PanGEO
INCORPORATED
Geotechnical & Earthquake
Engineering Consultants

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Project No. 23-223

Rahul Pathak and Severine Kelley
8541 Southeast 82nd Street
Mercer Island, Washington 98040

Subject: Geotechnical Report
Proposed Remodel and Alterations
8541 Southeast 82nd Street, Mercer Island, Washington

Dear Rahul and Severine:

As requested, PanGEO has completed a geotechnical study for the proposed remodel and alterations to your residence at 8541 Southeast 82nd Street 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 7 to 8 feet of very loose to medium dense silty sand which we interpreted as fill or colluvium, overlying medium dense sand with gravel which we interpreted as native Advance outwash deposits. Groundwater was not encountered in our test borings at the time of drilling.

Based on the conditions encountered at our exploration locations, from a geotechnical engineering perspective, new footings may consist of conventional shallow footings. Several feet of footing over-excavation to remove the existing fill/colluvium below the footing subgrade elevations may be needed to reach the competent bearing soils. Alternatively, small diameter driven pipe (pin) piles or helical piers may be used to support the footings and eliminate the need for extensive over-excavations and backfill.

Based on our understanding of subsurface conditions and the project, in our opinion the proposed alterations will not adversely impact the steep slope on the south side of the existing house.

We appreciate the opportunity to assist you with this project. Should you have any questions, please do not hesitate to call.

Sincerely,

Siew L. Tan, P.E.
Principal Geotechnical Engineer
stan@pangeoinc.com

Encl.: Geotechnical Report

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GEOTECHNICAL REPORT
PROPOSED REMODEL AND ADDITIONS
8541 SOUTHEAST 82ND STREET
MERCER ISLAND, WASHINGTON

1.0 INTRODUCTION

PanGEO, Inc. is pleased to present this geotechnical report to support the design and construction of the proposed remodel and alterations to your residence at 8541 Southeast 82nd Street in Mercer Island, Washington. This study was performed in general accordance with our mutually agreed scope of services outlined in our proposal dated June 21, 2023 which was subsequently signed on July 10, 2023. Our scope of services included reviewing readily available geologic and geotechnical data, conducting a site reconnaissance, drilling 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 an approximately 8,000 square feet lot located at 8541 Southeast 82nd Street in Mercer Island, Washington, approximately as shown on the attached *Figure 1, Vicinity Map*. The site is roughly trapezoidal in shape, and is bordered to the north by Southeast 82nd Street, to the east and west by single-family residences, and to the south by an undeveloped city parcel which contains a heavily vegetated steep slope. The upper (northern) portion of the subject site is currently developed with a single-story residence with a daylight basement, attached carport, and asphalt driveway. The layout of the site is shown on the attached *Figure 2, Site and Exploration Plan*.

Based on review of the project topographic survey, the site gradient in the northern portion of the site slopes relatively gently down from the northeast towards the south/southwest at an average gradient of about 10 to 11 percent. However, approximately 20 to 25 feet south of the existing house, the site grade descends steeply to the south at gradients exceeding 40 percent (see topographic contours on *Figure 2*). Total vertical relief across the site is on the order of about 30 feet. Beyond the south property line, the slope continue to descend an additional 140 feet down to East Mercer Way.

The site is generally vegetated with mature evergreen trees up to 40 inches in diameter, lawns, ivy, and small landscaping trees and shrubs. Plates 1 and 2 on the next page show current site conditions at the site.

Plate 1: *View of the north side of the site looking south from Southeast 82nd Street.*

Plate 2: *View of south side of the existing residence looking from south to north.*

We understand that you plan to remodel and make alterations to your existing residence at the site. Potential alterations include converting the existing carport to a new attached garage, and adding some new post footings and moment frame columns below the existing deck on the southwest corner of the residence. The proposed alterations are shown on the attached *Figure 2*.

We understand that the alterations would not significantly increase the footprint of the overall structure. Finally, we anticipate that the proposed project would involve cuts and fills on the order of about 5 to 6 feet, depending on actual footing elevations.

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 August 14, 2023. Borings PG-1 and PG-2 were drilled to depths of about 20 and 31½ feet below existing grades, respectively. The approximate boring locations were taped relative to existing features and are shown on the attached *Figure 2*.

The drill rig was equipped with 5-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), and as shown on *Plate 3* on Page 7, the surficial geologic unit at the site consist of Vashon advance outwash (Geologic Map Unit Qva). 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.

The steep slope south of the site is also identified as being underlain by landslide deposits and Vashon Lawton Clay (Qvlc). Landslide deposits are described as very loose to very dense and soft to hard, diamict of broken to internally coherent surficial deposits transported down slope *en masse* by gravity. Lawton Clay typically consists of very stiff to hard, laminated to massive silt and clay deposited in proglacial lakes early in the Vashon glaciation.

In addition, a landslide scarp is mapped at the top of the steep slope on the south side of the site.

4.2 Soil Conditions

In general, our test borings encountered colluvium comprised of very loose to medium dense silty sand overlying medium dense undisturbed Advance outwash sand, which is generally consistent with the mapped geology. None of our test borings at the site encountered the Lawton Clay deposits mapped in the vicinity of the site.

The subsurface conditions encountered in our borings and the topography were used to develop the generalized subsurface profile included as *Figure 3, Generalized Subsurface Profile A-A'*. It should be noted that the stratigraphic contacts indicated on the boring logs and our subsurface profile represent the approximate depth to boundaries between soil units. Actual transitions between soil units may be more gradual or occur at different elevations.

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.

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

Fill/Colluvium – Below the topsoil, we encountered very loose to medium loose silty sand with varying amounts of gravel and organics. Based on the relatively loose consistency and disturbed or discolored texture we interpreted this soil to consist of either uncontrolled fill or colluvium, which is soil that has been deposited at the base of a slope by mass wasting and erosional processes. This soil unit ranged from 7 feet thick at Boring PG-2 to 8 feet thick at Boring PG-1.

Advance Outwash (Qva) – Below the fill/colluvium layer, our test borings encountered medium dense, silty sand and poorly-graded sand with silt with varying amounts of gravel that extended to the termination depth of 20 feet and 31½ feet below existing grades in PG-1 and PG-2, respectively. We interpreted this soil to be consistent with the Advance outwash deposits mapped at the site. The Advance outwash deposits generally exhibited a massive (i.e., featureless) soil structure. Test boring PG-1 met practical refusal on a large rock or a cobble at 20 feet depth in this soil unit.

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

Groundwater was not observed within the maximum depths of our test borings at the time of drilling. Additionally, during our field exploration we did not note the presence of hydrophytic or water loving plants like horsetails at the site. However, the City of Mercer Island has mapped a line of springs along the slope on the southeast portion of the island, just south of the subject site. The approximate location of the spring line is shown on *Plates 4 and 5* on Page 8. Based on the geologic map and our experience in the area, and as shown on *Figure 3*, we infer that the spring line is located at the contact between the Advance outwash and the underlying Lawton clay (Qvlc), which is layer of fine-grained silts and clays characterized by its low permeability characteristics that typically acts like an aquitard in the Seattle area. However, based on our test borings and anticipated excavation depths, we do not anticipate that groundwater will result in significant construction related issues.

The designers and contractor should 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 located in a prehistoric landslide. The approximate extent of the landslide relative to the site is shown in *Plate 3* below.

Plate 4, on the following page, shows the approximate extent of landslide hazard areas mapped by the City of Mercer Island. Based on review of the City's mapping, the majority of the site is located in a landslide hazard area and potential landslide hazard area. Additionally, the slope on the south side of the property and beyond the property line is also mapped as a 40 percent and steeper slope area. The approximate extent of the 40 percent slopes mapped by Mercer Island are shown on *Plate 5*, also on the following page.

To the best of our knowledge, there are no documented past known slides at the subject site. An identified landslide is mapped at the property to the east of the subject site (8551 Southeast 82nd Street). According to the City's records, there is no documentation for the identified landslide, and PanGEO was not able to visually inspect the area to confirm the presence of a landslide. However, the mapped landslide appears to be located approximately 130 feet east of the proposed construction area.

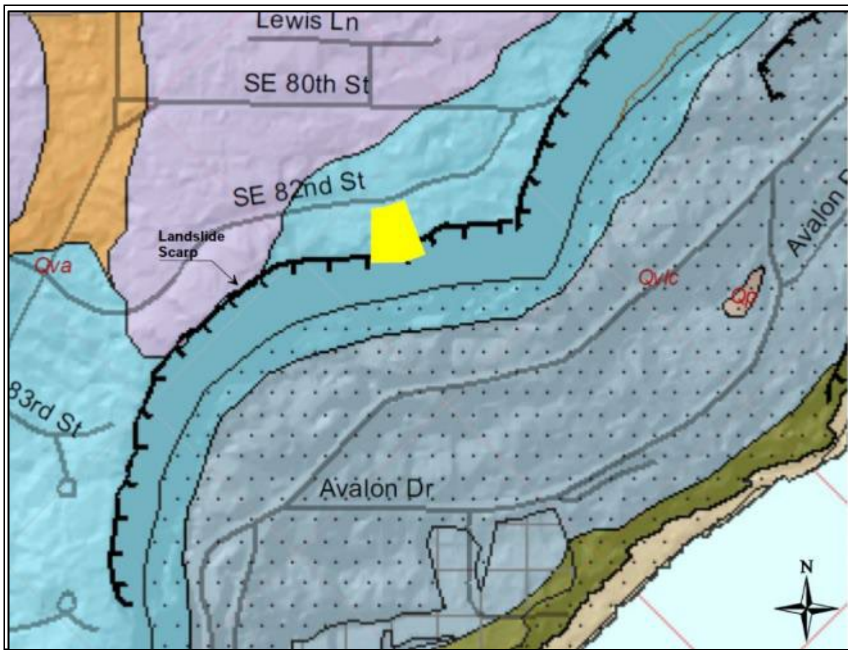


Plate 3: *Geologic map of the site vicinity showing the prehistoric (ancient) landslide scarp on south side of site.*

The approximate site location is delineated by a yellow rectangle.

-- image source Geologic Map of Mercer Island, Washington

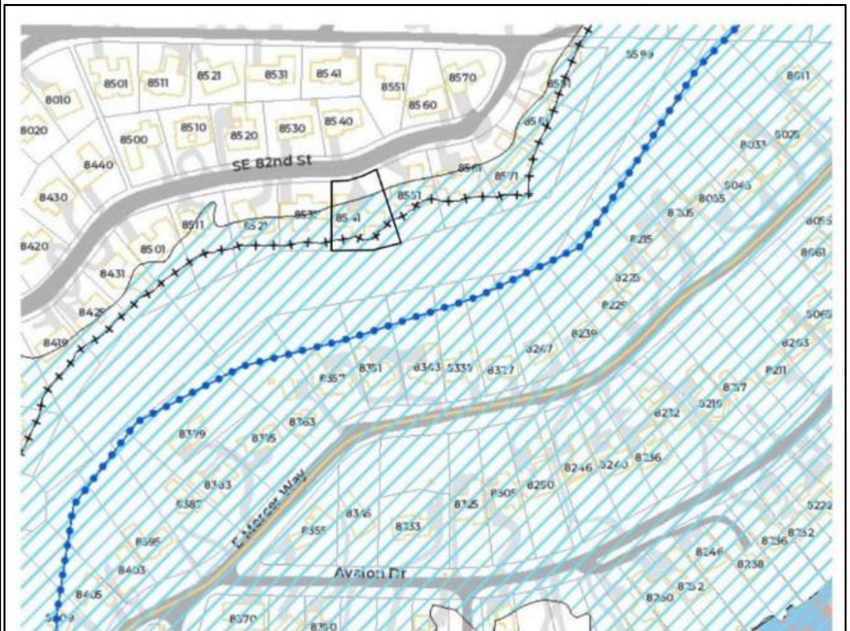


Plate 4: *The blue hatched area is the approximate extent of the Landslide and Potential Landslide Area identified by Mercer Island.*

The site is outlined in black. Approximately south three quarters of the site is located within the potential landslide area mapped by the City of Mercer Island.

The blue line of dots shows springs mapped by the City of Mercer Island.

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 the site and vicinity is included in *Plate 6* below.

In the LiDAR imagery, the ridge line outlining the top of the prehistoric 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 shadowed arcuate shape on the slope below the subject site is likely a more recent slope failure. However, we did not see evidence of recent landslides within the property limits.

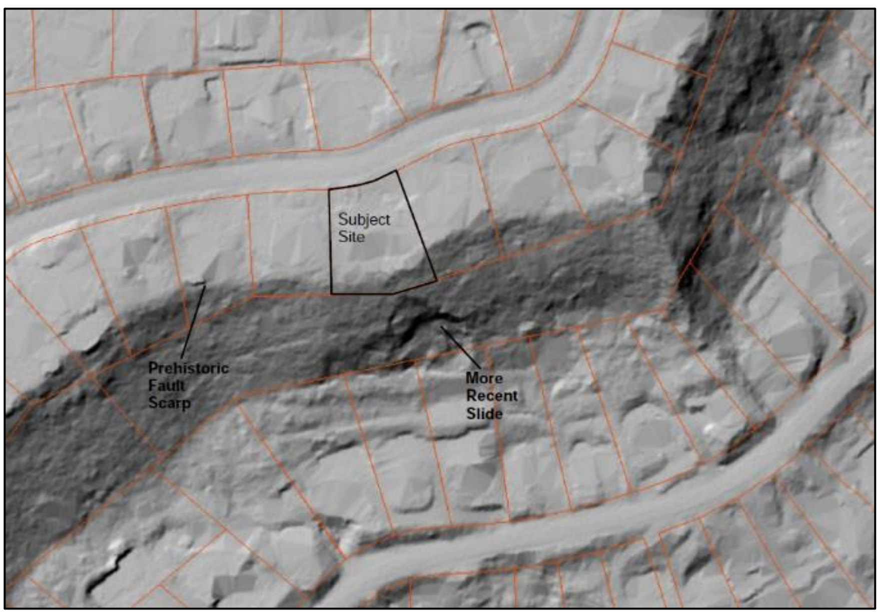


Plate 6: *LiDAR imagery for the site and vicinity.*

The site is outlined in black.

-imagery modified from Washington State Department of Natural Resources LiDAR Portal.

5.1.3 Site Reconnaissance

We conducted a reconnaissance of the site and site slopes on August 14, 2023. 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:

- During our site reconnaissance we did not observe evidence of recent or ongoing instability in the project area, such as tensions cracks or breaks in vegetation at the top of the steep south slope.

- The small block wall constructed at the top of the south slope (see Plate 2 on Page 2) appeared to be in fair condition and we did not observe any obvious evidence of wall movement or instability (i.e., tilting or toppling of the wall).
- The south slope was densely vegetated with mature Douglas fir, bigleaf 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.
- The majority of the mature trees appeared to be generally straight, indicating that the slope is relatively stable.
- We did not observe groundwater seepage or hydrophytic or water loving plants at the site.
- The existing building foundation and deck were observed to be in fair condition.

5.1.4 Landslide Hazard Summary

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

Although we did not observe indications of recent slope movement affecting the subject site, the site is located adjacent to a large mapped prehistoric landslide that may be susceptible to slope movement in the future. It would not be economically feasible or practical 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 prehistoric slide, especially during a strong seismic event.

In our opinion, the larger risk comes from surficial slope failures in the upper several feet of loosened soil, as is common on most slopes of this steepness. However, as currently planned, new deck post footings and moment frame column foundations are generally located about 25 feet from the top of the slope, within the footprint of the existing developed area, and the existing carport is more than 50 feet from the top of the slope (see *Figures 2 and 3*), which should not be impacted by surficial slope instabilities. Additionally, as currently planned, we understand that the new deck and moment frame foundations will be anchored to the existing foundations with very limited earthworks needed. As such, in our opinion, the development as currently planned will not adversely impact the subject property or adjacent properties or critical areas, provided that the recommendations presented in this report are properly incorporated into the design and construction of the project.

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:

- Earthwork should be limited to the area of the proposed additions. 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 sloped at a 1H:1V (Horizontal:Vertical) during excavation or supported using temporary shoring consisting. If needed, PanGEO can provide recommendations for temporary shoring.
- 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. Under no circumstances should collected stormwater be allowed to discharge onto the steep slope.
- All disturbed areas outside of the building footprints should be covered in hardscaping or planted with an appropriate species of vegetation to reduce erosion and improve stability of the surficial layer of soil.

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 following page.

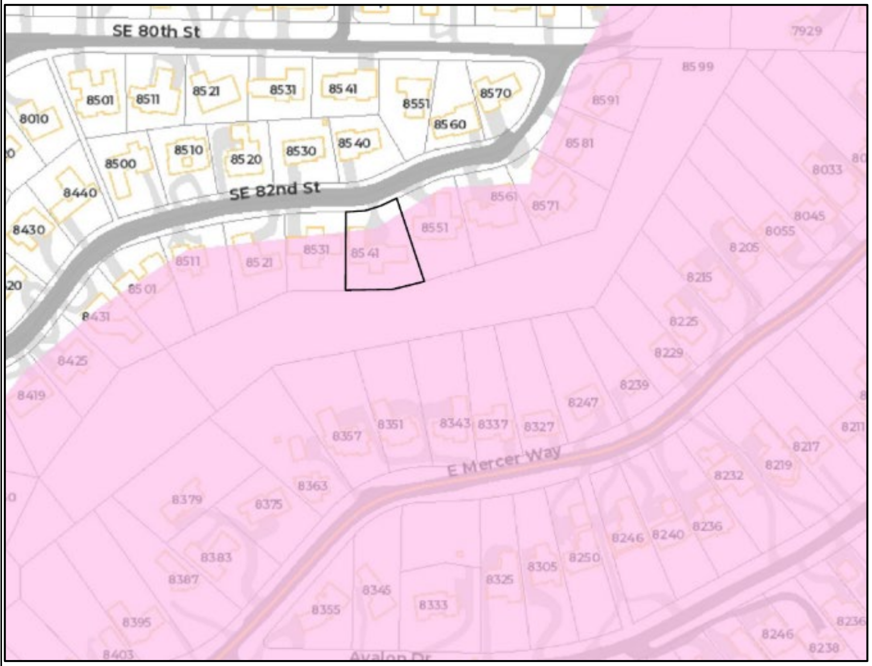


Plate 7: *Approximate extent of the seismic hazard areas identified by the City of Mercer Island.*

The site is outlined in black

Based on the soil observed in our test borings and the absence of a groundwater table, in our opinion, the potential for soil liquefaction is low, and design considerations associated with soil liquefaction are not needed.

It should also be noted that the site is located within a prehistoric 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 prehistoric landslide, which could impact the existing and proposed improvements. However, considering the scale of the pre-historic landslide and the height of the slope, it is not practical to mitigate the risk of such an event.

5.3 Erosion Hazards

The majority of the 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 following 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 7.5 of this report.

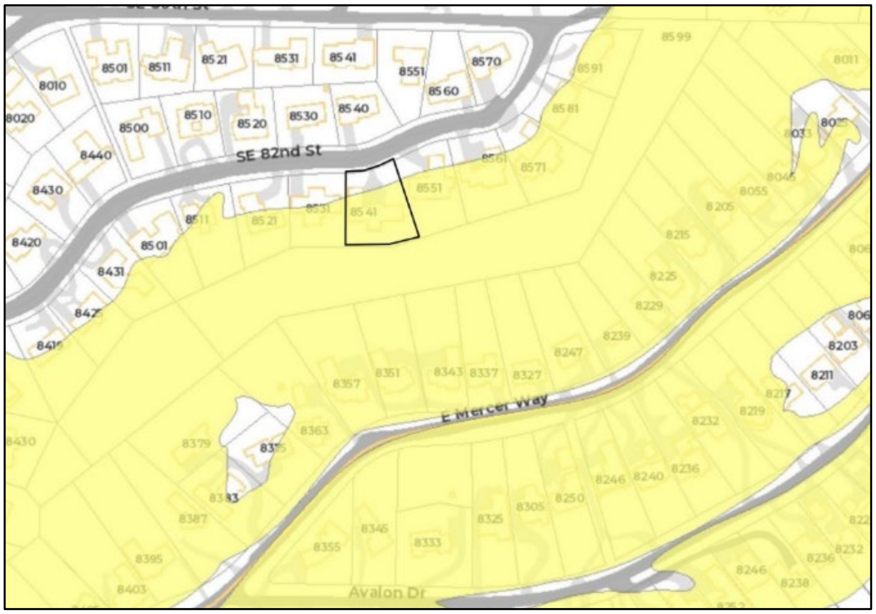


Plate 8: *Approximate extent of erosion hazard areas identified by the City of Mercer Island.*

The site is outlined in black.

5.4 STATEMENT OF RISK

The Mercer Island Municipal Code Section 19.07.160(B)(2) states:

Alteration of landslide hazard areas and seismic hazard areas and associated buffers may occur if the critical area study documents find that the proposed alteration:

> *a. Will not adversely impact other critical areas;*
>
> *b. Will not adversely impact the subject property or adjacent properties;*
>
> *c. Will mitigate impacts to the geologically hazardous area consistent with best available science to the maximum extent reasonably possible such that the site is determined to be safe; and*
>
> *d. Includes the landscaping of all disturbed areas outside of building footprints and installation of hardscape prior to final inspection.*

As previously discussed above, it is opinion that criteria *a*, *b*, and *c* are met for this project, provided that the project will be designed and constructed in accordance with our recommendations. PanGEO will review the final design plan when the design document is completed, to verify that criterion *d* is met.

Additionally, the Mercer Island Municipal Code Section 19.07.160(B)(3) states:

Alteration of landslide hazard areas, seismic hazard areas and associated buffers may occur if the conditions listed in subsection (B)(2) of this section are satisfied and the geotechnical professional provides a statement of risk matching one of the following:

> *a. An evaluation of site-specific subsurface conditions demonstrates that the proposed development is not located in a landslide hazard area or seismic hazard area;*
>
> *b. The landslide hazard area or seismic hazard area will be modified or the development has been designed so that the risk to the site and adjacent property is eliminated or mitigated such that the site is determined to be safe;*
>
> *c. Construction practices are proposed for the alteration that would render the development as safe as if it were not located in a geologically hazardous area and do not adversely impact adjacent properties; or*
>
> *d. The development is so minor as not to pose a threat to the public health, safety and welfare.*

The proposed development as currently planned is very small and we anticipate limited ground disturbance. As such, based on the above criteria and our understanding of the geologic hazards mapped at the site, as well as the site soil conditions and the current plans, it is our opinion that criteria *d* is applicable to the project. In our opinion, the development is so minor as not to pose a threat to public health, safety and welfare, provided that the recommendations presented in this report are properly incorporated into the design and construction of the project.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 SEISMIC SITE CLASS

We assume the seismic design of the proposed structure will be accomplished in accordance with the 2018 or 2021 International Building Code (IBC), which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years). Based on the results of our test borings and the geology at the site, it is our opinion that Site Class D (Stiff Soils) is considered appropriate for determining the site coefficients for the seismic design of the proposed additions.

6.2 BUILDING FOUNDATIONS

6.2.1 Foundation Options

Garage Foundations – Based on the results of our test borings, about 7 to 8 feet of very loose to medium dense fill and/or colluvium soils are present at the subject site. Based on the site soil conditions, it is our opinion that new footings for the garage may be supported on conventional footings bearing on the native sandy soils (recompacted Advance outwash), or on properly compacted structural fill placed on the recompacted native sandy soils.

The foundation subgrade soils should be recompacted to a firm/dense condition prior to placing structural fill or footing construction. Depending on the actual footing subgrade elevation and the variation of soil conditions, several feet of foundation soil over-excavation may be required to reach native bearing soils. The over-excavation should extend horizontally out from the edge of the footing a distance equal to half of the over-excavation depth. Over-excavation should be backfilled with properly compacted granular structural fill, as described in Section 7.3 of this report.

Alternatively, in lieu of over excavations to remove as much as 8 feet of marginal fill and colluvium soils, small diameter pipe piles (often referred to as pin piles) or helical piles may be used to transfer the building loads to the native soils to mitigate the risk for differential settlements. If pipe piles will be used, it is our opinion that 2- to 4-inch diameter steel pipe piles are appropriate.

Post and Moment Frame Foundations - We anticipate that the new post and moment frame footings below the exiting deck may bear on loose fill similar to what was encountered in our adjacent exploration at PG-2. However, we understand that the post and moment frame foundations are not load bearing and will be anchored to the existing adjacent house footings. As such, it is our opinion that the footings may be constructed as conventional footings with the same bottom elevation as the existing footings, as currently planned.

Prior to constructing the new footings, the bottom of the excavation should be compacted with a jumping jack-type compactor to a firm and unyielding condition. If the existing soils cannot be compacted to a firm and unyielding condition, they should be over-excavated a minimum of 1-foot and replaced with properly compacted structural fill (discussed in Section 7.3 below). The structural fill should extend horizontally a minimum of 12 inches beyond the edge of the foundation.

6.2.2 Conventional Footings

Allowable Bearing Pressure – Conventional continuous and individual (spread) footings constructed as described above may be sized using a maximum allowable bearing pressure of 2,500 psf. For allowable stress design, the recommended allowable bearing pressure may be increased by 1/3 for transient conditions such as wind and seismic loadings. Continuous and individual spread footings should have minimum widths of 18 and 24 inches, respectively. Footings should be placed at least 18 inches below final exterior grade. Interior footings should be placed at least 12 inches below the top of slab.

Where space may be limited for an unsupported open cut, it may be necessary to use L-shaped perimeter footings in order to conserve space and to allow the temporary excavations to be made within the property limits.

Where new footings will be constructed adjacent to the existing basement walls (see *Figure 3*), we recommend the footings be located below a 1H:1V (Horizontal:Vertical) projection from the base of the existing basement footings to avoid surcharging the basement walls. If needed, the potential surcharge pressures can be evaluated using a lateral pressure coefficient of 0.35 (i.e., lateral pressure equal to about 35 percent of the vertical pressure)

Lateral Resistance – Lateral forces from wind or seismic loading may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundations and walls, and by friction acting on the base of the foundations.

- Passive resistance values may be determined using an equivalent fluid weight of 250 pounds per cubic foot (pcf). This value includes a factor safety of at least 1.5 assuming that densely compacted structural fill will be placed adjacent to the sides of the foundation, and level ground surface within 5 feet of the footings. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches of soil should be neglected.
- A friction coefficient of 0.4 may be used to determine the frictional resistance at the base of the foundation. This coefficient includes a factor of safety of approximately 1.5.

Foundation Performance – Total and differential settlements are anticipated to be within tolerable limits for foundation designed and constructed as discussed above. For the proposed building supported by conventional footings bearing on competent native soils and structural fill/lean-mix concrete, the building settlement under static loading conditions is estimated to be less than approximately one inch, and differential settlement should be on the order of about ½ inch or less. Most settlement should occur during construction as loads are applied.

Footing Excavation and Subgrade Protection – All footing subgrades should be carefully prepared. Any loose or softened soil should be removed from the footing excavations and replaced with granular structural fill such as crushed rock or recycled concrete. The exposed footing subgrades should be observed by PanGEO to confirm that the subgrade is consistent with the expected conditions and adequate to support the proposed residence.

Some of the site soils are moisture sensitive, and can be easily disturbed when exposed to moisture. Wet weather and construction activities could soften/loosen the exposed subgrades. As a result, depending on the weather condition at the time of footing construction, it may be necessary to place 2 to 3 inches of lean-mix concrete or 4 to 6 inches of clean crushed rock on the exposed footing subgrades to protect against moisture and disturbance.

Perimeter Footing Drain – We recommend that a 4-inch diameter perforated pipe embedded in pea gravel or washed rock and wrapped in geotextile filter fabric be installed at the base of the footings to direct collected water to an appropriate outlet. Under no circumstances should roof downspout drain lines be connected to the footing drain system. Roof downspouts must be separately tightlined to an appropriate discharge. Cleanouts should be installed to allow for periodic maintenance of the footing drain and downspout tightline systems.

6.2.3 Driven Pipe Piles (Pin Piles)

As previously mentioned, 7 to 8 feet of fill and colluvium unsuitable for foundation bearing was encountered in our test borings. For areas where it is not practical to over-excavate the fill soils or where trying to limit the amount of ground disturbance, driven 2- to 4-inch diameter steel pin piles may be used to support the foundations. Where equipment access is limited, the use of 2-inch pin piles may be more appropriate since it can be installed with hand-held equipment.

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.

Pin Pile Capacity – In our opinion, 2-, 3- or 4-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)
2	3
3	6
4	10

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.

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

1. 2-inch diameter piles should consist of Schedule-80, ASTM A-53 Grade "A" pipe.
2. 3-inch and 4-inch diameter piles should consist of Schedule-40, ASTM A-53 Grade "A" pipe.
3. 2-inch piles shall be driven to refusal with a minimum 90-lb jackhammer. Refusal is defined as no more than 1 inch of penetration for 1 minute of continuous driving. Please note that the City requires load testing if a differential driving criteria is used for a different hammer size.
4. 3-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 (see discussion in Item 7).

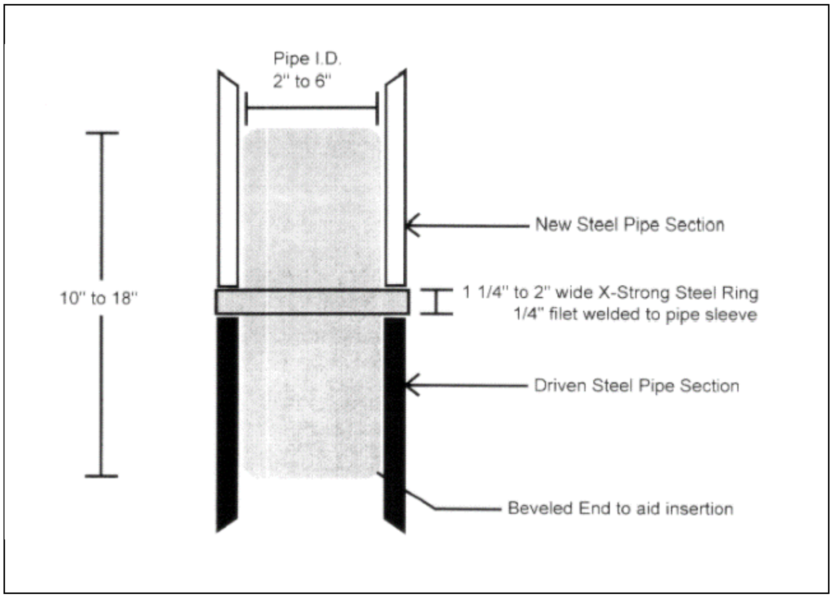
5. 4-inch piles shall be driven to refusal with a minimum 850-lb hydraulic hammer. We recommend the following refusal criteria based on the size of hammer utilized:

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 (see discussion in Item 7).

6. Piles shall be driven in nominal sections and connected with compression fitted sleeve couplers (see detail on following 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.



7. At least 3 percent (but no more than 5) of the 3-inch and 4-inch 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 2 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.

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

Lateral 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 250 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.

Friction resistance at the bottom of pile-supported footings should be ignored in design calculations.

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 the actual driving conditions encountered. For planning and cost estimating purposes, we estimate that the pile lengths, on average, may range from 25 to 30 feet

Obstructions – Obstructions may be encountered within the upper fill or disturbed soils. 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.

6.2.4 Helical Piers

Helical piles may also be used in lieu of driven pin piles. Installation of helical piers is a quieter operation. Helical piers typically consist of one or more helix-shaped bearing plates affixed to a central shaft. The helical piles are installed by rotating the lead section and subsequent extensions with a hydraulic driving motor. For 2 3/8- and 2 7/8-inch diameter shafts with an 8-inch and 10-inch diameter double helix lead section, a maximum allowable axial compression capacity of 15 and 20 kips may be used for design, respectively. For planning purposes, we estimate that 30- to 35-foot-long helical piles will be needed to achieve the required design compression capacity.

All helical piles should be installed to a torque that provides an ultimate load that is at least twice the design load. The helical pile assembly and installation system to be used by the contractor should be submitted to the geotechnical engineer for review.

To verify the capacity of the helical piles and the adequacy of the installation method, a minimum of 3 percent of the piles should be load tested to at least 200% of the design loads. The tests should be performed in general accordance with ASTM Quick Test (ASTM D1143-81).

6.2.5 Construction Monitoring

The geotechnical engineer of record or his/her representative shall provide full time observation of driven pin piles or helical pier installation to verify that the piles/piers have been driven to adequate refusal within the anticipated bearing stratum.

6.3 Concrete Slab-on-Grade

Floor slabs may be constructed using conventional concrete slab-on-grade floor construction. The floor slabs should be constructed on a minimum 4-inch thick capillary break. If loose soils are encountered at the proposed design slab subgrade elevations, we recommend that the loose soils be removed and replaced with at least one foot of properly compacted structural fill below the capillary break.

The capillary break material should consist of at least of 4 inches of pea gravel or compacted ¾-inch, clean crushed rock (less than 3 percent fines). The capillary break material should also have no more than 10 percent passing the No. 4 sieve and less than 5 percent by weight of the material passing the U.S. Standard No. 100 sieve. The capillary break should be placed on the subgrade that has been compacted to a dense and unyielding condition. A 10-mil polyethylene vapor barrier should also be placed directly below the slab. We also recommend that construction joints be incorporated into the floor slab to control cracking.

6.4 Retaining Wall Design Parameters

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

Lateral Earth Pressures – Concrete walls that are free to rotate should be designed for an equivalent fluid pressure of 35 pcf for level backfills behind the walls assuming the walls are free to rotate. If walls are to be restrained at the top from free movement, such as below-grade and basement walls, equivalent fluid pressures of 50 pcf should be used for level backfills behind the walls. Retaining walls with a maximum 2H:1V backslope should be designed for an active and at rest earth pressure of 55 and 65 pcf, respectively.

For the seismic condition, we recommend a uniform lateral earth pressure of 10H psf (where H is the wall height) be added to the static pressure for sizing the retaining and basement walls. The recommended lateral pressure assumes that adequate wall drainage will be incorporated into the design and construction of the walls to prevent the development of hydrostatic pressure.

Wall Surcharge – The retaining and basement walls should be designed to accommodate traffic surcharge pressures if the traffic load is located within the height dimension of the wall. Similarly, surcharge loads from construction equipment or soil/material stockpiles should be considered in the retaining and basement wall design. We recommend that 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 wall height.

Wall Drainage – Provisions for wall drainage should consist of a 4-inch diameter perforated drainpipe behind and at the base of the wall footings, embedded in 12 to 18 inches of clean crushed rock and pea gravel wrapped with a layer of filter fabric. Where applicable, in-lieu of conventional footing drains, weep holes (2” diameter of 10 feet on center) may be used for site retaining walls. 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.

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, 2023) 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 PERMANENT DRAINAGE AND INFILTRATION CONSIDERATIONS

Permanent control of surface water and roof runoff should be incorporated in the final grading design. In addition to these sources, irrigation and rainwater infiltrating into the proposed landscaped and planter areas adjacent to paved areas or building foundations should also be controlled. All collected runoff should be directed into conduits that carry the water away from the pavement or structure and into storm drain systems or other appropriate outlets and should not be discharged onto the slope. Adequate surface gradients should be incorporated into the grading design such that surface runoff is directed away from structures.

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, most likely due to the steep slopes, and the City’s concerns about compromising the stability of the slopes due to infiltration. As a result, for planning purposes, non-infiltrating alternatives will be needed for stormwater mitigation.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 TEMPORARY EXCAVATIONS

Based on our understating of the site soil conditions and the current building setbacks, we anticipate that unsupported slope cuts may be incorporated into the excavation design. All temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring.

In general, temporary excavations deeper than a total of 4 feet should be sloped or shored. However, excavations less than 4 feet deep, if located along or near property lines, will also need to be sloped or supported if sufficient space is not available to lay back the excavations without encroaching into neighboring properties.

Based on the soil conditions at the site, for planning purposes, it is our opinion that temporary excavations for the proposed construction may be sloped as steep as 1H:1V (Horizontal:Vertical). Based on the current design plans, unsupported open cuts appear to be feasible for foundation construction for the buildings. In the event that sufficient space is not available for unsupported open cuts, PanGEO can provide temporary shoring recommendation if requested. Where space may be limited, the use of L-shaped footings may be required to conserve space for the temporary cuts.

For planning purposes, the temporary unsupported excavation may be sloped as steep as 1H:1V (Horizontal: Vertical). The cut slopes may also need to be flattened in the wet reasons and should be covered with plastic sheets. We also recommend that heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a distance equal to 1/3 the slope height from the top of any excavation.

7.2 MATERIAL REUSE

In the context of this report, structural fill is defined as compacted fill placed under footings, concrete stairs and landings, and slabs, or other load-bearing areas. The contractor should be aware that the site soils are poorly graded and may be difficult to compact to the requirements of structural fill. As a result, the excavated site materials may not be suitable for use as structural backfill, particularly during periods of wet weather. For planning and budgeting purposes, we recommend granular import fill such as the City of Seattle Type 2 or 17 Mineral Aggregates (Section 9.03.10 (1) of the 2023 Seattle Standard Specifications), Gravel Borrow (Section 9.03.14 (1) of the 2023 WSDOT Standard Specifications), recycled crushed concrete, or approved equivalent.

Well-graded recycled concrete may also be considered as a source of structural fill. Use of recycled concrete as structural fill should be approved by the geotechnical engineer. The on-site soil can be used as general fill in the non-structural and landscaping areas. If use of the on-site soil is planned, the excavated soil should be stockpiled and protected with plastic sheeting to prevent softening from rainfall in the wet season.

7.3 Structural Fill and Compaction

Structural fill should be moisture conditioned to within about 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 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).

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 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.

7.5 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 Rahul Pathak, Severine Kelley, 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,

June 5, 2024

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SHarrington@pangeoinc.com

Siew L. Tan, P.E.
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10.0 REFERENCES

ASTM D1557-12e1, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft3 (2,700 kN-m/m3))*, ASTM International, West Conshohocken, PA, 2012, www.astm.org

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

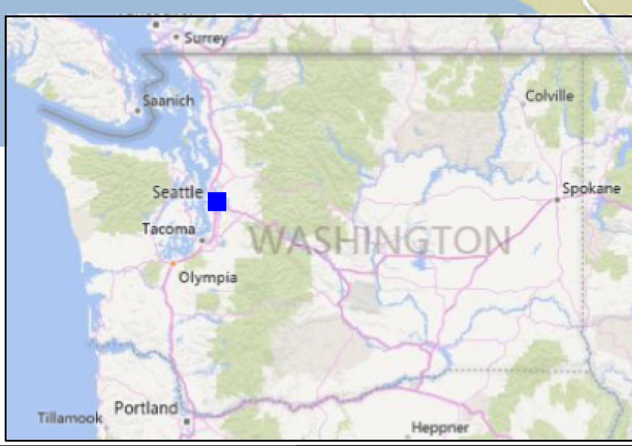
City of Seattle, 2023, *Standard Specifications for Road, Bridges, and Municipal Construction*.

International Building Code (IBC), 2018 and 2021, *International Code Council.*

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

Washington Administrative Code (WAC), 2019, Chapter 296-155 - *Safety Standards for Construction Work, Part N - Excavation, Trenching, and Shoring*, Olympia, Washington.

WSDOT, 2023, *Standard Specifications for Road, Bridge and Municipal Construction, M 41-10, Washington State Department of Transportation.*



Base Map: WSDOT GeoPortal

Not-To-Scale

PanGEO INCORPORATED

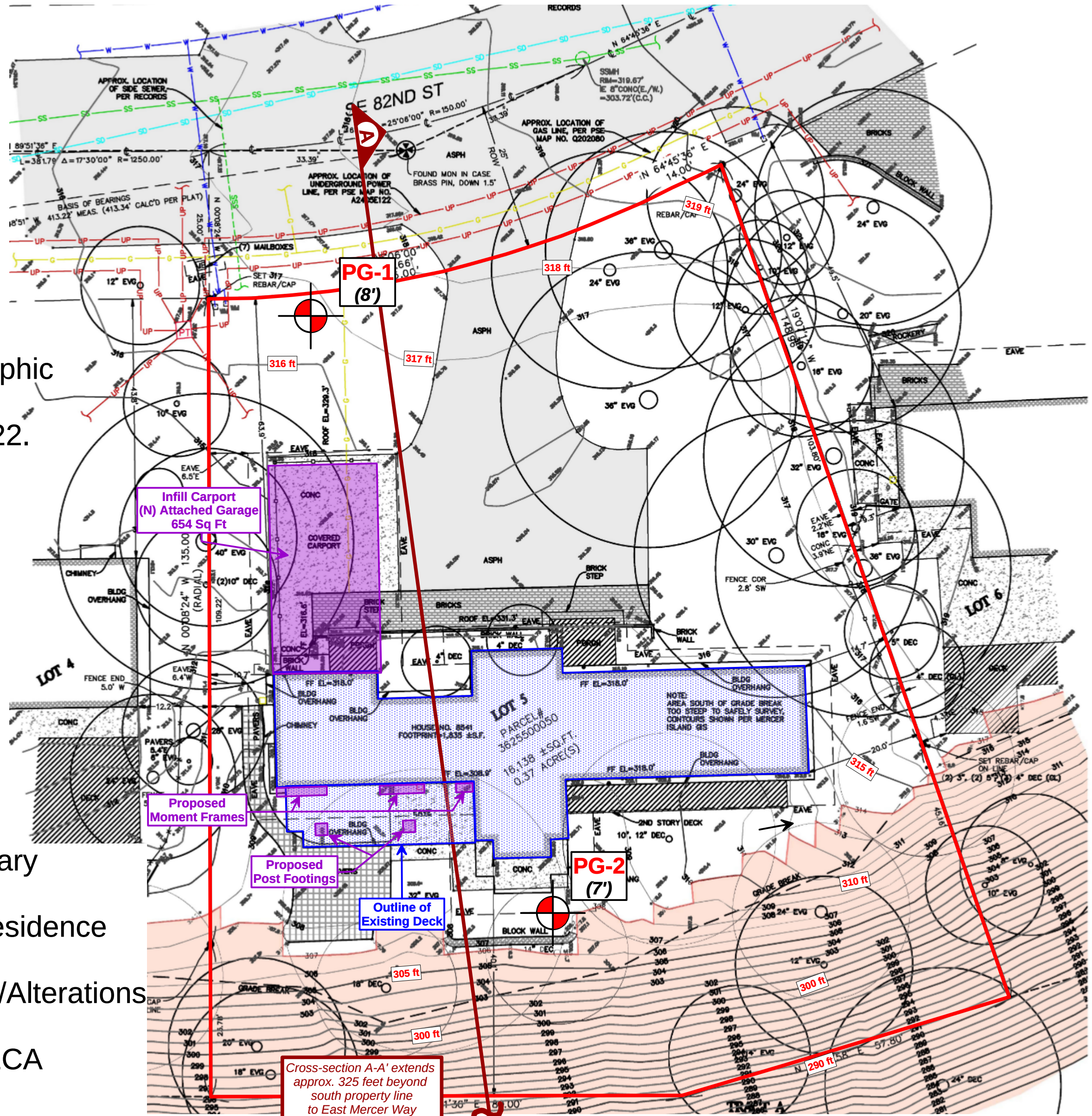
Proposed Remodel/Alterations
8541 SE 82nd Street
Mercer Island, WA

VICINITY MAP

Project No. **23-223**

Figure No. **1**

Note:
Base map modified from Topographic & Boundary Survey prepared by Terrane, dated November 11, 2022.
Vertical Datum: NAVD 88



Legend:

- Subject Site Boundary
- Approx. Existing Residence
- Proposed Remodel/Alterations
- 40% Steep Slope ECA *(mapped by Terrane)*
- PG-1 (8') Approx. Boring Locations PanGEO, Inc., 2023 *(Approx. Depth to Native Soil in Feet)*
- A Study Cross Section

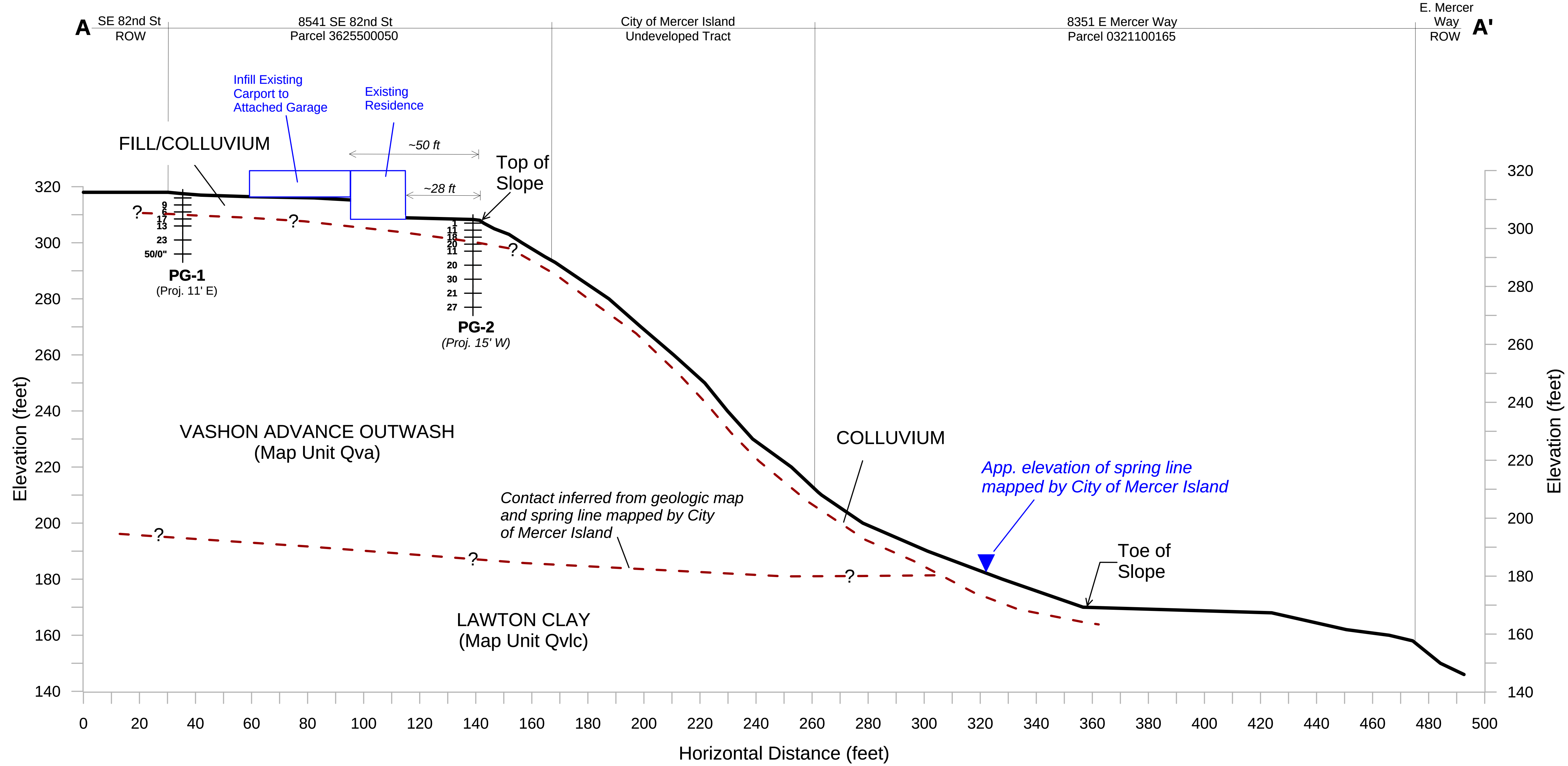
Approx. Scale
1" = 30'

PanGEO
INCORPORATED

Proposed Remodel/Alterations
8541 SE 82nd Street
Mercer Island, WA

SITE AND EXPLORATION PLAN

Project No.	Figure No.
23-223	2



Horizontal & Vertical Scale:
1 inch = 40 feet

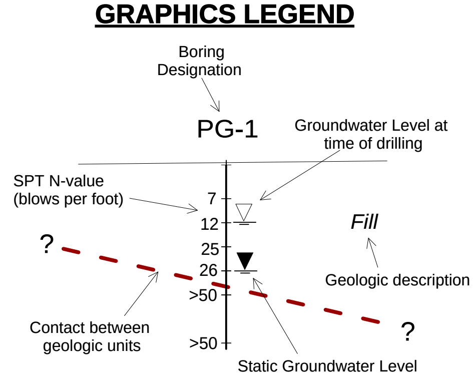
Description of Soil Units:

FILL/COLLUVIUM: Very loose to medium dense silty SAND with varying amounts of gravel and organics; moist, generally disturbed or discolored

ADVANCE OUTWASH (Qva): Medium dense, silty SAND and poorly-graded SAND with silt, with varying amounts of gravel; moist, generally massive soil texture

LAWTON CLAY (Qvlc): Very stiff to hard, laminated to massive silt and clay. *Not encountered in our test borings*

GRAPHICS LEGEND



Notes:
1. Existing ground profile on property and slope based on the topographic survey of 18541 SE 82nd St, Mercer Island, prepared by Terrane, dated November 11, 2022.
2. Ground surface topography from site property line down to East Mercer way estimated from elevation contours obtained from City of Mercer Island GIS.
3. See Figure 2 for location of Section A-A'.
4. See report text for a detailed description of the subsurface conditions across the site.

PanGEO INCORPORATED	**Proposed Remodel/Alterations 8541 SE 82nd Street Mercer Island, WA**	**GENERALIZED SUBSURFACE PROFILE SECTION A-A'**
		Project No. **23-223** / Figure No. **3**

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

LOG KEY 13-104_LOGS.GPJ PANGEO.GDT 6/18/13

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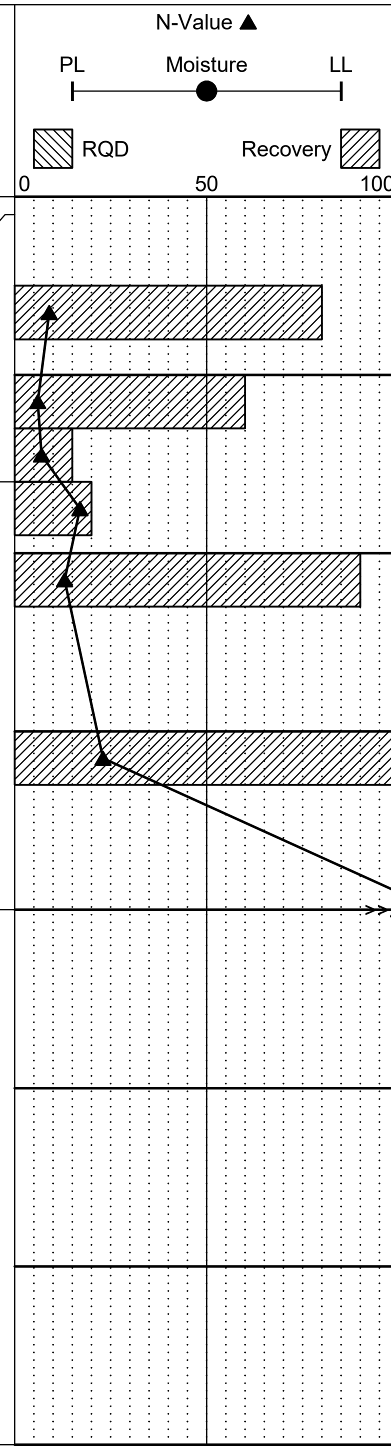
Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	Proposed Remodel & Alterations	Surface Elevation:	~316 ft
Job Number:	23-223	Top of Casing Elev.:	N/A
Location:	8541 NE 82nd Street, Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.53033, Easting: -122.22436	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						Approximately 4 inches of mulch over topsoil.
						FILL/COLLUVIUM Loose, tan to brown silty, fine to medium SAND with gravel and inorganic debris; moist, disturbed soil texture.
	S-1		3 5 4			*-- Gravelly drilling between about 3 and 3.5 feet and 5.5 to 7 feet.*
5.0	S-2		3 2 4			
	S-3		12 3 4			
	S-4		11 9 8			**ADVANCE OUTWASH (Qva)** Medium dense, gray, silty, fine to medium SAND with trace to some gravel; moist, massive soil structure.
10.0	S-5		2 5 8			
15.0	S-6		6 10 13			
						-- Boring met refusal on large rock or cobble at 20 feet.
20.0	S-7		50/0			Boring terminated at about 20 feet below ground surface. No groundwater observed within the termination depth at the time of drilling.
25.0						
30.0						
35.0						



Completion Depth:	20.0ft
Date Borehole Started:	8/14/23
Date Borehole Completed:	8/14/23
Logged By:	E. Eckles
Drilling Company:	CN Drilling

Remarks: Borings drilled using limited-access Acker drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Elevations are estimated from topographic contours from Site Plan by CAST Architecture, dated March 10, 2023. **Vertcal datum: NAVD88**

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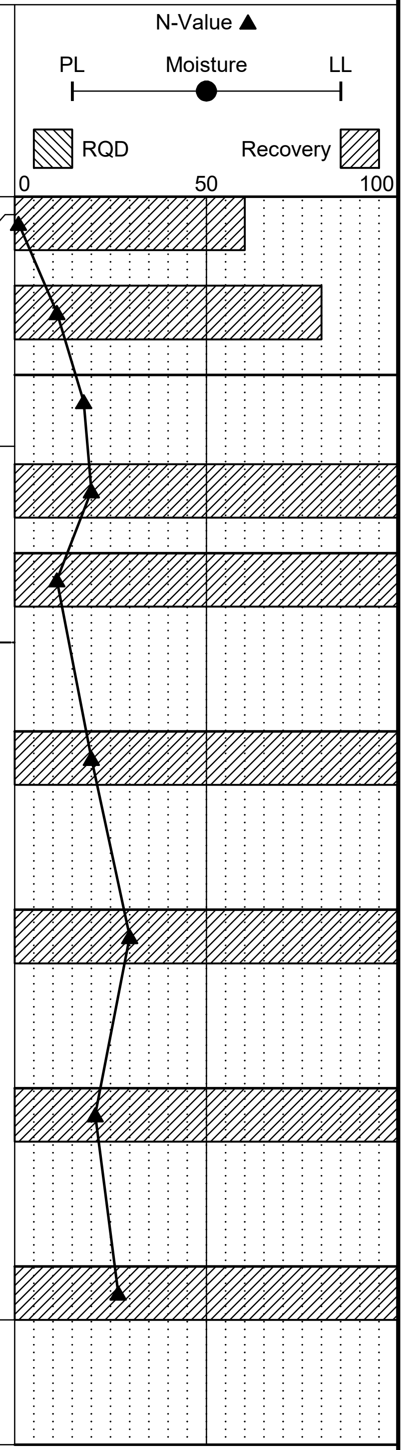
LOG OF TEST BORING PG-1

Figure A-2

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

Project:	Proposed Remodel & Alterations	Surface Elevation:	~307 ft
Job Number:	23-223	Top of Casing Elev.:	N/A
Location:	8541 NE 82nd Street, Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.53009, Easting: -122.22411	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0	S-1	X	1 0 1			Approximately 6 inches of grass over topsoil. **FILL/COLLUVIUM** Very loose to medium dense, tan to brown silty, fine to medium SAND, trace gravel and organics; moist, disturbed/discolored soil texture.
	S-2	X	1 2 9			*-- Cobbles encountered from 4 to 6 feet.*
5.0	S-3	X	16 10 8			*-- No sample recovery at 5 feet; pushing cobble. SPT N-Value likely overstated.*
	S-4	X	6 8 12			**ADVANCE OUTWASH (Qva)** Medium dense, brown, slightly silty, fine to medium SAND, scattered gravel; moist, minor iron-oxide staining, massive soil texture.
10.0	S-5	X	3 5 6			-- Increase in iron-oxide staining/weathering.
						Medium dense, gray-brown, poorly-graded, medium SAND with silt; moist, massive soil texture.
15.0	S-6	X	5 8 12			
20.0	S-7	X	9 14 16			-- Becomes medium dense to dense; scattered fine, sub-rounded gravel.
25.0	S-8	X	5 10 11			-- Becomes gray.
30.0	S-9	X	6 15 12			
						Boring terminated at about 31.5 feet below ground surface. No groundwater observed within the termination depth at the time of drilling.
35.0						



Completion Depth:	31.5ft
Date Borehole Started:	8/14/23
Date Borehole Completed:	8/14/23
Logged By:	E. Eckles
Drilling Company:	CN Drilling

Remarks: Borings drilled using limited-access Acker drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Elevations are estimated from topographic contours from Site Plan by CAST Architecture, dated March 10, 2023. **Vertcal datum: NAVD88**

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LOG OF TEST BORING PG-2

Figure A-3

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