular in shape and is bordered by 3rd Avenue West to the east, West Republican Street to the south, an alley to the west, and a surface parking lot to the north. The layout of the site is shown in the attached Figure 2, Site and Exploration Plan. The site is currently occupied by a two-story building and one one-story building. An approximately three- to seven-foot-tall brick retaining wall is located along the south portion of the east property line.

Plates 1 and 2 on the next page show the general site conditions.

The site grade slopes down from northeast to southwest with an average gradient of about 6 percent and a total vertical relief of about 8 feet. The site is not mapped as containing any Seattle Department of Construction and Inspection defined environmentally critical areas.

We understand it is planned to develop the site with an 8-story apartment building. The proposed building will have a finished floor elevation of 89 feet and a construction subgrade elevation of about 87½ feet. The building will be benched into the sloping grade with a cut of up to about 9½ feet deep along the north side of the site that daylights to the south. The excavation will be accomplished using a combination of open cuts and temporary shoring to support a vertical excavation along the north property line. The maximum shoring height will be about 9½ feet.

Plate 1: *View of the existing building and parking lot on the east parcel. Looking northwest from the corner of 3rd Ave West and West Republican Street.*

Plate 2. *View of the existing building on the west parcel. Looking approximately northeast from the corner of West Republican Street and the alley along the west property line.*

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

3.0 SUBSURFACE EXPLORATIONS

3.1 Field Exploration

Four test borings (PG-1 through PG-4) were drilled at the site on April 4, 2019, using a track mounted drill rig. The borings were drilled to maximum depths of about 31½ to 41½ feet below existing grade. The approximate boring locations were located in the field using a handheld GPS and by measuring from property corners and site features, and are shown on Figure 2, Site and Exploration Plan. Two borings (PG-1 and PG-3) were converted to groundwater monitoring wells.

The drill rig was equipped with 8-inch outside diameter hollow stem augers. Soil samples were obtained from the borings at 2½-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, to assist in sampling, and to describe and document the soil samples obtained from the borings. The soils were logged in general accordance with the system summarized on Figure A-1, Terms and Symbols for Boring and Test Pit Logs. Summary test boring logs are included as Appendix A.

3.2 Laboratory Testing

Grain size distribution, natural moisture contents, and Atterberg Limits tests were conducted on selected representative soil samples obtained from the borings. The test results from the moisture content tests are indicated at the appropriate depths on the boring logs. The grain size distribution and Atterberg Limits test results are included in Figures B-1 through B-3 in Appendix B.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 SITE GEOLOGY

Based on review of *The Geologic Map of Seattle – a Progress Report* (Troost, et al., 2005), the site is underlain by Vashon till (Qvt), with Lawton clay deposits (Qvlc) mapped one block to the north and Ice-contact deposits (Qvi) mapped in the site vicinity. A brief description of each mapped soil unit, from youngest to oldest, follows:

> Ice-contact deposits (Qvi) typically consist of loose to very dense, intercalated, irregularly shaped bodies of till and outwash. Ice-contact deposits may or may not be glacially overridden and can be loose to dense.
>
> Vashon till (Qvt) consists of an unsorted mixture of clay, silt, sand and gravel that is directly deposited below a glacier. This soil unit has been glacially overridden; as such it is typically dense to very dense.
>
> Subglacial meltout till (Qvtm) is a sub-unit of Vashon till, described by Troost et al. as compact diamict with large, often tabular, sand and gravel bodies that may exceed 50% of the volume of the deposit. Subglacial meltout till is locally gradational with Vashon till and Vashon advance outwash, and is locally identified as “sandy till”.
>
> Lawton clay (Qvlc) is described 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.

4.2 ORIGINAL STREET GRADING PROFILE

Based on our review of the historic street grading profiles obtained from the City of Seattle archives, original grades along the south property line (West Republican Street) varied from about three feet higher than current grades near the southwest property corner to three feet lower than current grades at the southeast property corner. Original grades along the east property line (3rd Avenue West) were originally up to nine feet lower than current grades, which is consistent with the existing retaining wall height along the east property line, that was likely constructed to retain the roadway fill. 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.

4.3 Soil Conditions

The following is a generalized description of the soils encountered in the borings. For a detailed description of the subsurface conditions encountered at each exploration location, please refer to our boring logs provided in Appendix A. The stratigraphic contacts indicated on the boring logs represent the approximate depth to boundaries between soil units. Actual transitions between soil units may be more gradual or occur at different elevations. The descriptions of groundwater conditions and depths are likewise approximate.

> ***UNIT 1: Fill –*** Below the concrete and asphalt surfaces, fill consisting of loose to dense, poorly graded gravelly and silty sand with organics was encountered to about 5 feet in borings PG-1 and PG-2, to about 4 feet in boring PG-3, and to about 8½ feet in PG-4. We interpret this unit as fill based on its loose condition and disturbed appearance.
>
> ***UNIT 2***: ***Vashon Ice-Contact Deposits –*** Below Unit 1, borings PG-2 and PG-3 encountered very stiff to hard silt with iron oxide staining extending to about 6 feet depth at PG-2 and to about 10 feet depth at PG-3. We interpret this unit as ice-contact deposits. This unit was not encountered in PG-1 or PG-4.
>
> ***UNIT 3***: ***Vashon Subglacial Meltout Till –*** Below Unit 1, borings PG-1 and PG-4 encountered medium dense sand with gravel that extended to about 7½ feet at PG-1 and to 13 feet below the surface at PG-4. We interpret this unit as subglacial meltout till. This unit was not encountered at PG-2 or PG-3.
>
> ***UNIT 4***: ***Vashon Till –*** Below Unit 2 at PG-2 and PG-3, and below Unit 3 at PG-1, the borings encountered very dense, grey-brown, silty fine sand with gravel. We interpret this unit as the mapped Vashon till deposits. This unit extended to about 29 feet below the surface at PG-1, to the bottom of exploration at 35½ feet at PG-2, and to about 20 feet at PG-3. This unit was not encountered at PG-4.
>
> ***UNIT 5***: ***Lawton Clay –*** Below Unit 4 at PG-1 and PG-3, and below Unit 3 at PG-4, hard silt and clay were encountered that extended to the bottom of the borings at about 31½ to 41½ feet below the surface in PG-1, PG-3, and PG-4. We interpret this unit as the Lawton Clay deposits mapped nearby.

Our subsurface descriptions are based on the conditions encountered at the time of our exploration. Soil conditions between our exploration locations may vary from those encountered. The nature and extent of variations between our exploratory locations may not become evident until construction. If variations do appear, PanGEO should be requested to reevaluate the recommendations in this report and to modify or verify them in writing prior to proceeding with earthwork and construction.

4.4 Groundwater

At the time of drilling, perched groundwater was observed from about 20 to 29 feet below grade in PG-1 and from about 8½ to 13 feet depth in PG-4, and minor perched groundwater seepage was encountered between about 10 to 11 feet at PG-2 and at about 15.25 to 20 feet at PG-3. After drilling, groundwater was measured at about 19 feet depth in the groundwater monitoring well PG-1 and no groundwater was measured in the groundwater monitoring well PG-3.

It should be noted that groundwater elevations and seepage rates are likely to vary depending on the season, local subsurface conditions, and other factors. Groundwater levels and seepage rates are normally highest during the winter and early spring.

5.0 GEOTECHNICAL RECOMMENDATIONS

5.1 Seismic Design Parameters

The seismic design should be performed using the 2018 edition 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). Based on the site soil conditions, it is our opinion that Site Class D should be used.

Liquefaction Potential: Liquefaction is a process that can occur when soils lose shear strength for short periods of time during a seismic event. Ground shaking of sufficient strength and duration results in the loss of grain-to-grain contact and an increase in pore water pressure, causing the soil to behave as a fluid. Soils with a potential for liquefaction are typically cohesionless, predominately silt and sand sized, must be loose, and be below the groundwater table. The site is underlain by medium dense to very dense silty sand without a defined groundwater table. Based on these conditions, in our opinion the liquefaction potential of the site is negligible and design considerations related to soil liquefaction are not necessary for this project.

5.2 BUILDING FOUNDATIONS

5.2.1 Foundation Type and Allowable Bearing Pressures

The excavation to achieve construction subgrade elevations will range from 9½ feet deep along the north side of the site that daylights to the south. At our boring locations, we encountered four to 8½ feet of fill and native loose soils below the site. If fill or loose soil is encountered at the construction subgrade elevation, the foundations should be extended through the fill/loose soil to bear on the underlying competent native soils or the fill/loose soil overexcavated and replaced with structural fill or lean-mix concrete/controlled density fill. If lean mix/CDF are used, we recommend the use of a 1½ sack mix minimum. We anticipate that foundation soil over-excavations of up to four feet below the construction subgrade elevation may be needed to reach the native bearing soils.

We recommend that an allowable soil bearing pressure of 6,000 psf be used for sizing footings bearing on the undisturbed, dense glacial soil and/or lean-mix concrete/CDF placed above the undisturbed, dense native glacial soils. An allowable soil bearing pressure of 4,000 psf may be used for footings bearing on structural fill (Seattle Type 2) placed over undisturbed, dense glacial soils. For allowable stress design, the recommended bearing pressure may be increased by one-third for transient loading, such as wind or seismic forces.

Continuous and isolated column footings should have minimum widths of 24 inches and 48 inches, respectively. 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.

5.2.2 Foundation Performance

Footings designed and constructed in accordance with the above recommendations should experience total settlement of about one inch or less, and differential settlement of less than ½ inch. Most of the anticipated settlement should occur during construction as dead loads are applied.

5.2.3 Lateral Resistance

Lateral loads on the structure may be resisted by passive earth pressure developed against the embedded portion of the foundation system and by frictional resistance between the bottom of the foundation and the supporting subgrade soils. Footings bearing on the medium dense to dense sandy native soil or structural fill may be designed using a frictional coefficient of 0.4 to evaluate sliding resistance developed between the concrete and the subgrade soil. Passive soil resistance may be calculated using an equivalent fluid weight of 350 pcf, assuming foundations are backfilled with structural fill. The above values include a factor of safety of 1.5. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches of soil should be neglected.

5.2.4 Footing Subgrade Preparation

Foundation subgrades should be carefully prepared. It should be noted that the site soil is moisture sensitive and can become disturbed or softened when exposed to moisture. As a result, it may be necessary to place a "rat slab" to protect the subgrade on an as-needed basis. The proper measures needed to protect the subgrade will be in part depend on the actual soil conditions exposed at the bottom of the excavation, and the contractor's construction methods and sequence.

The adequacy of footing subgrade should be verified by a representative of PanGEO, prior to placing forms or rebar. The footing subgrades at the basement level should be in a dense condition prior to concrete pour. Any loose/soft soils at the foundation level, if present, should be over-excavated and should be backfilled with lean-mix concrete/CDF. Footing excavations should be observed by PanGEO to confirm that the exposed footing subgrades are consistent with the expected conditions and adequate to support the design bearing pressure.

5.3 FLOORS SLABS

The floor slabs for the proposed building may be constructed using conventional concrete slab-on-grade floor construction. The floor slab should be supported on recompacted native soil or on structural fill. Any loose soil encountered at the slab subgrade should be either recompacted to a dense condition or over-excavated to expose dense native soils. Over-excavation should be replaced with compacted structural fill.

Interior concrete slab-on-grade floors should be underlain by a capillary break consisting of at least of 4 inches of pea gravel or compacted ¾-inch, clean crushed rock (less than 3 percent fines). The capillary break material should meet the gradational requirements provided in Table 2, below.

Table 2 – Capillary Break Gradation

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

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

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

5.4 RETAINING WALL DESIGN PARAMETERS

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

5.4.1 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 below-grade and building and basement walls, equivalent fluid pressures of 45 pcf should be used for level backfills behind the walls. For basement walls constructed against shoring walls, the permanent basement walls may be designed for an earth pressure based upon an equivalent fluid weight of 40 pcf. Walls with a maximum 2H:1V backslope should be designed for an active and at rest earth pressure of 45 and 55 pcf, respectively.

Permanent walls should be designed for an additional uniform lateral pressure of 9H 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.

5.4.2 Wall Surcharge

Surcharge loads, where present, should also be included in the design of retaining 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 the wall height.

5.4.3 Lateral Resistance

Lateral forces from 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 wall foundation. Passive resistance values may be determined using an equivalent fluid weight of 350 pcf. This value includes a factor of safety of 1.5, assuming the footing is backfilled with structural fill. A friction coefficient of 0.4 may be used to determine the frictional resistance at the base of the footings. The coefficient includes a factor of safety of 1.5.

5.4.4 Wall Drainage

Provisions for permanent control of subsurface water should be incorporated into the design and construction of the basement walls. We recommend that prefabricated drainage mats, such as Mirafi 6000 or equivalent, be installed behind the basement walls and the collected water should be directed inside the building beneath the floor slab and tight lined to an appropriate outlet.

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

5.4.5 Wall Backfill

The existing on-site soil contains excessive fines and is not suitable as wall backfill. Wall backfill should consist of imported, free draining granular soils, such as Seattle Mineral Aggregate Type 17 (2011 City of Seattle Standard Specifications, 9-03.12(2)).

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.

5.5 TEMPORARY EXCAVATION AND SHORING

Based on our current understanding of the project, temporary excavations for the below-grade portion of the building will extend up to about 9½ feet below the existing surface. We anticipate the excavations to encounter up to about 8½ feet of fill overlying dense to very dense sand and silty sand and very stiff to hard silt/clay. 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.

Based on anticipated excavations depths and the proposed building footprint, temporary shoring is planned along the north side of the property. Based on the soil conditions encountered at the site, it is our opinion that a shoring system consisting of soldier pile walls with timber lagging are appropriate to support the excavations.

5.5.1 Unsupported Open Cuts

Unsupported open cuts may be sloped as steep as 1H:1V (Horizontal:Vertical). Where groundwater seepage is encountered during construction, the temporary slopes may need to be flattened. The temporary excavations and cut slopes should be re-evaluated in the field during construction based on actual observed soil conditions, and may need to be flattered in the wet seasons and should be covered with plastic sheets. 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.

5.5.2 Soldier Pile Walls

A soldier pile wall consists of vertical steel beams, typically spaced from 6 to 8 feet apart along the proposed excavation wall, spanned by timber lagging. Prior to the start of excavation, the steel beams are installed into holes drilled to a design depth and then backfilled with lean mix or structural concrete. As the excavation proceeds downward and the steel piles are subsequently exposed, timber lagging is installed between the piles to support the soils between piles. Based on the proposed excavation depth, it is our opinion that either cantilevered soldier pile walls or soldier pile walls with one level of tiebacks may be used to support the excavations.

Design Lateral Pressures – In general, tiebacks are typically used for wall heights greater than about 12 feet to achieve a more economical design. We recommend that the lateral earth pressures depicted on Figure 3 be used for design of soldier pile walls and tiebacks.

Above the bottom of excavation, the recommended active earth and surcharge pressures should be applied over the full width of pile spacing. Below the bottom of excavation, the active pressures should be applied over one pile spacing, and the passive resistance should be applied over two times the pile diameter. We also recommend that the lagging be sized using an earth pressure equivalent to 50% of the design earth pressure shown in Figure 3 to account for the arching effects.

Soldier Pile Vertical Capacity – We recommend the vertical capacity of the soldier piles be determined 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 20 ksf.

Timber Lagging – Lagging design recommendations are presented on Figure 3.

Surcharge – Surcharge loads including but not limited to street traffic, adjacent buildings, and construction equipment should be considered in the shoring design. Surcharge from vehicle traffic along Streets and alleyway may be considered equivalent to 2 feet of soils and could be designed for an equivalent fluid weight of 80 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. Input from the contractor will be needed to determine if such design considerations will be necessary.

5.5.3 Groundwater and Caving Soil Conditions

The drilling of soldier piles is anticipated to encounter fill over dense to very dense silty sand and very stiff to hard clay. Obstructions, such as bricks and concrete, may exist in the fill. Cobbles and boulders may exist in the glacial soils. If the obstructions are located at depths that cannot be practically removed with a backhoe/excavator or coring, the soldier pile location may be revised as directed by the shoring designer.

Caving in fill and sandy soils could occur during drilling. As a result, the drilling contractor should be prepared to stabilize the holes by using temporary casings, hydrostatic pressures (i.e., flooding the hole), or drilling fluids, to prevent potential bottom heaving.

Perched groundwater/seepage will be present during soldier pile installation. If groundwater is present in the drilled holes, lean concrete or structural concrete backfill should be placed with tremie pipes from bottom up.

When placing timber lagging, the height of each lift may need to be limited to up to 5 feet deep. The actual allowable vertical cut for timber lagging placement should be determined in the field, based on the actual conditions observed.

Caving of the drilled holes may occur during drilling of tiebacks. If hole caving is observed during construction, temporary casing should be used during installation to keep the drilled holes open, and to minimize the risk of potential ground loss and off-site settlement.

It should also be noted that excessive disturbance of loose soils during drilling tiebacks could potentially lead to settlements. As a result, it is important to control the air flows if compressed air is used to flush the cuttings. Alternatively, the use of water may be incorporated into drilling/flushing of the tieback or soil nail holes to mitigate such risks. Where feasible, a soil plug may also be maintained at the head of the casings to reduce air flows beyond the casings and into the surrounding soils.

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 SDCI and SDOT should be performed. As a minimum, optical survey points should be established at:

- The top of every other soldier pile. These monitoring points should be monitored twice a week as required by the SDOT;
- The top of soil nail walls, space no more than 20 feet apart. These monitoring points should be monitored twice a week as required by the SDOT. The monitoring frequency may be reduced based on the monitoring results;
- The curbs and the centerlines of adjacent streets. These monitoring points should be spaced no more than 20 feet apart. These monitoring points do not need to be regularly surveyed unless the top of wall deflections exceed about ½ inch; and
- Adjacent buildings. Monitoring points should be established on the adjacent buildings.

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.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 SITE PREPARATION

Site preparation for the proposed project mainly includes removing the existing structures, site clearing, and excavations to the design subgrade. All debris resulted from site clearing should be hauled away from the site. The stripped surface materials should be properly disposed off-site or be “wasted” on site in non-structural landscaping areas.

Following site clearing and 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, may need to be over-excavated and replaced with compacted structural fill or lean-mix concrete. The need for over-excavation should be determined by PanGEO in the field.

6.2 Structural Fill 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 to 10 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.

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

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

6.3 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 soil is not suitable to be reused as structural fill. For planning purpose, structural fill should consist of imported, well-graded, granular material, such as City of Seattle Type 2 or 17, or WSDOT Gravel Borrow (WSDOT, 2023). 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.

6.4 Wet Weather Construction

In our opinion, the proposed site construction may be accomplished during wet weather (such as in winter) without adversely affecting the site stability, provided that surface water and erosion control will be properly managed. However, earthwork construction performed during the drier summer months likely will be more economical. Winter construction will require the implementation of best management erosion and sedimentation control practices to reduce the chance of off-site sediment transport. Some of the site soils contain a high percentage of fines and are moisture sensitive. Any footing subgrade soils that become softened either by disturbance or rainfall should be removed and replaced with structural fill, Controlled Density Fill (CDF), or lean-mix concrete. General recommendations relative to earthwork performed in wet conditions are presented below:

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

6.5 Erosion Considerations

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

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

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 the stability of open cut slopes;
- Observe shoring installation;
- Verify adequacy of foundation 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 CLOSURE

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

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

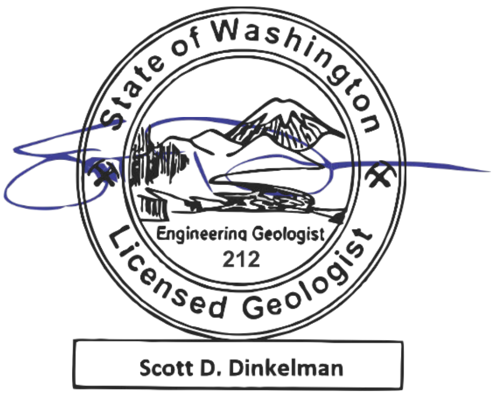
The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design. Additionally, the scope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

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

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

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

Sincerely,



Scott D. Dinkelman, LEG
Principal Engineering Geologist

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

9.0 REFERENCES

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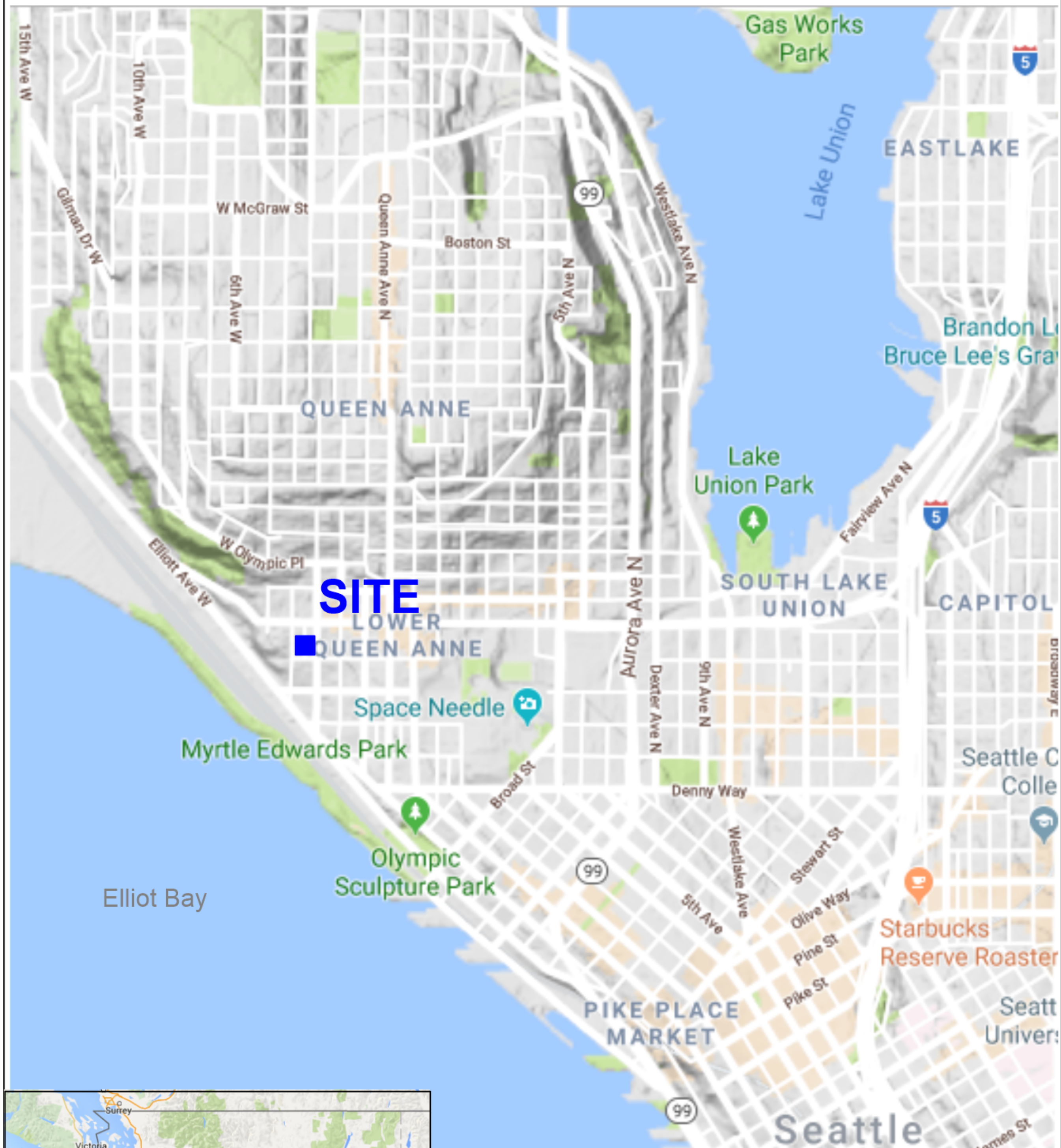
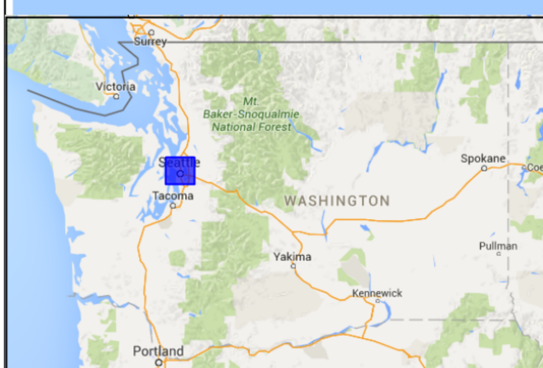


Base Map: Google Maps

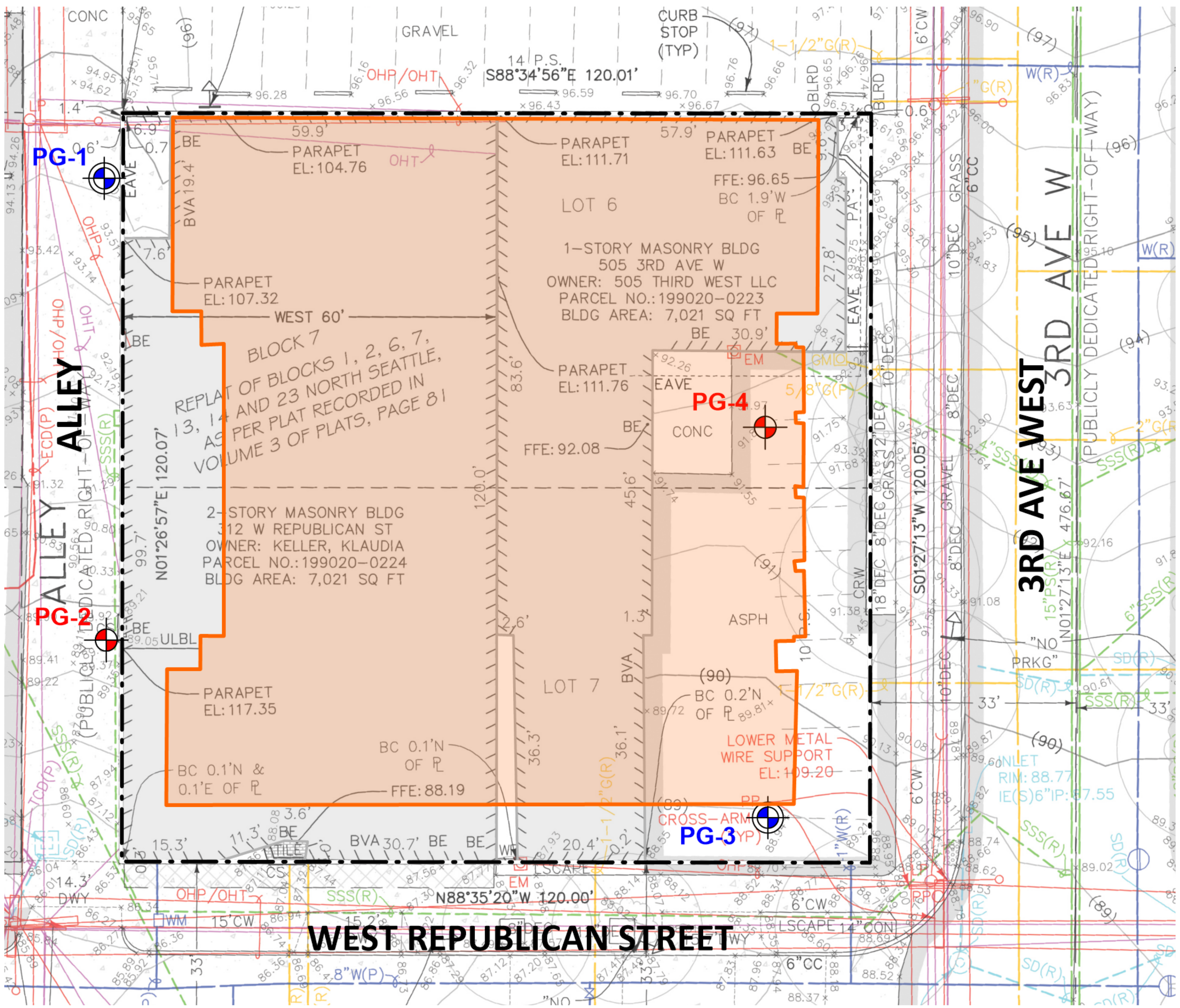


NORTH

Not-To-Scale

PanGEO INCORPORATED	Proposed Apartments 300 W Republican Seattle, Washington	VICINITY MAP Project No. **19-061.200** Figure No. **1**

Vicinity_Map.grf 3/5/23 (7:14:31) BW



LEGEND:

- Subject Site
- Existing Structures
- Proposed Apartment Building
- Approximate Boring Location, PanGEO, Inc., April 2019
- Approximate Boring Location with Standpipe Piezometer, PanGEO, Inc., April 2019

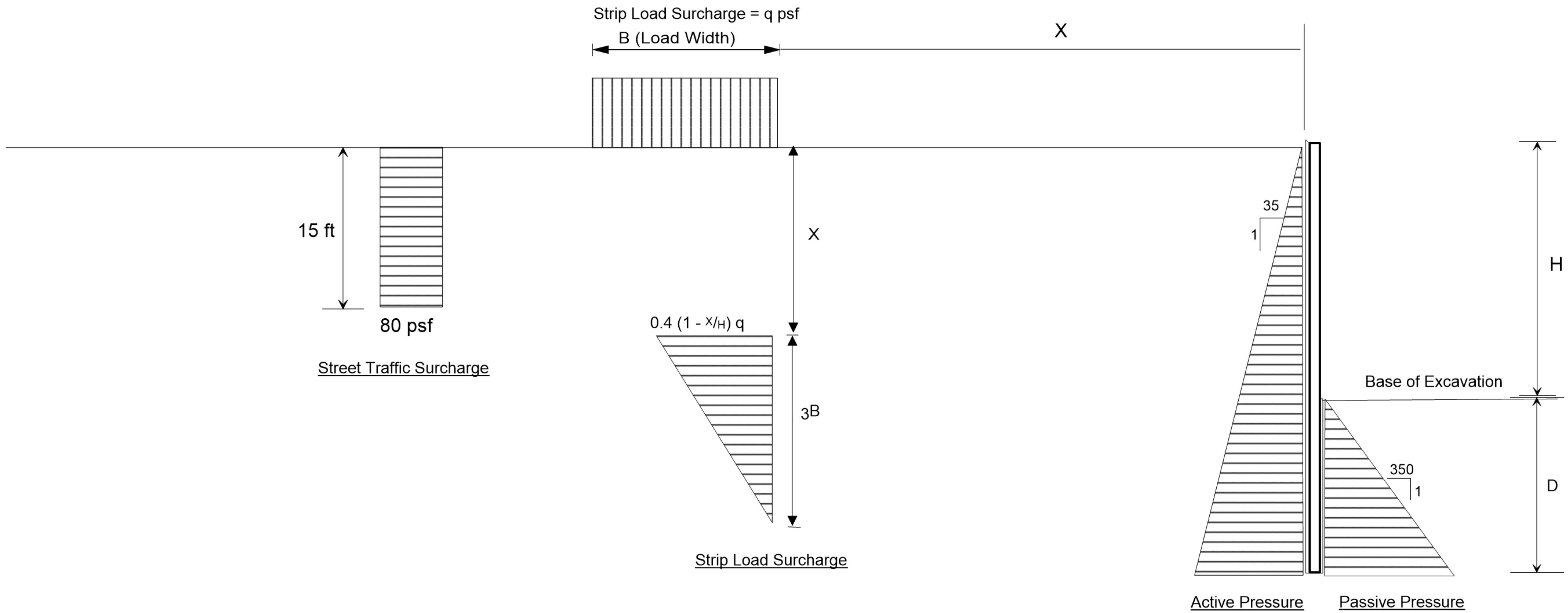
Approx. Scale (feet)

0 15 30

NORTH

PanGEO INCORPORATED	Proposed Apartments 300 W Republican Seattle, Washington	SITE AND EXPLORATION PLAN
		Project No. 19-061.200 — Figure No. 2

13-011_Fig 2 Site & Exploration Plan.grf 3/5/23 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, or pile center-to-center spacing, whicever is less.
6. For lagging design, the active pressures may be reduced by 50% for soldier piles spaced at 8' or less.
7. Refer to report text for additional discussions.

PanGEO INCORPORATED	Proposed Apartments 300 W Republican Street Seattle, Washington	DESIGN LATERAL PRESSURES SOLDIER PILE WALL CANTILEVERED
		Project No. 19-061.200 / Figure No. 3

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
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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Phone: 206.262.0370

Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	505 3rd Avenue W	Surface Elevation:	95.0ft
Job Number:	19-061	Top of Casing Elev.:	95.0ft
Location:	505 3rd Ave W & 312 W Republican St	Drilling Method:	HSA
Coordinates:	Northing: 47.62369, Easting: -122.36129	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						5-inches concrete alley.
						Dense (based on probing with 1/2" steel rod), grey, gravelly SAND; moist; poorly graded, angular gravel [Basecourse/Fill].
	S-1		1 1 2			
5	S-2		8 6 7			Very loose, mixed grey-brown, orange-brown, and dark brown, silty SAND trace gravel; moist; poorly graded, iron oxide staining, disturbed appearance [Fill].
	S-3		16 36 50/5			Medium dense, orange-brown, slightly gravelly, slightly silty to silty, fine to medium SAND; moist; well graded, iron oxide staining [Weathered Vashon Subglacial Meltout Till].
10	S-4		50/6			Very dense, grey-brown, silty fine SAND trace gravel; moist; gap-graded, partially cemented [Qvt - Vashon Till].
15	S-5		41 50/4			
						--water measured in piezometer at 18.9' below grade on 4/9/2019.
20	S-6		50/6			--becomes wet at about 20' during drilling.
						--becomes slightly gravelly, silty fine to medium SAND.
25	S-7		26 50/6			
						--driller notes change in drilling difficulty at about 29'.
30	S-8		6 12 20			Hard, grey, CLAY; moist; low plasticity [Qvlc - Lawton Clay].
35	S-9		10 16 23			
40	S-10		8 13 16			--trace fine sand.
						Boring terminated at about 41.5 feet below grade. Perched groundwater was observed from about 19 to 29 feet below grade during and after drilling.
45						

N-Value ▲

PL Moisture LL

RQD Recovery

0 50 100

Instrument

Completion Depth:	41.5ft
Date Borehole Started:	4/4/19
Date Borehole Completed:	4/4/19
Logged By:	B Weitering
Drilling Company:	Boretec, Inc.

Remarks: Borings drilled using an EC-95 track-mounted 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 SDCI GIS maps. Well tag BJG 141.

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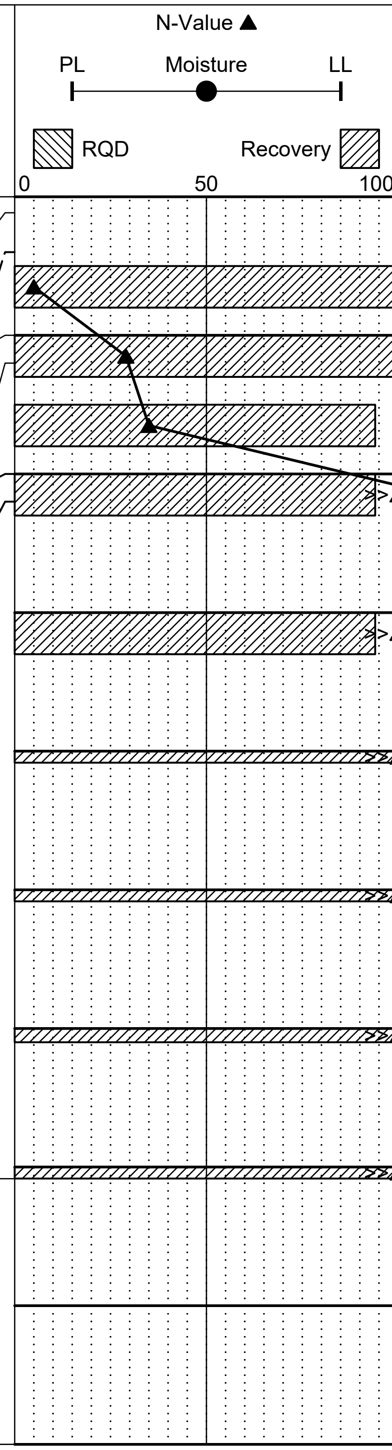
LOG OF TEST BORING PG-1

Figure A-2

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

Project:	505 3rd Avenue W	Surface Elevation:	89.0ft
Job Number:	19-061	Top of Casing Elev.:	N/A
Location:	505 3rd Ave W & 312 W Republican St	Drilling Method:	HSA
Coordinates:	Northing: 47.62345, Easting: -122.36129	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						7-inches concrete alley.
						Dense (based on probing with 1/2" steel rod), grey to grey-brown, gravelly SAND; moist; poorly graded, angular gravel [Basecourse/Fill].
	S-1		2 2 3			Medium stiff, orange-grey to grey-brown, SILT with few ~2" sand interlayers; moist; no to low plasticity, iron oxide staining [Fill].
5	S-2		6 11 18			Very stiff, grey-brown to grey, SILT; moist; low plasticity, trace iron oxide staining [Qvi - Ice Contact Deposits].
	S-3		12 19 16			Dense, grey-brown, slightly gravelly, silty fine to medium SAND; moist; gap-graded, partially cemented, trace iron oxide staining [Qvt - Vashon Till].
10	S-4		12 12 50/6			Medium dense, grey-brown, slightly gravelly, slightly silty to silty SAND; wet; poorly graded [Qvt - Vashon Till].
						Very dense, grey-brown, slightly gravelly, silty fine to medium SAND; moist; gap-graded, partially cemented [Qvt - Vashon Till].
15	S-5		24 25 50/6			
20	S-6		50/5			--broken cobble.
25	S-7		50/5			
30	S-8		50/6			
35	S-9		50/5			Boring terminated at about 35.4 feet below grade. Minor perched groundwater was observed from about 10 to 11 feet below grade during drilling.
40						
45						



Completion Depth:	35.4ft	Remarks: Borings drilled using an EC-95 track-mounted 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 SDCI GIS maps.
Date Borehole Started:	4/4/19	
Date Borehole Completed:	4/4/19	
Logged By:	B Weitering	
Drilling Company:	Boretec, Inc.	

PanGEO INCORPORATED

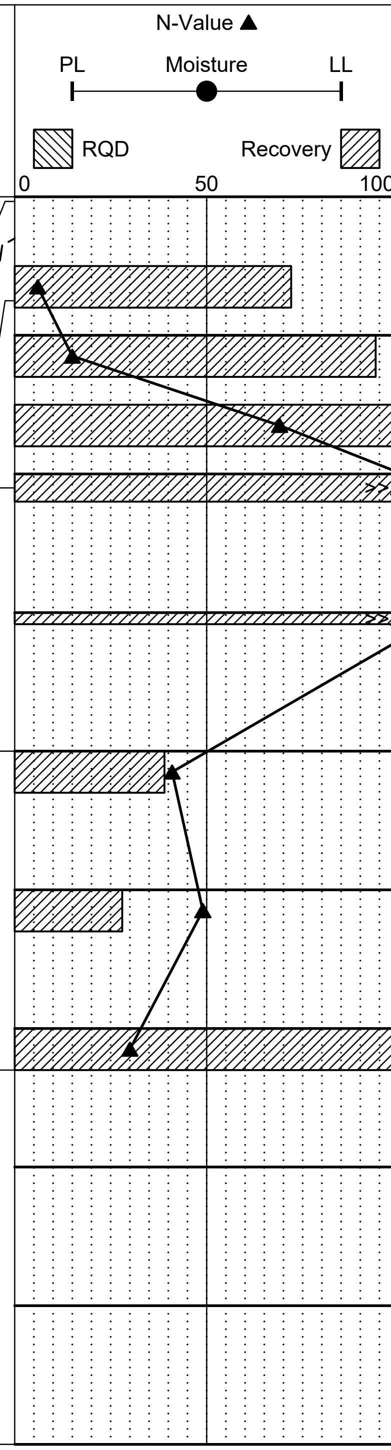
LOG OF TEST BORING PG-2

Figure A-3

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

Project:	505 3rd Avenue W	Surface Elevation:	90.0ft
Job Number:	19-061	Top of Casing Elev.:	N/A
Location:	505 3rd Ave W & 312 W Republican St	Drilling Method:	HSA
Coordinates:	Northing: 47.62339, Easting: -122.36086	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						2-inches asphalt parking lot.
						Dense (based on probing with 1/2" steel rod), brown, gravelly SAND; moist; poorly graded, angular gravel [Basecourse/Fill].
	S-1		2 3 3			
5	S-2		3 6 9			Loose, mixed dark brown and grey-brown, slightly gravelly silty SAND; moist; poorly graded, iron oxide staining [Fill].
	S-3		15 27 42			Very stiff to hard and medium dense to very dense, orange-grey and grey-brown, interlayered SILT and slightly gravelly silty SAND; moist; no to low plasticity, poorly graded sand, iron oxide staining [Qvi - Ice Contact Deposits].
10	S-4		19 50/6			Very dense, grey-brown, slightly gravelly, silty fine SAND; moist; gap-graded, partially cemented [Qvt - Vashon Till].
15	S-5		50/5			--3" slightly silty to silty interlayer with minor perched water.
20	S-6		18 24 17			Hard, grey, CLAY; moist; low plasticity [Qvlc - Lawton Clay].
25	S-7		17 22 27			
30	S-8		10 14 16			
						Boring terminated at about 31.5 feet below grade. Minor perched groundwater was observed at about 15.25 feet below grade during drilling. No groundwater was observed in the piezometer on 4/9/2019.
35						
40						
45						



Instrument

Completion Depth:	31.5ft
Date Borehole Started:	4/4/19
Date Borehole Completed:	4/4/19
Logged By:	B Weitering
Drilling Company:	Boretec, Inc.

Remarks: Borings drilled using an EC-95 track-mounted 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 SDCI GIS maps. Well tag BJG 142.

PanGEO INCORPORATED

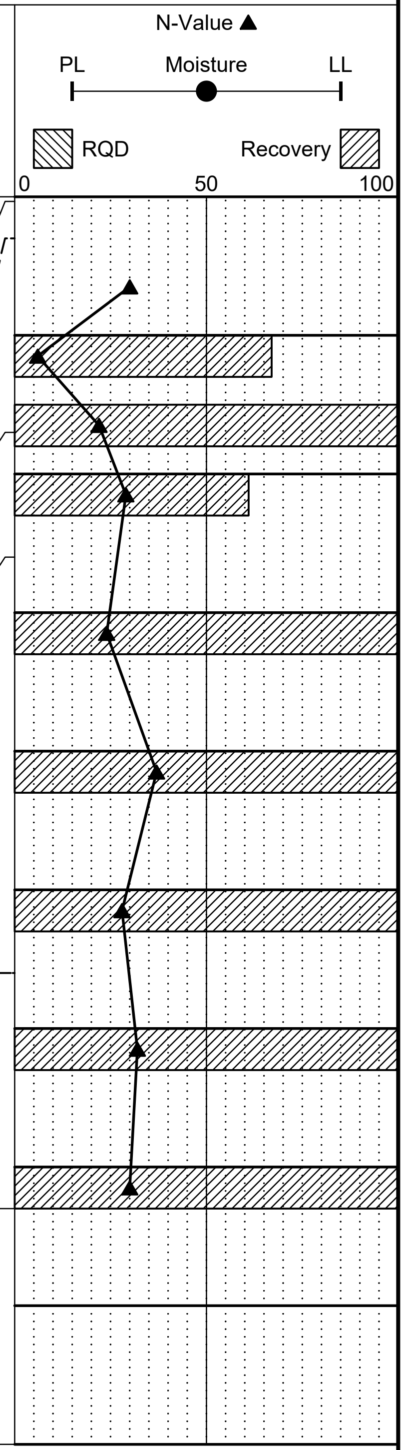
LOG OF TEST BORING PG-3

Figure A-4

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

Project:	505 3rd Avenue W	Surface Elevation:	92.0ft
Job Number:	19-061	Top of Casing Elev.:	92.0ft
Location:	505 3rd Ave W & 312 W Republican St	Drilling Method:	HSA
Coordinates:	Northing: 47.62358, Easting: -122.36086	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						2-inches asphalt parking lot.
						Dense (based on probing with 1/2" steel rod), brown, gravelly SAND; moist; poorly graded, angular gravel [Basecourse/Fill].
	S-1		16 13 17			Loose, mixed dark brown and grey-brown, slightly gravelly silty SAND; moist; poorly graded, trace organics and iron oxide staining [Fill].
5	S-2		3 3 3			--no recovery in S-1; blow count likely inflated; lithology inferred from cuttings and S-2.
	S-3		5 9 13			--becomes medium dense and very moist.
10	S-4		10 14 15			Medium dense, orange-grey and dark grey, slightly silty and gravelly SAND; wet; poorly graded, iron oxide staining [Qvtm - Vashon Subglacial Meltout Till].
						--driller notes change in drilling difficulty at about 13'.
						Very stiff to hard, grey, CLAY; moist; low plasticity [Qvlc - Lawton Clay].
15	S-5		6 9 15			
20	S-6		10 14 23			--becomes hard.
25	S-7		7 11 17			--becomes very stiff with few very moist to wet ~1-2" silty fine sand interlayers.
						Hard, grey, SILT; very moist to wet; no to low plasticity [Qvlc - Lawton Clay].
30	S-8		6 14 18			
35	S-9		8 13 17			
						Boring terminated at about 36.5 feet below grade. Perched groundwater was observed from about 8.5-13 feet and very moist to wet soils were observed from about 25 feet to the bottom of exploration during drilling.
40						
45						



Completion Depth:	36.5ft
Date Borehole Started:	4/4/19
Date Borehole Completed:	4/4/19
Logged By:	B Weitering
Drilling Company:	Boretec, Inc.

Remarks: Borings drilled using an EC-95 track-mounted 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 SDCI GIS maps.

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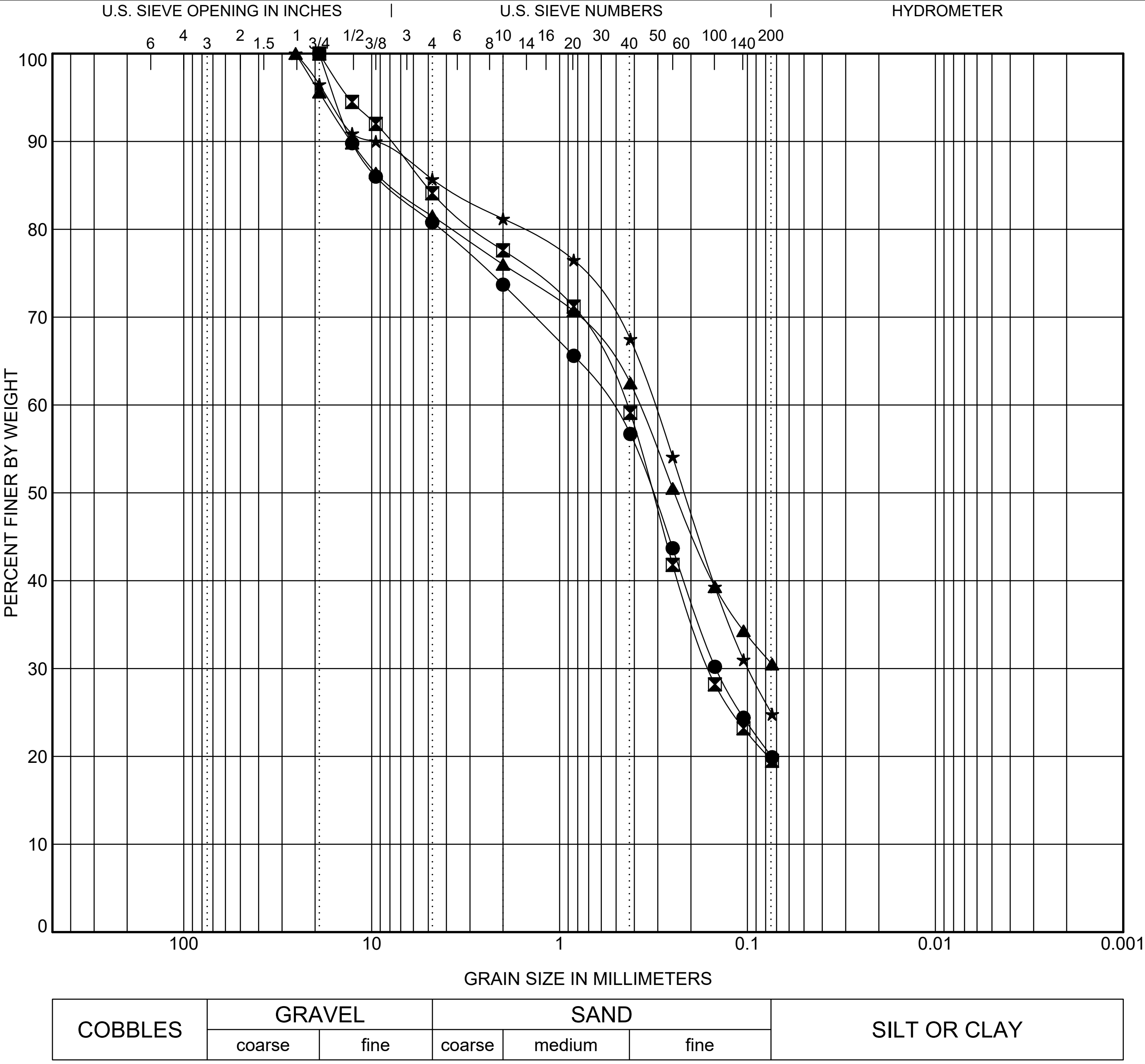
LOG OF TEST BORING PG-4

Figure A-5

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

APPENDIX B

SUMMARY LABORATORY TEST RESULTS



	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	**PG-1**	**@ 20.0 ft.**	**SILTY SAND with GRAVEL (SM)**					
⧗	**PG-1**	**@ 25.0 ft.**	**SILTY SAND with GRAVEL (SM)**					
▲	**PG-2**	**@ 10.0 ft.**	**SILTY SAND with GRAVEL (SM)**					
★	**PG-2**	**@ 15.0 ft.**	**SILTY SAND with GRAVEL (SM)**					

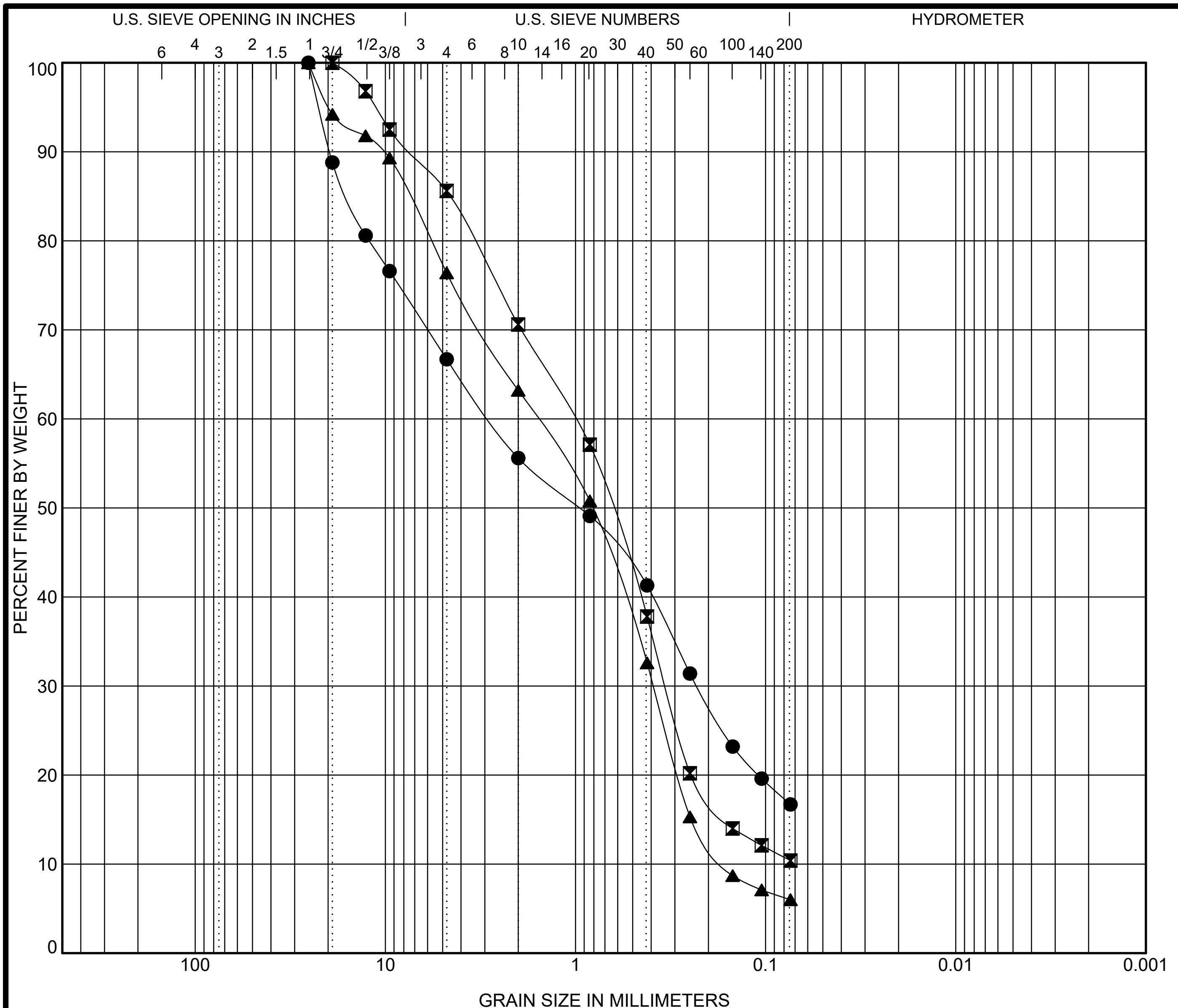
	Specimen Identification		D100	D90	D60	D10	%Gravel	%Sand	%Silt	%Clay
●	**PG-1**	**20.0**	**19**	**12.801**	**0.543**		**19.2**	**60.7**	**20.1**	
⧗	**PG-1**	**25.0**	**19**	**7.982**	**0.442**		**15.9**	**64.4**	**19.6**	
▲	**PG-2**	**10.0**	**25.4**	**12.878**	**0.377**		**18.5**	**50.8**	**30.6**	
★	**PG-2**	**15.0**	**25.4**	**9.51**	**0.314**		**14.3**	**60.7**	**25.0**	

GRAIN SIZE DISTRIBUTION

PanGEO INCORPORATED
Phone: 206.262.0370

Project: 505 3rd Avenue W
Job Number: 19-061
Location: 505 3rd Ave W & 312 W Republican St

Figure B-1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	PG-3	@ 15.0 ft.	**SILTY SAND with GRAVEL (SM)**					
⧗	PG-4	@ 8.0 ft.	**WELL-GRADED SAND with SILT (SW-SM)**				1.61	14.86
▲	PG-4	@ 10.0 ft.	**POORLY GRADED SAND with SILT and GRAVEL (SP-SM)**				0.57	9.69

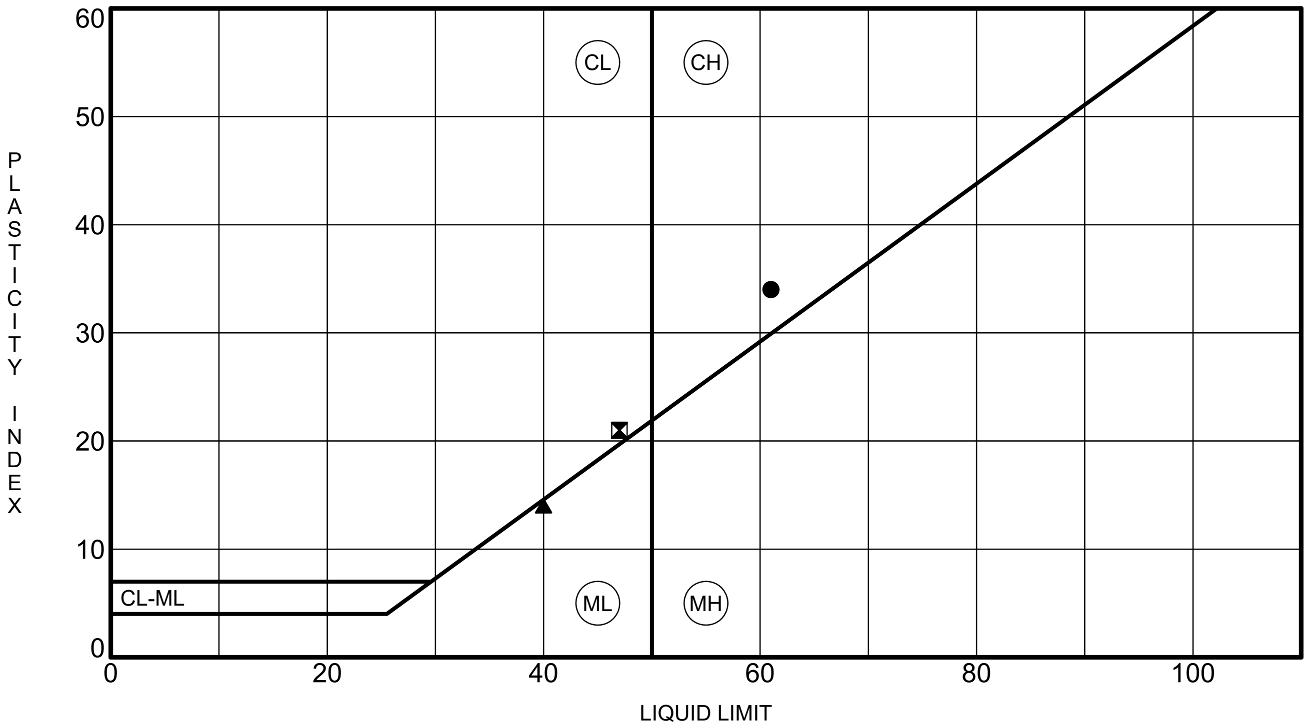
	Specimen Identification		D100	D90	D60	D10	%Gravel	%Sand	%Silt	%Clay
●	PG-3	15.0	25.4	19.6	2.82		33.3	49.9	16.8	
⧗	PG-4	8.0	19	7.401	1.013		14.4	75.1	10.5	
▲	PG-4	10.0	25.4	10.312	1.599	0.165	23.6	70.3	6.0	

GRAIN SIZE DISTRIBUTION

PanGEO INCORPORATED
Phone: 206.262.0370

Project: 505 3rd Avenue W
Job Number: 19-061
Location: 505 3rd Ave W & 312 W Republican St

Figure B-2



	Specimen Identification		M	LL	PL	PI	Fines	Classification
●	**PG-1**	**30.0**	**24**	**61**	**27**	**34**		**High Plasticity Clay**
⧗	**PG-4**	**25.0**	**29**	**47**	**26**	**21**		**Low Plasticity Clay**
▲	**PG-4**	**30.0**	**27**	**40**	**26**	**14**		**Silt**

PanGEO INCORPORATED
Phone: 206.262.0370

ATTERBERG LIMITS

Project: 505 3rd Avenue W
Job Number: 19-061
Location: 505 3rd Ave W & 312 W Republican St

Figure B-3