October 30, 2017  
PanGEO Project No. 17-297 (REV2)

Mr. Anthony Bridgewater  
Enterprise Community Partners, Inc.  
2025 1st Avenue, Suite 1250  
Seattle, Washington 98121

Subject: Preliminary Geotechnical Report  
Proposed Residential Development  
607 Second Avenue North, Seattle, Washington

Dear Mr. Bridgewater:

As requested, PanGEO, Inc. is pleased to present this preliminary geotechnical report to assist the project team with the planning and design of the proposed development at 607 Second Avenue North, Seattle, Washington.

In preparing this report, we observed and logged the drilling of two test borings, reviewed subsurface information from the site and vicinity, and conducted our engineering analyses. In summary, the site may be developed generally as planned.

We encountered potentially liquefiable loose to medium dense sand below the site. In order to mitigate the potential for liquefaction induced settlement, it is our opinion that building support should be provided using a structural mat foundation or a spread footing foundation system after a program of ground improvement.

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

Sincerely,

Siew L. Tan, P.E.
Principal Geotechnical Engineer
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Figure 1    Vicinity Map
Figure 2    Site and Exploration Location Plan
Figure 3    Geologic Map
Figure 4    Design Lateral Pressures, Soldier Pile Wall Cantilevered or with One Tieback

Appendix A  Boring Logs
Figure A-1    Terms and Symbols for Boring and Test Pit Logs
Figures A-2 and A-3    Logs of Test Borings PG-1 and PG-2
1.0 GENERAL

As requested, PanGEO, Inc. is pleased to present this preliminary geotechnical report to assist the project team with the design and construction of the proposed seven story residential development planned for 607 Second Avenue North in Seattle, Washington. This study was performed in general accordance with our mutually agreed scope of services outlined in our proposal dated September 26, 2017. Our scope of services included reviewing readily available geologic and geotechnical data, drilling two borings, conducting a site reconnaissance, and evaluating the feasibility of developing the site as planned.

2.0 SITE AND PROJECT DESCRIPTION

The subject site is located at 607 Second Avenue North, in the northeast corner of the intersection of Mercer Street and Second Avenue North in Seattle, Washington. The approximate location of the site is shown on Figure 1, Vicinity Map and Plate 1, below.

Plate 1: Aerial view of site.
The proposed development area comprises one rectangular lot (Tax Parcel No. 5457800296) with an area of about 11,000 square feet. The site is bordered to the north by an asphalt paved parking lot, to the west by Second Avenue North, to the east by a vacant lot, and to the south by Mercer Street. The layout of the site is shown on Figure 2, Site and Exploration Plan.

The site is currently occupied by a small park and contains paved walking paths around landscaping beds containing ornamental trees and shrubs. The topography is relatively flat, with less than five feet of elevation change across the width of the property. Plate 1, above and Plate 2, below illustrate the general site conditions.

Plate 2: View from northeast to southwest showing the general site conditions.

We understand it is planned to develop the site with a seven-story residential building over one level of below-grade parking. An alternate development proposal is also being considered which will not include the below-grade parking level.
We anticipate the below-grade parking and two lower levels will be of concrete construction, while the upper five levels of residential space will be of wood frame construction.

If the proposed development will include one level of basement, in order to achieve construction subgrade elevations for the below grade parking, we anticipate an excavation extending to a depth of 10 to 12 feet below grade will be needed. The base of the excavation will extend up to the surrounding property limits as a zero-lot line excavation. Temporary shoring will be used to support the excavation during 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. 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 SITE GEOLOGY

Based on our review of The Geologic Map of Seattle – A Progress Report (Troost, et. al. 2005), the geologic units in the vicinity of the site consist of Vashon Ice Contact deposits (Geologic Map Unit Qvi), Lawton Clay (Qvlc) and Vashon Till (Qvt). A portion of the geologic map for that includes the subject site is included as Figure 3, Geologic Map. Ice Contact Deposits consist of interlayered till and outwash deposits. The outwash consists of sand and gravel with varying amounts of silt. These units may or may not have been glacially overridden. Where overridden, the ice contact deposits are typically dense to very dense. Where they have not been overridden, the ice contact deposits may range from loose to dense.

Vashon Till consists of an unsorted deposit (diamict) of clay, silt, sand and gravel that has been glacially transported and deposited. Vashon Till has been glacially overridden and is typically dense to very dense.

Lawton Clay consists of a lacustrine silt and clay deposited in a meltwater lake that formed in front of the Vashon Glacier. This deposit has also been glacially overridden and is typically hard.
3.2 SUBSURFACE EXPLORATION

Two borings (PG-1 and PG-2) were drilled on September 27, 2017. The borings were drilled using an EC-95 track-mounted drill rig operated by Boretec, Inc. under subcontract to PanGEO and were logged by an engineer with our firm. The borings were drilled to depths of 41½ and 46½ feet below existing grade. The approximate boring locations were located in the field by measuring from property corners and site features and are shown on Figure 2, Site and Exploration Plan.

Standard Penetration Tests (SPT) were performed at 2½- to 5-foot depth intervals using a standard, 2-inch diameter split-spoon sampler. The sampler was advanced with a 140-pound drop hammer falling a distance of 30 inches for each strike, in general accordance with ASTM D-1586, Standard Test Method for Penetration Test and Split Barrel Sampling of Soils.

The soils were logged in general accordance with ASTM D-2487 Standard Practice for Classification of Soils for Engineering Purposes and the system summarized on Figure A-1, Terms and Symbols for Boring and Test Pit Logs.

3.3 SOIL CONDITIONS

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. The following is a generalized description of the soils encountered in the borings.

**Topsoil:** Both borings were located in landscaped areas and encountered a surface layer of topsoil. The topsoil layer was 6 to 8 inches thick and consisted of silty sand with gravel and organic material.

**Fill:** Below the topsoil, we encountered fill. The fill ranged from 8 feet thick at Boring PG-1 to 9½ feet thick at Boring PG-2. The fill consisted of grey silty fine to medium sand with gravel and ranged from loose to medium dense. The fill was characterized by its homogenous to disturbed soil structure, angular gravel, and the presence of organics including topsoil and fine roots.

**Vashon Ice Contact Deposits:** Underlying the fill, we encountered an interlayered deposit of silt, silty clay, sand, and silty sand with varying amounts of gravel. The silt and clay
deposits were typically medium stiff while the coarse-grained sand and silty sand deposits ranged from loose to dense.

We classified these interlayered soils as being consistent with ice contact deposits. The composition and density of this deposit may vary significantly over a small distance. The ice contact deposits extended to a depth of about 38 feet below grade in both of our borings.

**Pre-Frasier Glaciation Fine-Grained Deposits:** Below the ice contact deposits, we encountered hard grey silt. We identified this soil as consistent with Pre-Frasier Glaciation Fine-Grained Deposits. This deposit first encountered at about 38 feet deep in both PG-1 and PG-2, and extended to the maximum exploration depth of 46½ feet below grade. A review of other nearby test borings indicate that this deposit may be significantly shallower in the southeast and northwest corners of the site.

Our descriptions of subsurface conditions 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.

### 3.4 Groundwater

Wet to water bearing soils were encountered in both of our borings at about 14 feet below grade while drilling. Based on the planned one level of below grade parking with a cut of 10 to 12 feet deep, we do not anticipate groundwater will result in significant construction related issues. However, there will be fluctuations in seepage depending on the season, amount of rainfall, surface water runoff, and other factors. Generally, the water level is higher and seepage rates are greater in the wetter, winter months (typically October through May).

### 4.0 Geotechnical Recommendations

#### 4.1 General

The proposed project will occupy the southwest quadrant of the block defined by Mercer Street to the south, Second Avenue North to the west, Third Avenue North to the east, and Roy Street to the north. Another development is planned for the other three quadrants of the block and is expected to be constructed before the subject site.
The adjacent project will include a shored excavation with temporary tiebacks that will extend below the subject site. The temporary tiebacks will be abandoned in place after the construction of the adjacent project is completed. If the tiebacks are encountered during excavation at the subject site, they can be removed (if relatively shallow). Tiebacks that are abandoned below the elevation of the building at the subject site should not adversely impact the foundation bearing soils.

The foundation for the subject site will impart a surcharge load on the completed building walls for the adjacent building. In order to mitigate for the surcharge loads, this project may need to extend the foundation loads deeper such that they will not impart a surcharge on the adjacent building. This could be accomplished by stepping the building foundation down or transferring the building loads down through the use of a deep foundation system. The deep foundation options will need to be coordinated with tieback locations so that the deep foundation installation will not encounter the tiebacks. Recommendations for deep foundations, if needed, can be coordinated during final design.

We recommend coordinating the design and construction of this project with the adjacent project in order to address potential conflicts from temporary shoring elements and the impacts from foundation surcharge loads that may affect the constructability of this project.

4.2 Seismic Design Parameters

The 2015 International Building Code (IBC) seismic design section provides a basis for seismic design of structures. Because the some of the sand layers underlying the site are prone to soil liquefaction, Site Class F should be assumed for the seismic design of the project. With Site Class F, a site-specific ground response analysis will be required unless the natural period of the building is less than 0.5 second.

Assuming a building period of less than 0.5 seconds, Table 1 below provides seismic design parameters for the site that are in conformance with the 2015 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.
Table 1 – Seismic Design Parameters

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Spectral Acceleration at 0.2 sec. [g]</th>
<th>Spectral Acceleration at 1.0 sec. [g]</th>
<th>Site Coefficients</th>
<th>Design Spectral Response Parameters</th>
<th>Control Periods [sec.]</th>
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</table>

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.

For the seismic design of the building, if the fundamental period of the proposed building exceeds 0.5 seconds (as determined by the structural engineer) and the risk of soil liquefaction is present, a site-specific ground response analysis will likely be required. PanGEO will be available to perform the analysis if needed.

**Liquefaction Potential:** Liquefaction occurs when saturated sands are subjected to cyclic loading which causes the pore water pressure to increase in the sand thereby reducing the inter-granular stresses. As the inter-granular stresses are reduced, the shearing resistance of the sand decreases. If pore pressures develop to the point where the effective stresses acting between the grains become zero, the soil particles will be in suspension and behave like a viscous fluid. Typically loose, saturated, clean granular soils, that have a low enough permeability to prevent drainage during cyclic loading, have the greatest potential for liquefaction, while more dense soil deposits with higher silt or clay contents have a lesser potential. Potential effects of soil liquefaction include temporary loss/reduction of foundation capacity and settlement.

Based on the conditions encountered at our boring locations, there is the potential for soil liquefaction below the site, specifically in the loose to medium dense sand encountered between 20- to 30-feet below grade in Boring PG-1 and between 15- and 20-feet and 30- to 35-feet below grade in Boring PG-2. We estimate the potential liquefaction induced settlement may be as much as 5 to 6 inches.

With the plan for one level of below grade parking, the building foundation will be located at about 10 to 12 feet below grade. As such, the building foundation will be above the potentially liquefiable soils. In order to mitigate the potential for liquefaction induced settlement damage to
the structure we recommend a program of ground improvement with aggregate piers (e.g., Geo piers, etc) or supporting the building on a relatively stiff structural mat foundation.

4.3 BUILDING FOUNDATIONS

Based on our understanding of the proposed development, the project will include one level of below grade parking with a foundation subgrade elevation of about 10 to 12 feet below grade. An alternate development plan is also being considered, which will not include the below grade parking. Based on the conditions encountered in our borings, the ice contact deposits below the site contain loose to medium dense water bearing sand lenses that may be susceptible to liquefaction induced settlement.

In order to mitigate the impacts from liquefaction, it is our opinion one of the following options should be used for supporting the proposed building:

- **Mat Foundation:** With this option, the building foundation would be supported on a structural mat bearing on the medium dense to dense ice contact deposits underlying the fill.

- **Conventional Foundation on Improved Ground:** This option would consist of mitigating the liquefaction potential using ground improvement, such as aggregate piers. The building can then be supported on a conventional foundation bearing on the improved soils.

Due to the presence of 8 to 9½ feet of loose fill underlying the site, the mat foundation option would not be applicable for the development option that will not include a below grade parking level. If the option without below grade parking is pursued, it should be planned to support the building on conventional foundations after ground improvement.

4.3.1 Mat Foundation

With this option, building support can be provided using a structural mat foundation bearing on the ice-contact deposits that should be encountered 10 to 12 feet below grade. The ice-contact deposits may consist of medium dense to dense sand, and medium stiff to stiff silt and clay. The existing fill, which was about 8 to 9 feet thick at our test boring locations, are not suitable for supporting the mat foundation.

The structural mat foundation should be designed so that is sufficiently stiff to spread the concentrated column loads out over a wide area. The mat foundation can be evaluated using a
modulus of subgrade reaction of 150 pounds per cubic inch (pci). Local bearing pressures below concentrated loads can be evaluated using an allowable soil bearing pressure of 3,000 psf.

We estimate total settlement of the mat foundation due to dead plus live loads will be about 1-inch, with differential settlement of ½-inch.

We estimate total liquefaction induced settlements could be on the order of 5 to 6 inches. However, because the site soils in our test borings are quite uniform, it is our opinion the settlement from the potential soil liquefaction will likely be generally uniform across the site, and the potential differential settlements will likely be less than half of the estimated total settlement. Some architectural or structural damage could occur as a result of the settlement, however, from a life safety standpoint, the structure would not collapse and entrance or egress from the building should not be significantly impeded.

4.3.2 Conventional Footings on Improved Ground

A commonly-used ground improvement method in the local area is aggregate piers, such as Geopiers or stone columns. This type of ground improvement would consist of compacting columns of well-graded crushed rock to increase the bearing capacity of the loose, potentially liquefiable soils and reduce the potential for settlement. After the aggregate piers are installed, spread footings and slab-on-grade floors can be supported directly on the improved ground.

Because aggregate piers are a proprietary system, the installation contractor is responsible for the design of the aggregate piers, based on the performance criteria provided by the structural engineer. Based on the performance criteria provided, the aggregate pier contractor will determine the allowable bearing pressures, improved soil characteristics and anticipated settlements.

4.3.3 Lateral Resistance

Lateral loads on the structure may be resisted by passive earth pressure developed against the embedded portion of the foundation and by frictional resistance between the bottom of the foundation and the supporting subgrade soils. For footings bearing on the medium dense to dense sand and silty sand, a frictional coefficient of 0.35 may be used to evaluate sliding resistance developed between the concrete and the subgrade soil. Passive soil resistance may be calculated using an equivalent fluid weight of 300 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.
4.3.4 Perimeter Footing Drains

Footing drains should be installed around the perimeter of the at-grade portion of the building, at or just below the invert of the footings. Under no circumstances should roof downspout drain lines be connected to the footing drain systems. Roof downspouts must be separately tiedlined to appropriate discharge locations. Cleanouts should be installed at strategic locations to allow for periodic maintenance of the footing drain and downspout tightline systems.

4.3.5 Foundation Subgrade Preparation

The foundation subgrade soils should be in a firm and unyielding condition prior to setting forms and placing reinforcing steel. If loose soils are encountered at the foundation subgrade elevation, the loose soils should be removed from the footing excavations and replaced with structural fill.

4.4 Floor Slabs

Floor slabs with the ground improvement option may be constructed using conventional concrete slab-on-grade floor construction. The floor slabs should be supported on competent native soil or on structural fill. Any overexcavation, if needed, should be backfilled with structural fill.

If heated space or spaces that are sensitive to moisture intrusion are planned for the parking garage level, the concrete floors should be underlain by a capillary break meeting the gradational requirements provided in Table 2, below.

The capillary break material should meet the gradational requirements provided in Table 2, below.

<table>
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<th>Sieve Size</th>
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<tr>
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<td>0 – 5</td>
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<tr>
<td>No. 200</td>
<td>0 – 3</td>
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The capillary break should be placed on subgrade soils that have 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.

4.5 RETAINING WALL DESIGN PARAMETERS

Cast-in-place concrete retaining and basement walls should be designed to resist the lateral earth pressures exerted by the soils behind the wall. Proper drainage provisions should also be provided to intercept and remove groundwater that may be present behind the walls.

Cantilever walls should be designed for an equivalent fluid pressure of 35 pcf for a level backfill condition and assuming the walls are free to rotate. If the walls are restrained at the top from free movement, such as basement walls with a floor diaphragm, an equivalent fluid pressure of 45 pcf should be used for a level backfill condition behind the walls. Permanent walls should be designed for an additional uniform lateral pressure of 7H psf for seismic loading, where H corresponds to the height of the buried depth of the wall.

The recommended lateral pressures assume the backfill behind the walls consists of free draining and properly compacted fill with adequate drainage provisions.

4.5.1 Surcharge

Surcharge loads, where present, should also be included in the design of retaining walls. We recommend that a lateral load coefficient of 0.3 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.

4.5.2 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 300 pcf. This value includes a factor of safety of 1.5, assuming the footing is backfilled with structural fill. A friction coefficient of 0.30 may be used to determine the frictional resistance at the base of the footings. The coefficient includes a factor of safety of 1.5.
4.5.3 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. pea gravel or washed 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. The drainpipe at the base of the wall should be graded to direct water to a suitable outlet.

4.5.4 Wall Backfill

Wall backfill should consist of free draining granular material. The site soils are relatively silty and would not meet the requirements for wall backfill. We recommend importing a free draining granular material, such as Gravel Borrow as defined in Section 9-03.14(1) of the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction (WSDOT, 2016). In areas where space is limited between the wall and the face of excavation, pea gravel may be used as backfill without compaction.

Wall backfill should be moisture conditioned to near optimum moisture levels, placed in loose, horizontal lifts less than 8 inches in thickness, and systematically compacted to a dense and relatively unyielding condition and to at least 95 percent of the maximum dry density, as determined using test method ASTM D-1557 (Modified Proctor). Within 5 feet of the wall, the backfill should be compacted with hand-operated equipment to at least 90 percent of the maximum dry density.

4.6 Permanent Cut and Fill Slopes

Permanent cut and fill slopes should be inclined no steeper than 2H:1V. Permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve stability of the surficial layer of soil.

5.0 Excavations and Temporary Shoring

In order to achieve construction subgrade elevations for the below grade parking level, we anticipate an excavation extending to a depth of 10 to 12 feet below grade is planned. The planned excavation will extend up the site boundaries as a zero lot line excavation.
excavation will be accomplished using a combination of open cuts with temporary slopes and vertical cuts supported with temporary shoring.

5.1 **TEMPORARY EXCAVATIONS**

Temporary excavations should be constructed 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 the soil conditions encountered in the test borings, it is our opinion that temporary excavations may be cut at a maximum 1H:1V inclination. If sufficient space is not available, temporary excavation shoring will be needed.

Temporary excavations should be evaluated in the field during construction based on actual observed soil conditions. If seepage is encountered, excavation slope inclinations may need to be reduced. During wet weather, the cut slopes may need to be flattened to reduce potential erosion or should be covered with plastic sheeting.

5.2 **TEMPORARY SHORING**

Where there is not sufficient room to make conventional open cuts temporary shoring will be needed. In our opinion, temporary shoring consisting of a soldier pile wall with timber lagging is likely the most cost-effective shoring option.

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

5.2.1 **Temporary Soldier Pile Shoring Design Parameters**

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 flanges of the piles to further stabilize the walls of the excavation.
5.2.2 Wall Design Parameters

The earth pressures depicted on Figure 4, *Design Lateral Pressures, Soldier Pile Wall, Cantilevered* be used for design of soldier pile walls for this project. Our shoring design parameters assume the excavation is fully dewatered and do not include hydrostatic pressures from groundwater.

5.2.3 Lagging

Lagging design recommendations for general conditions are presented on Figure 4. Lagging located within 10 feet of the top of the shoring which may be subjected to surcharge loads from construction equipment or material storage should be designed for an additional uniform lateral surcharge pressure of 200 psf. This pressure approximately corresponds to a vertical uniform surcharge load of 500 psf at the top of the wall for general construction surcharge. Point loads located close to the top of the wall, such as outriggers of heavy cranes, may apply additional loads to the lagging. These loads may need to be individually analyzed. However, lagging designed for a uniform load of 600 psf in the top 10 feet of the wall should be able to accommodate most crane outrigger loads.

We recommend voids behind the lagging be backfilled with CDF.

The lagging will be faced by the permanent basement walls of the buildings. In order to maintain drainage, a continuous layer of a geocomposite drainage, such as Miradrain 6000, should be placed between the lagging and basement walls. The geocomposite drainage should be connected to a tightline collection pipe on the interior of the building.

5.2.4 Baseline Survey and Monitoring

Ground movements may be experienced as a result of excavation activities. As such, ground surface elevations of the adjacent properties and city streets should be documented prior to commencing earthwork to provide baseline data. As a minimum, optical survey points should be established at the following locations:

- The top of every other soldier pile. These monitoring points should be monitored twice a week. The monitoring frequency may be reduced based on the monitoring results.

- Adjacent structures located within 25 feet of the shoring walls.
• The curbs and the centerlines of adjacent streets should be monitored by establishing a set of baseline point spaced no more than 20 feet apart. These monitoring points should not need to be regularly surveyed after the baseline is established unless the soldier pile wall monitoring indicates deflections exceeding one inch.

The monitoring program should include monitoring for changes in both the horizontal (x and y directions) and vertical deformations. The monitoring should be performed by the contractor or the project surveyor, and the results should 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 the existing conditions along the public right-of-way and the adjacent private properties be photo-documented prior to commencing earthwork at the site.

6.0 EARTHWORK CONSIDERATIONS

6.1 STRUCTURAL FILL AND COMPACTION

Structural fill, if needed, should consist of City of Seattle Type 17, crushed surfacing base course as specified in WSDOT Section 9-03.9(3) (WSDOT 2014), or an approved similar material.

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 compacted to at least 95 percent maximum density, determined using ASTM D-1557 (Modified Proctor). The procedure to achieve proper density of a compacted fill depends on the size and type of compacting 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.
6.2 MATERIAL REUSE

The native soils underlying the site are moisture sensitive, and will become disturbed and soft when exposed to inclement weather conditions. We do not recommend reusing the native soils as structural fill. If it is planned to use the native soil in non-structural areas, the excavated soil should be stockpiled and protected with plastic sheeting to prevent it from becoming saturated by precipitation or runoff.

6.3 WET WEATHER CONSTRUCTION

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below. The following procedures are best management practices recommended for use in wet weather construction:

- 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.

6.4 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

To confirm that our recommendations are properly incorporated into the design and construction of the proposed development, PanGEO should be retained to conduct a review of the final project plans and specifications, and to monitor the construction of geotechnical elements. The City of Seattle, as part of the permitting process, will also require geotechnical construction inspection services. PanGEO can provide you a cost estimate for construction monitoring services at a later date.

8.0 CLOSURE

We have prepared this report for Enterprise Community Partners, Inc. 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,

PanGEO, Inc.

Scott D. Dinkelman, LEG, LHG
Senior Engineering Geologist

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


INCORPORATED

PROJECT LOCATION

ELLIOTT BAY

Source: Open Street Map

Proposed Residential Development
607 2nd Avenue N
Seattle, Washington

VICINITY MAP

Approximate Scale: 1" = 1000'

PanGEO INCORPORATED

17-297

10/2/2017
1. Aerial imagery & topography obtained from Seattle DPD GIS website.
2. Location of borings are approximate and based on the relative locations of known site features.
3. Vertical Datum: NAVD '88

NOTES

PROJECT LOCATION

LEGEND
PG-#  Soil Boring by PanGEO, Inc. (2017 - Appendix A)
GEOLOGIC UNITS

- QI Lake deposits
- Qvr Vashon recessional outwash deposits
- Qvrl Vashon recessional lacustrine deposits
- Qvi Vashon ice-contact deposits
- Qvt Vashon advance outwash deposits
- Qvlc Vashon lawton clay deposits
- Qpf Pre-Fraser glaciation age deposits
- Qpfn Pre-Fraser, nonglacial deposits
- Qob Olympia beds
- Qpo Deposits of pre-olympia age

NOTES

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

BORING LOGS
### RELATIVE DENSITY / CONSISTENCY

<table>
<thead>
<tr>
<th>SAND / GRAVEL</th>
<th>SILT / CLAY</th>
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<tr>
<td><strong>Density</strong></td>
<td><strong>SPT N-values</strong></td>
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<tr>
<td>Very Loose</td>
<td>&lt;4</td>
</tr>
<tr>
<td>Loose</td>
<td>4 to 10</td>
</tr>
<tr>
<td>Med. Dense</td>
<td>10 to 30</td>
</tr>
<tr>
<td>Dense</td>
<td>30 to 50</td>
</tr>
<tr>
<td>Very Dense</td>
<td>&gt;50</td>
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<td></td>
<td></td>
</tr>
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</table>

### UNIFIED SOIL CLASSIFICATION SYSTEM

#### MAJOR DIVISIONS
- **Gravel**
  - 50% or more of the coarse fraction retained on the #4 sieve. Use dual symbols (eg. GM-GP) for 5% to 12% fines.
  - GRAVEL (<5% fines)
  - GRAVEL (>12% fines)
- **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)
  - SAND (>12% fines)
- **Silt and Clay**
  - 50% or more passing #200 sieve
  - Liquid Limit < 50
  - Liquid Limit > 50
  - Highly Organic Soils

#### GROUP DESCRIPTIONS
- **GW**: Well-graded GRAVEL
- **GP**: Poorly-graded GRAVEL
- **GM**: Silty GRAVEL
- **GC**: Clayey GRAVEL
- **SW**: Well-graded SAND
- **SP**: Poorly-graded SAND
- **SM**: Silty SAND
- **SC**: Clayey SAND
- **ML**: Silty CLAY
- **OL**: Organic SILT or CLAY
- **ML**: Elastic SILT
- **OH**: Organic Silt or CLAY
- **PT**: PEAT

### 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 the core axis

### COMPONENT DEFINITIONS
- **Boulder**: > 12 inches
- **Cobbles**: 3 to 12 inches
- **Gravel**: 3 to 3/4 inches
  - Coarse Gravel: #4 to #10 sieve (4.5 to 2.0 mm)
  - Fine Gravel: #40 to #200 sieve (0.04 to 0.074 mm)
- **Sand**: 3/4 inches to #4 sieve
  - #4 to #10 sieve (4.5 to 2.0 mm)
  - #10 to #40 sieve (2.0 to 0.42 mm)
- **Silt and Clay**: 0.074 to 0.002 mm
  - Silt: 0.074 to 0.002 mm
  - Clay: <0.002 mm

### TEST SYMBOLS

<table>
<thead>
<tr>
<th>SYMBOLS</th>
<th>SAMPLE/IN SITU TEST TYPES AND INTERVALS</th>
</tr>
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<tbody>
<tr>
<td>ATT</td>
<td>Atterberg Limit Test</td>
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<tr>
<td>Comp</td>
<td>Compaction Tests</td>
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<tr>
<td>Con</td>
<td>Consolidation</td>
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<td>DD</td>
<td>Dry Density</td>
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<td>DS</td>
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<td>%F</td>
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<td>GS</td>
<td>Grain Size</td>
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<td>Permeability</td>
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<td>Pocket Penetrometer</td>
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<td>Specific Gravity</td>
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<td>Torvane</td>
</tr>
<tr>
<td>TXC</td>
<td>Triaxial Compression</td>
</tr>
<tr>
<td>UCC</td>
<td>Unconfined Compression</td>
</tr>
</tbody>
</table>

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

### MOISTURE CONTENT
- **Dry**: Dusty, dry to the touch
- **Moist**: Damp but no visible water
- **Wet**: Visible free water

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**Terms and Symbols for Boring and Test Pit Logs**

Figure A-1
Approximately 8-inch thick layer of landscaping bark and organic rich topsoil.

UNIT 1: FILL / MODIFIED LAND

Medium dense, grey, silty fine to medium SAND with scattered subangular to subrounded gravel; dry, non-plastic fines.

-- becomes loose, grey with faint bands of iron-oxide staining, silty fine SAND with scattered fine to coarse roots; dry to damp, non-plastic fines, poorly graded sand fraction, homogeneous soil structure.

-- becomes grey, slight increase in fine fraction and moisture content. Observed an approximately 0.1-inch thick layer of black, organic material mid-sample.

UNIT 2: VASHON ICE-CONTACT DEPOSITS

Medium stiff, grey with black and rust colored mottling, silty CLAY with scattered one-inch thick layers of silty sand; moist, low to medium plasticity, disrupted and blocky soil texture.

Medium dense, grey, fine to coarse SAND with gravel to poorly graded, sandy Gravel; wet, subangular to subrounded fine to coarse gravel.

Medium dense, dark grey with light bluish cast, silty SAND with abundant fine subangular gravel and scattered coarse gravel; wet, non-plastic fines, homogenous color.

Medium dense, grey, poorly graded, medium to coarse SAND; wet, trace silt, homogenous color and soil structure.

Remarks: 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 a City of Seattle’s GIS Website. This information is provided for relative information only and is not a substitution for field survey. Vertical Datum: NAVD88

The stratification lines represent approximate boundaries. The transition may be gradual.
Medium dense, grey, poorly graded, medium to coarse SAND; wet, trace silt, homogenous color and soil structure. (Continued)

Medium dense brownish-grey, silty very fine to fine SAND; wet, non-plastic to low plasticity fines, homogenous color and soil structure.

Medium dense, light grey, poorly graded medium to coarse SAND; wet, homogenous color and soil structure.
-- observed sampler bouncing on gravel while sampling. Blowcounts may be overstated due to the presence of gravel. Sample description based on recovered material.
-- driller reports it feels like the soil is squeezing against the outside of the augers while advancing the augers between 30 to 35 feet bgs. However, the inside of the hollow stem augers appeared to remain open and no heaving was observed.

Medium dense, fine to coarse SAND with some silt; wet, non-plastic fines, homogenous color and soil structure.

UNIT 3: Pre-Fraser Glaciation Fine-grained Deposits
Hard, grey, SILT; moist, non-plastic to low plasticity fines, finely bedded, massive.

-- same as above.

Boring terminated approximately 46.5 feet below the surface.
Groundwater was encountered approximately 14-ft below the ground surface.

Remarks: EC-95 track-mounted drill rig. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope an cathead mechanism. Surface elevation estimated based on a City of Seattle's GIS Website. This information is provided for relative information only and is not a substitution for field survey. Vertical Datum: NAVD88
LANDSCAPING / BARK TOPSOIL
Approximately 6-inch thick layer of landscaping bark and organic rich topsoil.

UNIT 1: FILL / MODIFIED LAND
Very loose, grey, silty fine to medium SAND with subangular to subrounded gravel; dry, non-plastic fines.
-- poor recovery. Soil description based on visual observation of soil cuttings.
-- becomes intermixed grey and brown, silty fine SAND with scattered fine gravel; very moist, disrupted soil structure.
-- becomes grey with faint iron-oxide staining, moist, slight increase in fines fraction, scattered fine rootlets.
Medium stiff, very dark grey, sandy SILT with organics; very moist, non-plastic fines, strong organic odor.

UNIT 2: VASHON ICE-CONTACT DEPOSITS
Medium stiff, grey with black and rust colored mottling, silty CLAY; moist, low to medium plasticity, disrupted and blocky soil texture.
Loose, grey with a bluish cast, very silty, very fine to medium SAND with scattered subangular to subrounded gravel; wet, non-plastic to low plasticity fines, homogenous color.

Dense, grey, medium to coarse SAND with scattered subangular gravel; wet, trace silt, poorly graded, homogenous color and soil structure.
-- observed sampler bouncing on gravel while sampling. Blowcounts may be overstated due to the presence of gravel.

Dense, grey with strong uniform iron-oxide staining, silty fine to medium SAND; wet, homogenous soil structure.

Remarks: EC-95 track-mounted drill rig. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope an cathead mechanism. Surface elevation estimated based on a City of Seattle's GIS Website. This information is provided for relative information only and is not a substitution for field survey. Vertical Datum: NAVD88.
Dense, grey with strong uniform iron-oxide staining, silty fine to medium SAND; wet, homogenous soil structure. 
-- observed sampler bouncing on gravel while sampling. Blowcounts may be overstated due to the presence of gravel. 

Medium dense, light grey, poorly graded medium to coarse SAND; wet, homogenous color and soil structure.
-- observed sampler bouncing on gravel while sampling. Blowcounts may be overstated due to the presence of gravel. Sample description based on recovered material.
-- driller reports it feels like the soil is squeezing against the outside of the augers while advancing the augers between 30 to 35 feet bgs. However, the inside of the hollow stem augers appeared to remain open and no heaving was observed.
-- becomes loose, grey, very silty, very fine SAND with numerous fine angular gravel; wet, homogenous soil structure. Observed an approximately 4-inch thick layer of grey, non-plastic SILT mid-sample.
-- driller reports increased drilling effort below 30 feet bgs.

UNIT 3: Pre-Fraser Glaciation Fine-grained Deposits
Hard, grey, SILT; moist, non-plastic to low plasticity fines, finely bedded, massive.
-- same as above.

Boring terminated approximately 41.5 feet below the surface.
Groundwater was encountered approximately 14-ft below the ground surface.