PRELIMINARY GEOTECHNICAL REPORT
PROPOSED SHORT PLAT AND RESIDENCE
8247 East Mercer Way
Mercer Island, Washington

PROJECT NO. 22-256

August 2022

Prepared for:

Peak Builders, Inc.

PanGEO INCORPORATED

Geotechnical & Earthquake
Engineering Consultants

PanGEO
INCORPORATED
Geotechnical & Earthquake Engineering Consultants

August 17, 2022
Project No. 22-256

Mr. Jeff Rudd
Peak Builders, Inc.
PO Box 328
Mercer Island, Washington 98040

Subject: Preliminary Geotechnical Report
Proposed Short Plat and Residence
8247 East Mercer Way, Mercer Island, Washington

Dear Mr. Rudd:

As requested, PanGEO has completed a geotechnical engineering study for the proposed short-plat and residence at 8247 East Mercer Way in Mercer Island, Washington. In preparing this report, we performed a reconnaissance of the site, drilled two test borings, and conducted our engineering analyses. The results of our study and our design recommendations are presented in the attached report.

At our exploration locations, we encountered an 8½ to 11½ foot thick water bearing layer of very loose to loose silty sand which we interpreted to consist of colluvium overlying very stiff to hard silt and clay that we interpreted to consists of intact slump blocks associated with a large prehistoric slope failure.

The steep slope to the north of the site is marginally stable and in our opinion is prone to shallow surficial debris flow type slope failures. To protect the planned improvements from potential instability, we recommend a catchment wall to mitigate the impacts of debris flows descending the north slope.

Based on the conditions encountered at our exploration locations, from a geotechnical engineering perspective, the proposed residence can be constructed generally as planned. Support for the residence can be provided using small diameter pipe piles or pin piles driven through the colluvium and bearing in the underlying silt and clay.

Due to the site geology, foundation excavations will likely need to be supported with soldier piles extending sufficiently into the silt and clay. Control of groundwater during construction and post construction will be needed.

We appreciate the opportunity to assist you with this project. Because details of the of the proposed residence are not available at this time, the recommendations outlined in this report should be considered preliminary. Additional geotechnical engineering analysis and input will be needed during the final design and permitting phase of the project. The recommendations outlined in this report may also need to be revised, depending on the final design concept.

Please call if you have any questions.

Sincerely,

Scott D. Dinkelman, LEG
Principal Engineering Geologist
SDinkelman@pangeoinc.com

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ATTACHMENTS:

PRELIMINARY GEOTECHNICAL REPORT
PROPOSED SHORT PLAT AND RESIDENCE
8247 EAST MERCER WAY
MERCER ISLAND, WASHINGTON

1.0 INTRODUCTION

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 residence at 8247 West Mercer Way, Mercer Island, Washington. This study was performed in general accordance with our mutually agreed scope of services outlined in our proposal dated April 29, 2022. Our scope of services included reviewing readily available geologic and geotechnical data, conducting a site reconnaissance, observing drilling of two test borings at the site, and developing the conclusions and recommendations presented in this report.

2.0 SITE AND PROJECT DESCRIPTION

The subject site is located at 8247 East Mercer Way in Mercer Island, Washington, approximately as shown on the attached Figure 1, Vicinity Map.

The approximately rectangular-shaped site comprises about 17,550 square feet and is bordered to the north by a vacant wooded parcel, to the east and west by residences and to the south by East Mercer Way. In the south portion of the site is a two-story single-family residence with a daylight basement that was constructed in 1961. The layout of the site is shown on the attached Figure 2, Site and Exploration Plan while Plate 1 on the next page shows an aerial view of the site. Plate 2 on the next page is a ground level view of the site.

Based on review of the project topographic survey and our observations while on site, the site and surrounding area generally slopes down from northwest to southeast with about 65 feet of elevation change between the north property boundary and East Mercer Way to the south. The site slopes are moderately sloping with slope gradients of 20 to 40 percent with localized areas of slopes in excess of 40 percent. The approximate extents of the 40 percent steeper slopes are shown on the attached Figure 2.

The site slopes extend beyond the north property boundary increasing in gradient to north. The topography beyond the north property boundary is shown on Plate 3, Regional Topography and Geology. The slope to the north has a gradient of 60 to 70 percent and is on the order of 100 to 110 feet high. Above the north slope is a gently rolling plateau.

In the south portion of the site is an existing residence constructed in 1961. The site is vegetated with Douglas fir, big leaf maple, and fruit trees, lawns, ivy and landscaping trees and shrubs.

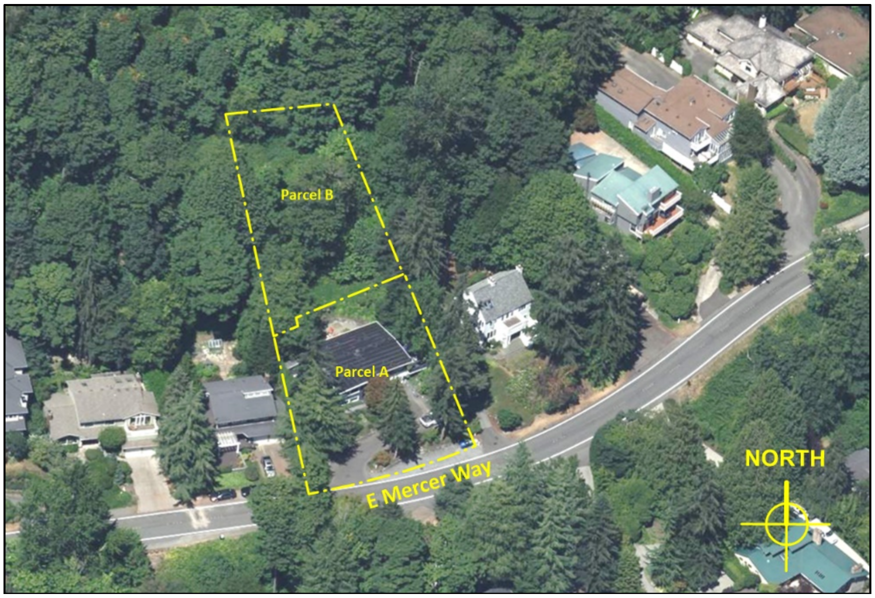


***Plate 1:** An aerial view of the project site.*

The site is approximately outlined in yellow dashed line.

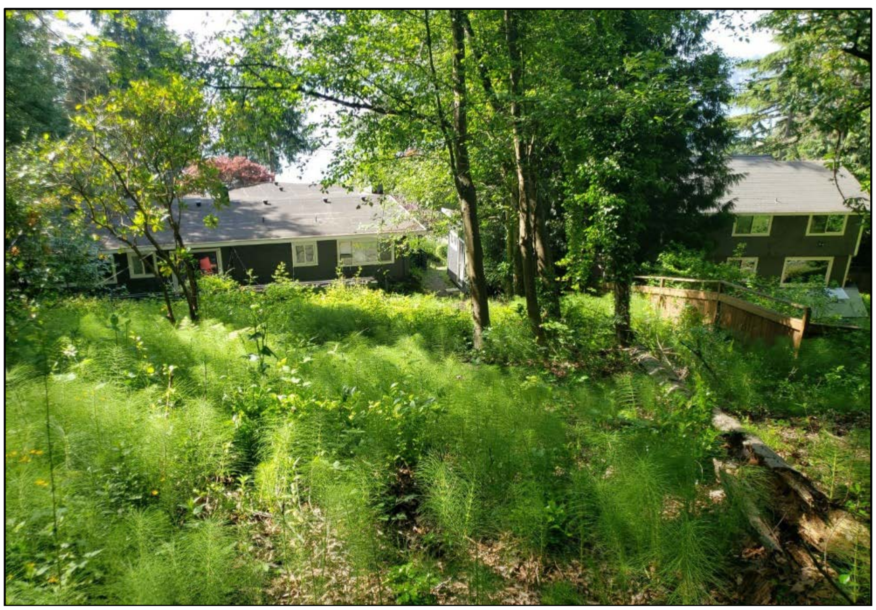


***Plate 2:** View of Parcel B and proposed residence location looking from north to south.*

The existing residence in visible in the background on the left side of the site.

We understand it is planned to divide the site into two lots which have been identified as Parcel A and Parcel B. Parcel A will comprise the south half of the site and will contain the existing residence. Parcel B will comprise the north half of the site and will be developed with a new residence. The design of the proposed residence is not available at this time, however in general, we understand the proposed residence will be two to three stories in height and will be benched into the sloping grade, with the lower level comprised of a daylight basement.

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 EXPLORATION

Two test borings (PG-1 and PG-2) were advanced at the site on June 21, 2022. Borings PG-1 and PG-2 were drilled to depths of about 21½ and 26½ feet below existing grades, respectively. The approximate boring locations were taped relative to existing features and are shown in the attached Figures 2 and 3.

The drill rig was equipped with 4-inch outside diameter hollow stem augers. Soil samples were obtained from the borings at 2½- and 5-foot 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 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. The completed borings were backfilled with drill cuttings and bentonite chips.

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 summary boring logs are included in Appendix A, Figures A-2 and A-3. The soil samples were described using the Modified Unified Soil Classification System outlined on Figure A-1 in Appendix A.

4.0 SUBSURFACE CONDITIONS

4.1 SITE GEOLOGY

Based on review of *The Geologic Map of Mercer Island* (Troost and Wisher, 2006) the surficial geologic units in the vicinity of the site consist of landslide deposits derived from Vashon advance outwash and Lawton clay (Geologic Map Unit Qvlc). Vashon advance outwash consists of silt and sand deposited by meltwater streams in front of the advancing glacier during the Vashon Stade of the Fraser glaciation. This soil was subsequently overridden by several thousand feet of glacial ice and is dense to very dense in its undisturbed state.

Lawton clay typically consists of laminated to massive silt and clay deposited in proglacial lakes early in the Vashon glaciation. This deposit has also been glacially overridden and is typically very stiff to hard. The presence of sand over clay geologic contact is typically considered prone to slope instabilities.

4.2 SOIL CONDITIONS

In general, our test borings encountered colluvium comprised of very loose to loose silty sand overlying silt and clay. The following is a generalized description of the soils encountered in the borings. For a more detailed description of the subsurface conditions encountered at each exploration location for this study, please refer to our boring logs provided in Appendix A.

It should be noted that 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.

Topsoil – At both of our boring locations, we encountered a surficial layer of topsoil. The topsoil ranged from six to eight inches thick and consisted of dark brown silty sand with organics.

Colluvium – Below the topsoil, we encountered very loose to loose silty sand with trace amounts of gravel with a blocky and broken texture. Based on the very loose to loose consistency and blocky and broken texture we interpreted this soil to consist of colluvium, which is soil that has been deposited at the base of a slope by mass wasting and erosional processes. The colluvium ranged from 8½ feet thick at Boring PG-1 to 11½ feet thick at Boring PG-2.

Lawton Clay (Qvlc) – Below the topsoil, we encountered laminated to massive, very stiff to hard silt and clay deposits. We interpreted this soil to be consistent with Lawton Clay which is mapped in this area. The clay ranged from massive to blocky. Both borings were terminated in massive silt and clay at 21½ to 26½ feet below grade.

The subsurface conditions encountered in our borings and the LiDAR derived topography were used to develop the generalized subsurface profile included as Figure 4, which shows the relationship between the encountered soil conditions, and the topography.

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.

4.3 Groundwater

Perched groundwater seepage was encountered at about two feet below grade in Borings PG-1 and PG-2. During our field exploration we noted the site is vegetated with horsetails, a hydrophytic or water loving plant. There is also a wetland that has been delineated in the northwest portion of Parcel B. The City of Mercer Island has also interpreted a line of springs along the southeast portion of the island. The approximate location of the spring line is shown in the attached Figure 3.

Based on the shallow groundwater conditions, consideration for collecting and removing groundwater from around the proposed residence will need to be a consideration in the project design. The designers and contractors should also be aware there will be fluctuations in groundwater conditions 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).

5.0 GEOLOGICALLY HAZARDOUS AREAS CONSIDERATIONS

Geologically Hazardous Areas are identified in the City of Mercer Island Municipal Code (MIMC) Chapter 19.07.160 as lands that are susceptible to erosion, landslides, seismic events, or other factors as identified by Washington Administrative Code (WAC) 365-190-120. Based on our review of the MIMC, the site contains erosion, landslide, and seismic hazards. The City's criteria for these hazard areas and our assessment of the hazard areas with respect to the subject site are provided in the following sections of this report.

5.1 Landslide Hazards

The City of Mercer Island defines landslide hazard area as those areas subject to landslides based on a combination of geologic, topographic, and hydrologic factors, including:

1. *Areas of historic failures;*
2. *Areas with all three of the following characteristics:*
    a. *Slopes steeper than 15 percent; and*
    b. *Hillsides intersecting geologic contacts with a relatively permeable sediment overlying a relatively impermeable sediment or bedrock; and*
    c. *Springs or ground water seepage;*
3. *Areas that have shown evidence of past movement or that are underlain or covered by mass wastage debris from past movements;*
4. *Areas potentially unstable because of rapid stream incision and stream bank erosion; or*
5. *Steep slope. Any slope of 40 percent or greater calculated by measuring the vertical rise over any 30-foot horizontal run.*

In order to evaluate the presence of historical failures and geologic conditions that may identify the presence of landslide features at the site, we reviewed geologic maps, LiDAR imagery, Mercer Island mapping information, and conducted a reconnaissance of the site slopes.

5.1.1 Map Review

Based on review of the *Geologic Map of Mercer Island, Washington* (Troost, et al, 2006), the site is mapped as being located in a prehistoric landslide. The approximate extent of the landslide relative to the site are shown in Plate 3, below and in more detail in the attached Figure 3.

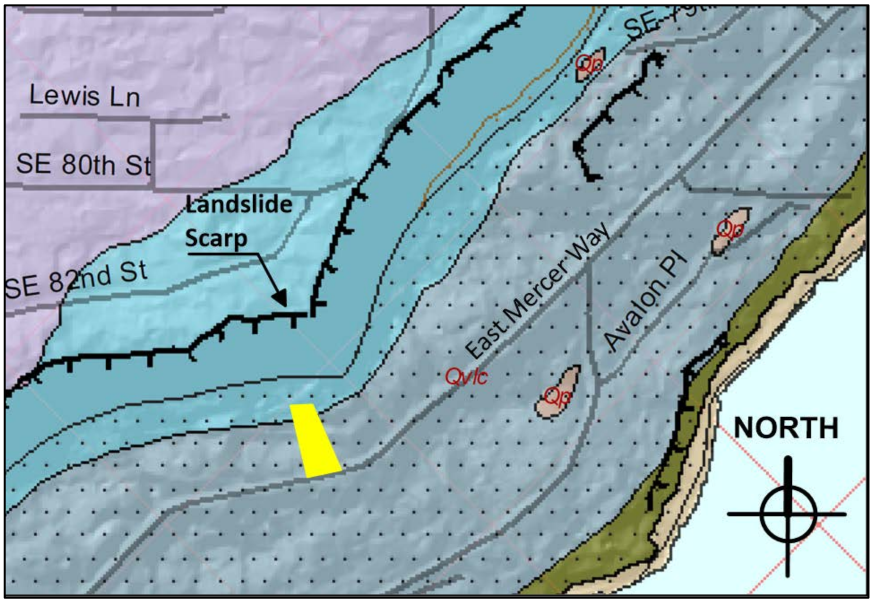


Plate 3: *Geologic map of the site vicinity showing the landslide scarp northwest of the site.*

The approximate site location is delineated by a yellow rectangle.

-- image source Geologic Map of Mercer Island, Washington

Plates 4 and 5 on the next page show the approximate extent of landslide hazard areas mapped by the City of Mercer Island. Based on review of the City's mapping, the entire site is located in a landslide hazard area and potential landslide hazard area. The slope to the north of the site is also mapped as a 40 percent and steeper slope area. The approximate extent of the 40 percent slopes are shown on Figure 2 and Plate 5.

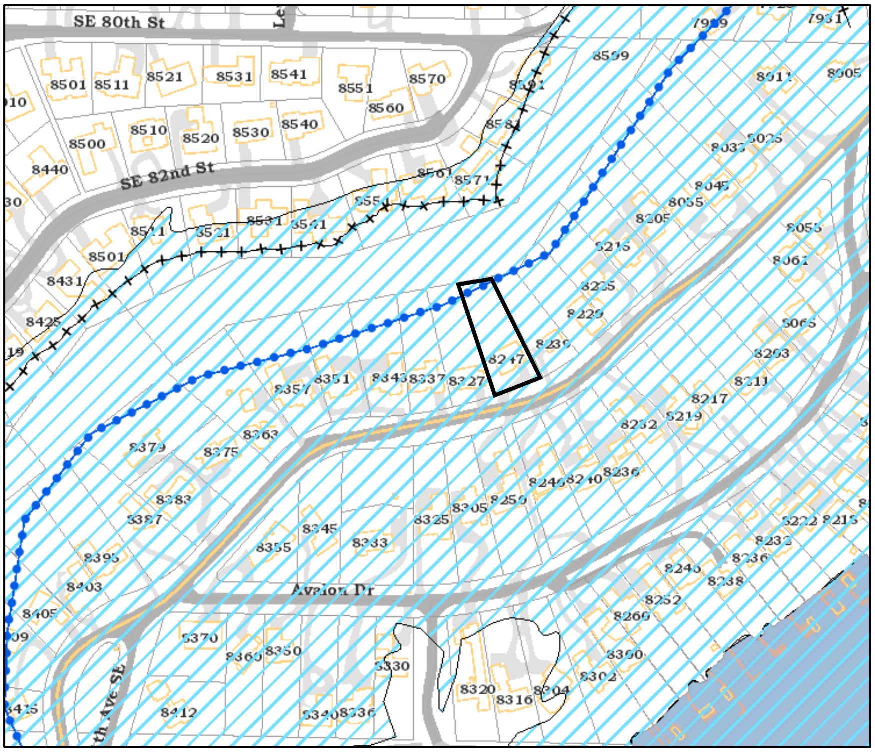


Plate 4: *The blue hatched area is the approximate extent of the Landslide and Potential Landslide Area identified by Mercer Island.*

The site is outlined in black.

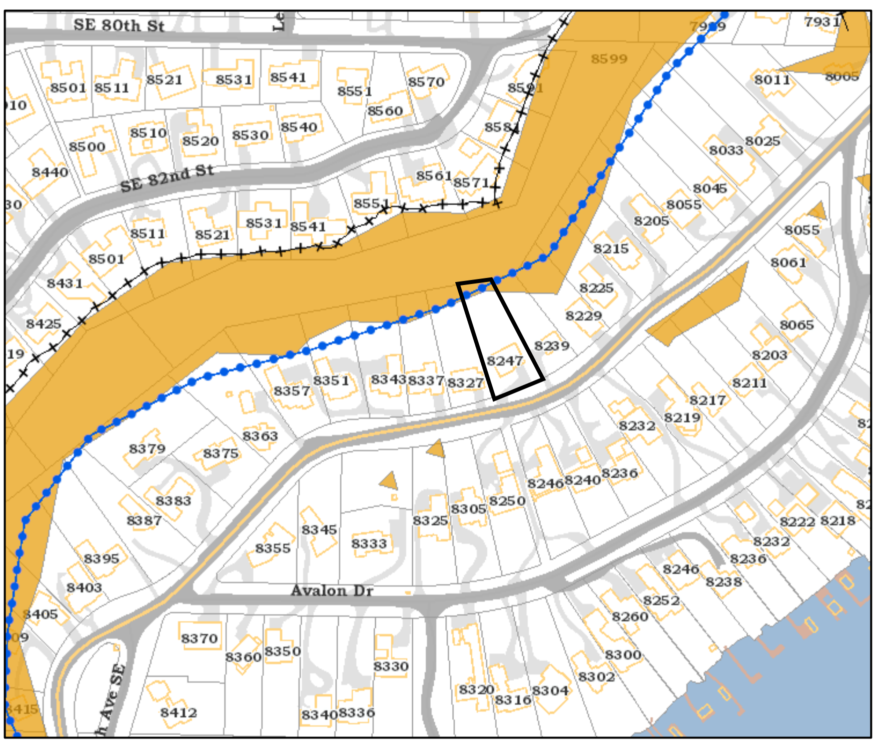


Plate 5: *The orange shaded area is the approximate extent of 40 percent and steeper slope identified by Mercer Island.*

The site is outlined in black.

5.1.2 LiDAR Review

The presence of landslide features for the site area was further evaluated by reviewing LiDAR (Light Detection and Radar) imaging for the site accessed through the Washington State Department of Natural Resources LiDAR Portal. LiDAR is a remote sensing technique that is used to produce high-resolution elevation data for use in mapping applications. LiDAR allows for digitally removing surface vegetation and manmade features, providing a bare earth image of the ground surface. We reviewed LiDAR mapping for the site from using the 2021 King County West data set which is the most recent imagery available. The LiDAR imagery for site is included in Plate 6 below and in more detail in the attached Figure 3.

In the LiDAR imagery, the ridge line outlining the top of the slope failure is visible as a well-defined series of arcuate-shaped scarps or scallops. The ground surface in the slopes below the scarps has a distinctive stippled pattern indicating uneven or hummocky topography, which is a characteristic of a landslide deposit. The arcuate shaped shadows and highlights within the stipple patterned area visible on the left side of Plate 6 are likely slump blocks within the larger landslide scarp.

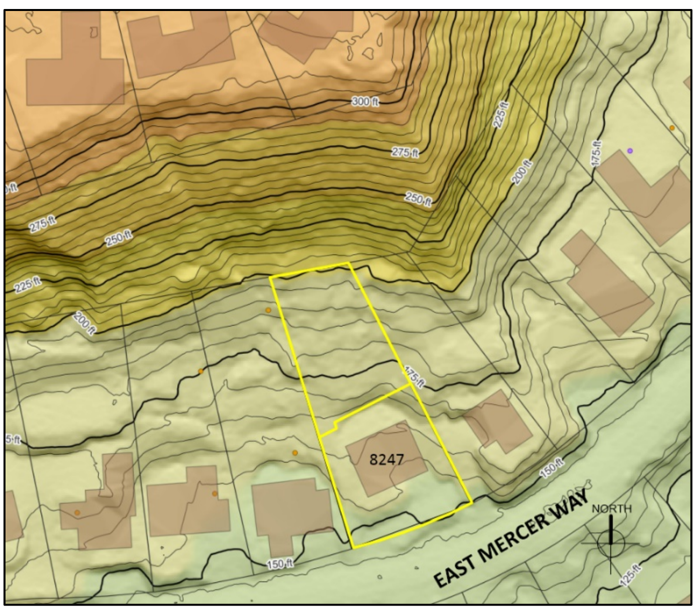


Plate 6: *LiDAR imagery for the site and vicinity.*

The site is outlined in yellow.

-imagery from 2021 King County West data set

5.1.3 Site Reconnaissance

We conducted a reconnaissance of the site and site slopes on June 21, 2022. The weather at the time our reconnaissance was cool and dry. The site boundaries were identified relative to fences and structures.

The purpose of our reconnaissance was to review the condition of the site slopes and identify indications of landslide features such as scarps, bowl-shaped depressions, hummocky topography, distressed vegetation and leaning or pistol butted trees. The following is a summary of our observations:

- The site boundaries in the north portion of the site were marked with stakes and flagged by others. The east and west property boundaries were delineated with fences.
- The north slope was densely vegetated with mature Douglas fir, cedar, and big maple trees and a dense understory of ferns, vine maple, and brush. At the time of our reconnaissance, the visibility of the ground surface was limited due to surface vegetation.
- We did not observe indications of recent or ongoing slope movement affecting the site, such as tension cracks or distressed vegetation.
- Slope gradients in the landslide scarp north of the site range from 60 to 70 percent.
- We observed wet areas and hydrophytic plants on the slope and there is a delineatd wetlands in the northeast portion of Parcel B.

5.1.4 Landslide Hazard Summary

Based on our review and the conditions observed during our reconnaissance, the site meets the criteria for a landslide hazard area.

Although we did not observe indications of recent slope movement affecting the subject site, the site is located within a larger mapped landslide that may be susceptible to movement in the future. It would not be economically feasible to stabilize the entire mapped landslide. Building in a mapped landslide such as this requires accepting a certain level of risk, including the risk of re-activation of the known slide.

The steep slope to the north of the site will be capable of generating debris flow types of slope failures. In order to protect the residence from potential instability, we recommend installing a catchment wall along the north side of the site to protect the future residence from debris flow types of failures. Recommendations for the catchment wall are provided in Section 6.2 of this report.

The following mitigation recommendations and the subsequent recommendations presented in this report should be implemented during design and construction to reduce potential risks at the site:

- We recommend locating and designing the residence in a manner to preserve the natural landforms and vegetation.
- Earthwork should be limited to the area of the proposed residence. Fill should not be placed on the site slopes or around the footprint of the residence.
- Clearing should be limited to the building footprint. If trees are to be removed, they should be stumped, leaving the roots intact.
- Cuts deeper than four feet should be supported during excavation using temporary shoring consisting of soldier piles with timber lagging.
- Earthwork construction should not take place during the wet season – October 1 through April 30.
- Surface water from impervious surfaces, such as roofs, driveways, patios, and walkways should be collected and discharged by tightline into the storm drainage system or to the base of the site slopes.

5.2 Seismic Hazard Areas

Seismic hazard areas are identified in the MIMC as the following:

> *…areas subject to severe risk of damage as a result of earthquake induced ground shaking, slope failure, settlement, soil liquefaction or surface faulting.*

Based on our review of the City of Mercer Island's Geologic Hazards Maps, the project site is mapped as a seismic hazard area. The City of Mercer Island Code defines seismic hazard areas as those areas subject to risk of damage as a result of earthquake-induced ground shaking, slope failure, soil liquefaction or surface faulting. The approximate extent of the mapped seismic hazard area is shown on Plate 7 on the next page. The very loose to loose water bearing silty sand below the site would likely be susceptible to liquefaction during a strong motion seismic event, which could lead to flow failure. Through the use of permanent dewatering, such as subsurface interceptor drains, the potential for liquefaction and flow failure can be reduced. Additional review and analysis should be conducted after the building design and elevations are established.

There will be a potential for off-site seismically induced slope instability to affect the site. Therefor we are recommending the use of a catchment to protect the proposed residence.

It is our opinion that the potential for seismic-induced slope failure is low within very stiff to hard silt and clay below the colluvium.

We have provided recommendations for mitigating the effects of liquefaction with the use of a small diameter pipe pile foundation extending through the colluvium and bearing in the underlying hard silt and clay.

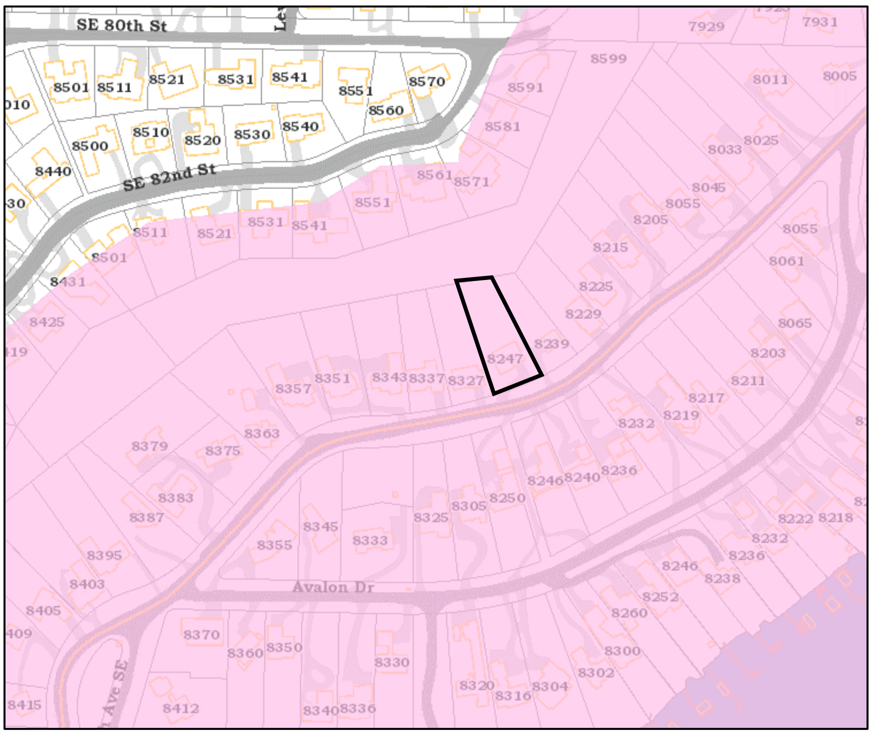


Plate 7: *Approximate extent of the seismic hazard areas identified by the City of Mercer Island.*

The site is outlined in black.

However, it should also be noted that the site is located within a historic landslide zone that encompasses a large area in the south end of Mercer Island. A strong seismic event consistent with the IBC design earthquake has the potential to re-activate the historic landslide. It is not practical to consider mitigating the risk of such an event.

5.3 Erosion Hazards

The entire site is mapped within a potential erosion hazard area according to the City of Mercer Island's Geologic Hazards Map, see Plate 8 on the next page. Based on soil conditions encountered in the borings, the near-surface site soils are likely to exhibit moderate to high erosion potential. In our opinion, the erosion hazards at the site can be effectively mitigated with the best management practice during construction and with properly designed and implemented landscaping for permanent erosion control. Recommendations for controlling erosion are provided in Section 7.6 of this report.

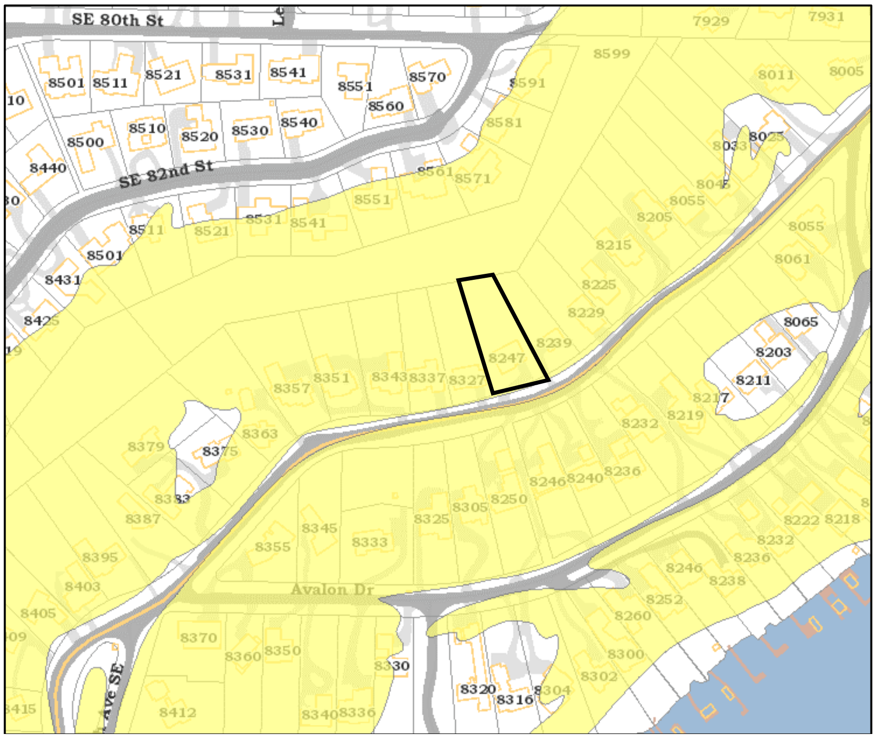


Plate 8: *Approximate extent of erosion hazard areas identified by the City of Mercer Island.*

The site is outlined in black.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 Seismic Site Class

The seismic design for the proposed residence should be accomplished in accordance with the 2018 International Building Code (IBC), which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years). The IBC seismic design parameters are in part based on the site soil conditions and site classifications defined in Chapter 20 of ASCE 7. According to Chapter 20 of ASCE 7, the site soil should be classified as Site Class F because of its liquefaction potential during a strong seismic event (see additional discussions regarding liquefaction potential in Section 5.2 of this report). Section 20.3.1 of ASCE 7-16 indicates that for Site Class F a site response analysis in accordance with Section 21.1 shall be performed unless the exception to Section 20.3.1 is applicable.

Section 20.3.1 of ASCE-7 states that "*For structures having fundamental periods of vibration equal to or less than 0.5s, site response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2.*" In other words, for structures with a period of vibration equal to or less than 0.5 second and situated on liquefiable soils, the ASCE-7 exception allows the values of F_a and F_v for liquefiable soils be taken equal to the values of site class determined without regard to soil liquefaction.

Based on our understanding of the proposed residence, it will likely consist of a lightly loaded, two to three story wood-frame structure and the vibration period for the residence should be less than 0.5 second.

For design purposes, we recommend assuming Site Class D for determining site coefficients for the seismic design of the proposed structures.

6.2 Catchment Wall Design

A debris flow catchment wall should be constructed on the uphill side of the proposed residence. We recommend the catchment wall have a minimum freeboard height of 10 feet. Periodic maintenance of the catchment wall be required to remove accumulated debris, as the function of the wall is related to the available catchment area behind the wall. Permanent access to the back of the catchment wall should be incorporated into the layout of the planned improvements to allow for periodic removal of accumulated debris from behind the wall to maintain the minimum freeboard.

Based on the soil conditions encountered on site, and the methods required to construct various wall types, in our opinion a soldier pile wall would be appropriate for the catchment wall. A cantilevered soldier pile wall consists of vertical steel beams, typically spaced from six to eight feet apart along the proposed wall alignment, spanned by timber lagging. The steel beams are installed into holes drilled to a prescribed design depth and then backfilled with lean mix or structural concrete. Timber lagging is installed between the piles to complete the wall. A variety of facing schemes, including pre-cast and cast in place concrete, can be applied to the face of the wall to give the wall a desired aesthetic appearance.

It should be noted that due to the presence of shallow groundwater, the contractor may need to utilize temporary casing to prevent caving of the holes prior to concrete placement. In addition, depending on the amount of water that has accumulated at the bottom of holes just prior to concrete placement, tremie methods of concrete placement may also be required.

The lateral pressures depicted on Figure 5 should be used for designing a soldier pile catchment wall for the project. Above the finished grade behind the wall, the recommended debris pressures should be applied over the full width of pile spacing. Below the bottom of the finished grade, the active pressures should be applied over one pile spacing, and the passive resistance should be applied over two times the pile diameter. The soldier piles should achieve a minimum embedment of 10 feet into the silt and clay underlying the colluvium. Deeper penetration may be needed based on the structural design.

Another soldier pile wall may also be needed along the downslope side of the proposed residence to prevent sliding of the colluvium. When the house elevations are determined, we should review the depth of excavation to determine if a downslope wall is necessary and can provide design parameters.

6.3 FOUNDATIONS

6.3.1 Driven Pipe Piles (Pin Piles)

Due to the presence of 8½ to 11½ feet of very loose to loose silty sand immediately below the site, we recommend supporting the proposed residence on a pipe pile foundation To mitigate the risk of excessive total and differential settlement and the need for extensive overexcavation, driven three-, four-, and six-inch diameter steel pipe piles may be used to support the addition. The principal advantages of driven pipe piles are that the pile lengths can be easily adjusted in the field, the speed of installation, and no spoils to be disposed of. The following sections present our recommendation for pin piles.

6.3.2 Pin Pile Capacity

In our opinion, three-, four- or six-inch diameter piles will likely be the most appropriate pile sizes. The number of piles required depends on the magnitude of the design load. Table 1 below shows the recommended capacities for pin piles with an approximate factor of safety of at least 2.0.

Table 1 – Pin Pile Capacities

Pile Diameter (in)	Allowable Axial Compression (tons)
3	6
4	10
6	30

Penetration resistance required to achieve the capacities will be determined based on the hammer used to install the pile.

The tensile capacity of pin piles should be ignored in design calculations.

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

6.3.3 Pin Pile Specifications

We recommend that the following specifications be included on the foundation plan:

- The three-inch, four-inch, and six-inch diameter piles should consist of Schedule-40, ASTM A-53 Grade "A" pipe.
- The three-inch piles shall be driven to refusal with a minimum 600-lb hydraulic hammer. We recommend the following refusal criteria based on the size of hammer utilized:

Table 2 – Three-Inch Pile Refusal Criteria

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

The driving criteria recommended in the table above will be verified by a static load test program as discussed below.

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

Table 3 – Four-Inch Pile Refusal Criteria

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

  The driving criteria recommended in the table above will be verified by a static load test program discussed below.

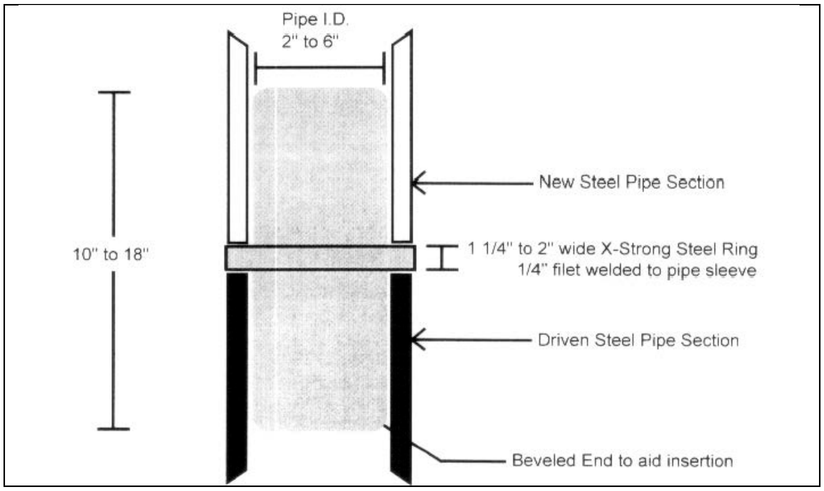
- The six-inch piles shall be driven to refusal with a minimum 2,000-lb hydraulic hammer. We recommend the following refusal criteria based on the size of hammer utilized:

Table 4 – Six-Inch Pile Refusal Criteria

Hammer Size	Approx. Blows per Minute	Refusal Criteria (4-inch pile)
2,000 lbs	600	10 seconds per inch
3,000 lbs	500	6 seconds per inch
4,700 lbs	500	4-6 seconds per inch

  The driving criteria recommended in the table above will be verified by a static load test program discussed below.

- Piles shall be driven in nominal sections and connected with compression fitted sleeve couplers (see detail on the next page – Courtesy of McDowell Pile King, Kent, WA). We discourage welding of pipe joints, particularly when galvanized pipe is used, as we have frequently observed welds broken during driving.



- At least 3 percent (but no more than 5) of the pin piles should be load tested. All load tests shall be performed in accordance with the procedure outlined in ASTM D1143. The maximum test load shall be two times the design load. The objective of the testing program is to verify the adequacy of the driving criteria, and the efficiency of the hammer used for the project.
- PanGEO should be on-site to observe of pile installation and testing.

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

6.3.4 Lateral Resistance

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

6.3.5 Estimated Pile Length

The required pile length in order to develop the recommended pile capacity is expected to vary across the footprint of the structure, depending on construction subgrades and the actual driving conditions encountered. For planning and cost estimating purposes, we estimate pile lengths will range from 15 to 25 feet.

6.3.6 Obstructions

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

6.4 FLOORS SLABS

Slab on grade floors should be supported on at least one foot of structural fill The exposed subgrade should be compacted to a firm and unyielding condition prior to placing the backfill or capillary break layer.

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 5/8-inch, clean crushed rock (less than 3 percent fines). The capillary break material should meet the gradational requirements provided in Table 5, below.

Table 5 – 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.

We recommend that 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.

6.4 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 40 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 55 pcf should be used for a level backfill condition behind the walls. Permanent walls should be designed for an additional uniform lateral pressure of 10H 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 structural fill with adequate drainage provisions.

6.4.1 Surcharge

Surcharge loads, where present, should also be included in the design of retaining walls. We recommend a lateral load coefficient of 0.4 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.

6.4.2 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. The passive resistance values provided in Section 6.3.4 Lateral Resistance may be used for retaining wall design.

6.4.3 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.

Where permanent retaining walls will be cast directly against temporary shoring, a geocomposite drain such as Miradrain 6000 may be used.

6.4.4 Wall Backfill

Where wall backfill will be needed, the backfill should consist of free draining granular soils such as WSDOT Gravel Borrow Section 9-03.9(3) (WSDOT, 2022) or an approved equivalent. On-site soils that are sandy or gravelly in nature may be re-used, provided they can be adequately compacted. The use of the on-site soils should be evaluated during construction by PanGEO. For cost estimating purposes, it may be more appropriate to assume that wall backfill, where needed, should entirely consist of imported soils.

Wall backfill should be moisture conditioned to near optimum moisture content, placed in loose, horizontal lifts less than 12 inches in thickness, and systematically compacted to a dense and relatively unyielding condition. If density tests will be performed, the test results should indicate at least 95 percent of the maximum dry density, as determined using test method ASTM D 1557. Within five feet of the wall, the backfill should be compacted to 90 percent of the maximum dry density.

6.5 On-Site Infiltration Considerations

Based on our review of the City of Mercer Island Low Impact Development (LID) infiltration feasibility map, the project site is located in an area where infiltration LID measures are not permitted.

6.6 Permanent Slope Inclinations

Permanent cut and fill slopes should be inclined no steeper than 2H:1V. Cut slopes should be observed by PanGEO during excavation to verify that conditions are as anticipated. Permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve stability of the surficial layer of soil.

6.7 Site Drainage

6.7.1 Surface Drainage

Permanent control of surface water should be incorporated into the final design. Surface water collected from impervious surfaces such as roofs, patios and walkways should be collected and separately tightlined to an appropriate discharge point. Cleanouts should be installed at strategic locations to allow for periodic maintenance of the footing drain and downspout tightline systems.

6.7.2 Subsurface Drainage

Groundwater was encountered during drilling in the upper very loose to loose silt sand soils underlying the site. With the plan to bench the proposed residence into the sloping grade, the control of groundwater will be critical element of the project design. We anticipate the use of subsurface drains, wall drainage, foundation drainage and underslab drains will need to be incorporated into the design.

- ***Subsurface Interceptor Drains:*** In order to collect and remove shallow groundwater seepage, subsurface interceptor drains should be considered. These drains would consist of a series of gravel filled trenches extending up and down the slope. The interceptor drains should be at least two feet wide and extend at least three feet below finished grade. It may be necessary extend the drains through the colluvium to adequately drain the colluvium.

  In order to prevent fines from migrating into and potentially clogging the drain, the trench should be lined with a filter fabric. For this application, the fabric should consist of Mirafi 140N or approved equivalent. A four-inch diameter perforated collection pipe should be placed in the bottom of the trench with the trench and pipe sloped to drain. The gravel backfill may consist of pea gravel or washed rock.

- ***Wall Drains:*** To reduce the potential for hydrostatic pressures to develop behind retaining walls, the retaining wall backfill should consist of a free-draining material that extends at least 18 inches behind the wall. Alternatively, a geocomposite drainage board can be used in place of the free draining backfill. Recommendations for wall drainage were provided in Section 6.4.3 of this report.

- ***Footing Drains:*** Footing drains should be installed around the perimeter of the residence, at or just below the invert of the footings. The footing drains should consist of a 4-inch diameter perforated drainpipe placed behind and at the base of the footings, embedded in 12 to 18 inches of clean crushed rock or pea gravel wrapped with a layer of filter fabric. Under no circumstances should roof downspout drain lines be connected to the footing drain systems. Roof downspouts must be separately tightlined to appropriate discharge locations. Cleanouts should be installed at strategic locations to allow for periodic maintenance of the footing drain and downspout tightline systems.

- ***Underslab Drains:*** Underslab drains should be considered below the lower level of the residence. These drains should consist of four-inch diameter perforated pipes placed in gravel filled trenches. The drains should be laid out with a horizontal spacing of about 10 feet. The trenches should be lined with a non-woven filter fabric, such as Mirafi 140N and the gravel should consist of pea gravel or washed rock.
- ***Final Site Grades:*** Site grades around the perimeter of the site should allow for drainage away from the residence foundations and away from the site slopes.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 TEMPORARY EXCAVATIONS

Temporary excavations should be constructed in accordance with Part N of the Washington Administrative Code (WAC) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring.

Temporary excavations should be evaluated in the field during construction based on actual observed soil conditions. For planning purposes, excavations should be sloped no steeper than 1H:1V (Horizontal:Vertical). 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.

Depending on the depth of the excavation to bench the residence into the site, temporary construction dewatering may need to be considered to relieve hydrostatic pressure and maintain a stable construction subgrade. A dewatering contractor should be hired during the design phase to provide a dewatering system design.

7.2 TEMPORARY SHORING

7.2.1 Soldier Pile Wall

Due to the presence of loose colluvium immediately underlying the site, we recommend temporary excavations deeper than four feet be supported using soldier piles with timber lagging. Soldier pile shoring consists of vertical steel beams, typically installed 6 to 8 feet apart along the proposed excavation, spanned by timber lagging. Prior to the start of excavation, the steel H-beams are installed in pre-drilled holes that are backfilled with structural concrete or lean mix concrete. As the excavation proceeds downward and the steel piles are exposed, timber lagging is installed between the flanges of the H-beams to support the soils exposed in the excavation. The basement walls may then be formed and cast directly against the shoring.

7.2.2 Design Lateral Pressures

The earth pressures depicted on Figure 6 should be used for design. The lateral earth pressures should be increased for surcharge loads resulting from adjacent structures, , construction equipment, or excavated soil stockpiles if they are located within the height dimension (H) of the wall.

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 diameter, and the passive resistance should be applied over two times the pile diameter. The upper two feet of the passive pressure should be neglected to allow for disturbance of the ground surface in front of the wall.

Voids behind the timber lagging should be backfilled with control density fill (CDF), sand, or on-site soils.

The joints between the lagging will allow groundwater to seep through the shoring. Drainage should be provided to collect seepage and prevent hydrostatic pressures from building up behind the wall and concrete facing. A sheet drain consisting of Miradrain 6000, or equivalent, should extend the full height of the wall and should be spaced evenly between the soldier piles. The sheet drain should be connected to a perforated collection pipe or weepholes placed along the base of the wall to remove seepage.

Vertical Capacity: We recommend the vertical capacity of the soldier piles be determined using an allowable skin friction value of 1.5 ksf for the portion of the pile below the bottom of the excavation, and an allowable end bearing value of 15 ksf.

7.2.3 Surcharge Loads

The shoring walls should be designed to accommodate surcharge pressures, including but not limited to footing loads from adjacent structures. The lateral pressure acting on the wall from surcharge loads may be calculated using the diagram shown in the attached Figure 7.

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. Surcharge loads including construction equipment or soil stockpile located within the height dimension of the wall should also be considered in the shoring design.

7.2.4 Timber Lagging

The design of timber lagging may be based on Section 5.4.2 of the *Geotechnical Engineering Circular No. 4 Ground Anchors and Anchored System* published by FHWA (1999). Where heavy surcharge loads are not present, the required minimum timber lagging thickness for different excavation depths and pile spacings are summarized in Table 5, below.

Table 5 – Minimum Timber Lagging Thickness

Depth Below Ground Surface	Thickness of Lagging (rough cut) for clear spans of:	
	6 feet pile spacing	8 feet pile spacing
0 to 25 feet	3-inch-thick lagging	3-inch-thick lagging

The tabulated values above are not applicable where heavy surcharge loads are present. In this event, the lagging design shall be evaluated on a case-by-case basis.

7.2.5 Baseline Survey and Monitoring

Ground movements will occur 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 during excavation. The monitoring frequency may be reduced once a week after the foundations are cast against the base of the walls and provided the surveyed shoring deflections are within tolerable limits.
- Adjacent buildings within 25 feet of the excavation and shoring.

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, 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 long the public right-of-way and the adjacent private properties be photo-documented prior to commencing on any earthwork at the site.

7.3 STRUCTURAL FILL AND COMPACTION

Structural fill should consist of a well-graded granular material such as WSDOT Gravel Borrow, 9-03.14(1) (WSDOT, 2022). Structural fill should be placed in 8- to 12-inch-thick loose lifts and compacted. If the fill will be tested for compaction, the fill should be compacted to at least 95 percent maximum dry density, per ASTM D-1557 (Modified Proctor). In non-structural areas, the recommended compaction level may be reduced to 90 percent of the Modified Proctor.

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.

The surficial topsoil layer is not suitable for use as structural fill, nor should it be mixed with materials to be used as structural fill.

7.4 MATERIAL REUSE

The native soils underlying the site are moisture sensitive will become disturbed and soft when exposed to inclement weather conditions and construction traffic. For planning purposes, 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.

7.5 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 3/4-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.

7.6 Erosion Considerations

Surface runoff can be controlled during construction by careful grading practices. The erosion control plan should include measures for reducing concentrated surface runoff and protecting disturbed or exposed surfaces by mulching and revegetation. The temporary erosion and sediment control (TESC) plan should include the following:

- Construction activity should be scheduled or phased as much as possible to reduce the amount of earthwork that is performed during the wet season – October through May.
- The TESC plan should include adequate ground cover-measures, access roads, and staging areas. The contractor should be prepared to implement and maintain the TESC measures to maximize the effectiveness of the TESC elements.
- Where practical, a buffer of vegetation should be maintained around cleared areas.
- The TESC measures should be installed in conjunction with the initial ground clearing. The recommended sequence of construction within a given area after clearing would be to install silt fences and straw waddles around the site perimeter prior to starting mass grading.
- In areas where grading is complete, hydroseed or straw mulch should be placed.
- During the wet season, or when large storm events are predicted during the summer months, work areas should be stabilized so that if showers occur, the work area can receive the rainfall without excessive erosion or sediment transport. Areas that are to be left un-worked for more than two days should be covered with straw mulch or plastic sheeting.
- Soils that are to be stockpiled on-site should be covered with plastic sheeting staked and sandbagged in place.

The erosion control measures should be reviewed, adjusted and maintain on a regular basis to verify they are functioning as intended.

8.0 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. PanGEO can provide you a cost estimate for construction monitoring services at a later date.

9.0 LIMITATIONS

We have prepared this report for use by Peak Builders and their designers and consultants. Conclusions and 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 work.

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 work specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances.

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.

We appreciate the opportunity to be of service.

Sincerely,

State of Washington
Engineering Geologist
212
Licensed Geologist
Scott D. Dinkelman

Siew Lan Tan
State of Washington
30342
Registered
Professional Engineer
August 17, 2022

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10.0 REFERENCES

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

Troost, K.G., and Wisher, A. P, 2006. *Geologic Map of Mercer Island, Washington, scale 1:24,000*.

WSDOT, 2022, Standard Specifications for Road, Bridges, and Municipal Construction.

Base Map: WSDOT GeoPortal

Not-To-Scale

PanGEO INCORPORATED

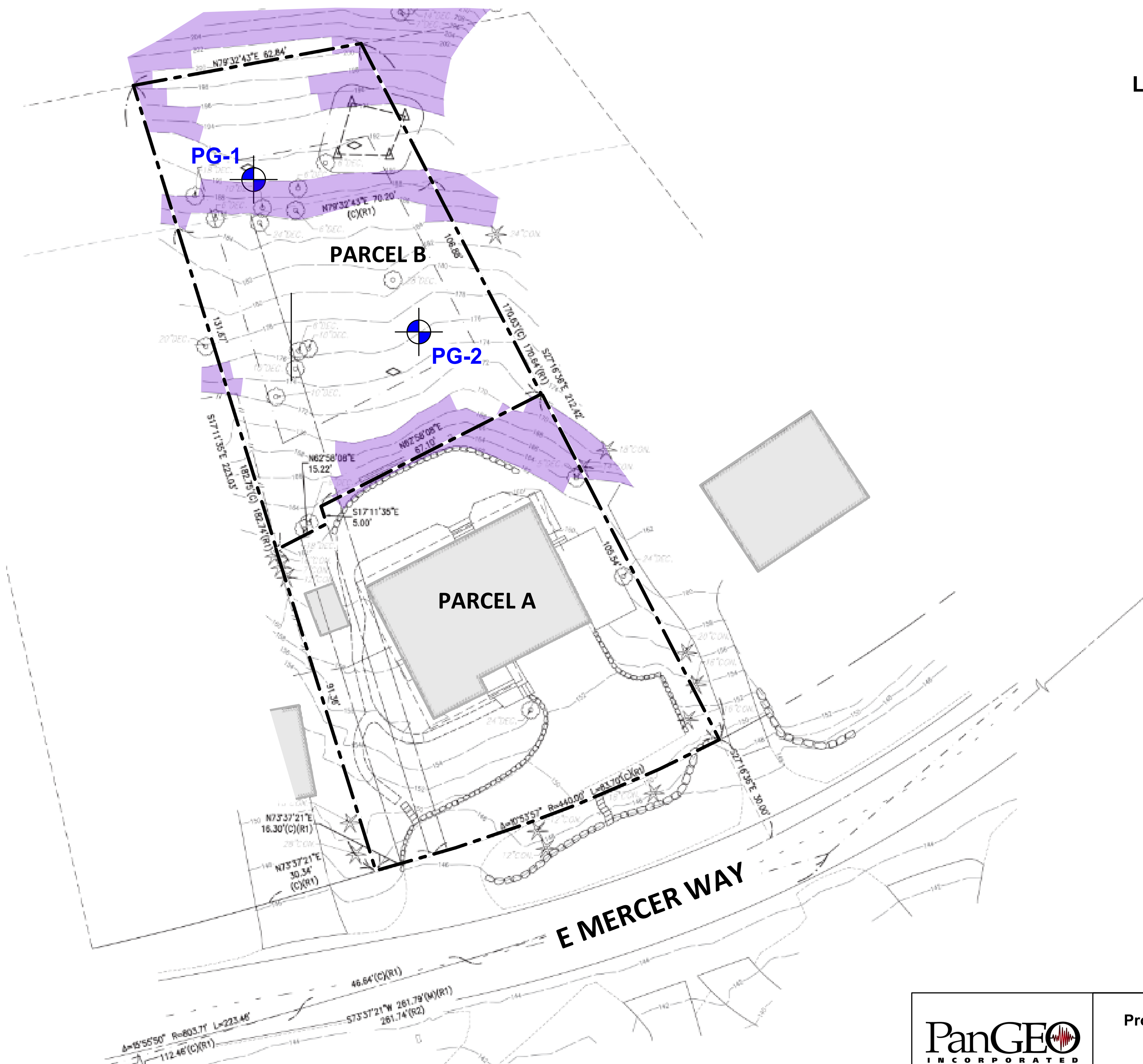
Proposed Short Plat and Residence
8247 E Mercer Way
Mercer Island, Washington

VICINITY MAP

Project No. **22-256**

Figure No. **1**

file.grf w/ file.dat 8/16/22 (3:07:14) SDD

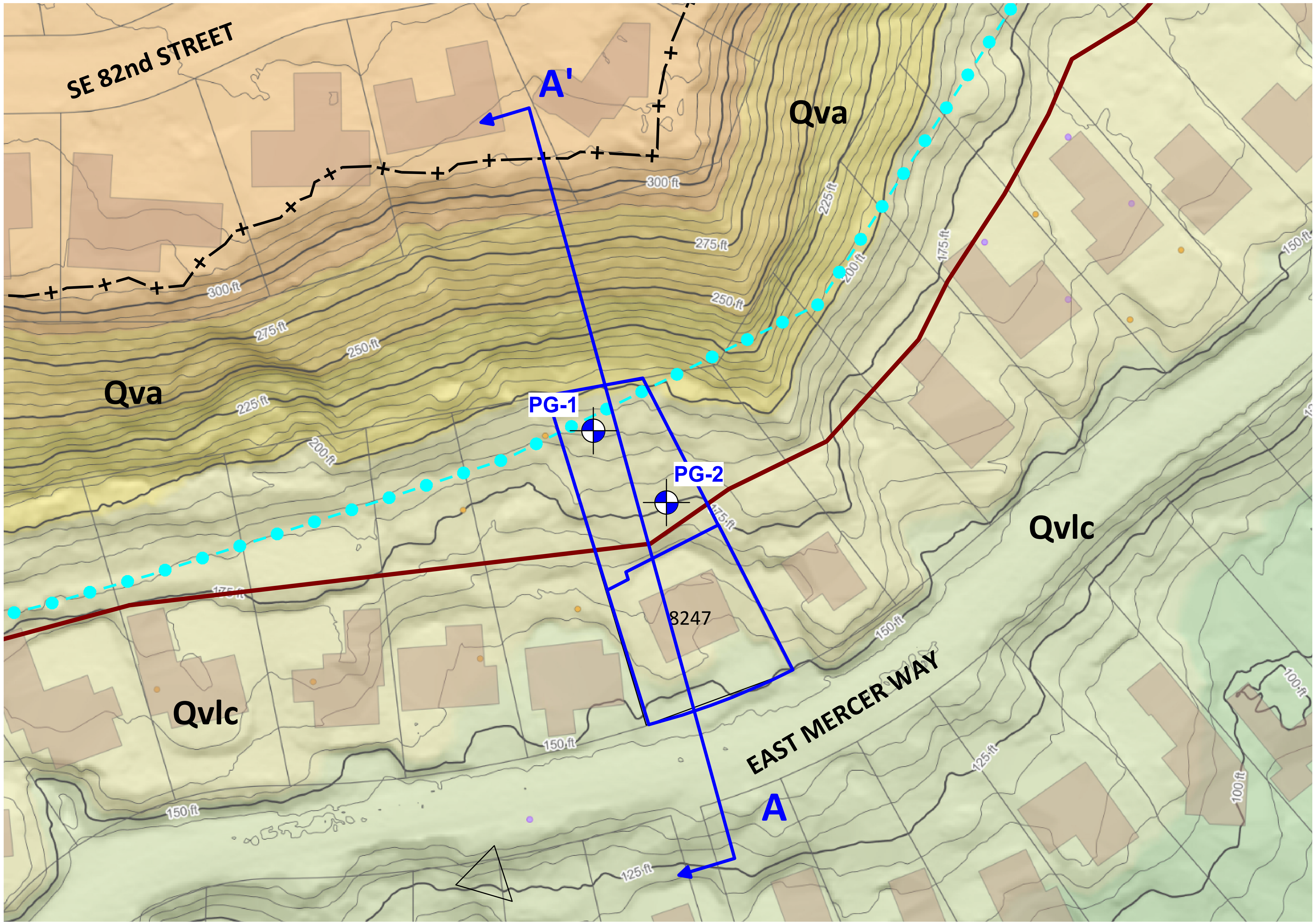


LEGEND:

- Subject Site
- Existing Structures
- Approximate Boring Location, PanGEO, Inc., June 2022

PanGEO INCORPORATED	Proposed Short Plat and Residence 8247 E Mercer Way Mercer Island, Washington	SITE AND EXPLORATION PLAN
		Project No. 22-256 / Figure No. 2

13-011_Fig 2 Site & Exploration Plan.grf 8/16/22 JCR



LEGEND:

Subject Site

GEOLOGY:
(Mapped by Mercer Island)

Scarp

Spring Line

Geologic Contact

Qva Vashon Advance Outwash, dense to very dense silt and sand

Qvlc Vashon Advance Outwash, dense to very dense silt and sand

Approx. Scale (feet)
0 35 70

NORTH

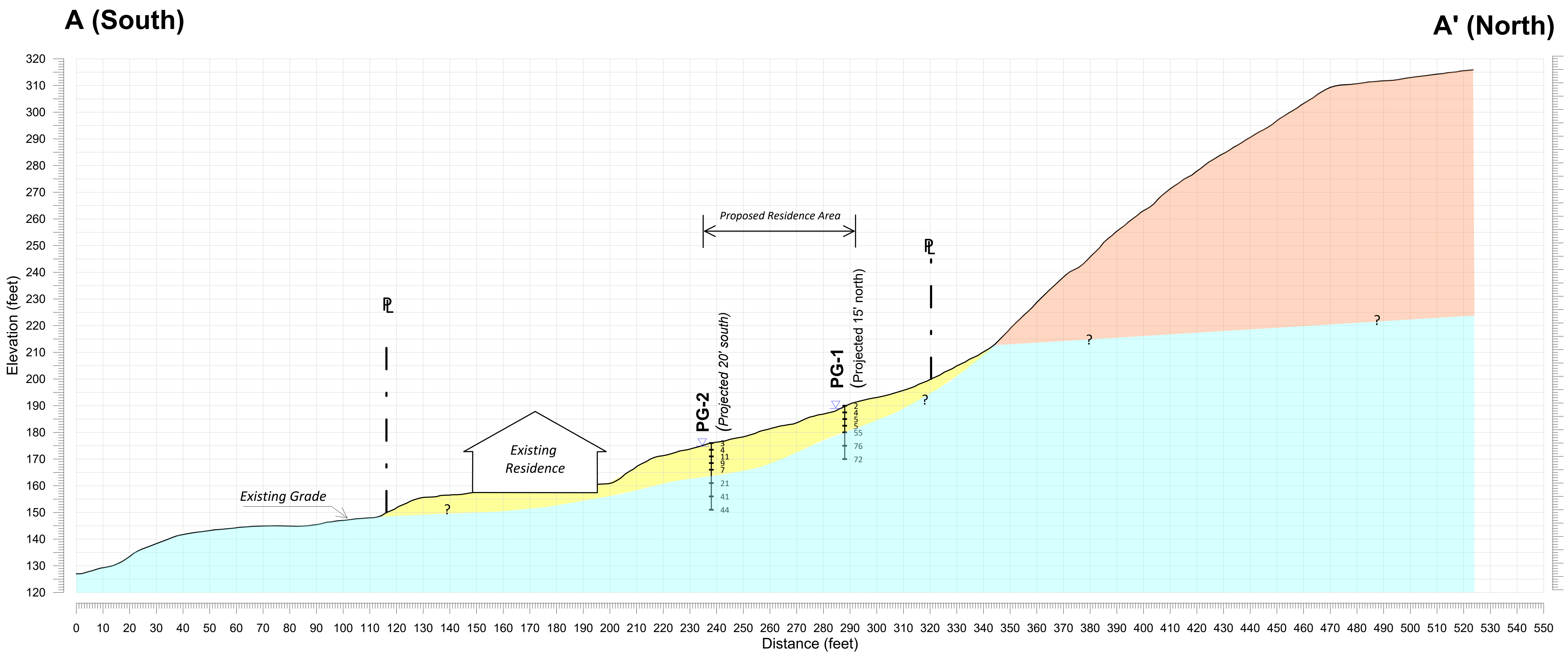
PanGEO INCORPORATED

Proposed Short Plat and Residence
8247 E Mercer Way
Mercer Island, Washington

REGIONAL TOPOGRAPHY AND GEOLOGY

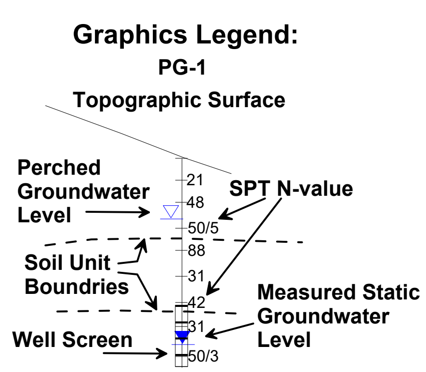
Project No.	Figure No.
21-559.200	3

13-011_Fig 2 Site & Exploration Plan.grf 8/16/22 JCR



Scale 1"=20'
Horizontal:Vertical

LEGEND:



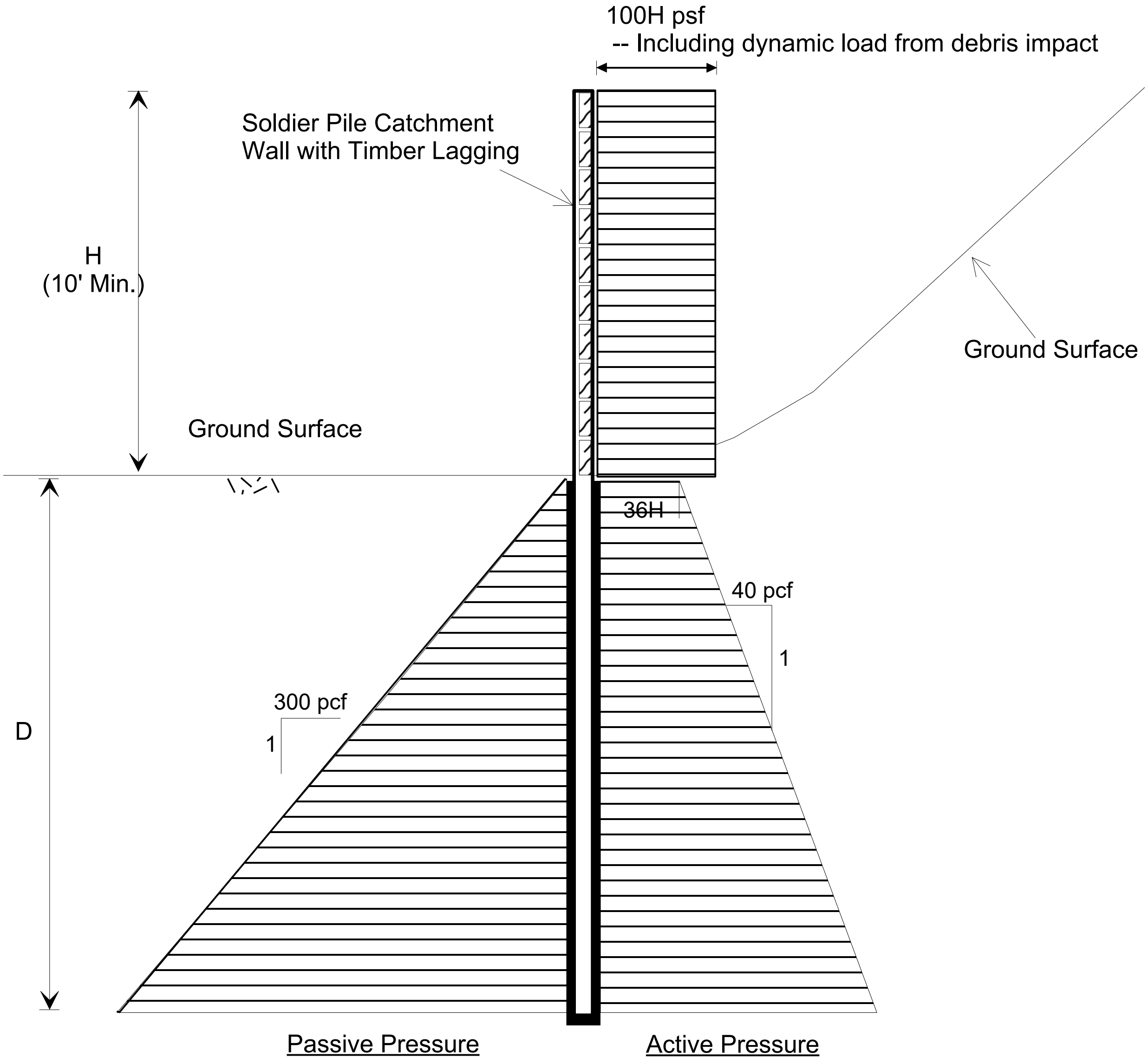
Subsurface Units:

- **Colluvium** - Very loose to loose, silty SAND
- **Advance Outwash** - Dense to very dense SAND with silt
- **Lawton Clay** - Very stiff, laminated to massive clayey SILT to silty CLAY

NOTES:

1. Elevations are based on the King County West 2021 LiDAR from WA DNR
2. Vertical datum NAVD88
3. See Geotechnical Report for detailed description of Soil Units

PanGEO INCORPORATED	Proposed Short Plat and Residence 8247 E Mercer Way Mercer Island, Washington	GENERALIZED SUBSURFACE PROFILE A-A'	
		Project No. 22-256	Figure No. 4

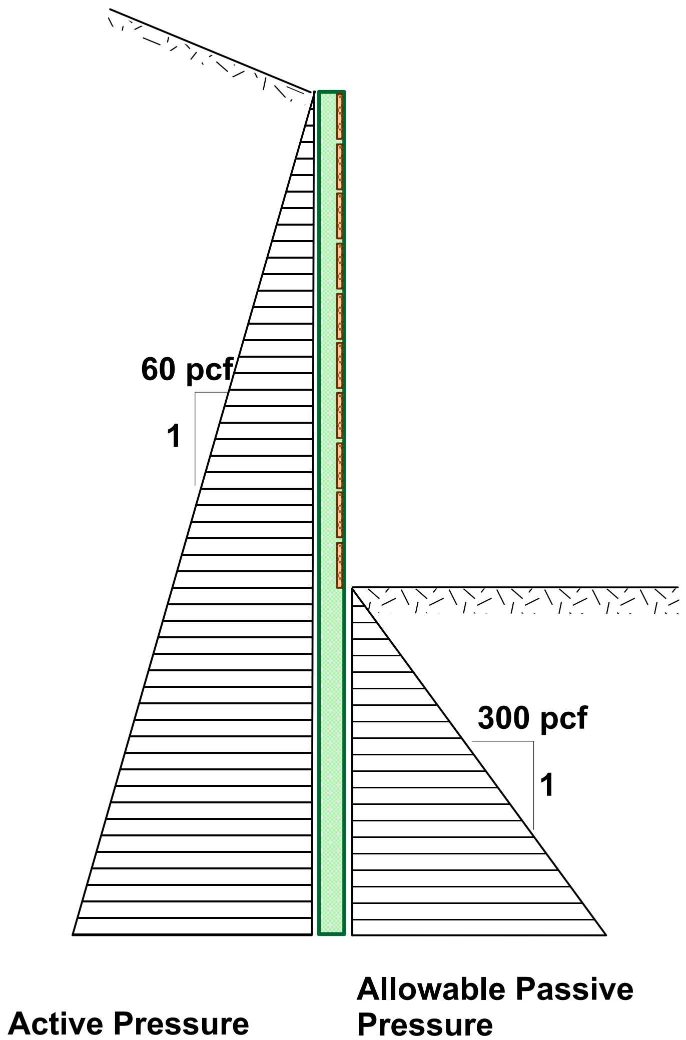


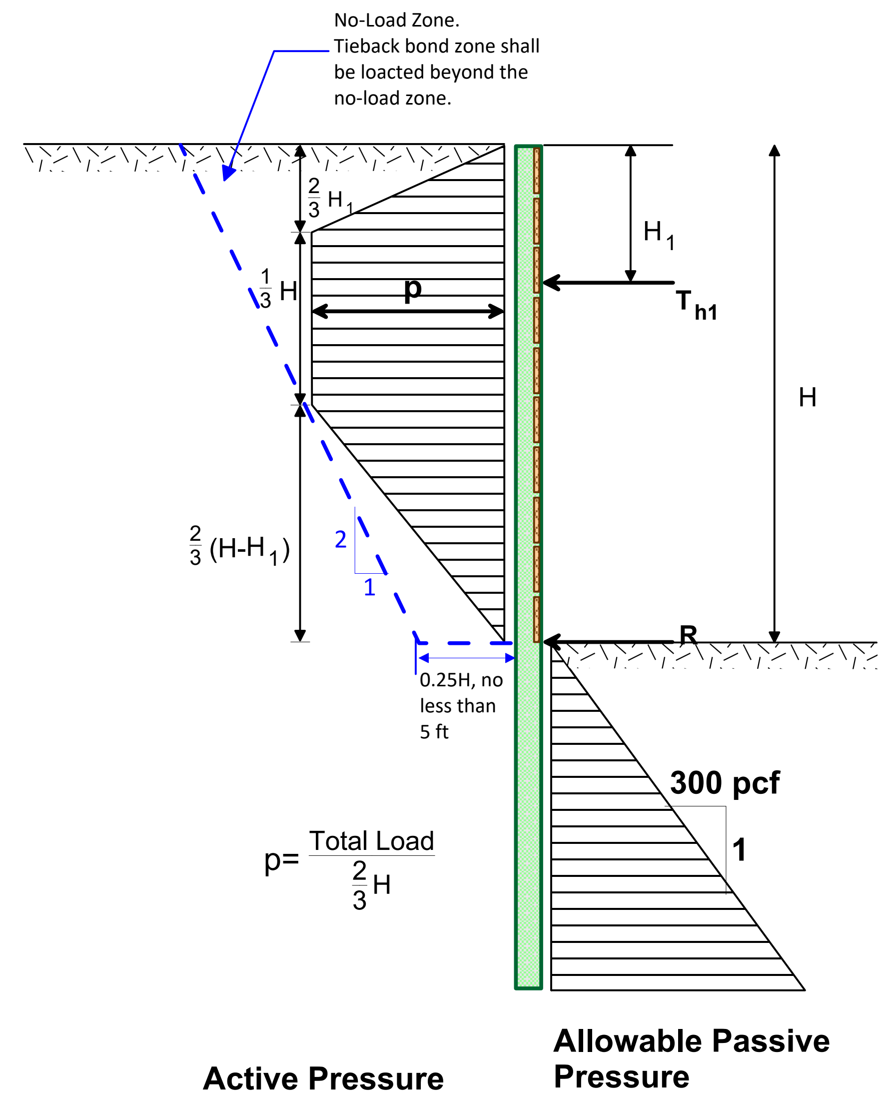
Notes:
1. Embedment (D) should be determined by summation of moments at the bottom of the soldier piles. Minimum embedment should be at least 15 feet.
2. A factor of safety of 1.5 has been applied to the recommended passive earth pressure values. No factor of safety has been applied to the recommended active earth pressure or debris pressure values.
3. Debris pressures should be applied over the full width of the pile spacing.
4. Active pressures should be applied over one pile diameter below the finished grade elevation.
5. Passive pressures should be applied to two times the diameter of the soldier piles.
6. Use full uniform earth pressure of 50H psf for lagging design.
7. Refer to report text for additional discussions.

Fig 4 EP diagram.grf 8/16/22 (3:07:51) JCR

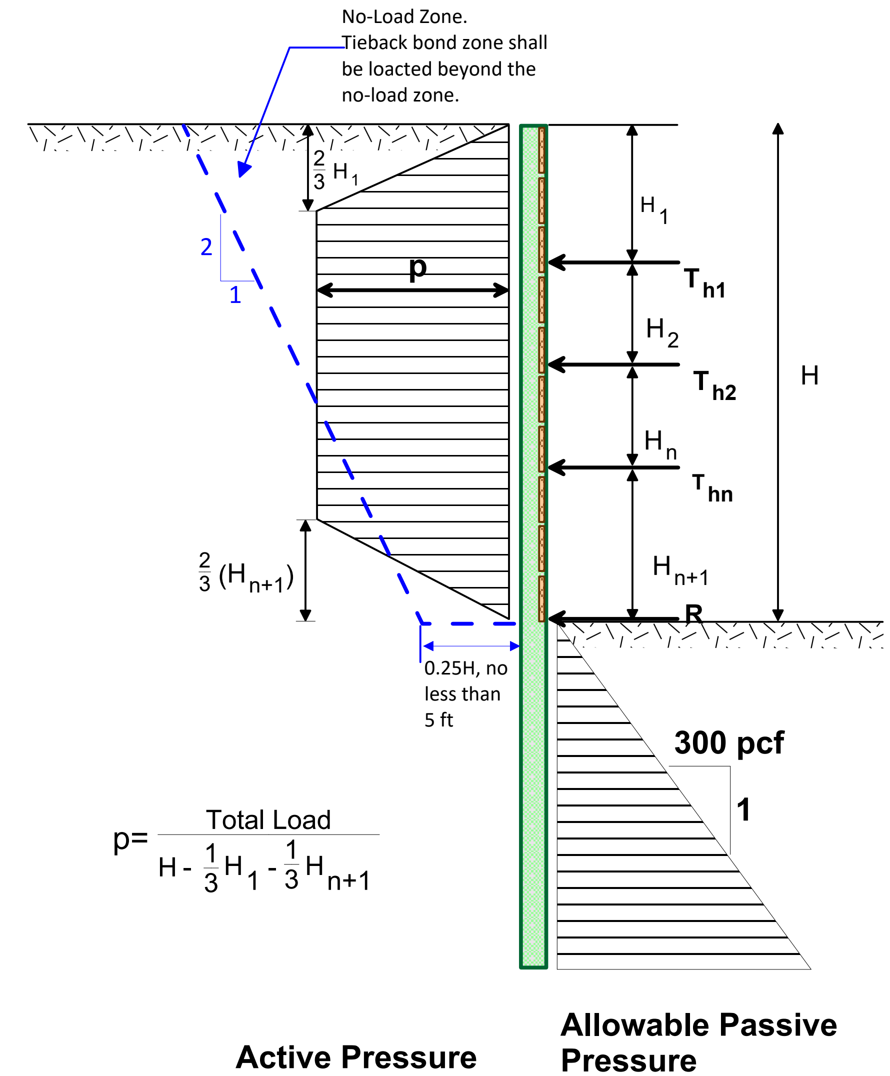
PanGEO INCORPORATED

PanGEO INCORPORATED	**Proposed Short Plat and Residence 8247 E Mercer Way Mercer Island, Washington**	**DESIGN LATERAL PRESSURES SOLDIER PILE CATCHMENT WALL** Project No. **22-256** Figure No. **5**





$\boxed{\text{Total Load} = 0.65\,K_A\,\gamma\,H^2 = 39H^2}$



$\boxed{\text{Total Load} = 0.65\,K_A\,\gamma\,H^2 = 39H^2}$

a) Cantilever Wall

b) Walls with One Level of Ground Anchors

c) Walls with Multiple Levels of Ground Anchors

Legend:

H_1 = Distance from the ground surface to uppermost ground anchor.

H_{n+1}= Distance from the base of excavation to lowermost ground anchor.

T_{hi}= Horizontal load in ground anchor i.

R= Reaction force to be resisted by subgrade (i.e., below the base of excavation).

p= maximum ordinate of diagram.

K_A= Active earth pressure coefficient.

γ = Soil unit weight (pcf).

PanGEO INCORPORATED	Proposed Short Plat and Residence 8247 E Mercer Way Mercer Island, Washington	Apparent Earth Pressures Temporary Soldier Pile Walls	
		Project No. 22-256	Figure No. 6

APPENDIX A

SUMMARY BORING LOGS

RELATIVE DENSITY / CONSISTENCY

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

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Notes:

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

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

DESCRIPTIONS OF SOIL STRUCTURES

Layered:	Units of material distinguished by color and/or composition from material units above and below	**Fissured:**	Breaks along defined planes
Laminated:	Layers of soil typically 0.05 to 1mm thick, max. 1 cm	**Slickensided:**	Fracture planes that are polished or glossy
Lens:	Layer of soil that pinches out laterally	**Blocky:**	Angular soil lumps that resist breakdown
Interlayered:	Alternating layers of differing soil material	**Disrupted:**	Soil that is broken and mixed
Pocket:	Erratic, discontinuous deposit of limited extent	**Scattered:**	Less than one per foot
Homogeneous:	Soil with uniform color and composition throughout	**Numerous:**	More than one per foot
		BCN:	Angle between bedding plane and a plane normal to core axis

COMPONENT DEFINITIONS

COMPONENT	SIZE / SIEVE RANGE	COMPONENT	SIZE / SIEVE RANGE
Boulder:	> 12 inches	Sand	
Cobbles:	3 to 12 inches	Coarse Sand:	#4 to #10 sieve (4.5 to 2.0 mm)
Gravel		Medium Sand:	#10 to #40 sieve (2.0 to 0.42 mm)
Coarse Gravel:	3 to 3/4 inches	Fine Sand:	#40 to #200 sieve (0.42 to 0.074 mm)
Fine Gravel:	3/4 inches to #4 sieve	Silt	0.074 to 0.002 mm
		Clay	<0.002 mm

TEST SYMBOLS

for In Situ and Laboratory Tests listed in "Other Tests" column.

ATT	Atterberg Limit Test
Comp	Compaction Tests
Con	Consolidation
DD	Dry Density
DS	Direct Shear
%F	Fines Content
GS	Grain Size
Perm	Permeability
PP	Pocket Penetrometer
R	R-value
SG	Specific Gravity
TV	Torvane
TXC	Triaxial Compression
UCC	Unconfined Compression

SYMBOLS

Sample/In Situ test types and intervals

- 2-inch OD Split Spoon, SPT (140-lb. hammer, 30" drop)
- 3.25-inch OD Spilt Spoon (300-lb hammer, 30" drop)
- Non-standard penetration test (see boring log for details)
- Thin wall (Shelby) tube
- Grab
- Rock core
- Vane Shear

MONITORING WELL

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

MOISTURE CONTENT

Dry	Dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

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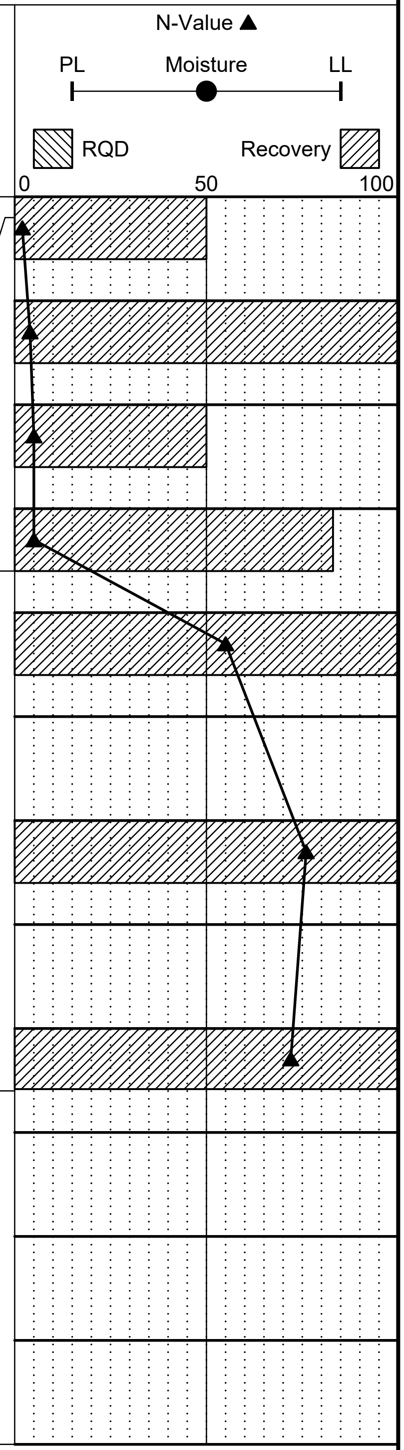
Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	8247 E Mercer Way, Mercer Island	Surface Elevation:	~190
Job Number:	22-256	Top of Casing Elev.:	N/A
Location:	8247 E Mercer Way, Mercer Island	Drilling Method:	HSA
Coordinates:	Northing: 47.52957, Easting: -122.22297	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0	S-1		1 1 1			Loose, dark brown, silty SAND with organics and trace fine gravel, moist to wet (Topsoil).
						[COLLUVIUM]
2.5	S-2		1 1 3			Very loose, grey, silty fine to medium SAND, trace gravel, trace iron-oxide staining, wood debris; moist to wet. *-- groundwater encountered near ground surface to approximately 10 feet below existing grade.*
5.0	S-3		2 2 3			*-- contains wood debris.* *-- 6-inch thick layer of organics at about 2.5 feet.* *-- contains silt pockets.*
7.5	S-4		2 2 3			
						[LAWTON CLAY - Qvlc]
10.0	S-5		9 21 34			Hard, grey, SILT, trace sand; laminated to blocky texture; non-plastic; wet to moist. *-- sample becomes moist below about 10 feet.*
12.5						
15.0	S-6		22 32 44			*-- sample becomes massive in texture.*
17.5						
20.0	S-7		9 22 50			*-- sample becomes laminated to massive in texture.*
						Boring terminated at 21.5 feet below grade. Groundwater encountered near existing ground surface to about 10 feet below grade at time of drilling.
22.5						
25.0						
27.5						
30.0						



Completion Depth:	21.5ft
Date Borehole Started:	6/21/22
Date Borehole Completed:	6/21/22
Logged By:	S. Scott
Drilling Company:	CN Drilling

Remarks: Borings drilled using Acker Soil Mechanic drill rig. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. This surface elevation is estimated from topographic survey prepared by Lanktree Land Surveying, Inc., dated January 12, 2022.**Vertical Datum: NAVD 88. Horizontal Datum: WGS 84**

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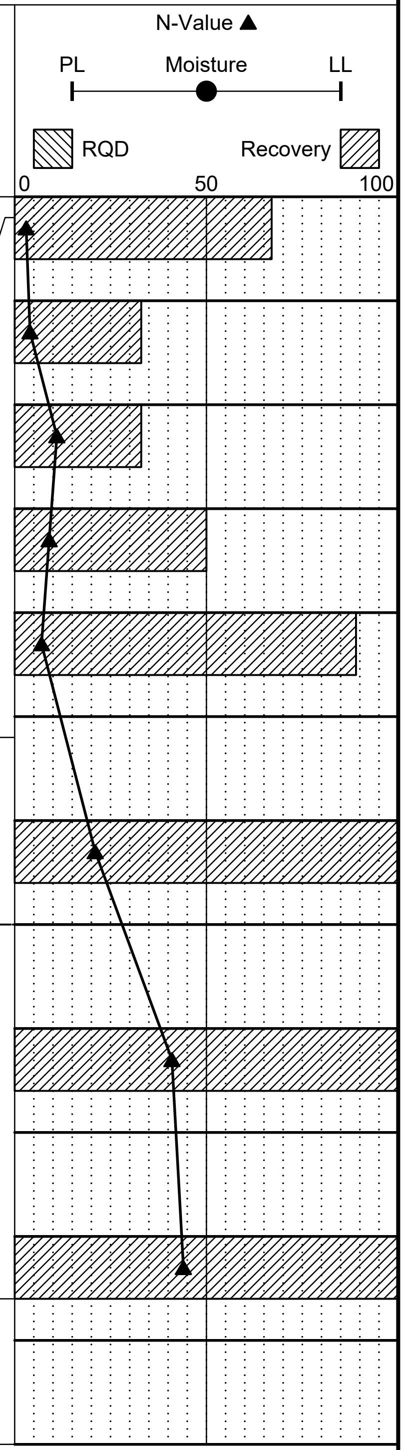
LOG OF TEST BORING PG-1

Figure A-2

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

Project:	8247 E Mercer Way, Mercer Island	Surface Elevation:	~176
Job Number:	22-256	Top of Casing Elev.:	N/A
Location:	8247 E Mercer Way, Mercer Island	Drilling Method:	HSA
Coordinates:	Northing: 47.52967, Easting: -122.22269	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0	S-1		1 2 1			Loose, dark brown, silty SAND with organics and trace fine gravel, moist (Topsoil).
						[COLLUVIUM] Very loose to loose, grey brown, silty fine to medium SAND, trace gravel; iron-oxide staining; blocky to disturbed texture; moist to wet.
2.5	S-2		2 2 2			*-- contains burnt wood fragments.*
						-- groundwater encountered beginning at approximately 2 feet to 11.5 feet below existing grade.
5.0	S-3		3 3 8			
7.5	S-4		2 4 5			
10.0	S-5		2 4 3			*-- sample becomes moist below about 11.5 feet.*
12.5						*-- drilling resistance increases at 13 feet.*
						[LAWTON CLAY - Qvlc] Very stiff, grey, clayey SILT to silty CLAY; laminated to massive texture; low plasticity; moist.
15.0	S-6		3 9 12			
17.5						Hard, grey, SILT, trace fine sand, massive texture; non-plastic; moist.
20.0	S-7		13 17 24			
22.5						
25.0	S-8		9 16 28			*-- sample becomes laminated in texture.*
27.5						Boring terminated at 26.5 feet below grade. Groundwater encountered from 2 feet below existing ground surface to about 11.5 feet below grade at time of drilling.
30.0						



Completion Depth:	26.5ft
Date Borehole Started:	6/21/22
Date Borehole Completed:	6/21/22
Logged By:	S. Scott
Drilling Company:	CN Drilling

Remarks: Borings drilled using Acker Soil Mechanic drill rig. Standard Penetration Test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. This surface elevation is estimated from topographic survey prepared by Lanktree Land Surveying, Inc., dated January 12, 2022.**Vertical Datum: NAVD 88. Horizontal Datum: WGS 84**

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LOG OF TEST BORING PG-2

Figure A-3

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