PanGEO
INCORPORATED
Geotechnical & Earthquake Engineering Consultants

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David and Karen Zimmer
4611 Forest Avenue SE
Mercer Island, WA 98040

Subject: **Geotechnical Report – Rev. 01**
Proposed Zimmer Residence
4661 Forest Avenue SE, Mercer Island, Washington 98040
King County Parcel # 404500-0065

Dear David and Karen,

Attached please find our geotechnical report for the proposed single-family residence project located at the above site in Mercer Island, Washington. This report documents the subsurface conditions at the site and presents our geotechnical engineering recommendations for the proposed development.

In summary, the project site is generally underlain by fill soils, disturbed native soils, colluvium, lake deposits, and slide deposits overlying glacially-overridden pre-Olympia nonglacial deposits (hardpan). In the waterfront area west of the existing house, we encountered two distinct groundwater conditions: an unconfined water table at 4.5 feet depth that is likely influenced by Lake Washington's water level, and a deeper confined groundwater table at 16 feet within granular zones of the pre-Olympia deposits. Due to the confined nature and topographic relief of the site, artesian pressures and heaving sands should be expected in excavations that extend below lake level into these deposits.

Based on the subsurface conditions encountered, we recommend supporting the proposed structures on 3- to 4-inch diameter steel pipe (pin) piles. The subsurface conditions vary significantly between the upper and lower portions of the site, with pile lengths expected to range from 20 to 30 feet based on our experience in the area. The piles should extend through the upper soil layers and be driven to refusal in the competent glacial deposits, providing allowable axial compression capacities of 6 tons for 3-inch piles and 10 tons for 4-inch piles.

Additionally, our analysis indicates that the steep slopes above and below the existing residence are marginally stable and are at risk of future instability, especially during a strong IBC level design earthquake event. To provide adequate factors of safety against potential slope instability, we recommend installing two stabilization walls: a permanent soldier pile catchment wall with timber lagging above the residence, and drilled cast-in-place concrete stabilization piles below the residence.

For the anticipated excavations up to 12 feet deep during construction, we recommend unsupported cuts no steeper than 1H:1V where space permits. In areas with limited space, soldier pile walls with timber lagging should be used. Ultrablock gravity walls may also provide a cost-effective temporary shoring option in suitable areas.

For permanent retaining walls, we recommend using expanded polystyrene (EPS) geofoam backfill to reduce lateral earth pressures. Proper drainage provisions and waterproofing measures should be implemented for all below-grade walls.

We appreciate the opportunity to assist with this project. Please contact us with any questions about our findings or recommendations.

Sincerely,

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

Encl.: Geotechnical Report

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GEOTECHNICAL REPORT – REV. 01
PROPOSED ZIMMER RESIDENCE
4661 FORREST AVENUE SE, MERCER, ISLAND, WASHINGTON 98040
KING COUNTY PARCEL # 404500-0065

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering study that was undertaken to support the design and construction of the proposed single-family residence at the above site in Mercer Island, Washington. Our service scope included reviewing available geologic and geotechnical data in the site vicinity, performing engineering analyses based on the 2024 revised conceptual plan set, and developing the geotechnical design recommendations presented in this report.

2.0 PROJECT AND SITE DESCRIPTION

The project site is an approximately 31,0981 square-foot waterfront lot located at 4661 Forest Avenue SE in Mercer Island, Washington (see *Vicinity Map, Figure 1*). The site is roughly trapezoidal in shape, and borders Forest Avenue SE to the east, Lake Washington to the west, and existing single-family residences to the north and south. A single-family house currently occupies the western portion of the site. Based on review of the topographic survey map, the site generally slopes down from east to west with an average gradient of about 27 percent and a total vertical elevation difference of about 80 feet. A driveway provides vehicle access from Forest Avenue SE to the existing house.

We understand that the proposed project, as currently envisaged, is to demolish the existing house and construct a new single-family residence (SFR) at approximately the same location. Based on the conceptual design plans, the proposed residence will consist of two main levels and a partial subbasement (*see Plate 1*). The proposed project will likely also include other site improvements, such as retaining walls and stairs. We anticipate temporary excavations up to approximately 12 feet below adjacent grades will be required to facilitate the construction of the basements. Due to the depth of the anticipated excavation and site constraints, temporary shoring may be required in areas where unsupported open cuts are not feasible / practical.

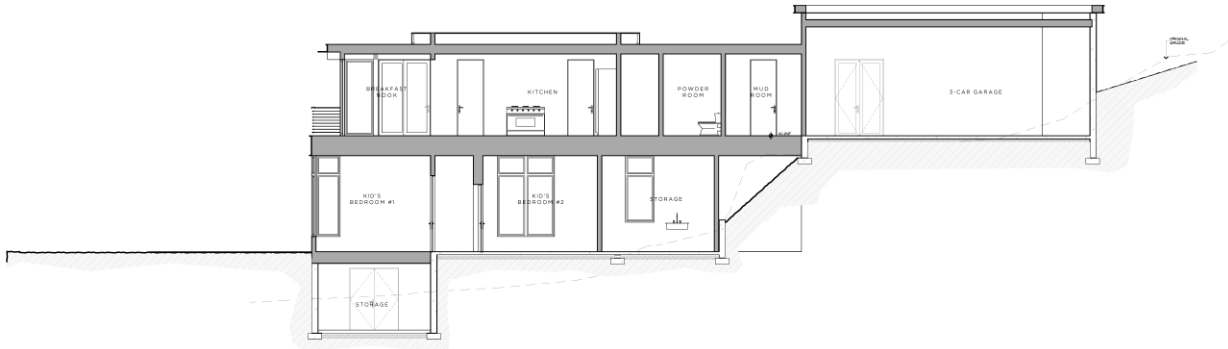


Plate 1. *Schematic west-east building section, looking north.*

The site is mapped with geologic hazards including potential slide, steep slope, seismic, and erosion hazards. Additionally, past known slides are mapped in the immediate site vicinity. As such, it is critical to explore the site subsurface conditions, to evaluate the potential geologic hazards, and provide geotechnical design recommendations for the proposed development.

Plates 2 through 5 below depict the current condition at the site.

Plate 2. *General conditions in western (water front) portion of the site, looking east.*

Plate 3. *General conditions between the existing residence and driveway apron, looking south.*

Plate 4. *General conditions of the driveway apron and existing garage, looking north.*

Plate 5. *General conditions of the shared driveway in the eastern (upper) portion of the site, looking west.*

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.

3.0 SUBSURFACE EXPLORATIONS

3.1 CURRENT SUBSURFACE EXPLORATIONS

PanGEO completed four test borings (PG-1 through PG-4) on December 15, 2021, to explore the subsurface conditions at the site. The approximate boring locations were taped from existing features at the site and are indicated on the attached *Figures 2A & 2B*. The borings were drilled to depths of about 16½ to 26½ feet below the existing grade, using a portable limited access acker drill and a mini track-mounted drill rig owned and operated by Geologic Drill Partners, Inc. of Fall City, Washington. The acker and track-mounted drill rigs were equipped with 4-inch and 6-inch outside diameter hollow stem augers, respectively.

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. The ground surface at the boring locations were restored.

A geologist from our firm was present throughout the field exploration to observe the drilling, assist in sampling, and to document the soil samples obtained from the borings. The soil samples were described using the system outlined on *Figure A-1* in *Appendix A*. Summary boring logs are included as *Figures A-2* through *A-5*.

3.2 PREVIOUS SUBSURFACE EXPLORATIONS

As part of our study, we reviewed summary logs of previous subsurface investigations in the vicinity of the site. The approximate locations of these previous explorations are indicated on the attached *Figures 2A & 2B*. Additionally, summary logs from previous explorations are included in *Appendices B* through *D* for reference. Specifically, the following previous subsurface data was reviewed for this study:

- Liu & Associates, Inc. (2010) previously completed four test borings (TH-1 trough TH-4) at the neighboring parcel to the south (4703 Forest Ave SE). These borings ranged from about 5 to 6 feet below the ground surface;

- James Easton, P.E. (1988) previously completed four testing borings (TH-1 through TH-4) at the neighboring parcel to the north (4651 Forest Ave SE). These borings ranged from about 11½ to 15½ feet below the ground surface. Please note that the resistance values indicated on the boring logs are non-standard Porter Penetration Resistance values. The logs and associated report did not specify the hammer size or drop height. However, Porter Penetration Resistance values are typically reported as the number of blows per 6 inches of penetration with a 40 lb weight dropped from a heigh of 18 inches.

The porter penetration resistance in blows per 6 inches approximately correlates to the Standard Penetration Resistance Values in blows per foot; and

- Golder & Associates (1999) previously installed a standpipe piezometer (PZ-1) at the neighboring parcel to the north (4649 Forest Ave SE). The piezometer extended approximately 15 feet below the surface.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 SITE GEOLOGY

According to the *Geologic Map of Mercer Island* (Troost and Wisher, 2006), the majority of the subject site is underlain by pre-Olympia nonglacial deposits (Qpon) and pre-Olympia fine grained nonglacial deposits (Qponf). The geologic map also indicates that Lake deposits (Ql) are also present along the waterfront margins of the site. The attached *Figure 3*, *Geologic Map*, presents the surficial geologic units mapped in the vicinity of the site. The following is a brief description of each relative geologic soil unit mapped in the vicinity of the site, from youngest to oldest.

- **Lake Deposits (Ql) –** Typically consists of very soft to soft silt and clay with local sand layers, peat, and other organic sediments, deposited in slow-flowing water. Most mapped areas are lake-bottom sediments exposed by the lowering of Lake Washington in 1916.

- **Pre-Olympia Nonglacial Deposits (Qpon)** – Is described by Troost, et al. as interbedded and intermixed sand, gravel, silt, clay and organic deposits of inferred nonglacial origin that were subsequently overridden during the Olympia Interglaciation. This unit typically ranges from very dense to hard.

- **Pre-Olympia Nonglacial Fine-Grained Deposits (Qponf)** – Is the predominantly fine-grained subunit of the geologic unit listed above. Nonglacial Fine-Grained deposits typically consist of laminated to massive, very stiff to hard, silt and clay and with occasional sandy interbeds and peat.

4.2 SOIL CONDITIONS

The current and previous subsurface explorations advanced at and near the project site generally confirmed the mapped stratigraphy. In general, the project site is underlain by a sequence of fill, disturbed native soils / colluvium / land slide deposits, overlying undisturbed, stiff to hard glacially overridden silt with variable amounts of sand, which we interpret as pre-Olympia Nonglacial deposits (Qpon).

For the purposes of this report, we have grouped the soils encountered in the borings into five engineering soil units (ESUs) based in part on their engineering properties, the composition of the soils, and anticipated engineering behaviors. The interpreted subsurface conditions are depicted in *Figures 5* and *6*, and brief descriptions of the generalized soil conditions encountered at the locations of the test borings advanced at the site are presented below. Please refer to the summary boring logs in *Appendices A* through *D* for more details.

Engineering Soil Unit (ESU)

ESU 1: Fill* **|** With the exception of test boring PG-3, a surficial layer of undocumented fill was encountered in all of the current test borings advanced at the site. The fill was observed to depths of 4½, 4½ and 5½ feet in PG-1, PG-2 and PG-4, respectively. The fill soil generally consisted of very loose to loose, silty sand with pockets of sandy silt and organic material. This unit was characterized by its low relative density, disrupted soil structure and the presence of organic debris.

ESU 2: Fill & Disturbed Native* **|** Test boring JE-TH2-88, previously advanced on the adjacent parcel to the north, encountered a two-foot-thick surficial layer of fill overlying sandy clayey silt with pockets of silty sand and 6-inch clasts of clay which extended up to about 8 feet below the existing ground surface. In general, the sand content and relative density of this unit increased with depth. This unit was characterized by its low to moderate relative density and disrupted soil structure.

ESU 3: Colluvium & Lake Deposits* **|** Underlying the existing fill, test boring PG-1, which was advanced in the lower portion of the site near the waterfront, encountered an 11-foot-thick layer of soft to medium stiff, silty clay with trace amount of sand and organics. We interpret this layer as Lake Deposits (Ql) based its low relative density, fine-grained soil structure, blueish-gray color and the presence of organic material. Portions of this unit may also be colluvium deposits (Qmw) originating from the upper portions of the site. This unit extended to about 15½ feet below the ground surface in PG-1.

ESU 4: Colluvium & Slide Deposits* | In the upper portions of the site, the majority of the test borings advanced at and near the site encountered a layer of stiff to very stiff sandy silt and silt with variable amounts of clay, gravel and organic debris. Based on the results of the current and previous test borings, we interpret this unit as colluvium (Qmw) and slide deposits (Qls). In general, this unit increased in thickness from west to east and increased in relative density with depth. The thickness of this unit ranged from about 7 to 10 feet below the existing ground surface. In test borings PG-3 and PG-4, which were advanced in the relatively undeveloped portions of the site, encountered this unit at the ground surface. The remaining explorations encountered this unit directly underlying the surficial fill soils.

This unit was characterized by its variable soil structure, moderate relative density, and presence of organic material and debris.

ESU 5: Pre-Olympia Nonglacial Deposits (Hardpan) | All of the test borings advanced at and near the site encountered a layer of very stiff to hard, silt with scattered sand seams which appears to be consistent with the pre-Olympia nonglacial deposits mapped at the site. Based on the current and previous test borings, this unit directly underlies the lake deposits (Ql) and slide deposits (Qls) in the lower and upper portions of the site respectively. This is the deepest soil unit encountered in all of the test borings.

In general, the relative density and drilling effort in this unit increased with depth. However, the upper approximately 2½ feet of this unit appears to have weathered to a medium dense state.

This unit is characterized by its interbedded soil structure and high SPT N-values which is an indication that this engineering soil unit was glacially-overridden.

* Possible debris or obstructions are likely present in Engineering Soil Units 1 through 4 due to layers of fill, slide deposits, and previous development.

4.3 GROUNDWATER

Test boring PG-1, which was advanced near the waterfront, encountered groundwater at 4½ and 16 feet below the ground surface at the time of drilling. The groundwater encountered at 4½ feet is likely groundwater perched above the relatively fine-grained lake deposits. We expect the level of this of this unconfined groundwater table to be heavily influenced by the level of the lake. The lower groundwater contact at 16 feet is likely a confined groundwater table within granular zones of predominantly fine-grained pre-Olympia nonglacial deposits. Due to confined nature and topographic relief of the site, artesian pressures and heaving sands should be expected in excavations that extend below the level of the lake and into this geologic unit.

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

5.0 GEOLOGIC HAZARDS ASSESSMENT & SITE STABILITY CONSIDERATIONS

Based on our review of Mercer Islands' Geologic Hazard Map, the site is mapped with geologic hazards including potential slide, seismic, and erosion hazards. Additionally, the topographic and boundary survey identifies portions of the site as having slopes steeper than 40%. The approximate boundaries of the critical areas, slide initiation points, as mapped by Mercer Island, are depicted on *Figure 4, Lidar and Critical Areas.* The approximate location of the steep slope critical areas is depicted on *Figure 2A, Site and Exploration Plan*.

5.1 EROSION HAZARDS

The entire site is mapped as a potential erosion hazard area in accordance with the City of Mercer Island's Geologic Hazards Map. Based on the USDA Soil Survey data and our test borings, the site soils are anticipated to exhibit moderate erosion potential when disturbed and left unprotected. However, 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. During construction, the temporary erosion hazard can also be effectively managed with an appropriate erosion and sediment control plan, including but not limited to installing a silt fence at the construction perimeter, placing quarry spalls or hay bales at the disturbed and traffic areas, covering stockpiled soil or cut slopes with plastic sheets, constructing a temporary drainage pond to control surface runoff and sediment trap, placing rocks at the construction entrance, etc.

Permanent erosion control measures should be applied to the disturbed areas as soon as feasible. These measures may include but are not limited to planting and hydroseeding. The use of permanent erosion control mats may also be considered in conjunction with planting/hydroseeding to protect the soil from erosion.

5.2 SEISMIC HAZARDS

Based on our review of the City of Mercer Island's Geologic Hazards Maps, the subject site is mapped within 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, and soil liquefaction or surface faulting.

Liquefaction is a process that can occur when the soil loses its shear strength for short periods of time during a seismic event. Ground shaking of sufficient strength and duration results in the loss of grain-to-grain contact and an increase in pore water pressure, causing the soil to behave as a fluid. Soils with a potential for liquefaction are typically cohesionless, predominately silt and sand sized, must be loose, and below the groundwater table.

Based on the current and previous test borings advanced at and near the site, only relatively minor amounts of perched groundwater are anticipated in the upper portions of the site. In the lower portion of the site, the shallow unconfined groundwater table is expected to be influenced by the level of the lake. However, the soils underlying this portion of the site are predominantly fine-grained deposits. Based on these conditions, in our opinion the liquefaction potential of the site is negligible and design considerations related to soil liquefaction are not necessary for this project.

Additionally, we recommend that pin piles be used to support the new footings, which will effectively mitigate the risk of the seismic hazard.

5.3 POTENTIAL LANDSLIDE HAZARDS & KNOWN SLIDES

The entire site is mapped as a potential landslide hazard area due to the underlying geology. Additionally, our research indicates that at least one known slides event occurred within 500 feet of the proposed residence in 1974. Our general review of documented slide in the site vicinity is summarized below.

> ***Nearby Slide | 81st Ave SE near Forest Ave SE -*** According to the City of Mercer Island Landslide Hazard Map, one known slide event (LS-1974-003) occurred at the parcel located approximately 250 feet northeast of the project site in January of 1974. The slide is described by Dames & Moore as being approximately 180 feet in length with a downset ranging from several inches to 3 feet. Ponded water was observed in the slide area for sever days after a period of heavy rainfall. Dames & Moore concluded that the landslide resulted from oversteepening and oversaturation of the relatively loose native materials and fill

soils. Furthermore, they concluded that *"it appears the problem is of local origin and not the postulated larger ancient landslide"*. Mitigation measures included the construction of interception trench drain.

Based on our review of the 2016 King County LiDAR data, the west-facing slope above the project site appears to be part of a larger ancient landslide system. Which is consistent with the slide debris encountered in some of the test borings. However, we have found no evidence of recent movement or reactivation of the ancient landslide. The recent small slides appear to be localized in nature and triggered by heavy rain falls, natural weathering of the exposed face and development activities.

It should be noted that additional slides may have occurred in the immediate vicinity of the site but may not have been reported or documented by the City.

5.4 SLOPE RECONNAISSANCE AND OBSERVATIONS

We also conducted a reconnaissance of the steep slope on November 23, 2021. The majority of the site contains heavily vegetated west-facing slopes immediately east of the developed portion portions of the site. The purpose of our reconnaissance was to review the condition of the slope and identify indications of potential historical slope instability, which may include:

- Bowl-shaped topography;
- Irregular or hummocky topography;
- Tension cracks, scarps, or other indicators of ground movement;
- Leaning or pistol-butted trees;
- Distressed vegetation;
- Vegetation of markedly different ages or types (i.e., a swath of young alders and blackberries in an otherwise mature forest);
- "Fresh" looking soil deposited at the base of steep slopes;
- Disturbed or destroyed anthropogenic features, such as fence lines that have been displaced;
- Hillside seeps or springs; and
- Ponding water/sag ponds.

As with most steep slopes, the surficial material is loose, and tends to slowly move downslope due to gravity over time, which is often referred to as "soil creep". Our observation of the general conditions of the area suggests that there is some minor evidence of soil creep on the subject slope, in the form of slightly leaning trees, or trees with bent trunks. Some irregularities were observed in the topography of the slope which may be attributed to undocumented shallow slides or slumps that have occurred on the property and adjacent lots. However, recent movements such as tension cracks and fresh-looking soil were not observed during our reconnaissance.

5.5 SITE STABILITY ANALYSIS

Based on our understanding of the subsurface conditions and site reconnaissance, it is our opinion that the steep slopes above and below the existing residence may be at risk of future instability, especially during a strong earthquake or prolonged rain events. Therefore, the stability of the site was evaluated to determine what stability measures are needed to adequately stabilize the developed portion of the site. Our evaluation was based on our understanding of subsurface conditions as described above, the topographic survey provided to us, as well as topographic information derived from 2016 King County LiDAR data obtained from the Washington DNR, the results of our site reconnaissance, and our understanding of the proposed project.

Approach to Global Stability Analyses – The stability of a slope depends on a variety of factors, including the geometry of the slope, the subsurface stratigraphy, material properties of the soils, the presence of groundwater, and the effects of surface loads.

Based on our interpretation of the current and previous subsurface explorations advanced at and near the project site; the topographic survey by Bush, Roed & Hitchins, Inc.; supplemental LiDAR data; and conditions observed during our site reconnaissance, we developed a generalized subsurface profile for one critical cross section which runs perpendicular to the steep slope, as shown *Figure 2A.*

We divided on-site soils into Engineering Soil Units (ESUs) for the slope stability analysis. The soil parameters for the ESUs were assigned based on observed soil types, empirical correlations using SPT blowcount values, our experience with similar soil conditions and engineering judgement, and published literature (e.g. Meyerhof, G. G., 1956; WSDOT, 2021; and USGS, 2006). The profiles and soil parameters used in our slope stability analysis are shown on *Figures 5* through *8*.

In order to evaluate the stability of the slope, as well as the design parameters for the stabilization piles and minimum depth of embedment, the 2D limit equilibrium slope stability analysis software Slide2 (RocScience) was used to perform the stability analyses. Search routines were used to identify the potential critical failure surface having the lowest static and pseudo-static factors of safety using the Spencer method of analysis. A factor-of-safety of 1.0 is equilibrium while a factor-of-safety of less than 1.0 indicates instability. The acceptable static and seismic factors of safety against global instability by current standard of practice and the City of Mercer Island, are 1.5 and 1.1, respectively.

For seismic analysis, a horizontal ground acceleration coefficient was determined based on a modified Peak Ground Acceleration (PGA_M) with a 2 percent of probability of exceedance in 50 years (i.e., a 2,475-year return period). Accordingly, a PGA_M value of 0.679g was calculated from the USGS seismic hazard deaggregation program. One-half of the expected design ground acceleration (i.e. half of PGA_M), or 0.3395g was used in our pseudo-static stability analysis.

Discussion of Results – As shown in *Figures 5* and *6*, our analysis indicates that the slope above and below the existing residence is marginally stable and is at risk of future instability, especially during a strong earthquake.

Without improvements/mitigations, the factor of safety against future slope instability under static and seismic loading conditions will not meet the current code of 1.5 and 1.1, respectively. To provide adequate factors of safety against future slope instability, we recommend stabilization walls downslope and upslope of the proposed residence. The upper wall will be just east of the proposed garage and driveway near the toe of the steep slope. This will also function as a catchment wall that will help mitigate the impact of debris flows coming down from the adjacent steep slope. Likewise, the lower wall will stabilize the lower portion of the site and provide additional protection to the structure. In order to facilitate the construction of the proposed daylight basement, a third wall may be need to be constructed along the eastside of main residence depending on the final finish floor elevations. Conceptional wall alignments are shown on *Figures 2A* and *2B.*

As shown in *Figures 7* and *8*, the permanent soldier pile catchment wall and lower concrete stabilization pile design concept will achieve adequate factors of safety in both the static and seismic condition, provided that the walls are designed with the recommendations provided in this report.

From a geotechnical perspective, it is our opinion that the proposed soldier pile catchment wall & lower concrete piles design concept provides adequate stability to the site.

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

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 SEISMIC SITE CLASS

We anticipate the seismic design of the proposed buildings will 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). For design purposes, a Site Class D (Stiff Soil) is considered appropriate for the seismic design for the project site.

6.2 BUILDING FOUNDATIONS

6.2.1 General

Based on the soil conditions, in our opinion, the proposed structures should be supported by deep foundations, such as small diameter steep pipes, to avoid excessive long-term and differential building settlement. The following sections present our recommendations for the pin pile foundations.

6.2.2 Pin Pile Foundations

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

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

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

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

Three- and four-inch diameter piles are typically installed using small (approximately 850 to 2,000 pound) hammers mounted to a small excavator. The criterion for driving refusal is defined as the minimum amount of time (in seconds) required to achieve one inch of penetration, and it varies with the size of hammer used for pile driving. For 3- and 4-inch pin piles, Table 1 is a summary of driving refusal criteria for different hammer sizes that are commonly used:

Table 1 - Summary of Commonly Accepted Driving Criteria for 3- & 4-inch Pin Pile with a 6, 10-ton Allowable Axial Compression Load

Hammer Model	Hammer Weight (lb) / Blows per minute	3” Pile Refusal Criteria (seconds per inch of penetration)	4” Pile Refusal Criteria (seconds per inch of penetration)
Hydraulic TB 325	850 / 900	10	16
Hydraulic TB 425	1,100 / 900	6	10
Hydraulic TB 725X	2,000 / 600	3	4

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

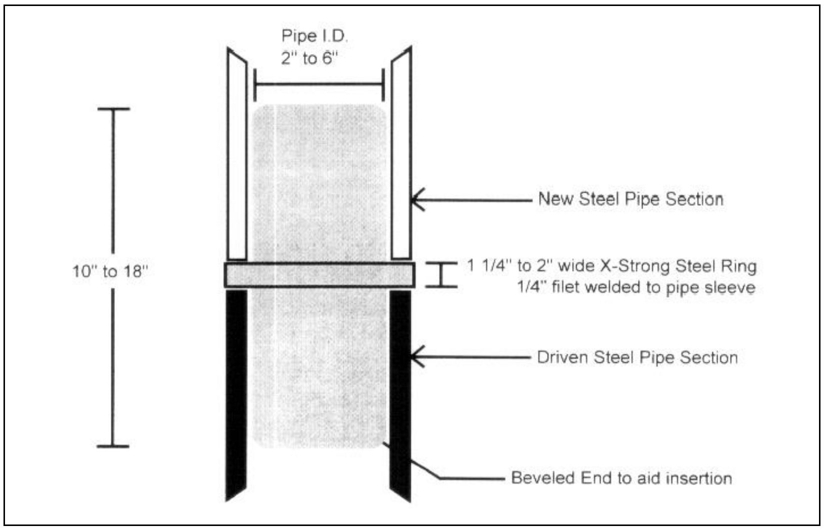
Pin Pile Specifications **-** We recommend that the following specifications be included on the foundation plan:

1. Three- and four-inch diameter piles should consist of Schedule-40, ASTM A-53 Grade “A” pipe.

2. The piles shall be driven to refusal as shown in Table 1 above.

3. Piles shall be driven in nominal sections and connected with compression fitted sleeve couplers (see typical detail on below) We discourage welding of pipe joints, particularly when galvanized pipe is used, as we have frequently observed welds broken during driving.

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



Typical Splicing Detail

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

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

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

Estimated Pile Length **–** The subsurface conditions at the site will likely vary substantially across the site. Based on the soil conditions at the site and our experience in the project area, for planning and cost estimating purposes, we estimate that pin pile lengths of about 20 to 30 feet.

Obstructions **–** Obstructions may be encountered during pile 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 the adjustment.

6.3 CONCRETE SLAB-ON-GRADE

The floor slabs for the proposed buildings may be constructed using conventional concrete slab-on-grade floor construction. The floor slabs should be supported on 12-inches of compacted structural fill over re-compacted native soil. If the native soils cannot be compacted to a firm condition, additional over-excavation may be needed. Any over-excavation should be replaced with compacted structural fill.

Interior concrete slab-on-grade floors should be underlain by a capillary break consisting of at least of 4 inches of pea gravel or compacted ¾-inch, clean crushed rock (less than 3 percent fines). The capillary break material should 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 GEOFOAM BACKFILL FOR BELOW-GRADE WALLS

Based on our review of the current plans, expanded polystyrene (EPS) geofoam will be used as backfill behind the concrete basement walls to reduce lateral earth pressures. Standard EPS geofoam blocks typically measure 4’x4’x4’ or 4’x4’x8’ but can be custom ordered in various dimensions. The geofoam should be a structural grade with a minimum of EPS 19, conforming to ASTM D6817.

The depth of geofoam backfill required to reduce active earth pressures should be determined by the structural engineer. For planning purposes, geofoam blocks should extend laterally with a 1:1 projection upward from the bottom row of blocks placed directly against the wall (see *Plate 6*, below). A minimum of 2 feet of granular structural backfill should be placed above the geofoam.

The depth of geofoam backfill needed to reduce the active pressure should be determined by the structural engineer. For planning purposes, geofoam blocks should extend laterally at a 1:1 projection upwards from the bottom row of geofoam blocks placed directly against the wall (see *Plate 6,* below). Above the geofoam, at least 2 feet of granular structural backfill should be placed. We also recommend that the lowest 18 inches of wall backfill consist of granular structural fill.

While active pressures are minimal within the geofoam section, we recommend that the active and seismic surcharge pressures shown in *Figure 9* be applied over the full wall height.

It is important to note that geofoam will disintegrate when exposed to petroleum products. Therefore, the entire geofoam backfill zone (not just the top) must be completely encased in a 15-mil membrane for protection. This membrane should overlap by at least 12 inches at the seams and be secured with the manufacturer’s specified tape. Grip plates between blocks should be used to minimize lateral movement, and any gaps between blocks should be filled with clean, crushed gravel.

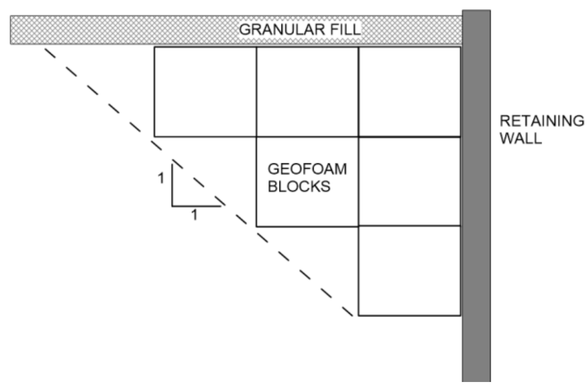


Plate 6. *Typical EPS geofoam block placement*

To ensure proper drainage, a drainage blanket of clean, crushed rock at least 12 inches thick should be installed beneath the encapsulated geofoam along the excavation line. A non-woven filter fabric should be placed between the drainage material and the native soil to prevent migration of fines.

6.5 UPPER – PERMANENT SOLDIER PILE & CATCHMENT WALLS

As discussed in *Section 5.5 – Site Stability Analysis* of this report, permanent stabilization piles will be needed east (upslope) of the improved area in order to meet the City of Mercer Island’s development requirements for slope stability. These stabilization piles/walls will also function as temporary shoring and a catchment wall that will help mitigate the impact of debris flows coming down from the adjacent steep slope.

The conceptual layout described in *Section 5.5* is based on our current understanding of the proposed development and subsurface conditions. The actual alignment of the piles should be determined by the project architect and owner based on the final design. Depending on the final finish floor elevations and configuration of the proposed site features, we anticipate excavations up to about 10 feet below adjacent grades will be required to facilitate the construction of the proposed garage and auto court. Due to the proximity of the excavation to an ascending steep slope, an unsupported open cut with temporary side slopes will not be feasible, and a shoring wall will be required.

Given the subsurface conditions at the site, in our opinion a soldier pile wall with timber lagging is likely the most cost-effective shoring option. The permanent soldier pile wall should have a minimum hole diameter of 2.5 feet with a maximum spacing of 8 feet on center. The following sections present our recommendations for the design of permanent soldier pile walls.

6.5.1 Soldier Pile and Lagging Wall

A soldier pile wall consists of vertical steel beams, typically spaced from 6 to 8 feet apart along the proposed wall alignment, spanned typically by timber lagging. The steel beams are installed into holes drilled to a design depth and then backfilled with lean-mix or structural concrete. In general, tiebacks are typically needed for wall heights greater than about 12 feet to achieve a more economical design.

Design Earth Pressures - For a cantilevered soldier pile wall, or a wall with one level of earth anchors or tiebacks (if needed), the earth pressures depicted on *Figures 11* and *12* should be used for design. Above the bottom of excavation, the recommended active earth pressure should be applied over the full width of the pile spacing. Below the bottom of excavation, the passive resistance should be applied over two times the pile diameter and the active pressure applied over one single pile diameter. The recommended passive earth pressure assumes a level ground surface at the bottom of the excavation.

Because the soldier piles will be used to permanently stabilize the slope, a uniform seismic pressure of 12H should also be included in the pile wall design, where H equals the distance between the top of the pile and the finish grade at the bottom front face of the wall. For the seismic condition, the recommended passive pressure may be increased by one third.

The minimum soldier pile embedment should be determined by the shoring wall designer. However, we recommend that the soldier piles extend at least 10 feet below the bottom of the excavation.

Catchment Wall Height & Pressures - Due to the proximity of the proposed garage addition and driveway apron to the steep slope, we recommend that the permanent soldier pile wall be designed to retain up to 6 feet of soil (i.e. catchment height of 6 feet) which may potentially accumulate behind the walls due to erosion or minor surficial sloughing of the slope.

The catchment wall should extend the entire length of the proposed garage. However, extending the wall further south past the garage addition will provide additional protection to the driveway apron and main residence. Conceptual wall alignments are depicted on *Figures 2A* and *2B*. However, the actual alignment should be determined by the project architect and owner based on the desired level of additional protection, aesthetics and access constraints. The earth pressures depicted on *Figures 11* and *12* should be used for the catchment wall design.

Periodic maintenance of the catchment wall will be required to remove accumulated debris, as the function of the wall is related to the available catchment area behind the wall, therefore, permanent access to the back of the catchment wall should be incorporated into the layout of the planned improvements to maintain the minimum freeboard.

Surcharge Loads - The permanent walls should be designed to accommodate surcharge pressures if surcharge loads are located within the height dimension of the wall which can be estimated using *Figures 11, 12* and *14*. Additionally, we recommend using a surcharge load 80 psf in areas with vehicles traffic.

It should be noted that 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.

Lagging - Lagging design recommendations for the anticipated conditions are presented on *Figures 11* and *12*. Lagging may consist of materials such as timber boards, precast concrete panels, cast-in-place concrete, or steel sheets. For the permanent condition, if timber lagging is utilized, treated timber should be specified, and the saw cut ends of the lagging should be treated on-site prior to lagging installation. It should be noted that even treated timber lagging will eventually deteriorate, and would need to be replaced. The lifespan on treated timber lagging may range from 15 to 25 years. The advantage of concrete or steel lagging is that they would be permanent. A permanent cast-in-place wall facing may also be constructed after the timber lagging has been installed.

The voids behind lagging may be backfilled with clean crushed rocks, such as Seattle Type 22 Mineral Aggregate (COS 2023, Section 9-03.14) if no severe sloughing occurs during excavations. If severe sloughing occurs during lagging installation, we recommend Control Density Fill (CDF) be used as backfill behind the lagging.

Vertical Capacity - The soldier piles for the lower basement wall and upper catchment wall may be designed using an allowable skin friction value of 1.0 ksf for the portion of the pile below elevations 22 and 38 feet, respectively. An allowable end bearing value of 30 ksf may be used for both walls.

Corrosion - Permanent soldier piles should be properly protected against corrosion. This may include corrosion resistant coatings or oversizing the piles to allow for a sacrificial layer of corrosion.

Wall Drainage - See *Section 6.8* for recommendations and additional discussion.

Performance / Pile Deflection - In general, the top of piles should be designed with one-inch deflection or less.

6.5.2 Permanent Ground Anchors (Tiebacks)

Tiebacks will likely be needed for wall heights greater than about 12 feet to improve performance of the wall and reduce the steel beam size. Excessive pile top deflections could occur before the tiebacks are installed. It may be necessary to limit the tiebacks to no more than 10 feet below the pile top unless steel beams of sufficient size will be used to limit the cantilever deflection.

Tieback Adhesion Estimate - The manner in which the tieback anchors carry load will depend on the type of anchor selected, the method of installation, and the soil conditions surrounding the anchor. Accordingly, we strongly recommend the use of a performance specification requiring the shoring contractor to install anchors capable of satisfactorily achieving the design structural loads, with a pullout resistance factor of safety of 2.0.

For planning purposes, the anchors may be sized assuming an allowable skin friction value of 2.0 kips/ft in the ESU 5 unit, assuming that small diameter (about 6 inches) pressure grouted tiebacks will be used. If the contractor utilizes one or multiple post-grouting, higher allowable skin friction values are achievable, which would result in shorter tiebacks. We recommend that the shoring contractor review this report and collaborate with the shoring designer, owner, and PanGEO to determine the most cost-effective tieback design, based on the planned method of tieback installation and grouting. We recommend that the allowable tieback loads be limited to approximately 100 kips per anchor.

***Tieback Testing* –** The actual capacity of the anchors should be checked with 200 percent verification tests. At least two 200% test should be performed. All production anchors should be proof tested to 130% of the allowable design load. The anchor installations should be conducted in accordance with the latest edition of the Post Tensioning Institute (PTI) "*Recommendations for Prestressed Rock and Soil Anchors*." Elements of the testing are as follows:

Verification Tests (200% Tests)

- Perform a minimum of two tests on each anchor type, installation method and soil type with the tested anchors constructed to the same dimensions as production anchors;
- Test locations to be determined in conjunction with and approved by the geotechnical engineer;
- Test anchors, which will be loaded to 200% of the allowable design load, may require additional prestressing steel (steel load not to exceed 80% of the ultimate tensile strength) or reinforcing of the soldier pile;
- Load test anchors to 150% load in 25% load increments, holding each incremental load for at least 5 minutes and recording deflection of the anchor head at various times within each hold to the nearest 0.01inch;
- At the 150% load, the holding period shall be at least 60 minutes;
- After completion of the 150% hold, load the anchor in 25% load increments to the 200% load, which shall be held for 10 minutes; and

- A successful test shall provide a measured creep rate of 0.04 inches or less at the 150% load between 1 and 10 minutes, and 0.08 inches between 6 and 60 minutes, and both shall have a creep rate that is linear or decreasing with time. The applied load must remain constant during all holding periods (i.e. no more than 5% variation from the specified load).

Proof Tests (130% load tests on all production anchors)

- Load test all production anchors to 130% of the allowable design load in 25% load increments, holding each incremental load until a stable deflection is achieved (record deflection of the anchor head at various times within each hold to the nearest 0.01inch);
- At the 130% load, the holding period shall be at least 10 minutes; and
- A successful test shall provide a measured creep rate of 0.04 inches or less at the 130% load between 1 and 10 minutes with a creep rate that is linear or decreasing with time. The applied load must remain constant during the holding period (i.e. no more than 5% variation from the 130% load). Anchors failing this proof testing creep acceptance criteria may be held an additional 50 minutes for creep measurement. Acceptable performance would equate to a creep of 0.08 inches or less between 5 and 50 minutes with a linear or decreasing creep rate.

Verification tested anchors or extended creep proof tested anchors not meeting the acceptance criteria will require a redesign by the contractor to achieve the acceptance criteria.

In the tieback construction, a bond breaker shall be constructed in the no load zone when the installation procedures use single stage grouting.

Tiebacks will need to be designed to provide adequate clearance from utilities, footings and other site improvements, if present behind the wall.

6.6 LOWER – PERMANENT DRILLED CAST-IN-PLACE STABILIZATION PILES

As discussed in *Section 5.5 – Site Stability Analysis* of this report, permanent stabilization piles will be needed along the west (downslope) side of the improved area in order to meet the City of Mercer Island's development requirements for slope stability.

Based on our experience with similar projects, drilled concrete shafts with steel reinforcing cages (drilled cast-in-place piles) will likely be the most economical solution. The stabilization piles, as currently envisaged, will extend through the fill / colluvium / lake deposit layers and into underlying competent pre-Olympia nonglacial (Hardpan) deposits.

These stabilization piles should be installed between the proposed site improvements and the waterfront. Two conceptual wall alignments consisting of a single row of drilled concrete piles with minimum hole diameters of 2.5 feet and maximum spacing of 8 feet on center are depicted on *Figures 2A* and *2B*. However, the actual alignment should be determined by the project architect and owner based on the desired level of additional protection, aesthetics and access constraints.

To evaluate the minimum depth of embedment needed to provide adequate stabilization of the developed area, we performed slope stability analyses using limit-equilibrium methods for both static and seismic loading conditions, as described above in *Section 5.5*.

Pile Size and Design - Our analysis assumed that the stabilization piles will have a minimum diameter of 2.5 feet, a maximum center-to-center spacing of 8 feet, and will provide a minimum ultimate (nominal) shear capacity of at least 430 kips per pile, which corresponds to approximately 53.75 kips per linear foot of wall. The factored strengths for the stabilization piles, used in both static and seismic stability models, were determined by applying partial factors of safety of 2.0 and 1.15, respectively. The factored strengths were calculated using the following relationship: *Factored strength = Ultimate shear capacity / Partial factor of safety.*

If piles with lower ultimate shear capacities are used, the spacing between the piles should be reduced to maintain an equivalent minimum ultimate shear capacity of 53.75 kips per linear foot of wall. The structural engineer must verify the shear capacity of the piles during the design phase by detailing the reinforcing steel and specifying the appropriate concrete strength.

Additionally, in order to provide adequate factors of safety against future instability for the developed portion of the site, the proposed stabilization piles located along the western edge of the developed area must extend to the following minimum tip elevations:

- Alternative 1 | Minimum elevation of +5 feet or lower.
- Alternative 2 | Minimum elevation of +2 feet or lower.

Design Earth Pressures – In addition to the minimum tip elevations listed above, the concrete piles should be designed to accommodate the recommended lateral earth pressures shown in

Figure 13, with a minimum cantilever height of 4 feet, plus the exposed retained height of the wall (if applicable). The 4-foot cantilever height was derived from foreslope stability analysis, which assumes that some soils in front of the piles may potentially slide or creep away.

Because the soldier piles will be used to permanently stabilize the slope, a uniform seismic pressure of 12H psf should also be included in the pile wall design, where H equals the distance between the top of the pile and the finish grade at the bottom front face of the wall.

Above the bottom of the front face of wall, the recommended active earth pressure should be applied over the full width of the pile spacing. Below the bottom of excavation, the passive resistance should be applied over two times the pile diameter and the active pressure applied over one single pile diameter. Additionally, we recommend factors of safety of 1.5 and 1.15 should be applied to the nominal passive earth pressure depicted on *Figure 13* for static and seismic loading conditions, respectively. No load factors have been applied to the recommended active and seismic earth pressures values depicted on *Figure 13*.

The stabilization piles also should be designed to accommodate surcharge pressures if surcharge loads are located within the exposed height dimension of the piles which can be estimated using *Figure 14*.

It should be noted that 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.

***Pile Top Elevation* -** For aesthetic reasons it may not be desirable to have the top of the stabilization piles protruding above the ground surface. In our opinion, the stabilization piles will still function as intended provided the top of the piles are located within 2 feet of the finished graded.

***Vertical Capacity* -** The stabilization piles may be designed using an allowable skin friction value of 1.0 ksf for the portion of the pile below the bottom front face of wall, and an allowable end bearing value of 30 ksf.

***Ground Anchors (Tiebacks)* –** In general, tiebacks are typically needed for wall heights greater than 12 feet to achieve a more economical design. Depending on the final location and finished grade of the proposed development, ground anchors may be required to reduce the steel reinforcement requirements and limit pile deflection. If tiebacks are needed, the recommendations provided in *Section 6.5.2* should be followed.

***Corrosion* -** Permanent stabilization piles should be properly protected against corrosion. This may include but not limited to providing adequate concrete coverage around the steel reinforcement cages.

***Sloughing Control* -** The potential for ground loss behind the piles due to the granular soils sloughing between the piles once the piles become exposed due to future slope loss/movements should be mostly controlled by the soil arching between the relatively tight pile spacing. Based on minimum pile diameter of 30-inches and center-to-center spacing of 8 feet, the clear spacing between the stabilization piles will be approximately 5½ feet and have a 'clear spacing vs diameter ratio' of 2.2D.

The potential for future ground loss behind the piles could be further mitigated by over-excavating behind the stabilization piles, installing lagging in the upper 5 feet, and backfilling behind the piles. Alternative sloughing mitigation measures include a trench of CDF/lean-mix concrete placed behind the stabilization piles or short unreinforced lean-mix shafts placed in between the stabilization piles (secant wall).

If timber lagging is utilized, treated timber should be specified, and the saw cut ends of the lagging should be treated on-site prior to lagging installation. It should be noted that even treated timber lagging will eventually deteriorate. The lifespan on treated timber lagging may range from 15 to 25 years. The advantage of concrete or steel lagging is that they would be permanent.

***Performance / Pile Deflection* -** In general, the top of piles should be designed with one-inch deflection or less under static loading conditions. Depending on the final proximity of the proposed stabilization piles to the residence, designing for smaller total deflections may be warranted so that pile movement will not impact the proposed structure.

***Wall Drainage* -** See *Section 6.8* for recommendations and additional discussion.

6.7 RETAINING AND BASEMENT WALL DESIGN RECOMMENDATIONS

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

Lateral Earth Pressures - Concrete cantilever walls without geofoam backfill should be designed for an equivalent fluid pressure of 35 pcf for level backfills behind the walls assuming the walls are free to rotate. If walls are to be restrained at the top from free movement, such as basement walls, equivalent fluid pressures of 45 pcf should be used for level backfills behind the walls. Walls with a maximum 2H:1V backslope should be designed for an active and at-rest earth pressure of 50 and 60 pcf, respectively. The recommended lateral pressures assume that the backfill behind the wall consists of free draining and properly compacted fill with adequate drainage provisions to prevent the development of hydrostatic pressure.

Permanent walls with a level ground behind the walls should be designed for an additional uniform lateral pressure of 12H psf for seismic loading, where H corresponds to the buried depth of the wall. The recommended lateral pressures assume that the backfill behind the wall consists of a free-draining and properly compacted fill with adequate drainage provisions.

Lateral earth pressures for concrete cantilever walls with geofoam backfill are provided in *Section 6.4* of this report. Additionally, recommended lateral earth pressures for the upper permanent soldier piles & catchment wall and lower stabilization piles, which run parallel to the face of the slope are provided in *Sections 6.5* and *6.6*, respectively.

Wall Surcharge - The retaining and below-grade walls should be designed to accommodate surcharge pressures, if present, within the height dimension of the wall. As a minimum, the traffic surcharge should be considered equivalent to 2 feet of soil surcharge (i.e., 75 psf of horizontal uniform pressure). Similarly, surcharge loads from adjacent footings, construction equipment, or soil/material stockpiles should be considered in the retaining and basement wall design. We recommend that a lateral load coefficient of 0.35 be used to compute the lateral pressure on the wall face resulting from surcharge loads located within a horizontal distance of one-half wall height. Alternatively, *Figure 14* be used to calculate the lateral pressure on the face of the wall face resulting from surcharge loading.

Lateral Resistance - Lateral forces from wind or seismic loading and unbalanced lateral earth pressures may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundation and friction acting on the base of the foundation. Passive resistance values may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf), assuming a level fore slope. A friction coefficient of 0.35 may be used to determine the frictional resistance at the base of the foundation. Both of these values include a safety factor of at least 1.5.

***Wall Backfill* -** In our opinion, the on-site excavated soils are not suitable for use as wall backfill. We recommended that the wall backfill should consist of free-draining granular structural fill as defined in *Section 7.3* of this report.

Wall backfill should be moisture conditioned to near its optimum moisture content, placed in loose, horizontal lifts less than 8 to 12 inches in thickness, and systematically compacted to a dense and relatively unyielding condition. If density tests will be performed, the results should indicate at least 95 percent of the maximum relative dry density, as determined using the test method ASTM D-1557 (Modified Proctor). Within 5 feet of the wall, the backfill should be compacted with hand-operated equipment to at least 90 percent of the maximum dry density.

***Wall Drainage* -** See *Section 6.8* for recommendations and additional discussion.

***Damp-proofing / Waterproofing* -** We recommend the designers consider utilizing a waterproofing material, such as prefabricated clay mats, or other measures, on the exterior of all below-grade walls to reduce the potential for moisture intrusion into the below-grade portion of the structures. We recommend that a waterproofing or building envelope specialty consultant be retained to provide details regarding waterproofing measures, as waterproofing is beyond the scope of our work.

6.8 SUBSURFACE DRAINAGE PROVISIONS

Provisions for permanent control of subsurface water should be incorporated into the design and construction of the below-grade walls. As a minimum, 4-inch diameter perforated drainpipes should be installed behind and at the base of the wall footings, embedded in 12 to 18 inches of crushed rock or washed gravel. The gravel should be wrapped in a geotextile filter fabric to prevent the migration of fines into the drain system. The drainpipe should be graded to direct water to a suitable outlet.

Under no circumstances should roof downspout drain lines be connected to the perorated footing/wall drain systems. Roof downspouts must be separately tight-lined to appropriate discharge locations. Cleanouts should be installed at strategic locations to allow for periodic maintenance of the footing drain and downspout tightline systems.

If the below-grade wall will be constructed against permanent or temporary shoring walls, we recommend that prefabricated drainage mats, such as Mirafi 6000 or equivalent, be installed behind the walls (full face coverage) and the collected water should be directed inside the building beneath the floor slab and tight lined to an appropriate outlet. Additionally, a perforated footing drain should be constructed on the interior of the perimeter footing to remove any groundwater seepage.

If steel sheets, concrete lagging or concrete facing over timber lagging is used, we recommend that 3-inch diameter weep holes should be installed at the bottom of each soldier pile bay to allow drainage at the base of the wall. If timber lagging is used, gaps in the timber lagging will provide an adequate drainage pathway.

6.9 PERMANENT CUT AND FILL SLOPES

Based on the anticipated soils that will be encountered in the proposed development area, we recommend permanent cut and fill slopes be constructed no steeper than 2H:1V (Horizontal:Vertical), with flatter slopes preferred to reduce the potential of erosion. Permanent fill slopes should be constructed of properly compacted structural fill. Permanent slopes should be covered with a thick layer of mulch or topsoil, and vegetated with an appropriate species of grass or vegetation to reduce the potential for erosion.

6.10 PERMANENT SURFACE DRAINAGE & 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 landscape and planter areas adjacent to paved areas or building walls should also be controlled. All collected runoff should be directed into conduits that carry the water away from the pavement, structure, and steep slope; and into appropriate/approved outlets. Under no circumstances should collected surface water or downspout drains be allowed to discharge onto open slopes or behind walls.

Adequate surface gradients should be incorporated into the grading design such that surface runoff is directed away from structures and steep slope. Furthermore, it is important to note that roof downspouts should be tightlined to a suitable outlet, and not discharged into the wall or perimeter footing drain system.

6.11 PERMANENT EROSION CONTROL CONSIDERATIONS

Permanent erosion control measures such as covering exposed ground surfaces with topsoil or mulch, and installing landscaping, should be performed as soon as possible after construction to limit the time the exposed surfaces are susceptible to erosion.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 SITE PREPARATION

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

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

7.2 MATERIAL REUSE

In the context of this report, structural fill is defined as compacted fill placed under footings, concrete stairs and landings, and slabs, or other load-bearing areas. In our opinion, the on-site soils are not suitable to be reused as structural fill. Structural fill should consist of imported, well-grade, granular material, such as WSDOT CSBC or Gravel Borrow, or approved equivalent. 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 PLACEMENT AND COMPACTION

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

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

7.4 SOLDIER PILE AND DRILLED CONCRETE PILE INSTALLATION

Soldier piles will be installed through up to about 20 feet of fill and disturbed soil deposits. It is important to note that caving of the fill and colluvium deposits is likely, especially if zones of seepage are encountered, and the contractor should be prepared to temporarily case the holes to maintain stability during drilling. Flooding the holes with water to maintain the stability of the drill holes is not recommended for this project due to the layers of relatively clean sand that could transmit water towards steep slopes.

Lean concrete or structural concrete backfill should be placed with tremie pipes from bottom up if more than 6 inches of groundwater is present in the drilled holes at the time of concrete placement.

Obstructions may be encountered within the upper fill or disturbed soils. Where possible, the obstructions should be removed to facilitate pile installation. If obstructions cannot be removed, the structural engineer of record should be notified to revise the pile layout.

7.5 TIEBACK INSTALLATION

The drilling for tiebacks may encounter wet sand layers where caving of the drilled holes may occur. As result, the contractor should be prepared to use temporary casing during installation to keep the drilled holes open, and to minimize the risk of potential ground loss.

7.6 TEMPORARY EXCAVATIONS

As currently planned, we anticipate that temporary excavations for the proposed project extending up to about 3 to 12 feet below adjacent grades will be required to facilitate the construction of the basement and stabilization piles. We anticipate the excavations will encounter mostly loose to medium-dense silty sand. All temporary excavations should comply with Part N of Washington Administrative Code (WAC) 296-155, which governs excavation safety standards in Washington State. The contractor is responsible for maintaining safe excavation slopes and/or shoring.

7.6.1 Unsupported Open Cuts

In general, temporary excavations deeper than 4 feet should be either sloped or shored. This also applies to shallower excavations (less than 4 feet) where they occur along property boundaries. For planning purposes, we recommend that temporary excavations be sloped no steeper than 1H:1V (Horizontal : Vertical). Where space may be limited, the use of L-shaped footings may be required to conserve space for the temporary cuts.

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.

During construction, actual soil conditions may require adjustments to the slope or support systems. If groundwater seepage is encountered the temporary slope will likely need to be cut to shallower angles to maintain stability. During wet weather, runoff water should be prevented from entering excavations and the exposed slopes should be covered with plastic sheeting.

Special considerations for upper soldier pile / catchment wall - We anticipate that excavations up to about 10 feet deep may be needed for the construction of the permanent catchment soldier pile wall located near the toe of the steep slope. Temporary excavations made near the toe the slope, if needed for pile installation, should consist of 3-foot-wide excavations on 8-foot centers (~ pile spacing). Adjacent excavations should not be made until piles have been installed in the previously excavated sections.

7.6.2 Temporary Shoring

Where sufficient space or easement is not available for unsupported open cuts, temporary shoring will be needed to support the excavations. In our opinion, Ultrablock walls may be considered to support temporary excavations where feasible.

The following recommendations should be incorporated into the design and construction of the Ultrablock wall:

- The maximum wall height of staggered blocks is 7½ feet (i.e., 3 blocks in height);
- The wall face may be constructed as steep as 10V:1H (vertical: horizontal);
- The unsupported portion of the excavation located above the blocks may be sloped as steep as 1H:1V but the backslope should be no taller than 5 feet;

- The subgrade at the base of the Ultrablock wall shall consist of dense native soil or compacted 5/8-inch minus crushed rock;
- No excavation shall be made until blocks are available on site;
- The width of the unsupported cut face for block placement shall be limited to no more than 15 feet at any given time;
- Blocks shall be placed immediately after the cut is made, otherwise the cut face shall be buttressed with on-site soils until the blocks can be placed;
- The blocks should be installed to avoid vertical seams between the two adjacent rows;
- The ground surface in front of the wall (i.e., at the base) shall be level and no excavation shall extend below the base of block wall;
- Any voids behind blocks shall be backfilled with crushed rock or on-site sand and gravel immediately after the block wall are installed (the use of pea gravel and rounded gravel such as drain rock as backfill is not allowed). The backfill should be tightly compacted with hand tools; and
- PanGEO shall provide observation during Ultrablock wall installation and may revise our recommendations based on the observed conditions.

7.7 TEMPORARY GROUNDWATER AND SURFACE WATER CONTROL

Based on the borings advanced at the site, we anticipate groundwater/seepage will be present within the proposed excavation depths. The contractor should be prepared to provide temporary groundwater control methods during excavation. In our opinion, a conventional dewatering system consisting of trenches, sumps and pumps will likely be adequate to control perched groundwater or runoff from heavy precipitation in the excavation.

7.8 TEMPORARY EROSION AND DRAINAGE CONSIDERATIONS

It should be noted that the site soils are prone to surficial erosion. Special care should be taken to avoid surface water on bare exposed subgrade and slope faces. As previously mentioned in *Section 7.6.1 – Unsupported Open Cuts*, we recommend that the exposed temporary slopes be covered with plastic sheeting.

Surface runoff can be controlled during construction by careful grading practices. Typically, this includes the construction of shallow, upgrade perimeter ditches or low earthen berms in conjunction with silt fences to prevent water from entering excavations or to prevent turbid runoff from leaving the work site.

Temporary erosion control may require the use of hay bales on the downhill side of the project to prevent water from leaving the site and potential stormwater detention to trap sand and silt before the water is discharged to a suitable outlet. All collected water should be directed under control to an appropriate/approved discharge point or outlet.

7.9 WET EARTHWORK RECOMMENDATIONS

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below:

- Earthwork should be performed in small areas to minimize subgrade exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.

- During wet weather, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight based on the portion passing ¾-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 strategically located to control erosion and the movement of soil. Erosion control measures should be installed along all the property boundaries.

- Excavation slopes and soils stockpiled on site should also be covered with plastic sheets.

8.0 ADDITIONAL SERVICES

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

9.0 LIMITATIONS

We have prepared this report for use by David and Karen Zimmer and the project design team. Recommendations contained in this report are based on a site reconnaissance, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of work.

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

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

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

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

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

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

Sincerely,

PanGEO, Inc.

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

10.0 REFERENCES

ASCE 2016, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI Standard 7-16.

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.

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

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

Meyerhof, G. G., *Penetration Tests and Bearing Capacity of Cohesionless Soils,* Journal of the Soil Mechanics and Foundations Division, ASCE, 1956.

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

United States Geological Survey, *Earthquake Hazards Program, Interpolated Probabilistic Ground Motion for the Conterminous 48 States by Latitude and Longitude, 2008 Data,* accessed via: https://seismicmaps.org/

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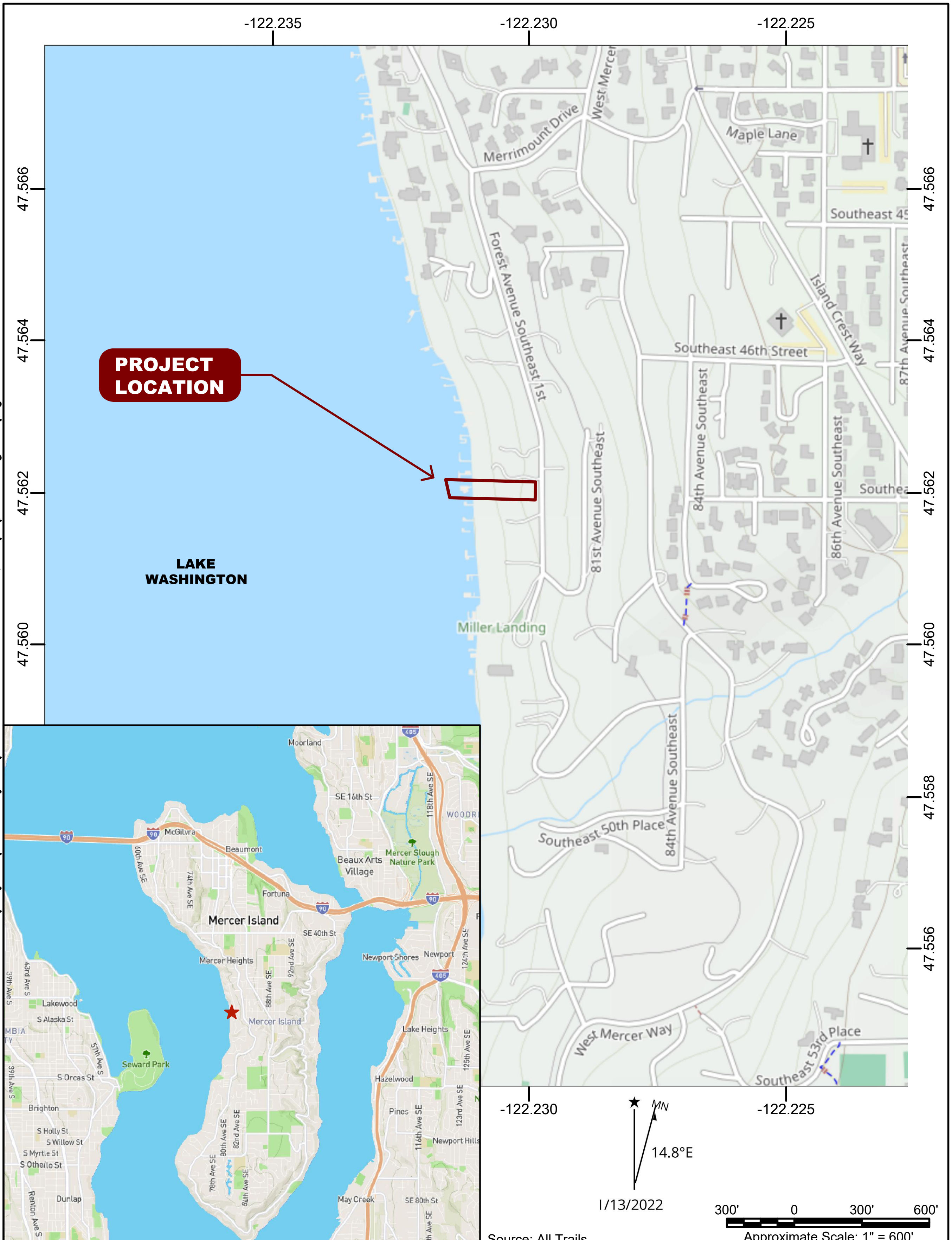
WSDOT, 2021, *Standard Specifications for Road, Bridge and Municipal Construction, M 41-10.*



Source: All Trails

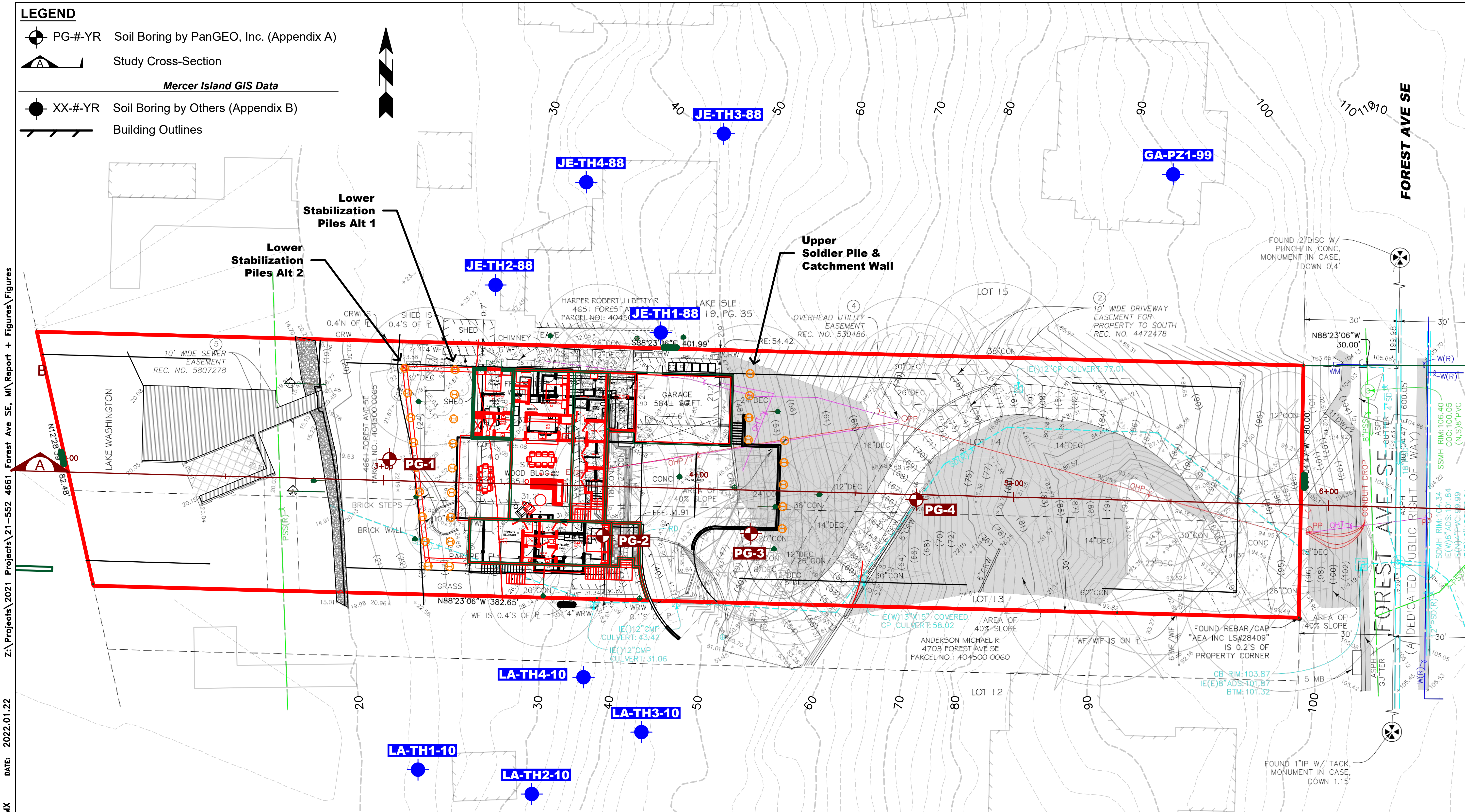
DRAWN BY: NTW CHECKED BY: HMX DATE: 2022.01.12 Z:\Projects\2021 Projects\21-552 4661 Forest Ave SE, MI\Report + Figures\Figures

PanGEO INCORPORATED

Zimmer Residence
4661 Forest Avenue SE
Mercer Island, Washington

VICINITY MAP

PROJECT NO.	FIGURE NO.
21-552	1



NOTES

1. Base map derived from Boundary and Topographic Survey, dated 6/4/2021, by Bush, Roed & Hitchings, Inc.
2. Topology beyond the extents of the project specific survey was derived from 2016 King County LiDAR data obtained from Washington DNR.
3. Proposed featured derived from conceptual drawings, dated 6/13/22, by Ripple Design Studio.
4. Additional features are based on GIS data obtained from Washington DNR and City of Mercer Island websites.
5. Supplemental Topology and features are provided for relative information only and are not a substitution for field survey.
6. Locations of subsurface explorations are approximate and based on the relative locations of known site features.
7. Vertical Datum: NAVD 88

15' 0 15' 30'
scale in feet
Scale: 1" =30'

PanGEO INCORPORATED

Zimmer Residence
4661 Forest Avenue SE
Mercer Island, Washington

SITE AND EXPLORATION PLAN
TOPOGRAPHIC SURVEY

PROJECT NO. 21-552

FIGURE NO. 2A

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DATE: 2022.01.22

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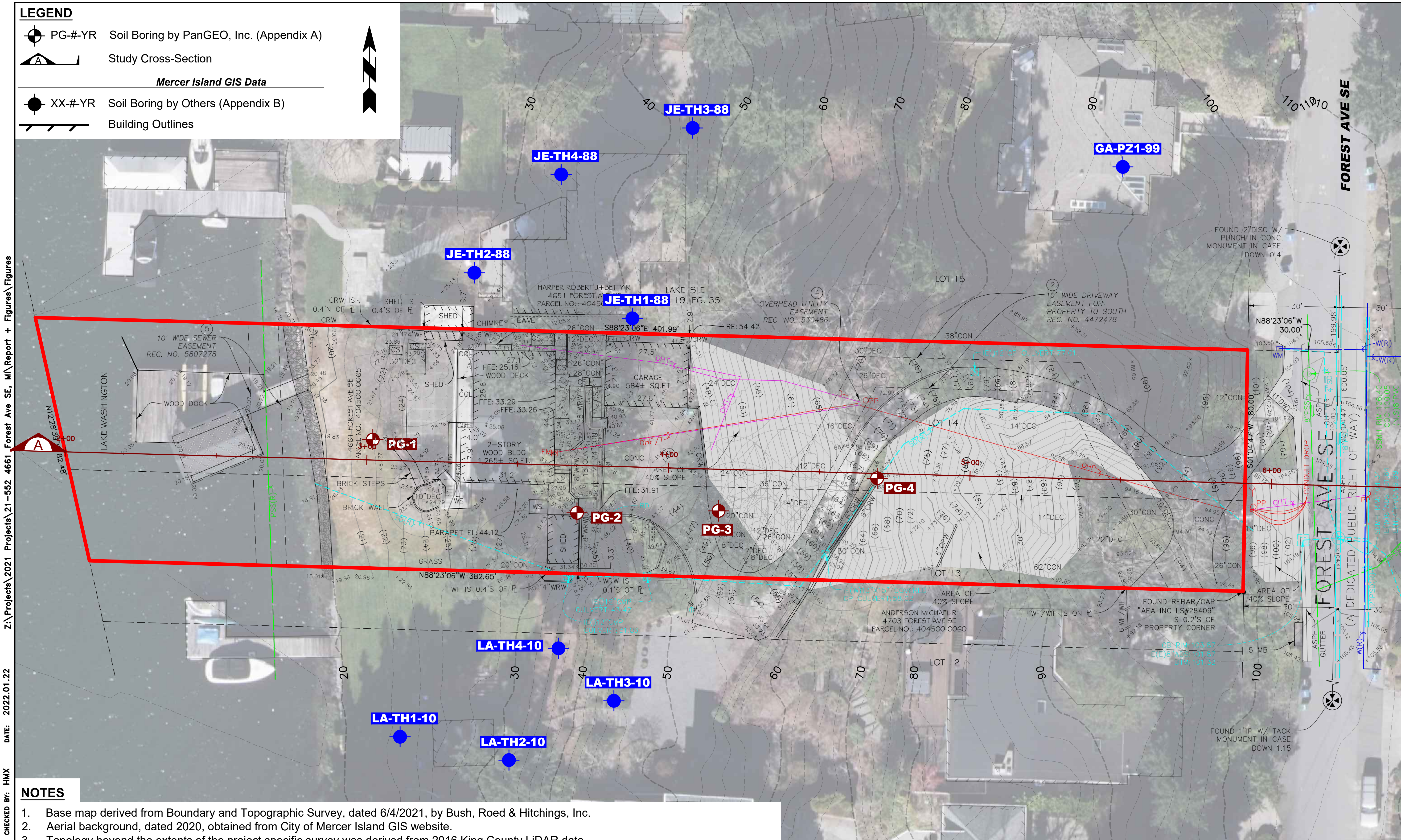
LEGEND

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

Mercer Island GIS Data

- XX-#-YR — Soil Boring by Others (Appendix B)
- Building Outlines



NOTES

1. Base map derived from Boundary and Topographic Survey, dated 6/4/2021, by Bush, Roed & Hitchings, Inc.
2. Aerial background, dated 2020, obtained from City of Mercer Island GIS website.
3. Topology beyond the extents of the project specific survey was derived from 2016 King County LiDAR data obtained from Washington DNR.
4. Additional features are based on GIS data obtained from Washington DNR and City of Mercer Island websites.
5. Supplemental Topology and features are provided for relative information only and are not a substitution for field survey.
6. Locations of subsurface explorations are approximate and based on the relative locations of known site features.
7. Vertical Datum: NAVD 88

15' 0 15' 30'
scale in feet
Scale: 1" =30'

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Zimmer Residence
4661 Forest Avenue SE
Mercer Island, Washington

SITE AND EXPLORATION PLAN
AERIAL IMAGERY

PROJECT NO. 21-552

FIGURE NO. 2B

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DATE: 2022.01.22

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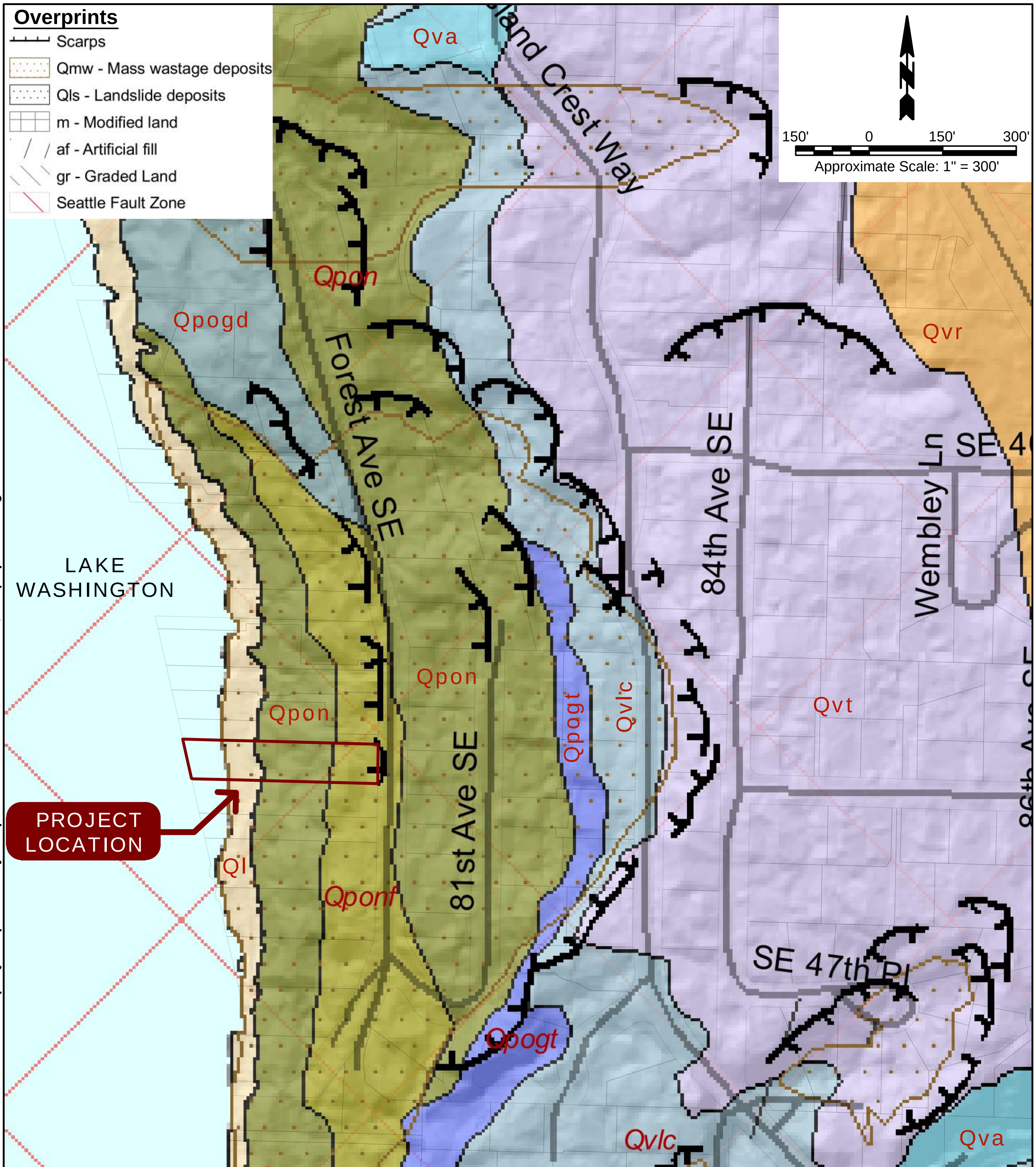

GEOLOGIC UNITS

Ql	Lake deposits
Qvr	Vashon recessional outwash deposits
Qvt	Vashon glacial till
Qva	Vashon advance outwash deposits
Qvlc	Lawton clay
Qpogt	Pre-Olympia glacial till
Qpogd	Pre-Olympia glacial diamict
Qpon	Pre-Olympia nonglacial deposits
Qponf	Pre-Olympia fine-grained nonglacial deposits.

NOTES

1. Derived from the Geologic Map of Mercer Island, Washington (Troost and Wisher, 2006)
2. Detailed descriptions of the geologic units can be found in the text of the report.
3. Only the applicable geologic units are listed.

DATE: 2022.01.22 Z:\Projects\2021 Projects\21-552 4661 Forest Ave SE, MI\Report + Figures

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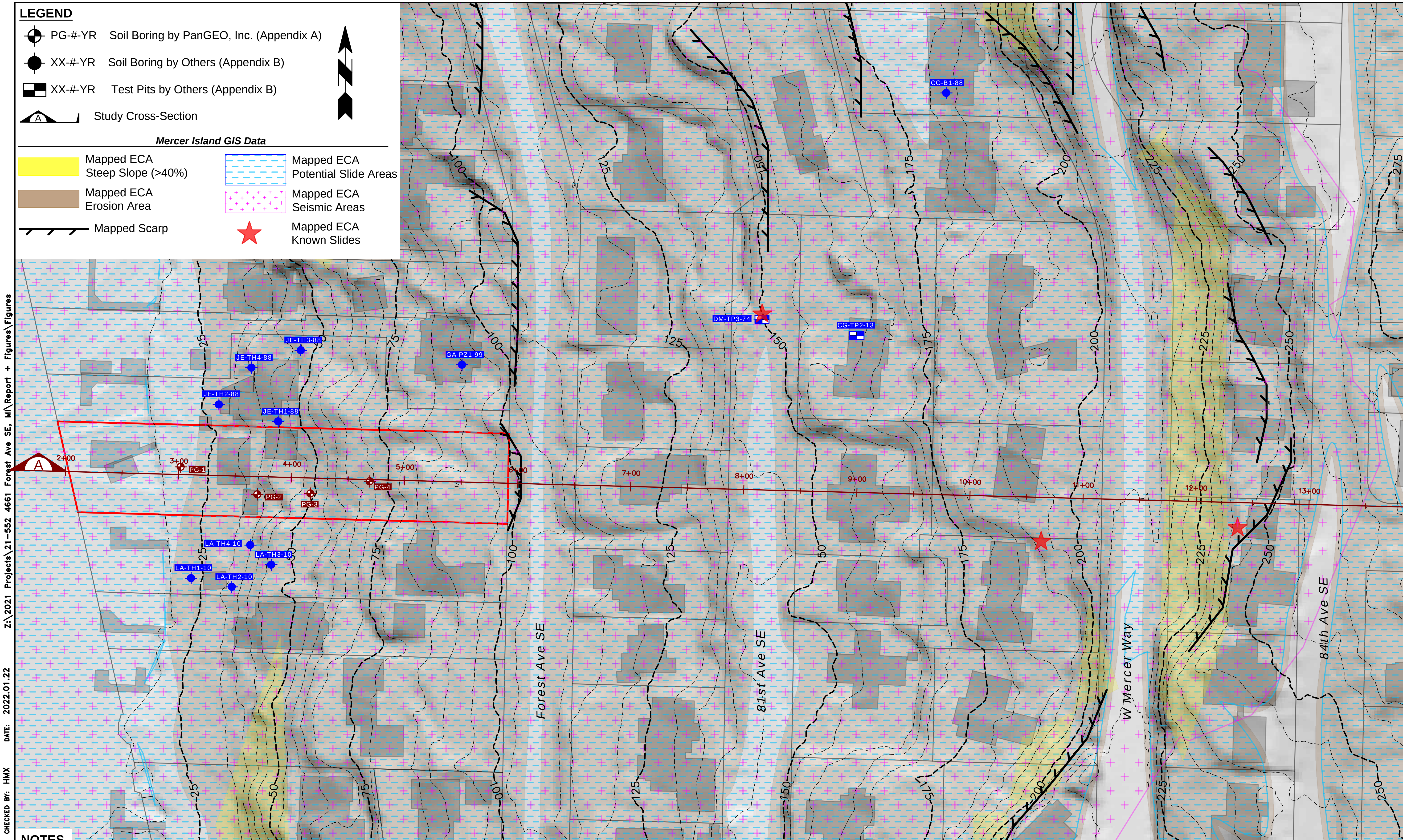
PanGEO INCORPORATED

Zimmer Residence
4661 Forest Avenue SE
Mercer Island, Washington

GEOLOGIC MAP

PROJECT NO. 21-552

FIGURE NO. 3



NOTES

1. Base map and features are based on 2016 King County Lidar and GIS data obtained from Washington DNR and City of Mercer Island websites. Features are provided for relative information only and are not a substitution for field survey.
2. Locations of borings are approximate and based on the relative locations of known site features.
3. Vertical Datum: NAVD 88

40' 0 40' 80'
scale in feet
Scale: 1" =80'

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Zimmer Residence
4661 Forest Avenue SE
Mercer Island, Washington

LIDAR AND CRITICAL AREAS

PROJECT NO.	FIGURE NO.
21-552	4

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DATE: 2022.01.22
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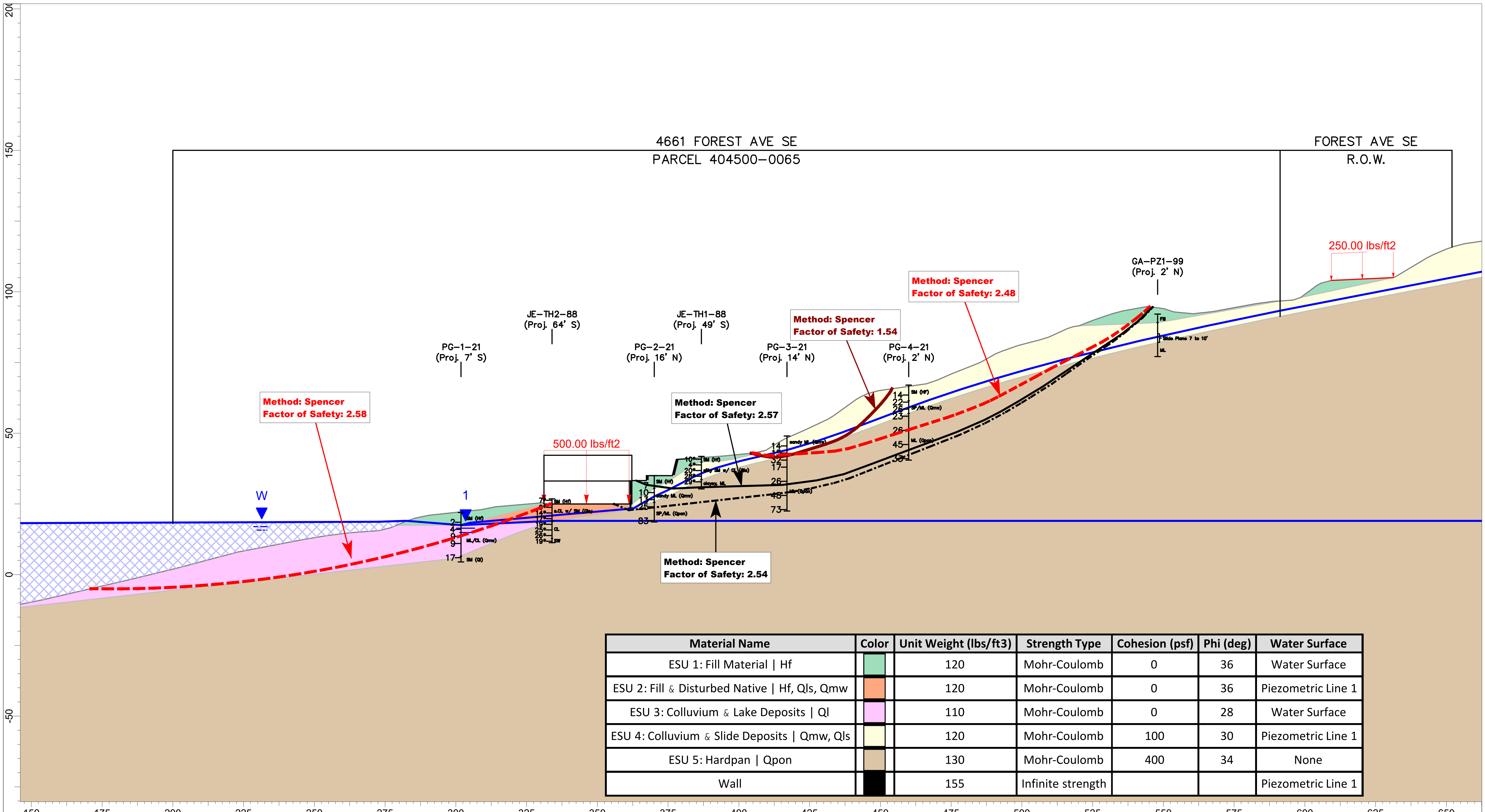


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
ESU 1: Fill Material | Hf		120	Mohr-Coulomb	0	36	Water Surface
ESU 2: Fill & Disturbed Native | Hf, Qls, Qmw		120	Mohr-Coulomb	0	36	Piezometric Line 1
ESU 3: Colluvium & Lake Deposits | Ql		110	Mohr-Coulomb	0	28	Water Surface
ESU 4: Colluvium & Slide Deposits | Qmw, Qls		120	Mohr-Coulomb	100	30	Piezometric Line 1
ESU 5: Hardpan | Qpon		130	Mohr-Coulomb	400	34	None
Wall		155	Infinite strength			Piezometric Line 1

PanGEO INCORPORATED

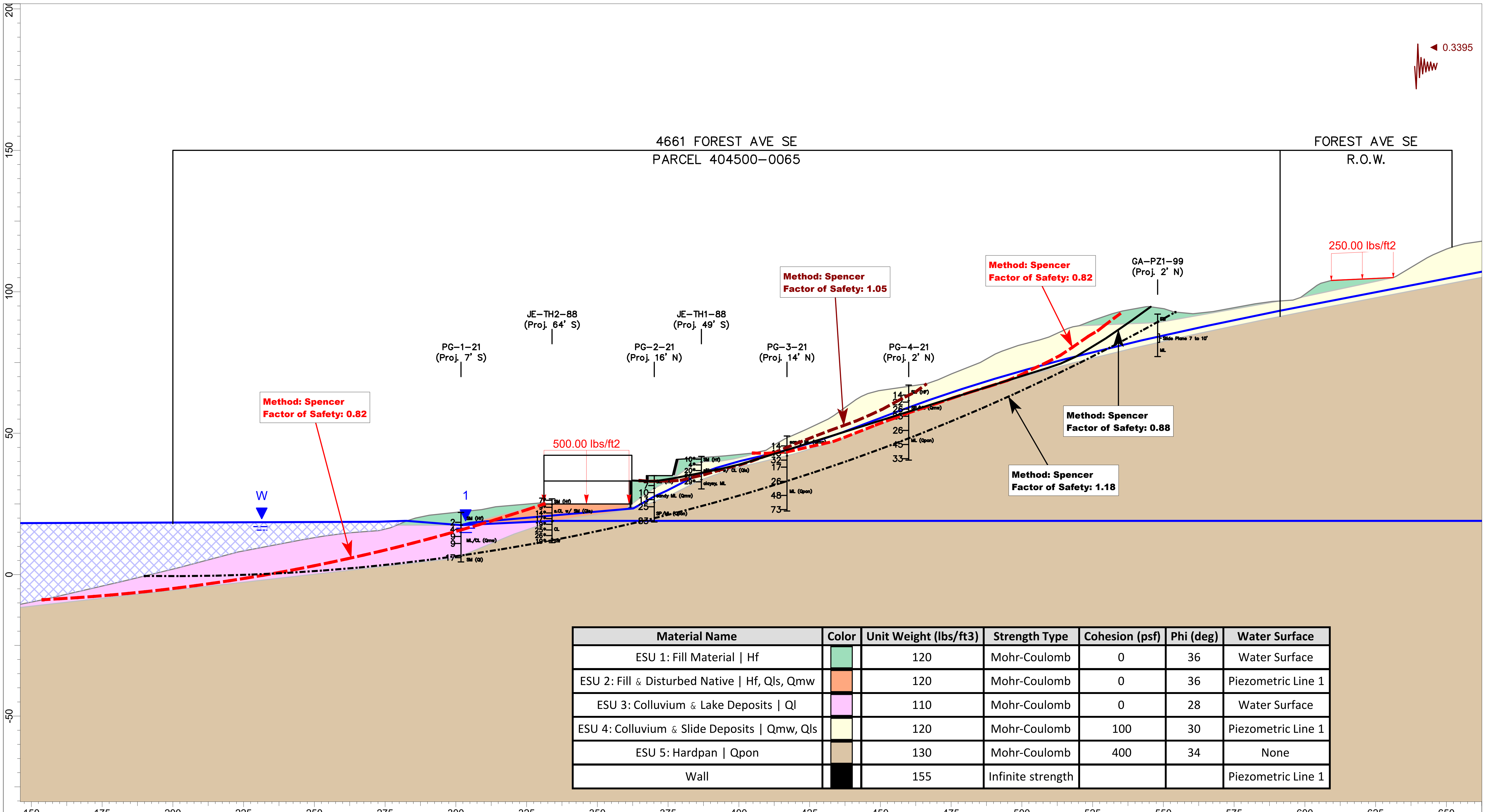
SLIDEINTERPRET 9.022

Zimmer Residence
4661 Forest Avenue SE
Mercer Island, WA

Static Global Stability Analysis

Section A | Existing Condition

Scale:	Project No.	Figure No.
1:380	21-552	5



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
ESU 1: Fill Material | Hf		120	Mohr-Coulomb	0	36	Water Surface
ESU 2: Fill & Disturbed Native | Hf, Qls, Qmw		120	Mohr-Coulomb	0	36	Piezometric Line 1
ESU 3: Colluvium & Lake Deposits | Ql		110	Mohr-Coulomb	0	28	Water Surface
ESU 4: Colluvium & Slide Deposits | Qmw, Qls		120	Mohr-Coulomb	100	30	Piezometric Line 1
ESU 5: Hardpan | Qpon		130	Mohr-Coulomb	400	34	None
Wall		155	Infinite strength			Piezometric Line 1

PanGEO INCORPORATED

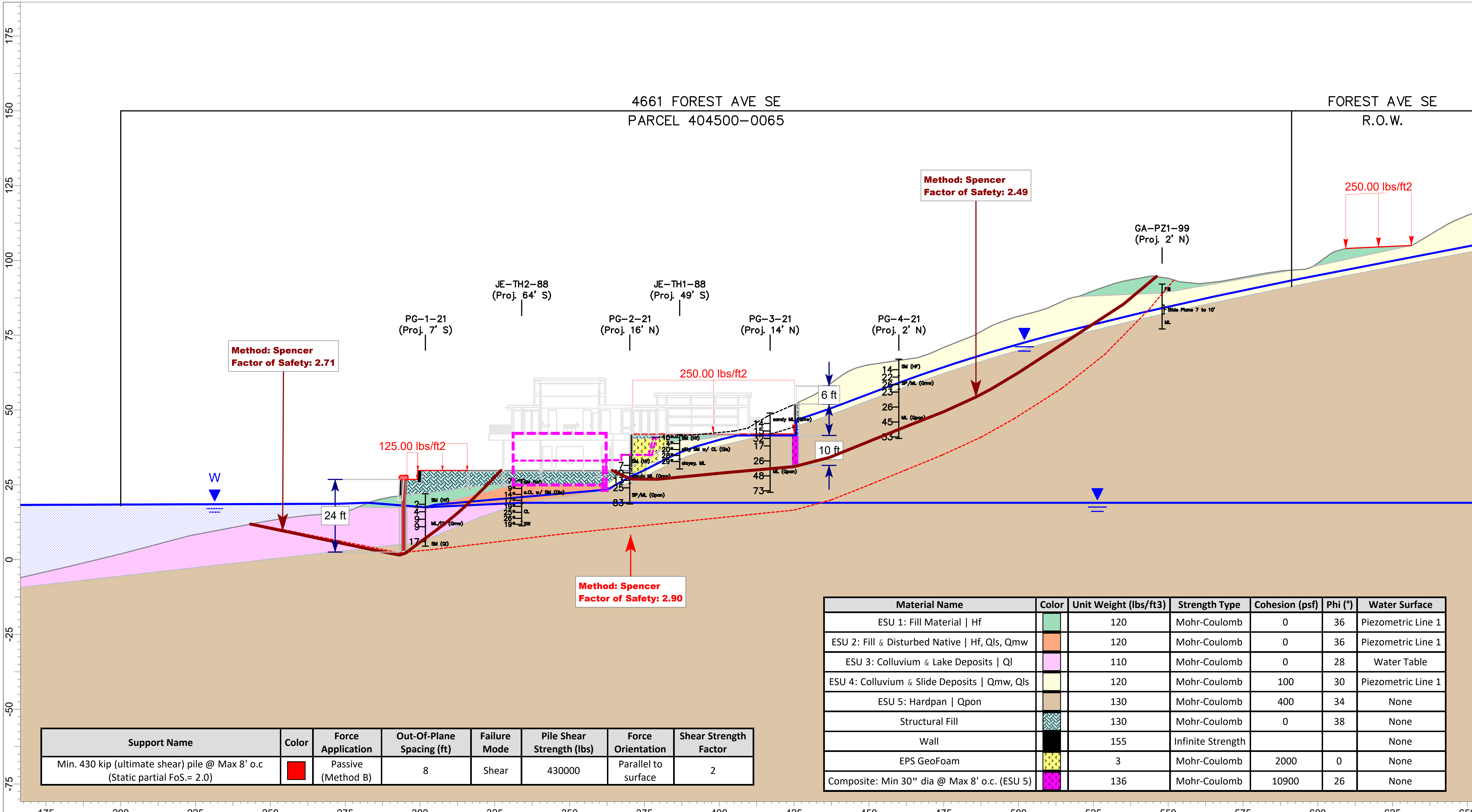
SLIDEINTERPRET 9.022

Zimmer Residence
4661 Forest Avenue SE
Mercer Island, WA

Pseudo-static (Seismic) Global Stability Analysis

Section A | Existing Condition

Scale:	Project No.	Figure No.
1:380	21-552	6



Support Name	Color	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation	Shear Strength Factor
Min. 430 kip (ultimate shear) pile @ Max 8' o.c (Static partial FoS.= 2.0)	(red)	Passive (Method B)	8	Shear	430000	Parallel to surface	2

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface
ESU 1: Fill Material | Hf		120	Mohr-Coulomb	0	36	Piezometric Line 1
ESU 2: Fill & Disturbed Native | Hf, Qls, Qmw		120	Mohr-Coulomb	0	36	Piezometric Line 1
ESU 3: Colluvium & Lake Deposits | Ql		110	Mohr-Coulomb	0	28	Water Table
ESU 4: Colluvium & Slide Deposits | Qmw, Qls		120	Mohr-Coulomb	100	30	Piezometric Line 1
ESU 5: Hardpan | Qpon		130	Mohr-Coulomb	400	34	None
Structural Fill		130	Mohr-Coulomb	0	38	None
Wall		155	Infinite Strength			None
EPS GeoFoam		3	Mohr-Coulomb	2000	0	None
Composite: Min 30" dia @ Max 8' o.c. (ESU 5)		136	Mohr-Coulomb	10900	26	None

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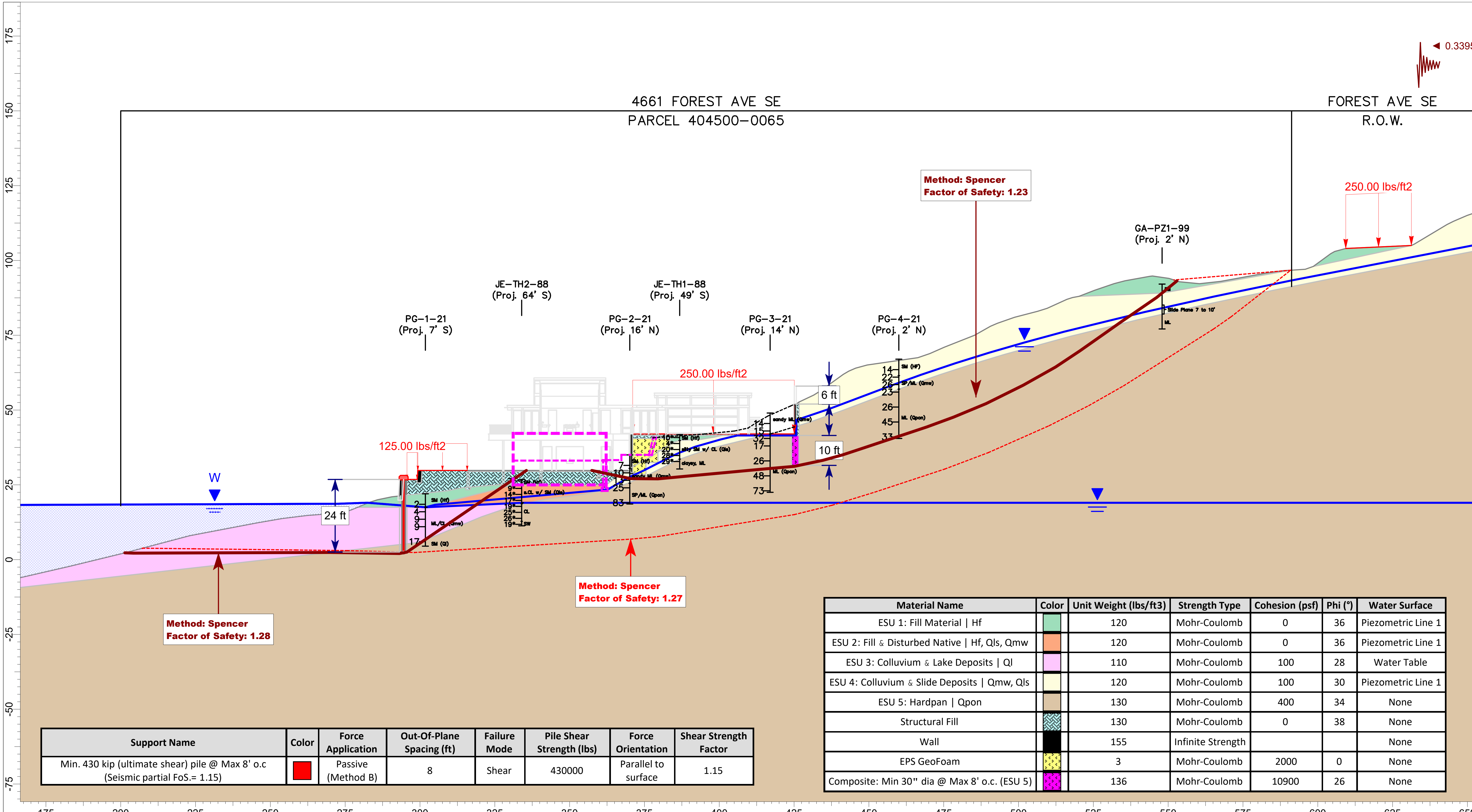
SLIDEINTERPRET 9.036

Zimmer Residence
4661 Forest Avenue SE
Mercer Island, Washington

Static Global Stability Analysis

Section A | Proposed Condition

Scale: 1:360 | Project No. 21-552 | Figure No. 7



Support Name	Color	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation	Shear Strength Factor
Min. 430 kip (ultimate shear) pile @ Max 8' o.c (Seismic partial FoS.= 1.15)		Passive (Method B)	8	Shear	430000	Parallel to surface	1.15

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface
ESU 1: Fill Material | Hf		120	Mohr-Coulomb	0	36	Piezometric Line 1
ESU 2: Fill & Disturbed Native | Hf, Qls, Qmw		120	Mohr-Coulomb	0	36	Piezometric Line 1
ESU 3: Colluvium & Lake Deposits | Ql		110	Mohr-Coulomb	100	28	Water Table
ESU 4: Colluvium & Slide Deposits | Qmw, Qls		120	Mohr-Coulomb	100	30	Piezometric Line 1
ESU 5: Hardpan | Qpon		130	Mohr-Coulomb	400	34	None
Structural Fill		130	Mohr-Coulomb	0	38	None
Wall		155	Infinite Strength			None
EPS GeoFoam		3	Mohr-Coulomb	2000	0	None
Composite: Min 30" dia @ Max 8' o.c. (ESU 5)		136	Mohr-Coulomb	10900	26	None

PanGEO INCORPORATED

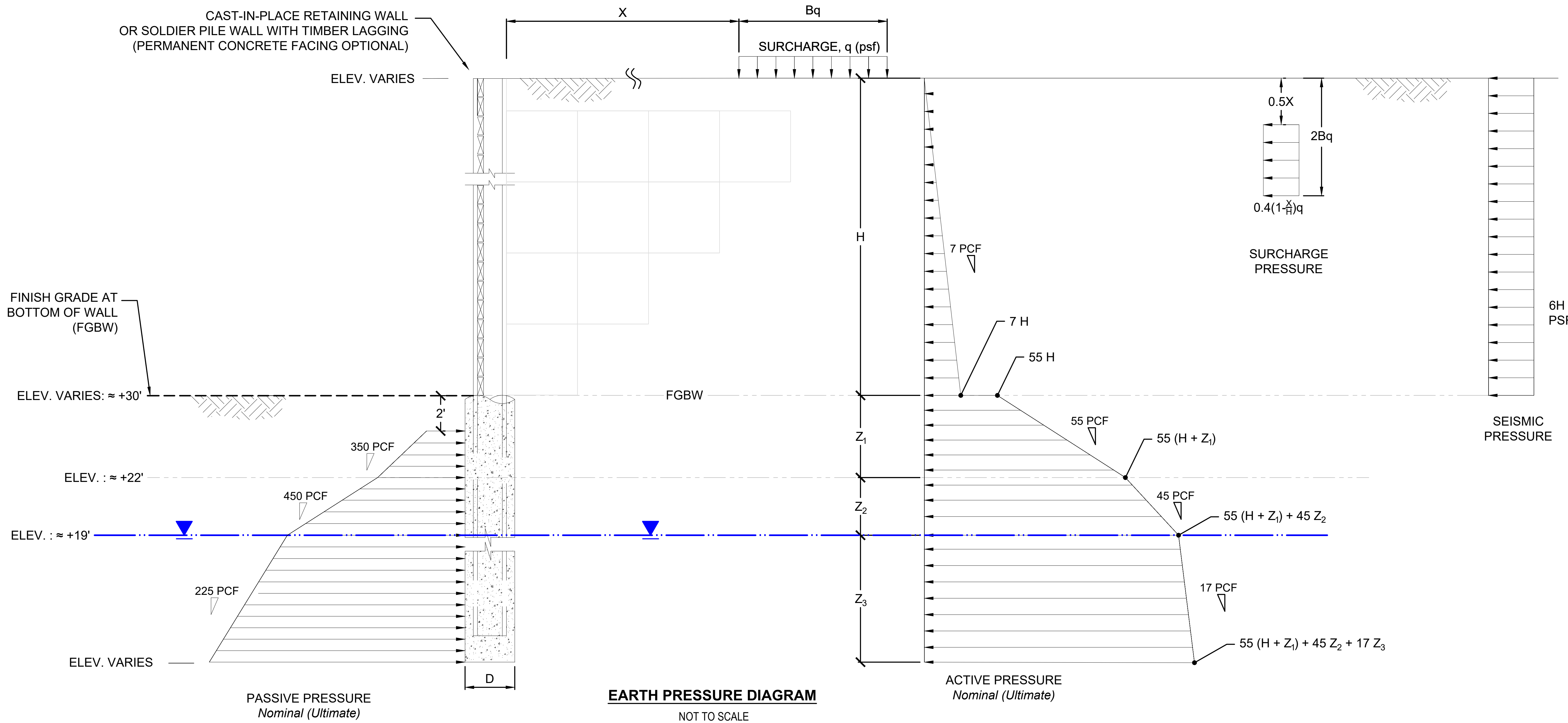
SLIDEINTERPRET 9.036

Zimmer Residence
4661 Forest Avenue SE
Mercer Island, Washington

Pseudo-static (seismic) Stability Analysis

Section A | Proposed Condition

Scale:	Project No.	Figure No.
1:360	21-552	8



EARTH PRESSURE DIAGRAM

NOT TO SCALE

NOTES:

1. The embedment (Z) for soldier piles should be determined by summation of moments at the bottom of the soldier piles or at ground anchor location if present. Minimum pile embedment should be at least 10 feet below bottom of excavation, or deeper as determined by structural analysis.

2. Active and Passive earth pressure values shown are nominal (ultimate) values. Factors of safety of 1.5 and 1.15 should be applied to the passive earth pressure values for static and seismic loading conditions, respectively. No load factors have been applied to the recommended active earth pressure values.

3. Apply active and surcharge pressures over the full width of the pile spacing above the base of the excavation and over one pile diameter below the base of the excavation.

4. Apply passive pressures over two times the pile diameter (D) below the base of the excavation.

5. Use 50% of the active and surcharge pressures for lagging design with soldier piles spaced at 8-ft or less.

7. Allowable vertical soldier pile capacity:
   Skin Friction = 1.0 ksf (Below Elev. 22 ft)
   End Bearing = 30 ksf

8. For seismic condition, combine the seismic pressure (psf) with the static (active and surcharge pressures).

9. Refer to the report text for geofoam recommendations and additional discussions.

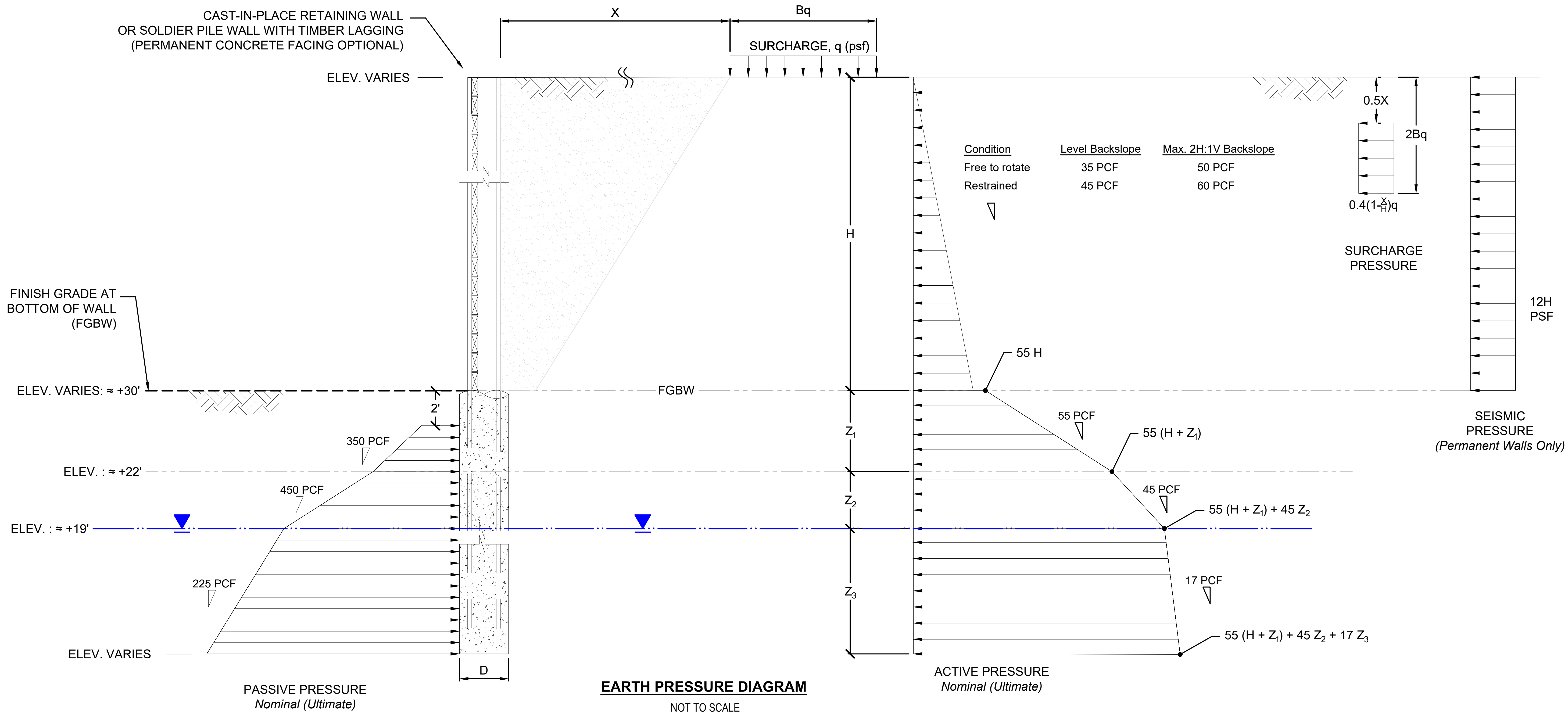
PanGEO INCORPORATED

Zimmer Residence
4661 Forest Avenue SE
Mercer Island, Washington

DESIGN LATERAL PRESSURES
SFR BASEMENT WALL
W/ GEOFOAM BACKFILL
STATIC & SEISMIC CONDITIONS

PROJECT NO.	FIGURE NO.
21-552	9

DRAWN BY: NTW CHECKED BY: HMX DATE: 2024.10.21



EARTH PRESSURE DIAGRAM

NOT TO SCALE

NOTES:

1. The embedment (Z) for soldier piles should be determined by summation of moments at the bottom of the soldier piles or at ground anchor location if present. Minimum pile embedment should be at least 10 feet below bottom of excavation, or deeper as determined by structural analysis.

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3. Apply active and surcharge pressures over the full width of the pile spacing above the base of the excavation and over one pile diameter below the base of the excavation.

4. Apply passive pressures over two times the pile diameter (D) below the base of the excavation.

5. Use 50% of the active and surcharge pressures for lagging design with soldier piles spaced at 8-ft or less.

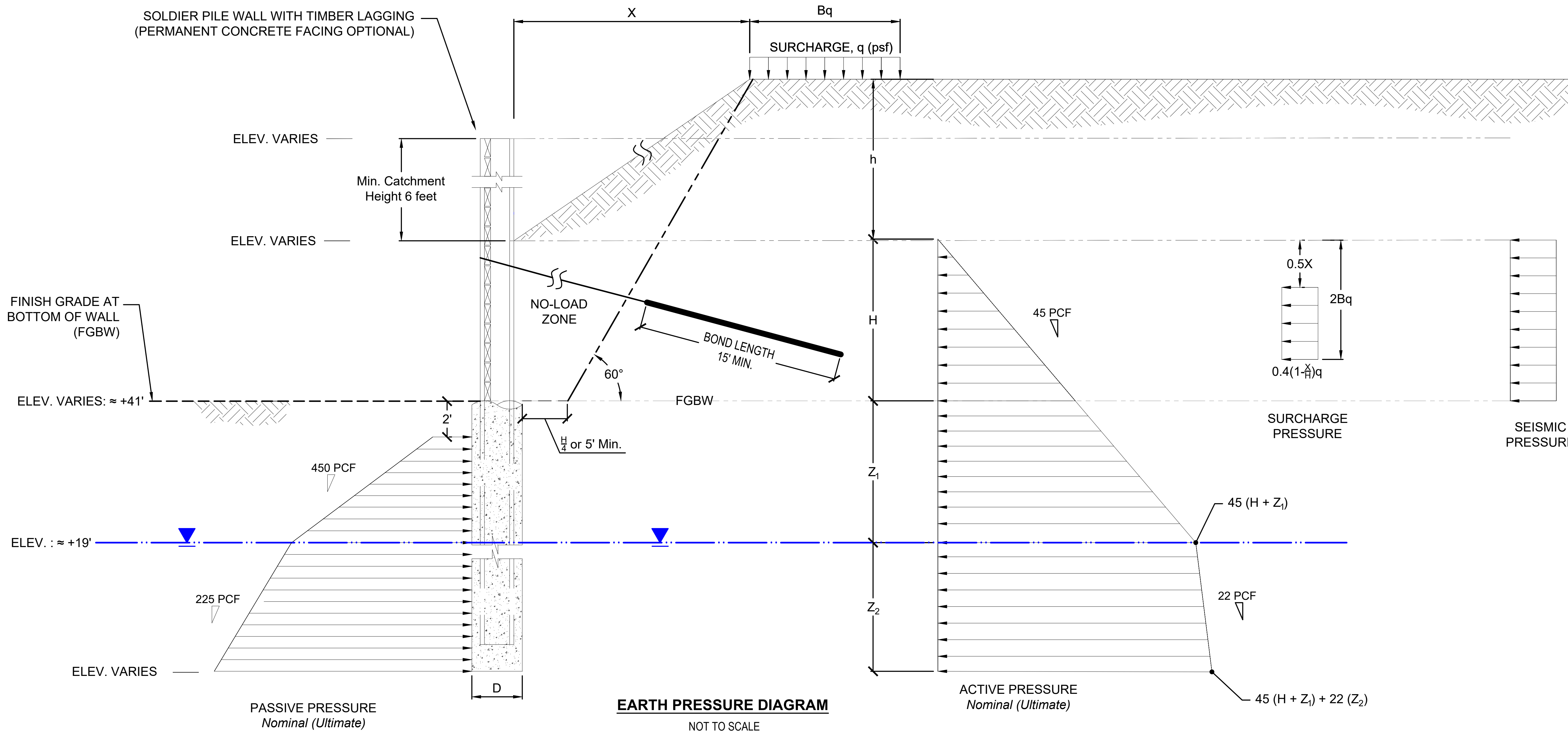
7. Allowable vertical soldier pile capacity:
   Skin Friction = 1.0 ksf (Below Elev. 22 ft)
   End Bearing = 30 ksf

8. For seismic condition, combine the seismic pressure (psf) with the static (active and surcharge pressures).

9. Refer to the report text for geofoam recommendations and additional discussions.

DRAWN BY: NTW CHECKED BY: HMX DATE: 2024.10.21

PanGEO INCORPORATED	Zimmer Residence 4661 Forest Avenue SE Mercer Island, Washington	DESIGN LATERAL PRESSURES SFR BASEMENT WALL WITHOUT GEOFOAM BACKFILL *STATIC & SEISMIC CONDITIONS* PROJECT NO. 21-552 FIGURE NO. 10



EARTH PRESSURE DIAGRAM

NOT TO SCALE

NOTES:

1. Embedment (Z) should be determined by summation of moments at the bottom of the soldier piles or at ground anchor location if present. Minimum pile embedment should be at least 10 feet below bottom of excavation, or deeper as determined by structural analysis.
2. Active and Passive earth pressure values shown are nominal (ultimate) values. Factors of safety of 1.5 and 1.15 should be applied to the passive earth pressure values for static and seismic loading conditions, respectively. No load factors have been applied to the recommended active earth pressures.
3. Apply active and surcharge pressures over the full width of the pile spacing above the base of the excavation and over one pile diameter below the base of the excavation.
4. Apply passive pressures over two times the pile diameter (D) below the base of the excavation.
5. Use 50% of the active and surcharge pressures for lagging design with soldier piles spaced at 8-ft or less.
6. Anchor design provided by others.
7. Allowable vertical soldier pile capacity:
   Skin Friction = 1.0 ksf (Below Elev. 38 ft)
   End Bearing = 30 ksf
8. For seismic condition, combine the seismic pressure (psf) with the static (active and surcharge) pressures.
9. Refer to the report text for anchor recommendations and additional discussions.

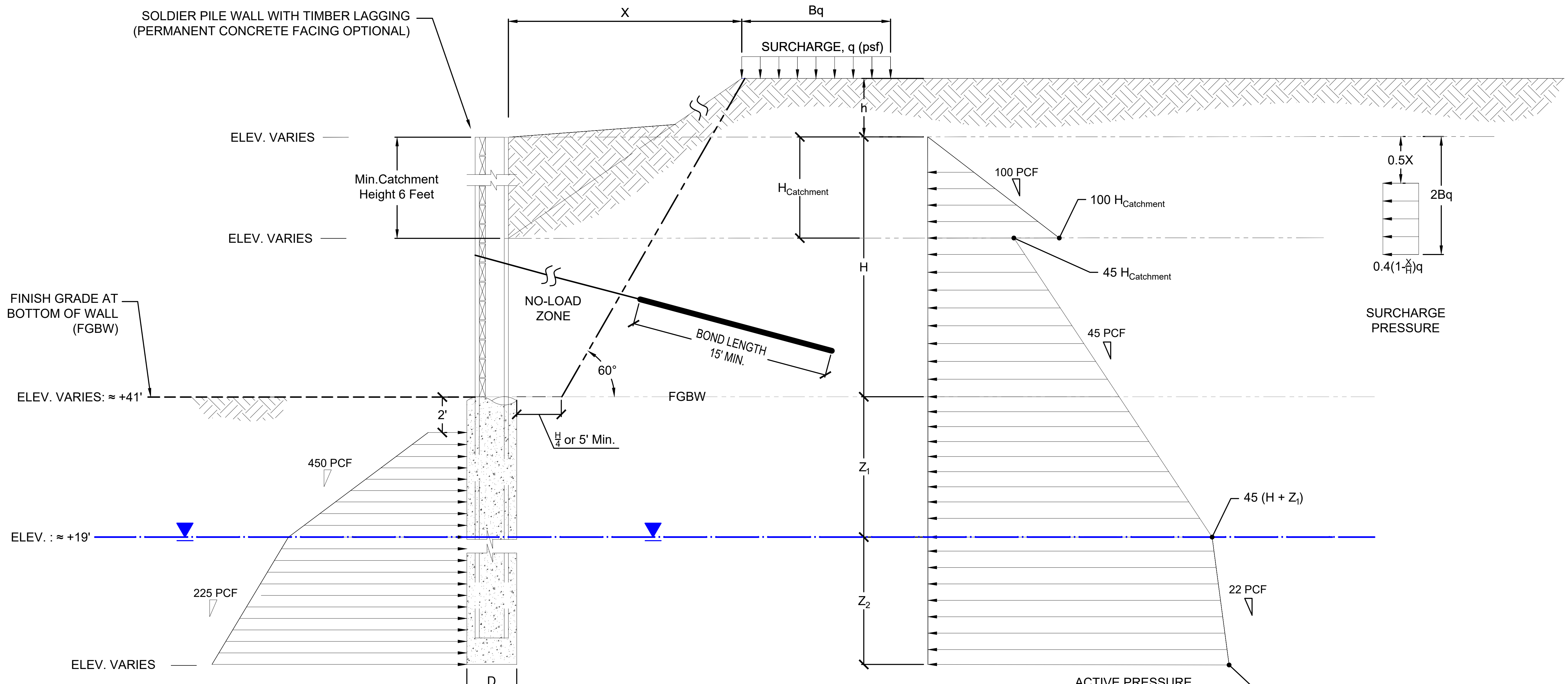
DRAWN BY: NTW CHECKED BY: HMX DATE: 2024.10.21

PanGEO INCORPORATED

Zimmer Residence
4661 Forest Avenue SE
Mercer Island, Washington

DESIGN LATERAL PRESSURES
PERMANENT SOLDIER PILE / CATCHMENT WALL
CANTILEVERED OR SINGLE TIEBACK
STATIC & SEISMIC CONDITIONS

PROJECT NO. 21-552 FIGURE NO. 11



EARTH PRESSURE DIAGRAM

NOT TO SCALE

NOTES:

1. Embedment (Z) should be determined by summation of moments at the bottom of the soldier piles or at ground anchor location if present. Minimum pile embedment should be at least 10 feet below bottom of excavation, or deeper as determined by structural analysis.
2. Active and Passive earth pressure values shown are nominal (ultimate) values. A factor of safety of 1.15 should be applied to the passive passive earth pressure values for the dynamic (moment of slide impact) loading condition. No load factor has been applied to the recommended active earth pressure values.
3. Apply active and surcharge pressures over the full width of the pile spacing above the base of the excavation and over one pile diameter below the base of the excavation.
4. Apply passive pressures over two times the pile diameter (D) below the base of the excavation.
5. Use 50% of the active and surcharge pressures for lagging design with soldier piles spaced at 8-ft or less.
6. Anchor design provided by others.
7. Allowable vertical soldier pile capacity:
   Skin Friction = 1.0 ksf (Below Elev. 38 ft)
   End Bearing = 30 ksf
8. Refer to the report text for anchor recommendations and additional discussions.

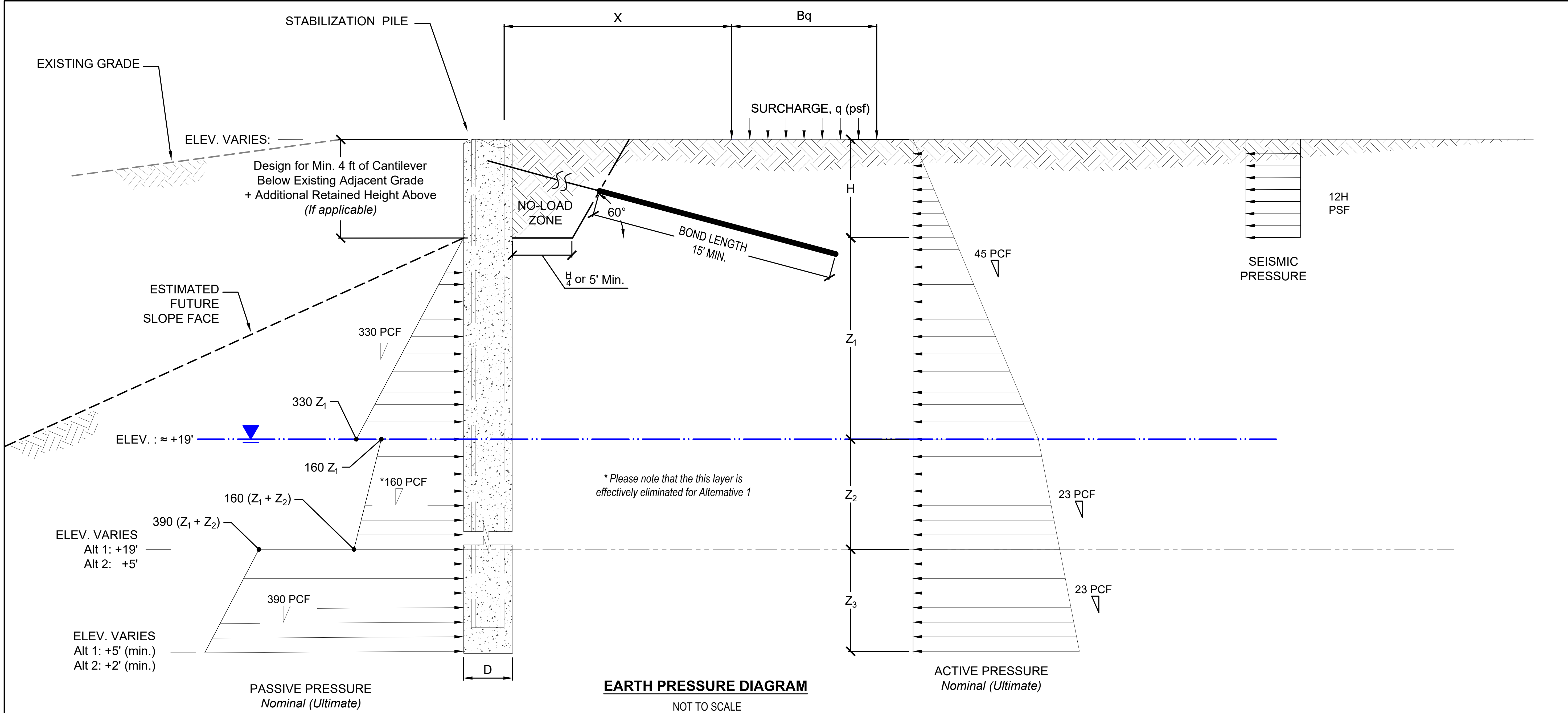
DRAWN BY: NTW CHECKED BY: HMX DATE: 2024.10.21

PanGEO INCORPORATED

Zimmer Residence
4661 Forest Avenue SE
Mercer Island, Washington

DESIGN LATERAL PRESSURES
PERMANENT SOLDIER PILE / CATCHEMENT WALL
CANTILEVERED OR SINGLE TIEBACK
DYNAMIC IMPACT SLIDE CONDITION

PROJECT NO.	FIGURE NO.
21-552	12



EARTH PRESSURE DIAGRAM

NOT TO SCALE

NOTES:

1. Embedment (Z) should be determined by summation of moments at the bottom of the piles or at ground anchor if present. Minimum pile tip elevation (Z) for the stabilization piles should be +5 feet (Alt 1), +2 feet (Alt 2); or deeper as determined by structural analysis.
2. Active and Passive earth pressure value shown are nominal values. Factors of safety of 1.5 and 1.15 should be applied to the passive earth pressure values for static and seismic loading conditions, respectively. No load factors have been applied to the recommended active earth pressures.
3. Apply active and surcharge pressures over the full width of the pile spacing above the base of the excavation and over one pile diameter below the base of the excavation.
4. Apply passive pressures over two times the pile diameter (D) below the base of the excavation.
5. Use 50% of the active and surcharge pressures for lagging design with soldier piles spaced at 8-ft or less.
6. Anchor design provided by others. Refer to the report text for anchor recommendations and additional discussions.
7. Allowable vertical soldier pile capacity:
   - Skin Friction = 1.0 ksf (Below the estimated future slope face: Approx. 4 feet below existing adjacent grade)
   - End Bearing = 30 ksf
8. For seismic condition, combine the seismic pressure (psf) with the static (active and surcharge pressures)
9. Vertical Datum: NAVD 88

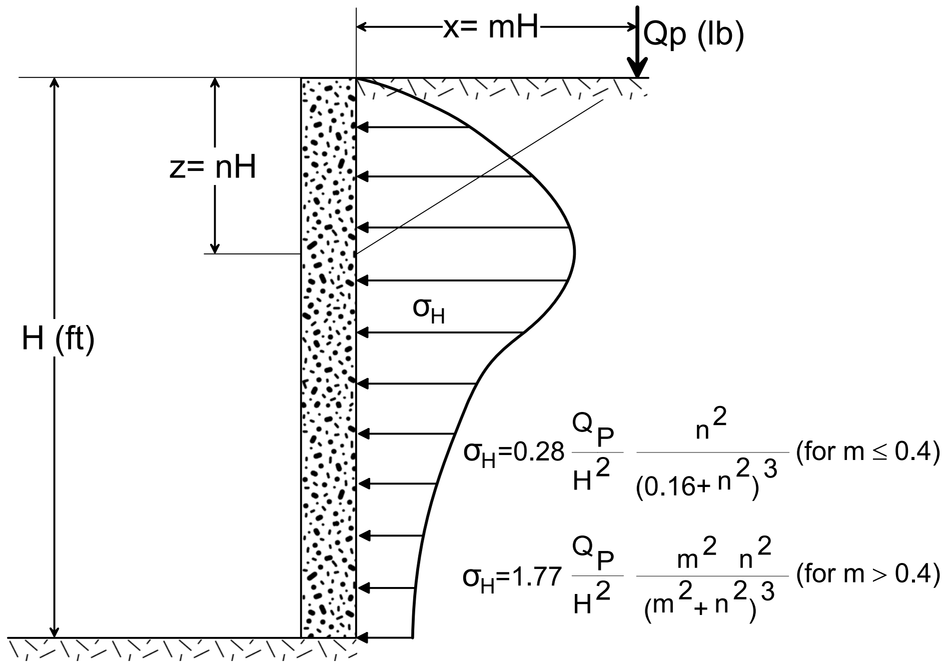
DRAWN BY: NTW CHECKED BY: HMX DATE: 2024.10.25

PanGEO INCORPORATED

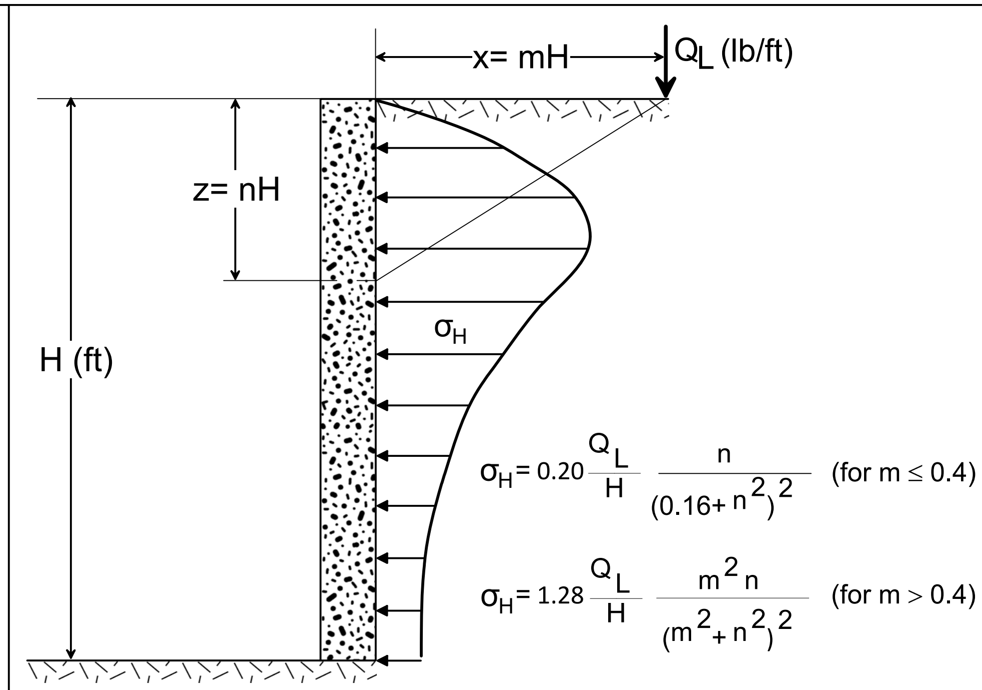
Zimmer Residence
4661 Forest Avenue SE
Mercer Island, Washington

DESIGN LATERAL PRESSURES
LOWER PERMANENT STABILIZATION PILES
CANTILEVERED OR SINGLE TIEBACK
STATIC & SEISMIC CONDITIONS

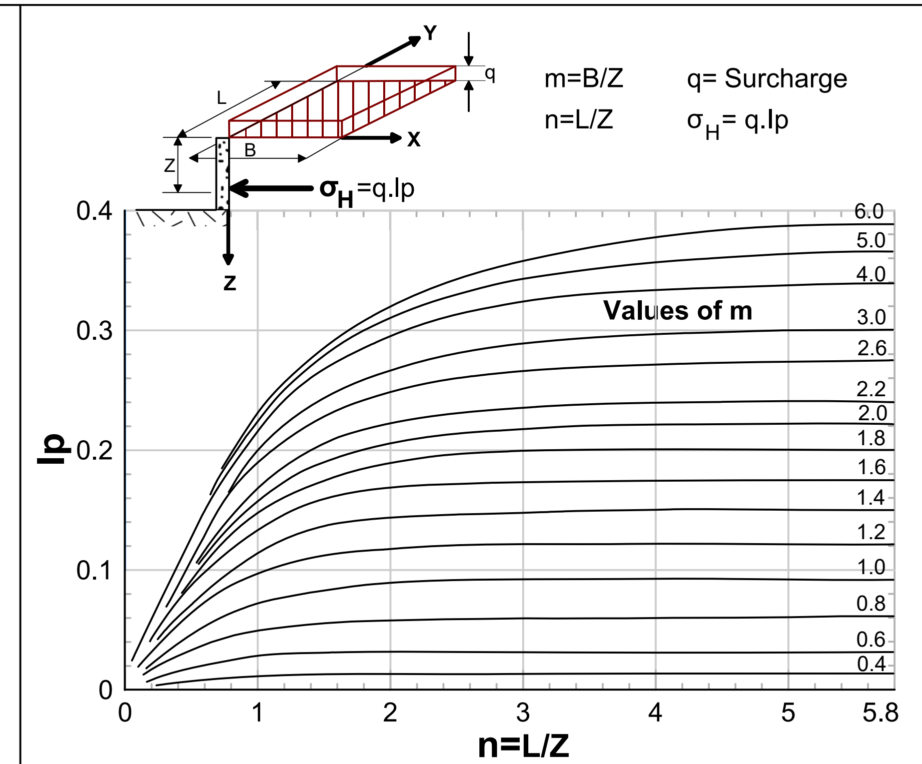
PROJECT NO.	FIGURE NO.
21-552	13



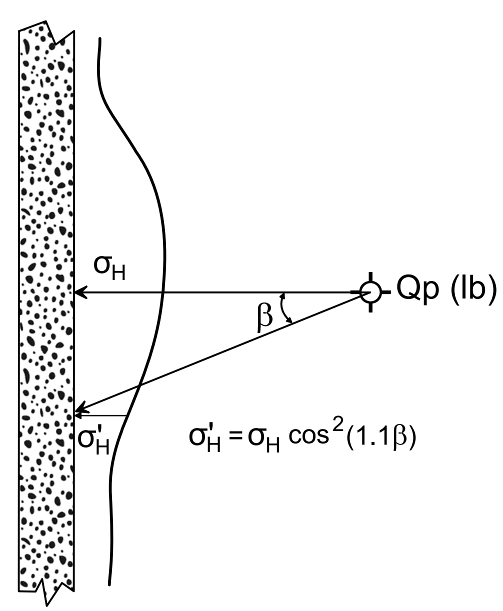
A-1) Lateral Pressure Due to Point Load- Elevation View



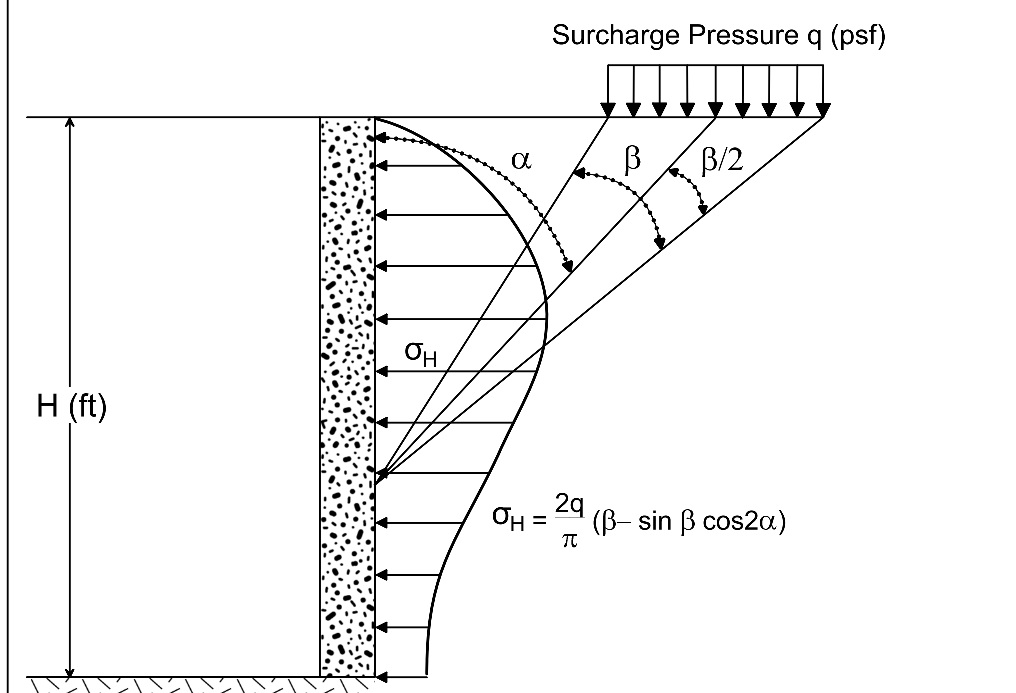
B) Lateral Pressure Due to Line Load-Parallel to the Wall



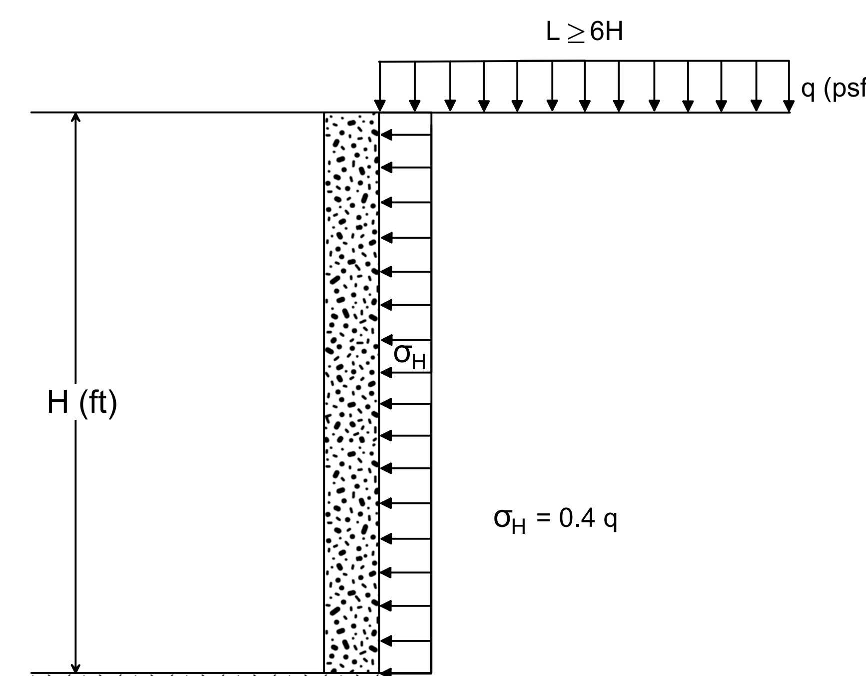
D) Lateral Pressure Due to Adjacent Footing



A-2) Lateral Pressure Due to Point Load- Plan View



C) Lateral Pressure Due to Strip Load-Perpendicular to the Wall



E) Lateral Pressure Due to Uniform Surcharge.
(For L ≤ 6H Use Chart D Above)

* σ_H in psf.

PanGEO INCORPORATED

Zimmer Residence
4661 Forest Avenue SE
Mercer Island, Washingotn

SURCHARGE LOADING ON WALLS

Project No.	Figure No.
21-552	14

APPENDIX A

CURRENT SUBSURFACE INVESTIGATION

4661 Forest Avenue SE | PanGEO, 2021

RELATIVE DENSITY / CONSISTENCY

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

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

DESCRIPTIONS OF SOIL STRUCTURES

Layered: Units of material distinguished by color and/or composition from material units above and below

Laminated: Layers of soil typically 0.05 to 1mm thick, max. 1 cm

Lens: Layer of soil that pinches out laterally

Interlayered: Alternating layers of differing soil material

Pocket: Erratic, discontinuous deposit of limited extent

Homogeneous: Soil with uniform color and composition throughout

Fissured: Breaks along defined planes

Slickensided: Fracture planes that are polished or glossy

Blocky: Angular soil lumps that resist breakdown

Disrupted: Soil that is broken and mixed

Scattered: Less than one per foot

Numerous: More than one per foot

BCN: Angle between bedding plane and a plane normal to core axis

COMPONENT DEFINITIONS

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

TEST SYMBOLS

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

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

SYMBOLS

Sample/In Situ test types and intervals

2-inch OD Split Spoon, SPT (140-lb. hammer, 30" drop)

3.25-inch OD Spilt Spoon (300-lb hammer, 30" drop)

Non-standard penetration test (see boring log for details)

Thin wall (Shelby) tube

Grab

Rock core

Vane Shear

MONITORING WELL

Groundwater Level at time of drilling (ATD)

Static Groundwater Level

Cement / Concrete Seal

Bentonite grout / seal

Silica sand backfill

Slotted tip

Slough

Bottom of Boring

MOISTURE CONTENT

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

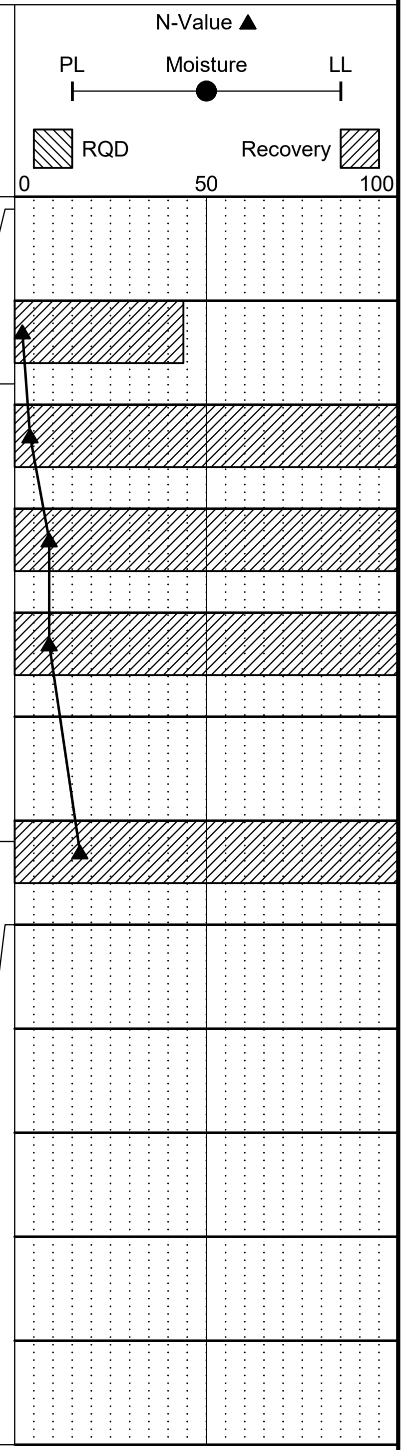
PanGEO INCORPORATED
Phone: 206.262.0370

Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	Zimmer Residence	Surface Elevation:	~22 ft
Job Number:	21-552	Top of Casing Elev.:	n/a
Location:	4661 Forest Ave SE, Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.56205, Easting: -122.23109	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						Grass sod over approximately 3 inches of topsoil (dark brown silty sand with organics).
						FILL [Hf] Very loose/soft, gray, brown to gray, silty SAND/sandy SILT, some organics (roots), trace gravel; moist to very moist, disturbed.
2.5	S-1		1 2 0			
5.0	S-2		2 2 2			**LAKE DEPOSITS (Ql) & COLLUVIUM (Qmw)** Soft to medium stiff, blue-gray, silty CLAY, trace sand and oganics; moist, massive, slightly mottled.
7.5	S-3		2 4 5			-- Becomes stiff, trace gravel; some iron-oxide staining.
10.0	S-4		3 4 5			-- Becomes brown silt with clay; slight blocky texture.
12.5						
15.0	S-5		8 8 9			**PRE-OLYMPIA NONGLACIAL DEPOSITS (Qpon)** Medium dense, brown, silty SAND with gravel; wet to water-bearing, iron-oxide staining, slightly laminated to massive, gravel poorly graded,.
17.5						*Driller reports approximately 3 feet of heaving sand in augers at 17.5 feet below grade. Unable to sample.*
						Boring terminated about 17.5 below grade.
						Minor perched groundwater encountered about 4.5 feet below grade at time of drilling. Groundwater encountered about 16 feet below grade at time of drilling.
20.0						
22.5						
25.0						
27.5						
30.0						



Completion Depth:	17.5ft
Date Borehole Started:	12/15/21
Date Borehole Completed:	12/15/21
Logged By:	S. Harrington
Drilling Company:	Geologic Drill Partners

Remarks: Hand-portable Acker drill rig used. 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 Bush, Roed & Hitchings, Inc., dated June 4, 2021.**Vertical datum: NAVD 88.**

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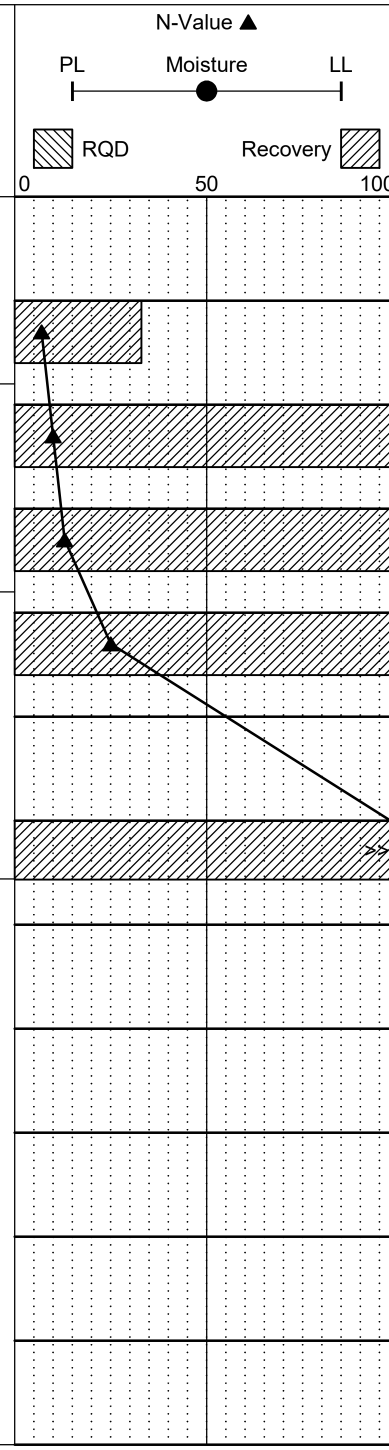
LOG OF TEST BORING PG-1

Figure A-2

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

Project:	Zimmer Residence	Surface Elevation:	~35 ft
Job Number:	21-552	Top of Casing Elev.:	n/a
Location:	4661 Forest Ave SE, Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.56201, Easting: -122.2308	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						**FILL (Hf)** Loose, dark brown to brown, very silty SAND with organics, trace gravel; moist, disturbed.
2.5	S-1		4 3 4			
5.0	S-2		4 6 4			**COLLUVIUM (Qmw)** Stiff, brown, sandy SILT with some organics (roots), trace clay, trace gravel; moist, minor iron-oxide staining, low plasticity.
7.5	S-3		4 6 7			-- Occasional poorly-graded sand layers.
10.0	S-4		6 12 13			**PRE-OLYMPIA NONGLACIAL DEPOSITS (Qpon)** Very stiff / medium dense, bown to grayish-brown, sandy SILT and poorly-graded, medium-grained SAND interbeds, trace gravel; moist.
12.5						
15.0	S-5		17 33 50/5			-- Becomes hard; some iron-oxide staining; grades to blue-gray silt about 16 feet below grade.
						Boring terminated about 16.4 below grade. Groundwater was not ecnountered at the time of drilling.
17.5						
20.0						
22.5						
25.0						
27.5						
30.0						



Completion Depth:	16.4ft
Date Borehole Started:	12/15/21
Date Borehole Completed:	12/15/21
Logged By:	S. Harrington
Drilling Company:	Geologic Drill Partners

Remarks: Hand-portable Acker drill rig used. 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 Bush, Roed & Hitchings, Inc., dated June 4, 2021.**Vertical datum: NAVD 88.**

PanGEO INCORPORATED

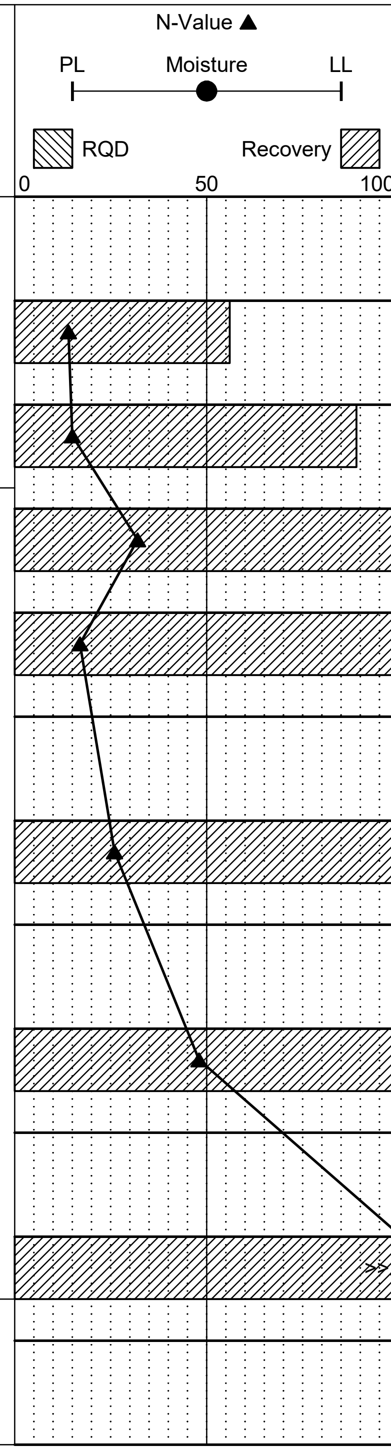
LOG OF TEST BORING PG-2

Figure A-3

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

Project:	Zimmer Residence	Surface Elevation:	~49 ft
Job Number:	21-552	Top of Casing Elev.:	n/a
Location:	4661 Forest Ave SE, Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.56196, Easting: -122.23067	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						**COLLUVIUM (Qmw)** Approximately 5 inches of duff over stiff, brown, sandy SILT with organics, trace gravel; moist, low plasticity.
2.5	S-1	SPT	5 6 8			
5.0	S-2	SPT	3 7 8			-- Occasional sand lenses.
7.5	S-3	SPT	8 14 18			**PRE-OLYMPIA NONGLACIAL DEPOSITS (Qpon)** Hard, grayish-brown, SILT, trace sand; moist, low to medium plasticity, slightly blocky to laminated structure, some iron-oxide staining.
10.0	S-4	SPT	5 7 10			-- Becomes very stiff, gray; massive.
12.5						
15.0	S-5	SPT	8 10 16			-- Trace gravel.
17.5						
20.0	S-6	SPT	12 20 28			-- Becomes hard, trace clay, trace fine sand lenses.
22.5						
25.0	S-7	SPT	22 23 50/6			-- Becomes brown, trace medium sand lenses; minor iron-oxide staining.
27.5						Boring terminated about 26.5 below grade. Groundwater was not ecnountered at the time of drilling.
30.0						



Completion Depth:	26.5ft
Date Borehole Started:	12/15/21
Date Borehole Completed:	12/15/21
Logged By:	S. Harrington
Drilling Company:	Geologic Drill Partners

Remarks: CAT-mounted track drill rig used. 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 Bush, Roed & Hitchings, Inc., dated June 4, 2021. **Vertical datum: NAVD 88.**

PanGEO INCORPORATED

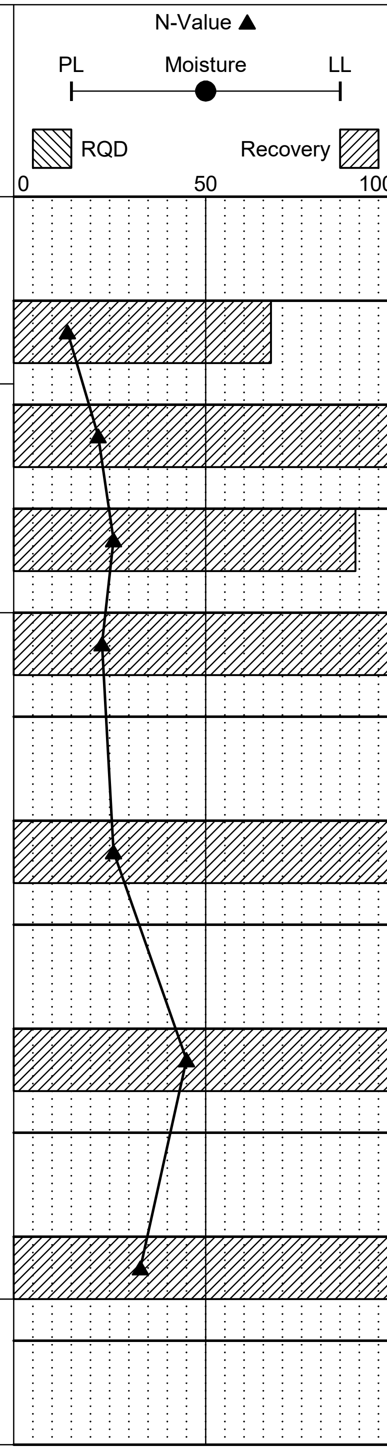
LOG OF TEST BORING PG-3

Figure A-4

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

Project:	Zimmer Residence	Surface Elevation:	~67 ft
Job Number:	21-552	Top of Casing Elev.:	n/a
Location:	4661 Forest Ave SE, Mercer Island, WA	Drilling Method:	HSA
Coordinates:	Northing: 47.56203, Easting: -122.23053	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						**FILL (Hf)** Approximately 6 inches of duff over medium dense, brown, silty fine- to medium-grained SAND, trace fine gravel; moist.
2.5	S-1		3 7 7			
5.0	S-2		4 8 14			**COLLUVIUM (Qmw)** Very stiff/medium dense, brown, SILT with fine- to medium-grained SAND pockets; moist, slightly jumbled texture.
7.5	S-3		5 9 17			-- Trace fine gravel.
10.0	S-4		10 12 11			**PRE-OLYMPIA NONGLACIAL DEPOSITS (Qpon)** Very stiff, brown, SILT, trace clay; moist, massive , minor iron-oxide staining.
12.5						
15.0	S-5		4 12 14			-- Becomes laminated, trace fine sand.
17.5						
20.0	S-6		13 21 24			-- Becomes hard, occasional fine- to medium-grained sand layers.
22.5						
25.0	S-7		7 15 18			-- Becomes gray.
27.5						Boring terminated about 26.5 below grade. Groundwater was not ecnountered at the time of drilling.
30.0						



Completion Depth:	26.5ft
Date Borehole Started:	12/15/21
Date Borehole Completed:	12/15/21
Logged By:	S. Harrington
Drilling Company:	Geologic Drill Partners

Remarks: CAT-mounted track drill rig used. 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 Bush, Roed & Hitchings, Inc., dated June 4, 2021. **Vertical datum: NAVD 88.**

PanGEO INCORPORATED

LOG OF TEST BORING PG-4

Figure A-5

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

APPENDIX B

PREVIOUS SUBSURFACE INVESTIGATION

4703 Forest Avenue SE | Liu & Associates Inc., 2010

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS MORE THAN 50% RETAINED ON THE NO. 200 SIEVE	GRAVEL MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE-GRAINED SOILS MORE THAN 50% PASSING ON THE NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50%	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILTY AND CLAY LIQUID LIMIT 50% OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC SILT, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

NOTES:

1. FIELD CLASSIFICATION IS BASED ON VISUAL EXAMINATION OF SOIL IN GENERAL ACCORDANCE WITH ASTM D2488-83.
2. SOIL CLASSIFICATION USING LABORATORY TESTS IS BASED ON ASTM D2487-83.
3. DESCRIPTIONS OF SOIL DENSITY OR CONSISTENCY ARE BASED ON INTERPRETATION OF BLOW-COUNT DATA, VISUAL APPEARANCE OF SOILS, AND/OR TEST DATA.

SOIL MOISTURE MODIFIERS:

DRY - ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH

SLIGHTLY MOIST - TRACE MOISTURE, NOT DUSTY

MOIST - DAMP, BUT NO VISIBLE WATER

VERY MOIST - VERY DAMP, MOISTURE FELT TO THE TOUCH

WET - VISIBLE FREE WATER OR SATURATED, USUALLY SOIL IS OBTAINED FROM BELOW WATER TABLE

LIU & ASSOCIATES, INC.

Geotechnical Engineering · Engineering Geology · Earth Science

UNIFIED SOIL CLASSIFICATION SYSTEM

PLATE 3

TEST HOLE NO. TH-1

Logged By: MK   Date: 8/31/2010   Ground El. ±

Depth ft.	USCS CLASS.	Soil Description	Sample No.	W %	Other Test
0–1	OL	Grass Sod and Topsoil			
1–4	SP	Gray, loose, gravelly, fine SAND, with some organics in the top portion and occasional cobble in the lower portion of layer, moist.			
4–6	ML	Light-brown with reddish oxidized stains, stiff, becomes hard at 5.0 ft, clayey SILT, with wood fragment and topsoil in the matrix, moist. (gray, medium-dense gravelly sand at 5.9 ft)			
6–	ML	Light-brown to gray, hard, clayey SILT with sand in matrix, moist			
		Test hole terminated @ 6.2 ft; groundwater not encountered.			

TEST HOLE NO. TH-2

Logged By: MK   Date: 8/31/2010   Ground El. ±

Depth ft.	USCS CLASS.	Soil Description	Sample No.	W %	Other Test
0–1	OL	Grass Sod and Topsoil			
1–2	ML	Brownish-gray, soft, clayey SILT, with organics and fractured light-brown clayey silt			
2–5.2	SM	Brownish-gray, loose to medium-dense, gravelly SAND, moist (reddish oxidized stains at 3.0 ft and with clay in sand matrix at bottom of hole. Refusal due to gravel at 5.2 ft)			
		Test hole terminated at 5.2 ft; groundwater not encountered.			

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Geotechnical Engineering · Engineering Geology · Earth Science

TEST HOLE LOGS
NEW CABANA - ANDERSON RESIDENCE
4703 FOREST AVENUE SOUTH
MERCER ISLAND, WASHINGTON

JOB NO. 10-033 | DATE 9/1/10 | PLATE 4

TEST HOLE NO. TH-3

Logged By: JSL   Date: 11/11/2010   Ground El. ±

Depth ft.		USCS CLASS.	Soil Description	Sample No.	W %	Other Test
			Shredded Bark and Topsoil			
1		SM	Brownish-gray, loose, fine gravelly SAND, moist (FILL)			
2, 3		SM	Brownish-gray to light-brown, medium-dense, silty fine SAND, moist			
4, 5		ML	Light-brown to light-gray, fine sandy SILT to clayey SILT, very-stiff to hard, moist			
6, 7, 8			Test hole terminated at 5.5 ft; groundwater not encountered.			

TEST HOLE NO. TH-4

Logged By: JSL   Date: 11/11/2010   Ground El. ±

Depth ft.		USCS CLASS.	Soil Description	Sample No.	W %	Other Test
			Shredded Bark and Topsoil			
1, 2, 3, 4		SM/ML	Brownish-gray to light-brown, loose to medium-dense, silty fine SAND to fine sandy SILT, trace gravel, moist			
5, 6		ML	Light-gray, very-stiff to hard, fine sandy to clayey SILT, moist			
7, 8			Test hole terminated at 5.8 ft; groundwater not encountered.			

LIU & ASSOCIATES, INC.

Geotechnical Engineering · Engineering Geology · Earth Science

TEST HOLE LOGS
NEW CABANA - ANDERSON RESIDENCE
4703 FOREST AVENUE SE
MERCER ISLAND, WASHINGTON

JOB NO. 10-032 | DATE 11/11/10 | PLATE 5

APPENDIX C

PREVIOUS SUBSURFACE INVESTIGATION

4651 Forest Avenue SE | James Eaton P.E., 1988

TEST HOLE LOGS

#1 — Southeast of 4 holes

0' -	(5)	Medium brown loam (fill)
	(9)	
1.2' -	(12)	
		Dark brown loam topsoil
1.8' -		
2.0' -	(4)	Medium brown F-M sand w/ silty pockets and occasional clasks of clay to 1' across, trace of gravel (ancient slide debris)
	(4)	
	(5)	
4.0' -	(8)	
	(19)	
	(21)	
6.0' -	(35)	
	(26)	
	(30)	
8.0' -	(35)	
8.2' -	(28)	
	(30)	Tan to gray clayey silt (horizontally layered, non-fissured hardpan)
11.5' -		Completed 3-1-88; backfilled 3-2-88; no groundwater encountered

#2

0' -	(6) (9)	Variable brown sandy loam and clayey loam (fill)
2.0' -	(3) (9) (9)	Medium brown sandy clayey silt w/ pockets of silty sand and clasks of clay to 6" across, occasional gravel (ancient slide debris)
4.0' -	(8) (13) (16)	
6.0' -	(6) (15) (20)	Sand grades out
8.0' -	(9) (15) (24)	Light gray silty clay below 8-1.2 ft.
10.0' -	(8) (21) (30)	
12.0' -	(12) (30) (22)	
14.0' -	(8) (13) (26)	Brown well graded sand
15.5' -		Completed 3-1-88; groundwater level at 7.8' 3-2-88' backfilled 3-2-88.

#3

0' -	Variable brown clayey loam (ancient slide debris) (generally soft)
8.1' -	Brown thinly and horizontally bedded silt (hardpan)
12.2' -	Completed and backfilled 3-9-88; no groundwater encountered

#4

0' -	Variable silt and clay w/ occasional bits of construction rubble (fill) (soft)
6.9' -	Variable brown clayey silt w/ traces of organic matter (slide debris) (generally soft)
13.0' -	Completed and backfilled 3-9-88; unable to reach deeper; no groundwater encountered

APPENDIX D

PREVIOUS SUBSURFACE INVESTIGATION

4649 Forest Avenue SE | Golder & Associates, Inc., 1999

14102

MI

6282

24-4-13

BORETEC, INC.
Drilling & Sampling

ENTE

PIEZOMETER WELL REPORT

24-4E-13R

START CARD NO. R03220

PROJECT NAME	Mercer Island	COUNTY:	King
WELL ID. NO.:	Piezometer #1	LOCATION:	SE, SE, SEC 13, T~~13~~ 24, R4E
DRILLER:	Ritch Gibson	STREET ADDRESS:	4649 FOREST AVE SE, Mercer Island
FIRM:	Boretec, Inc.		
SIGNATURE:		WATER LEVEL:	NO WATER'
CONSULTING FIRM:	Golder & Associates	DATE INSTALLED	12-20-99
REPRESENTATIVE	Shawn MacInnare	DEVELOPED:	No

WELL DATA