PanGEO
INCORPORATED
*Geotechnical & Earthquake
Engineering Consultants*

January 25, 2022
File No. 21-537

Mr. Kevin Leung
9102 SE 78th Place
Mercer Island, WA 98040

**Subject**: **Geotechnical Report**
**Leung Residence**
**9102 Southeast 78th Place, Mercer Island, Washington**
**Parcels: 919780-0070 and 19780-0060**

Dear Mr. Leung:

Attached please find our geotechnical report for the proposed garage addition and a detached auxiliary structure 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 approximately 20 to 30 feet of loose colluvial soils/slide deposits overlying dense/stiff, glacially-overridden sands and clays. Based on the soil conditions and anticipated finish floor elevation, in our opinion, the proposed structures will need to be vertically supported on small diameter (3- to 4-inch) pipe piles driven to the competent glacially overridden soils located up to approximately 20 to 30 feet below the existing ground surface.

Additionally, the steep slopes above and below the existing residence are marginally stable, especially during a strong IBC level design earthquake event. To provide adequate factors of safety against potential future slope instability, we recommend stabilization piles/walls be installed on the upper and lower lots near the toes of their respective steep slope areas. The upper wall 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 in combination with a fill slope or terraced landscape walls will help buttress the steep slope below the existing residence.

3213 Eastlake Avenue E East, Suite B
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We appreciate the opportunity to work on this project.  Please call if there are any questions.

Sincerely,

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

Encl.:  Geotechnical Report

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# GEOTECHNICAL REPORT
# LUENG RESIDENCE
# 9102 SE 78$^{TH}$ PLACE
# MERCER ISLAND, WASHINGTON

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## 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 garage addition and a detached auxiliary structure at the above site in Mercer Island, Washington. We completed our study in accordance with our proposal dated November 5, 2021, which was subsequently approved on November 12, 2021. Our service scope included reviewing available geologic and geotechnical data in the site vicinity, drilling two three borings at the site, performing engineering analyses, and developing the geotechnical design recommendations presented in this report.

## 2.0 PROJECT AND SITE DESCRIPTION

The project site consists of two adjoining single-family parcels located at 9102 SE 78th Place in Mercer Island, Washington (see Vicinity Map, Figure 1). The combined site is approximately 30,077 square feet in size, and borders SE 78th Place to the south, and existing single-family residences to the other three sides. A single-family house currently occupies the northwest portion of the site. Based on review of the topographic survey map, the site consists of two relatively level benches, separated by a steep slope area. The area to the west of the project site ascends to west with an average gradient of about 60 to 70 percent and a total vertical elevation difference of about 140 feet.

Based on the information provided to us, we understand that the major project elements consist of a garage addition at the southwest corner of the existing house and an auxiliary residential structure located on the lower parcel. Based on the conceptual design plans, the proposed garage addition will be one-story structures with concrete slabs on grade. Conceptual designs of the auxiliary structure were not available at the time this report was prepared but we envisage it will be a one-to two-story wood framed structure. Additionally, the proposed project will also include site walls and walkways. We anticipate that temporary excavations up to about 4 to 8 feet will likely be needed for the foundation and retaining wall construction.

The site is mapped with geologic hazards including potential slide, seismic, and erosion hazards. Additionally, past known slides and springs are also mapped at the site and in the immediately 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 1 through 3 below depict the current condition at the site.

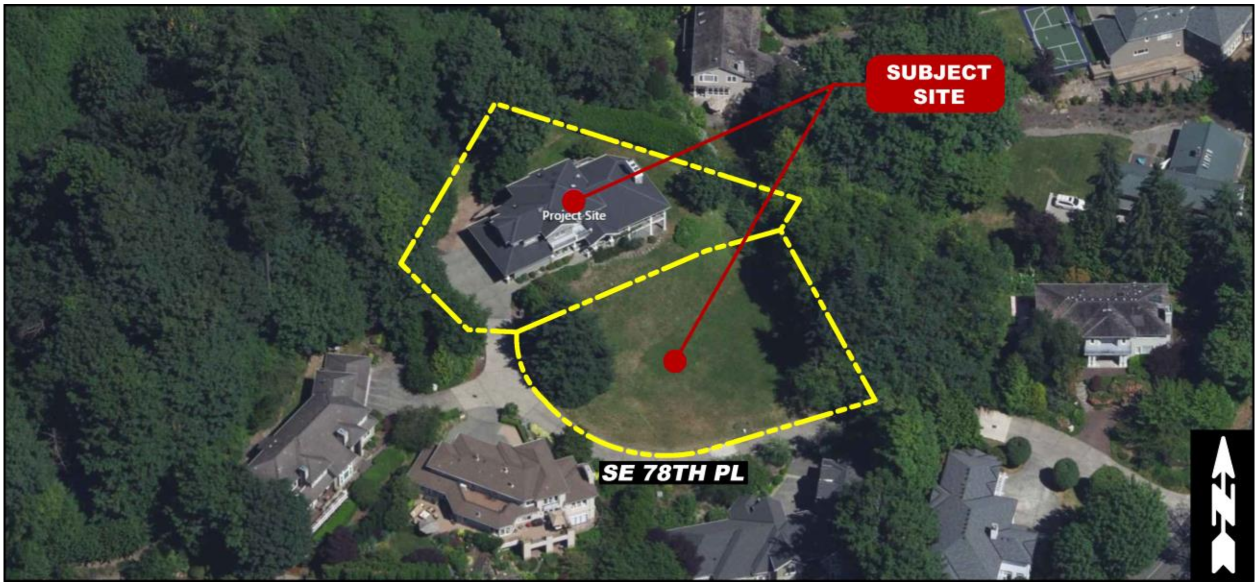


***Plate 1.*** *Oblique aerial view of the project site and surrounding parcels (Source: Bing Maps)*

***Plate 2.*** *General conditions in the area of proposed garage addition, looking north.*

***Plate 3.*** *General conditions of the lower parcel (19780-0060), looking north.*

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 three test borings (PG-1 through PG-3) on December 7, 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 25 to 36 feet below the existing grade, using a mini track mounted drill rig owned and operated by Geologic Drill Partners, Inc. of Fall City, Washington. The drill rigs were equipped with 6-inch outside diameter hollow stem augers.

Soil samples were obtained from the borings at 2½- and 5-foot intervals in general accordance with Standard Penetration Test (SPT) sampling methods (ASTM test method D-1586) in which the samples are obtained using a 2-inch outside diameter split-spoon sampler. The sampler was driven into the soil a distance of 18 inches using a 140-pound weight falling a distance of 30 inches. The number of blows required for each 6-inch increment of sampler penetration was recorded. The number of blows required to achieve the last 12 inches of sample penetration is defined as the SPT N-value. The N-value provides an empirical measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils. The completed borings were backfilled with drill cuttings and bentonite chips. 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-4.

### 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 these previous explorations are indicated on the attached *Figure 3, Lidar and Critical Areas*. Additionally, summary logs from previous explorations are included in Appendices A and B for reference. Specially, the following previous subsurface data that were reviewed for this study:

- PanGEO, Inc. (2020) previously completed two test borings (PG-1 and PG-2) approximately 240 north of the project site at 7710 - 89th Place SE. Both test borings extended about 32 feet below the ground surface;

- Applied Geotechnology, Inc. (1988) previously completed three test pits (TP-1 through TP-3) approximately 560 feet northeast of the project site at 7625 E Mercer Way. The test pit ranged from about 10 to 13 feet below the ground surface;

- Cascade Geotechnical (1987) previously completed two test pits (TP-E and TP-F) approximately 200 feet southwest of the project site at 9060 SE 79th Street. The test pit ranged from about 11 to 12 feet below the ground surface;

- Dames & More (1987) previously completed one test boring (B-3) and two test borings (B-1 and B-2) at the subject site and the neighboring parcel to the northeast, respectively. The test borings ranged from about 16½ to 32 feet below the ground surface;

- Earth Consultants (1986 - 1987) previously completed fifteen test pits and two test borings in the area approximately bounded by SE 78th Place to the south; E Mercer Way to the east; SE 77th Place to the north; and the tall east-facing slope to the west.

  Nine test pits (TP-1 through TP-9) ranging between 4 and 14 feet deep were excavated in 1986. The following year, six additional test pits (TP-101 through TP-106) ranging between 10 to 11 feet deep were excavated. Two test borings (B-1 and B-2) extending about 39 and 45 feet below the ground surface, respectively were also advanced in 1987.

- Earth Consultants (1995) previously completed two test pits (TP-1 and TP-2) at the neighboring parcel to the south of the project site (9103 SE 78th Place). Both test pits were excavated up to 7 feet below the ground surface.

- Geological Services (1987) previously completed four test pits (TP-1 through TP-4) approximately 180 north of the site at 7712 89th Place SE. The test pits ranged from about 10 to 11 feet below the ground surface.

- Hart Crowser (1988 - 1990) previously completed 7 test borings (HC-1 through HC-3, and HC-101 through HC-104) approximately 440 feet northeast of the project site at the Tarywood Park Stairway. The test borings ranged from about 5 to 36 feet below the ground surface.

- Shannon & Wilson (1997) previously completed 3 hand borings (HB-1 through HB-3) approximately 240 feet northwest of the project site at the 7800 89th Place SE. The hand borings ranged from about 5½ to 9 feet below the ground 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 subject site is underlain by Vashon Advance Outwash (Qva), Latwon Clay (Qvlc) and Landslide Deposits (Qls). The geologic map also indicates Peat Deposits (Qp) are also present in the vicinity of the subject site. The attached *Figure 4, 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.

- **Landslide Deposits (Qls) –** Diamict of surficial deposits transported downslope in mass by gravity. Typically consists of intermixed very loose to medium dense coarse-grained deposits and soft to stiff fine-grained deposits with voiding.

- **Vashon Advance Outwash (Qva)** – This deposit typically consists of sediment deposited in front of the advancing ice sheet by glacial melt water (glaciofluvial) and was subsequently overridden by the glacier. Advance outwash deposits are typically dense to very dense in density and typically exhibit low compressibility and high strength characteristic in its undisturbed state.

- **Lawton Clay (Qvlc)** – Typically consists of fine-grained glacial sediment that accumulated in a proglacial lake in the Puget Lowland. Deposits predominantly consist of laminated to massive, very stiff to hard, silty clay and clayey wilt with trace amounts of sand and gravel. Scattered to abundant sheared and slickensided zone are common within this unit. Gravel, cobbles and boulders (drop stones) may be encountered within this unit. Additionally, glaciolacustrine silts typically exhibits low compressibility and high strength characteristics in its undisturbed state. However, zones of low strength material have been well documented within the highly fractures zones with well-defined slickenside planes.

### 4.2 SOIL CONDITIONS

The 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 and colluvium / land slide deposits, overlying undisturbed, stiff to hard glacially overridden clay, which we interpret as Lawton Clay deposits (Qvlc). The interpreted subsurface conditions are depicted in the generalized subsurface profiles shown on *Figure 5*, 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 Appendix A for more details.

> ***Upper Parcel* |** Boring PG-1-21, which was located near the southwest corner of the existing residence, encountered an approximately 5-foot-thick layer of loose silty sand, overlying soft to medium stiff silt and sandy silt which extended up to 11 feet below the ground surface. Which we interpret as Colluvium / Land Slide Deposit.

Below a depth of 11 feet, PG-1-21 encountered loose, fine to medium sand with occasional gravels to maximum depth explored, which we interpreted to be disturbed Vashon Advance Outwash Deposits.

***Lower Parcel* |** Borings PG-2-21 and PG-3-21 were advanced in the vacant parcel below the existing residence. Both borings encountered a sequence of loose granular colluvium/landslide deposits; medium stiff fine-grained deposits; and stiff Lawton Clay. The granular colluvium/landslide deposits consisted of loose, silty sand with various amounts of organic material and had a disturbed soil structure/texture. In general, this layer ranged from a thin surficial layer in PG-3-21 to 9½ feet thick in PG-3-21 which was advanced further down the slope.

Below the granular colluvium/landslide deposits both borings encountered a layer of medium stiff silt with varying amounts of sand, organics, and scattered gravel. The thickness of this layer ranged from about 18 to 21 feet and is most likely native transitional beds typically encountered mantling Lawton Clay deposits.

Underlying the medium stiff fine-grained deposit, both borings encountered a layer of stiff to hard, high plasticity clay and silt which we interpret as Lawton Clay. This unit is characterized by its massive / fine-grained soil structure; and high SPT N-values which is an indication that this engineering soil unit was glacially-overridden. This is the deepest soil unit encountered in these test borings.

***Previous Boring:*** The previous boring DM-B3-99 generally encountered loose medium dense silty sand with interbedded layers of silt and gravel to about 12 feet below the ground surface. Below 12 feet, DM-B3-99 encountered a very stiff silt with occasional sand seams which extended to maximum depth explored of about 16½ feet. Additionally, test pits previously excavated at and adjacent to the project site by Earth Consultants in 1986-1986 generally encountered colluvium / landslide deposit within the depth of the excavations.

Based on the soil descriptions, the upper and lower units appear to be consistent with the ‘granular colluvium / landslide deposits’ and ‘medium stiff fine-grained deposits’ encountered in our test borings. The logs of the previous explorations are included in Appendix B for reference.

### 4.3 GROUNDWATER

Groundwater was encountered between 9½ and 20 feet below the surface during drilling. 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.

***Previous Boring:*** Boring DM-B3-99 previously advanced at the project site did not encounter groundwater at the time of drilling. However, test pits previously excavated at and adjacent to the project site by Earth Consultants in 1986-1986, encounter groundwater with 10 feet of the ground surface. Which is consistent with our test borings and mapped springs in the area. The logs of the previous explorations are included in Appendix B for reference.

## 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 steep slope, potential slide, seismic, and erosion hazards. Additionally, past known slides and springs are also mapped at the site and in the immediately site vicinity. The approximate boundaries of the critical areas, slide initiation points and springs, as mapped by Mercer Island, are depicted on *Figure 3, Lidar and Critical Areas.*

### 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 not limited to planting and hydroseeding. The use of permanent erosion control mat may also be considered in conjunction with planting/hydroseeding to protect the soils 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 soils lose shear strength for short periods of time during a seismic event. Ground shaking of sufficient strength and duration results in the loss of grain-to-grain contact and an increase in pore water pressure, causing the soil to behave as a fluid. Soils with a potential for liquefaction are typically cohesionless, predominately silt and sand sized, must be loose, and be below the groundwater table.

Current and previous subsurface explorations advanced at and near the subject site encountered layers of saturated loose to medium dense sand and medium stiff silt within 10 to 25 feet of the ground surface. which, in our opinion, have a moderate potential to liquefy during a 2,475-year code level earthquake.

However, in our opinion, liquefaction of these layers would not significantly impact the performance of the proposed soldier pile wall, which will be adequately embedded within non-liquefiable soils, and will also contain tiebacks that are anchored within non-liquefiable soils. As such, in our opinion, the design parameters recommended below will adequately address the potential for liquefaction of soils below the site. We recommend that pin piles be used to support the new footings, which will effectively mitigate the risk of the seismic hazard.

**Seismic Induced Settlement –** Ground settlement often occurs as a result of soil liquefaction. Based on the Tokimatus & Seed (1984) & Zhang (2002) methods, we estimate that the potential liquefaction-induced settlement during the IBC level earthquake will generally range from 2 to 4 inches, across the site. Due to the loose thick layer of colluvium / landslide deposits and liquefaction induced seismic settlement, we recommend that the proposed structures be supported on small diameter steel pipe piles (pin piles) that extend past the liquefiable layers to the competent bearings soils below.

**Seismic Induced Lateral Spreading –** Where sloping ground surface conditions are present, the occurrence of soil liquefaction could trigger lateral ground movements, or the horizontal flow of liquefied soils. This phenomenon is generally referred to as lateral spreading. The magnitude of the potential ground movements will largely depend on the earthquake magnitude, geometry of the slope, the depth and thickness of the liquefiable layer, the soil properties, and the distance from the slope face.

Based on our understating of the subsurface conditions, it is our opinion the steep slope area located on the eastside of existing residence has a moderate potential to experience seismic induced lateral spreading during a strong seismic event. Therefore, we recommend stabilization piles be installed near the toe of the steep slope area located between the upper and lower parcels.

### 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 indicated that several known slides were documented near the project site between 1960 and 2015. Our general review of documented slides in the site vicinity is summarized below

> ***Nearby Slide | 9040 SE 79th Street -*** According to the City of Mercer Island Landslide Hazard Map, one known slide events (LS-HMAP-017) occurred at the parcel located approximately 280 feet southwest of the project site sometime between 1960 and 2002. The records indicate the source document was lost and no additional details are provided.
>
> ***Nearby Slides | 7712 89th Avenue SE -*** According to the City of Mercer Island's GIS website, two known slide events (LS-2014-007 and LS-2015-002) occurred at the parcel located approximately 240 feet north of the project site in 2014 and 2015. The 2014 slide is described as the rear yard of the residence settling several inches with 2 to 3 feet from the crest of the slope. The subsequent slide in 2015 is described as a slump in medium dense to dense sand on the steep slope below the south end of the driven pile wall.

In general, the steep east-facing slope above project parcel has experience regular on-going sloughing since at least 1960. The trigger mechanisms are believed to be related to heavy rain falls, natural weathering of the exposed face, and over steepened slopes due to prior developments.

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 a heavily vegetated east-facing slopes immediately west 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 evidence of ongoing 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. Additionally, we observe some evidence of minor groundwater seepage near the toe of the steep slope. However, recent movement 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 describe 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 Terrane, supplemental LiDAR data, and conditions observed during our site reconnaissance, we developed a generalized subsurface profile for the critical cross section which runs perpendicular to the steep slope, just south of the existing residence, as shown as Section A on *Figure 3*.

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 6 through 11*.

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 failure. 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 0.686g was calculated from the USGS seismic hazard deaggregation program. One-half of the expected design ground acceleration (i.e. half of $PGA_M$), or 0.343g was used in our pseudo-static stability analysis.

***Discussion of Results*** – As shown in *Figures 6 and 7*, 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 be installed on the upper and lower lots near the toes of their respective steep slope areas. The upper wall 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 in combination with a fill slope or terraced landscape walls will help buttress the steep slope below the existing residence.

As shown in *Figures 8 through 11*, 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 wall is designed with the recommendations provided in this report.

From a geotechnical perspective, it is our opinion that the proposed soldier pile & lower concrete piles design concept provides adequate stability to the site. In addition, the concept will protect the future auxiliary structure on the lower parcel.

***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 structures 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). The IBC seismic design parameters are in part based on the site soil conditions and site classifications defined in Chapter 20 of ASCE 7. According to Chapter 20 of ASCE 7, the site soil should be classified as Site Class F because of its liquefaction potential during a strong seismic event (see additional discussions regarding liquefaction potential in Section 5.2 of this report). Section 20.3.1 of ASCE 7-16 indicates that for Site Class F a site response analysis in accordance with Section 21.1 shall be performed unless the exception to Section 20.3.1 is applicable.

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

Based on our understanding of the proposed lightly loaded, wood-frame structure, the vibration period for the buildings should be less than 0.5 second.

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

### 6.2 BUILDING FOUNDATIONS

#### *6.2.1 General*

Based on the soil conditions and liquefaction potential, in our opinion, the proposed structures should be supported by deep foundations, such as small diameter steep pipes, to avoid excessive long-term 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. Three, four-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-, 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 page 8). 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-, 4-inch pin piles, the 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-, 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.

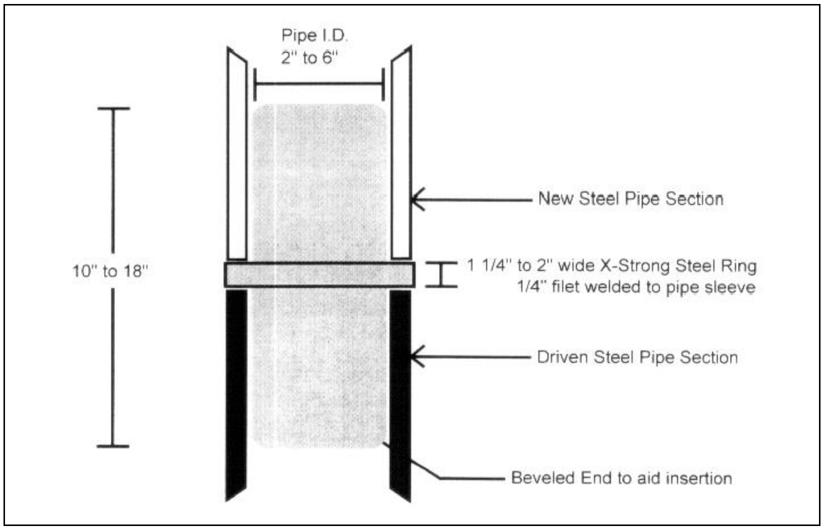
***Pin Pile Specifications*** - We recommend that the following specifications be included on the foundation plan:

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 30 to 45 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-inch compacted structural fill over re-compacted native soil. If the native soils cannot be compacted to a firm conditions, 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 SUBSURFACE DRAINAGE PROVISIONS

Footing drains should be installed around the perimeter of the building, at or just below the invert of the footings. The footing drains should consist of a 4-inch diameter perforated drainpipe placed behind and at the base of the footings, embedded in 12 to 18 inches of clean crushed rock or pea gravel wrapped with a layer of filter fabric.

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

### 6.5 PERMANENT SOLDIER PILE & CATCHMENT WALL | UPPER WALL

In order to bench the proposed garage addition into the sloping grade, excavations up to 8 feet deep will be needed. Due to the proximity of the excavation to an ascending steep slope, an 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 that a soldier pile wall with tiebacks and timber lagging is likely the most cost-effective shoring option.

Additionally, as discussed in the Section 5.5 of this report, a permanent soldier pile wall will be needed along the west property line in order to stabilize the site as part of the proposed project. The permanent soldier pile wall should have a minimum hole diameter of 2 feet with a maximum spacing of 6 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. Due to the proposed backslope and relatively high earth pressures on the wall, we anticipate that tiebacks will be needed for the proposed wall.

***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 *Figure 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 18H 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 the permanent stabilization wall extend to elevation 147 feet or lower.

***Catchment Wall Height & Pressures* -** Due to the proximity of the proposed garage addition and existing residence to the steep slope, we recommend that the permanent soldier pile wall be designed to retain up to 10 feet of soil (i.e. catchment height of 10 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 addition. However, extending the wall further north past the garage addition will provide additional protection to the existing residence. Two conceptual wall alignment alternatives 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 Figure 13 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 12 and 13*.

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 12 and 13*. 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.

***Vertical Capacity* -** Soldier piles may be designed using an allowable skin friction value of 1.0 ksf for the portion of the pile below elevation 152 feet, and an allowable end bearing value of 30 ksf.

***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* -** 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.

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

### *6.5.2 Tiebacks*

Tiebacks will likely be need for wall heights greater than about 8 to 10 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 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, 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.

The actual capacity of the anchors should be checked with 200 percent verification tests. At least two 200% tests should be performed. All production anchors should be proof tested to 130% of the 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:

***Tieback Testing*** **-** The actual capacity of the anchors should be checked with 200 percent verification tests. At least two 200% tests should

The actual capacity of the anchors should be checked with 200 percent verification tests. At least two 200% tests should be performed. All production anchors should be proof tested to 130% of the 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 each 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 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

- 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 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
- 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, if present behind the wall.

### 6.6 DRILLED CONCRETE STABILIZATION PILES & FILL BUTTRESS | LOWER WALL

As discussed in the Sections 5.2 and 5.5 of this report, permanent drilled concrete piles will be needed along the east side of the existing house to stabilize the site as part of the proposed development. The concrete piles should be installed between east building line and the toe of the steep slope below the existing residence. Two conceptual wall alignment alternatives 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. Conceptual Alternatives 1 and 2 are located approximately 30- and 42-feet downslope of the existing residence, respectively. Both alternatives consist of single of row of drilled concrete piles with minimum hole diameters of 2 feet and maximum spacing of 6 feet on center.

In order to achieve adequately factors of safety against future instability along the first conceptional wall alignment ‘Alt. 1’, the top of the piles or grade beam should be located 1 foot below the existing ground surface or higher.

Due to the geometry and orientation of the fore-slope along the second conceptual wall alignment ‘Alt.2’, the minimum top of pile elevations required to satisfy stability vary from just below the ground surface to approximately 5 feet above the existing grade on the west and east ends of the proposed wall, respectively (*see Figures 2A & 2B*). We recommend structural fill be used to backfill the void space between the existing slope and the top of the retaining wall along cantilevered portion of the wall. We also recommend that a fill slope, terraced landscape walls or similar concept be constructure above the top of the wall retaining wall to further buttress the steep slope below the existing residence. The fill slope, if selected, should have a minimum and maximum slope of 3H:1V and 2H:1V, respectively.

The minimum embedment of the drilled concrete piles should be determined by the designer and should extend at least elevation 133 feet to satisfy the global stability. Additionally, the concrete piles should also be designed for the recommended lateral earth pressures depicted in *Figure 14*.

The active earth pressure should be applied over the full width of pile spacing for the cantilever height specified in the earth pressure diagram (*Figure 14*), and over one pile diameter below the wall heights. The passive resistance should be applied over two pile diameters or one pile spacing, whichever is less. The concrete piles should have a minimum reinforcement ratio (e.g. 2%) or based on the structural calculation, whichever is more.

***Vertical Capacity*** **-** The vertical capacity of the drilled concrete piles may be designed using an allowable skin friction value of 0.5 ksf and an allowable end bearing of 15 ksf.

***Fill Buttress*** **-** We recommend granular import fill such as the City of Seattle Type 17 Mineral Aggregate, WSDOT Gravel Borrow, clean crushed rock, quarry spalls, or approved equivalent, to construct the fill buttress and fill the void space between the existing slope and proposed stabilization wall. If quarry spalls or clean crushed rock are used, a geotextile fabric should be placed below the material to prevent the migration of fines into the void space in the quarry spalls or crushed rock. In addition, a geotextile should be placed over the quarry spalls or clean crushed rock before placing granular fill soil such as Type 17 or gravel borrow.

Wall backfill should be moisture conditioned to within about 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and systematically compacted to a dense and relatively unyielding condition and to at least 95 percent of the maximum dry density, as determined using test method ASTM D 1557. Within 5 feet of the wall, the backfill should be compacted to 90 percent of the maximum dry density.

Based on our stability analysis, we recommend 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.

In lieu of the fill slope buttress, alternative design concepts such as terraced landscaping walls may also be used to buttress to the existing slope above the proposed tabulation wall. Design recommendations for retaining walls can be found in the following section.

### 6.7 RETAINING AND BASEMENT WALL DESIGN RECOMMENDATIONS

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

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

***Lateral Earth Pressures*** **-** Concrete cantilever walls should be designed for an equivalent fluid pressure of 35 pcf for level backfills behind the walls assuming the walls are free to rotate. If walls are to be restrained at the top from free movement, such as 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 53 and 60 pcf, respectively.

Permanent 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.

***Wall Surcharge*** **-** Surcharge loads, where present, should also be included in the design of basement walls. We recommend that a lateral load coefficient of 0.35 be used to compute the lateral pressure on the wall face resulting from surcharge loads located within a horizontal distance of one-half wall height.

***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 by 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). 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*** **-**. Retaining wall backfill should consist of free draining granular material. Based on the field exploration, the on-site soil would not be suitable for wall backfill due to its high fines content. We recommend importing a free draining granular material, such as Seattle Type 17 or a soil meeting the requirements of Gravel Borrow as defined in Section 9-03.14(1) of the *WSDOT Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT, 2021) as wall backfill. In areas where space is limited between the wall and the face of excavation, clean crushed rocks may be used as backfill without compaction.

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

Where the below-grade wall will be constructed against a soldier pile wall, 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.

***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 homes. 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 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.

### 6.9 PERMANENT 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 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.

Due to the proximity of the steep slope, infiltration of surface water should not be allowed through dispersion trenches, dry wells, or similar infiltration facilities.

### 6.10 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 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.

### 7.3 SOLDIER PILE AND DRILLED CONCRETE PILE INSTALLATION

Soldier piles will be installed through up to about 25 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

### 7.4 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..

### 7.5 TEMPORARY EXCAVATIONS

We anticipate that excavations less than about 8 feet deep may be needed for the construction of the permanent soldier pile wall. Temporary excavations made near the toe the slope, if needed for pile installation, should be consist of 3-foot-wide excavations on 6-foot centers (~ pile spacing). Adjacent excavations should not be made until piles have been installed in the previously excavated sections.

Temporary excavations greater than 4 feet deep should be properly sloped or shored. All temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring.

For planning purposes, we recommend that temporary excavations be sloped no steeper than 1H:1V (horizontal:vertical), and cuts should not be made into the toe of the steep slope. All cuts must be re-evaluated in the field during construction based on actual observed soil conditions and the presence of groundwater seepage. 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. We also recommend that heavy construction equipment, building materials and excavated soil should not be allowed within a distance equal to 1/3 the slope height from the top of any excavation.

### 7.6 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.

### 7.7 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.

### 7.8 EROSION AND DRAINAGE CONSIDERATIONS

Surface runoff can be controlled during construction by careful grading practices. Typically, this includes the construction of shallow, upgrade perimeter ditches or low earthen berms to collect runoff and prevent water from entering the excavation. All collected water should be directed to a positive and permanent discharge system such as a City of Mercer Island storm sewer.

It should be noted that the site soils are prone to surficial erosion. Special care should be taken to avoid surface water on open cut excavations. We recommend that the exposed temporary slopes be covered with plastic sheeting.

Permanent control of surface water and roof runoff should be incorporated in the final grading design. In addition to these sources, irrigation and rain water 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 or structure and into City of Mercer Island storm drain systems or other appropriate outlets. Adequate surface gradients should be incorporated into the grading design such that surface runoff is directed away from structures.

### 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 Kevin Leung 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.**

HONGBO MICHAEL XUE
35016

1/25/2022

Nick Weikel

Nicholas T. Weikel, E.I.T.
Project Geotechnical Engineer

Michael H. 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.

City of Seattle, 2020, *Standard Specifications for Road, Bridges, and Municipal Construction.*

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 Probabalisitic Ground Motion for the Conterminous 48 States by Latitude and Longitude, 2008 Data,* accessed via: https://seismicmaps.org/

Washington Administrative Code (WAC), 2013, Chapter 296-155 - *Safety Standards for Construction Work, Part N - Excavation, Trenching, and Shoring*, Olympia, Washington.

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

Source: Open Street Maps

Approximate Scale: 1" = 500'

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Leung Residence
9102 SE 78th Place
Mercer Island, Washington

VICINITY MAP

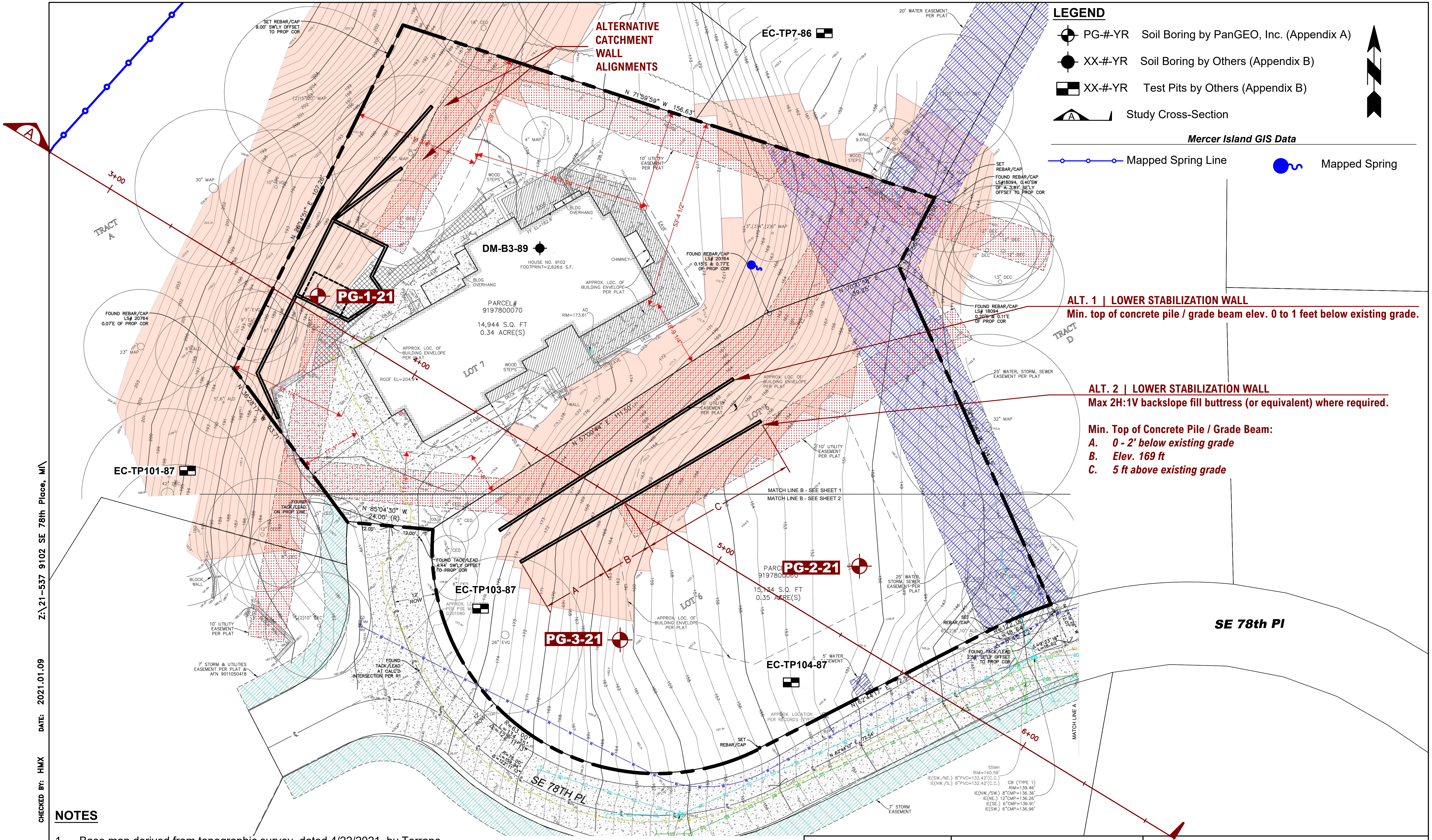
| PROJECT NO. | FIGURE NO. |
|---|---|
| 21-537 | 1 |

DRAWN BY: NTW

CHECKED BY: HMX

DATE: 2022.01.07

Z:\Projects\2020 Projects\20-236 7710 - 89th PL SE, Mercer Island\Figures\CADD\Vicinity Map.dwg



## NOTES

1. Base map derived from topographic survey, dated 4/22/2021, by Terrane.
2. Additional features are based on 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.
3. Locations of subsurface explorations are approximate and based on the relative locations of known site features.
4. Vertical Datum: NAVD 88

12.5' 0 12.5' 25'
scale in feet
Scale: 1" =25'

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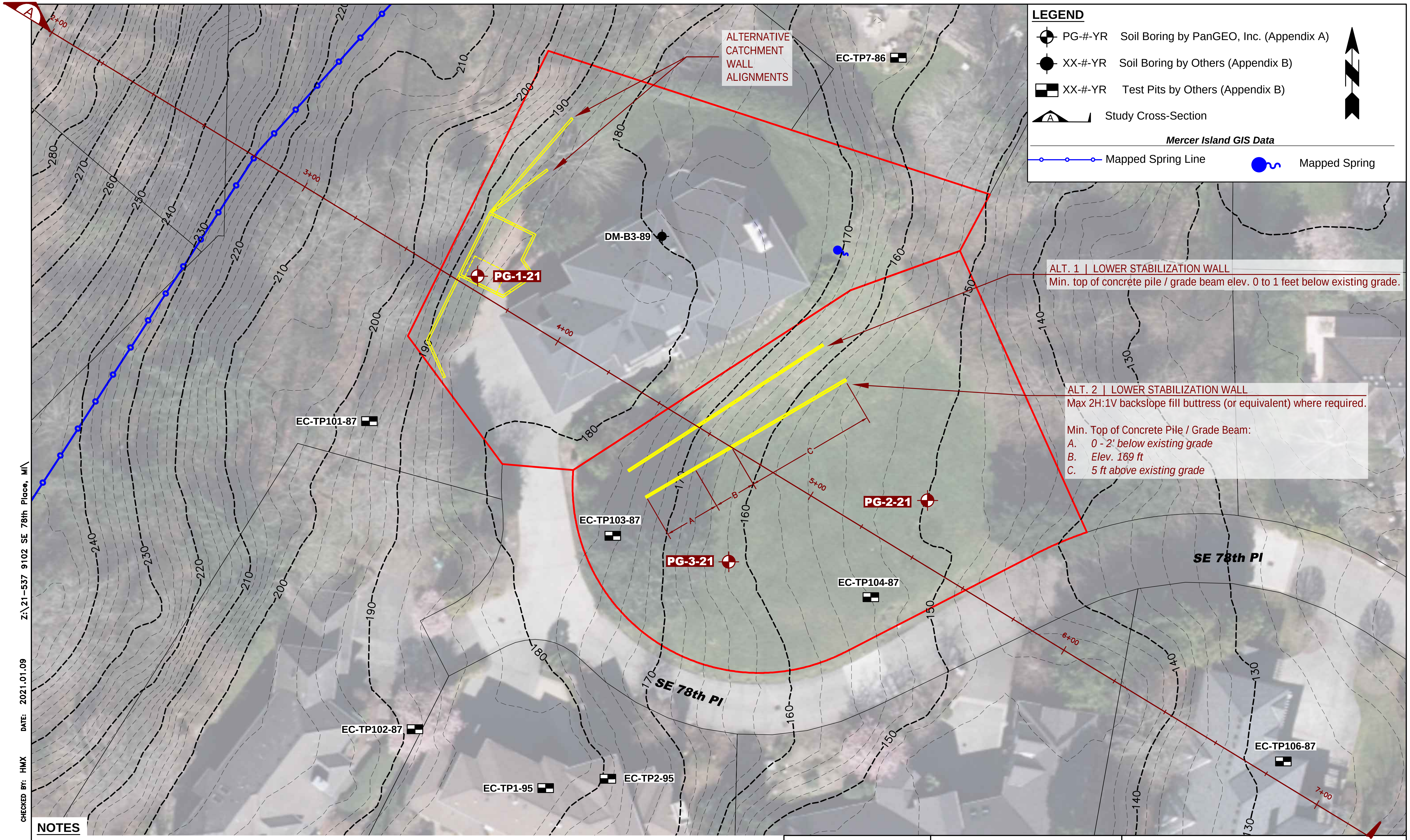
Mercer Island, Washington

SITE AND EXPLORATION PLAN
TOPOGRAPHIC SURVEY

PROJECT NO. 21-537
FIGURE NO. 2A

DRAWN BY: NTW
CHECKED BY: HMX
DATE: 2021.01.09
Z:\21-537 9102 SE 78th Place, MI\



Z:\21-537 9102 SE 78th Place, MI\

DATE: 2021.01.09

CHECKED BY: HMX

DRAWN BY: NTW

## NOTES

1. Base map and features are based on 2016 King County Lidar, 2020 Aerial Imagery, 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 subsurface explorations are approximate and based on the relative locations of known site features.
3. Vertical Datum: NAVD 88

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

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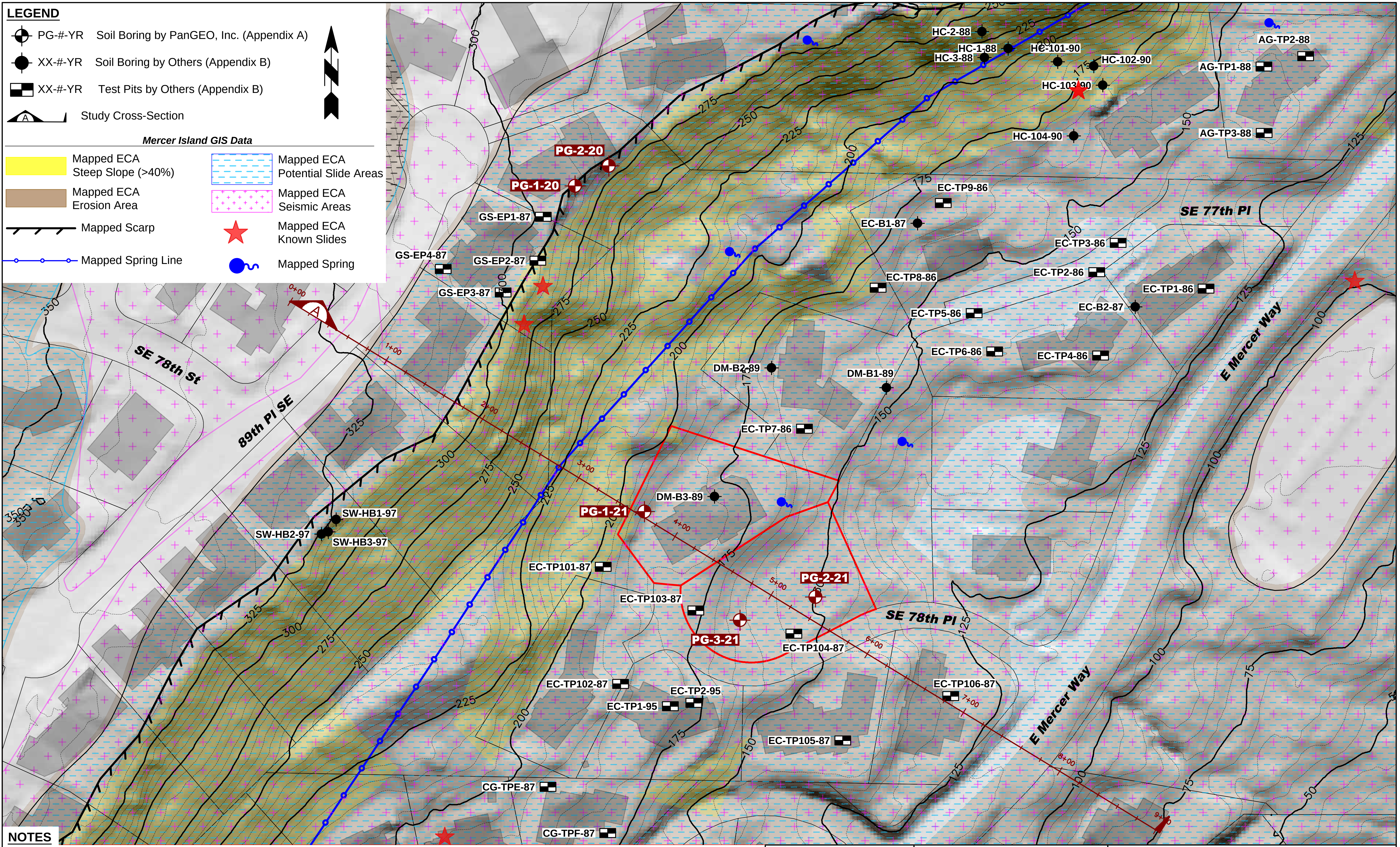
Mercer Island, Washington

SITE AND EXPLORATION PLAN
AERIAL

PROJECT NO. 21-537

FIGURE NO. 2B



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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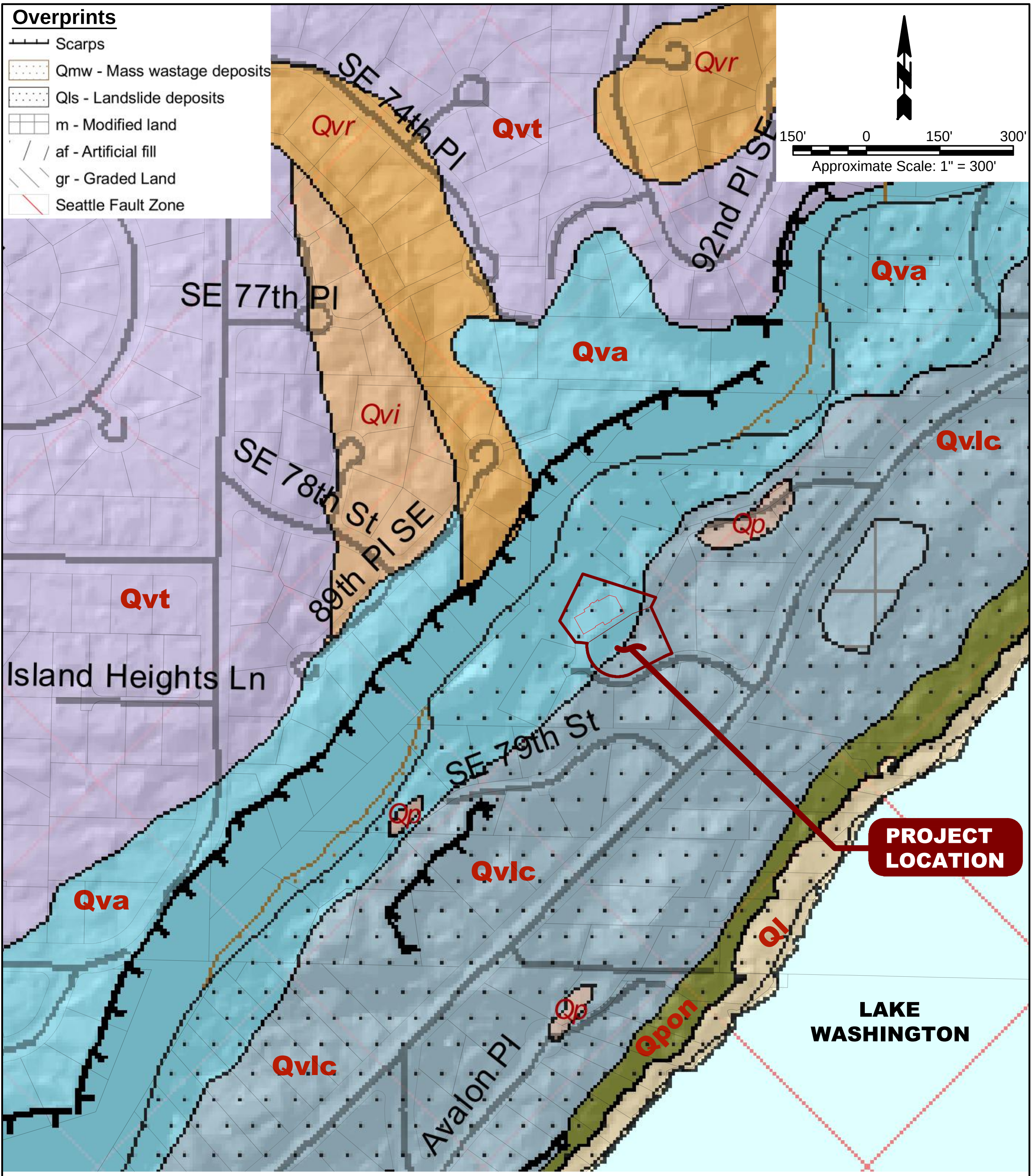
9102 SE 78th Place
Mercer Island, Washington

LIDAR AND CRITICAL AREAS

PROJECT NO. 21-537

FIGURE NO. 3

Z:\21-537 9102 SE 78th Place, MI\
DATE: 2021.01.09
CHECKED BY: HMX
DRAWN BY: NTW



## GEOLOGIC UNITS

| | |
|---|---|
| Qp | Peat |
| Ql | Lake deposits |
| Qvr | Vashon recessional outwash deposits |
| Qvi | Vashon ice-contact deposits |
| Qvt | Vashon glacial till |
| Qva | Vashon advance outwash deposits |
| Qvlc | Lawton clay |
| Qpogn | Pre-Olympia, 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.

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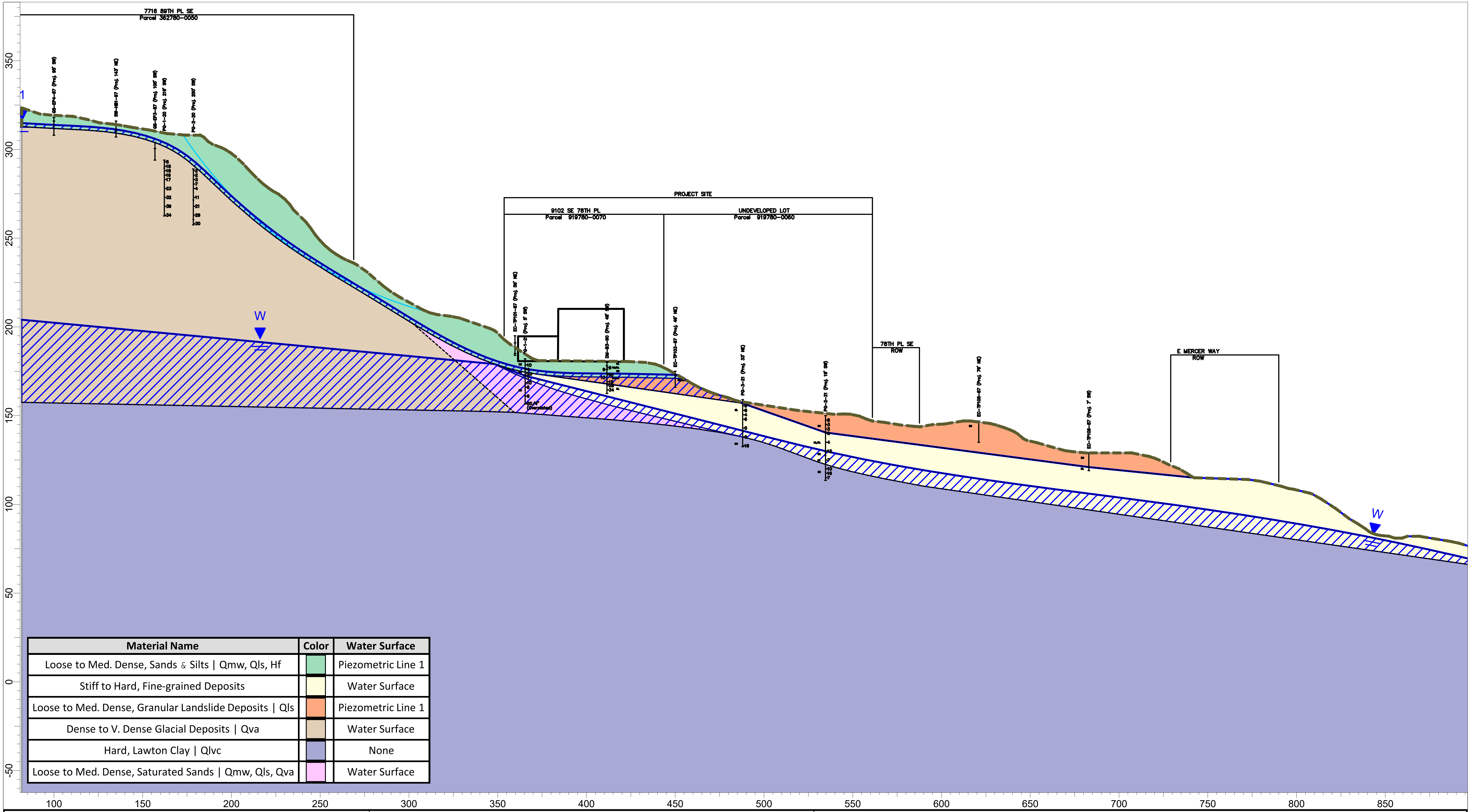
Leung Residence
9102 SE 78th Place
Mercer Island, Washington

GEOLOGIC MAP

PROJECT NO. 21-537

FIGURE NO. 4

Z:\21-537 9102 SE 78th Place, MI\
DATE: 2022.01.07
CHECKED BY: HMX
DRAWN BY: NTW



| Material Name | Color | Water Surface |
|---|---|---|
| Loose to Med. Dense, Sands & Silts \| Qmw, Qls, Hf | 🟩 | Piezometric Line 1 |
| Stiff to Hard, Fine-grained Deposits | ▭ | Water Surface |
| Loose to Med. Dense, Granular Landslide Deposits \| Qls | 🟧 | Piezometric Line 1 |
| Dense to V. Dense Glacial Deposits \| Qva | 🟫 | Water Surface |
| Hard, Lawton Clay \| Qlvc | 🟦 | None |
| Loose to Med. Dense, Saturated Sands \| Qmw, Qls, Qva | 🟪 | Water Surface |

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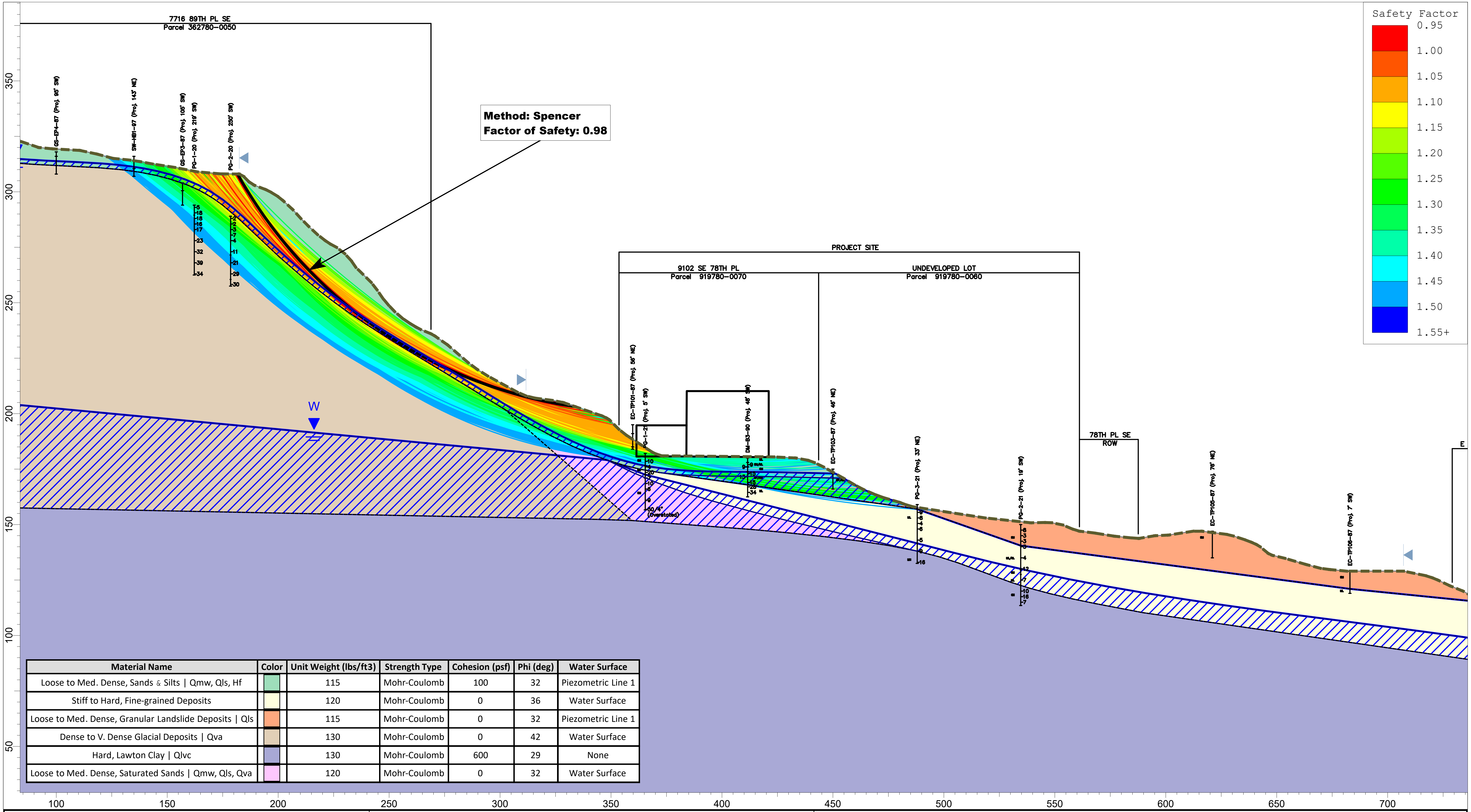
**Leung Residence**
**9102 SE 78th Place**
**Mercer Island, Washington**

**Generalized Subsurface Profile**

**Section A | Existing Condition**

| Scale: | Project No. | Figure No. |
|---|---|---|
| 1:600 | 21-537 | 5 |



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface |
|---|---|---|---|---|---|---|
| Loose to Med. Dense, Sands & Silts \| Qmw, Qls, Hf | 🟩 | 115 | Mohr-Coulomb | 100 | 32 | Piezometric Line 1 |
| Stiff to Hard, Fine-grained Deposits | ⬜ | 120 | Mohr-Coulomb | 0 | 36 | Water Surface |
| Loose to Med. Dense, Granular Landslide Deposits \| Qls | 🟧 | 115 | Mohr-Coulomb | 0 | 32 | Piezometric Line 1 |
| Dense to V. Dense Glacial Deposits \| Qva | 🟫 | 130 | Mohr-Coulomb | 0 | 42 | Water Surface |
| Hard, Lawton Clay \| Qlvc | 🟪 | 130 | Mohr-Coulomb | 600 | 29 | None |
| Loose to Med. Dense, Saturated Sands \| Qmw, Qls, Qva | 🟪 | 120 | Mohr-Coulomb | 0 | 32 | Water Surface |

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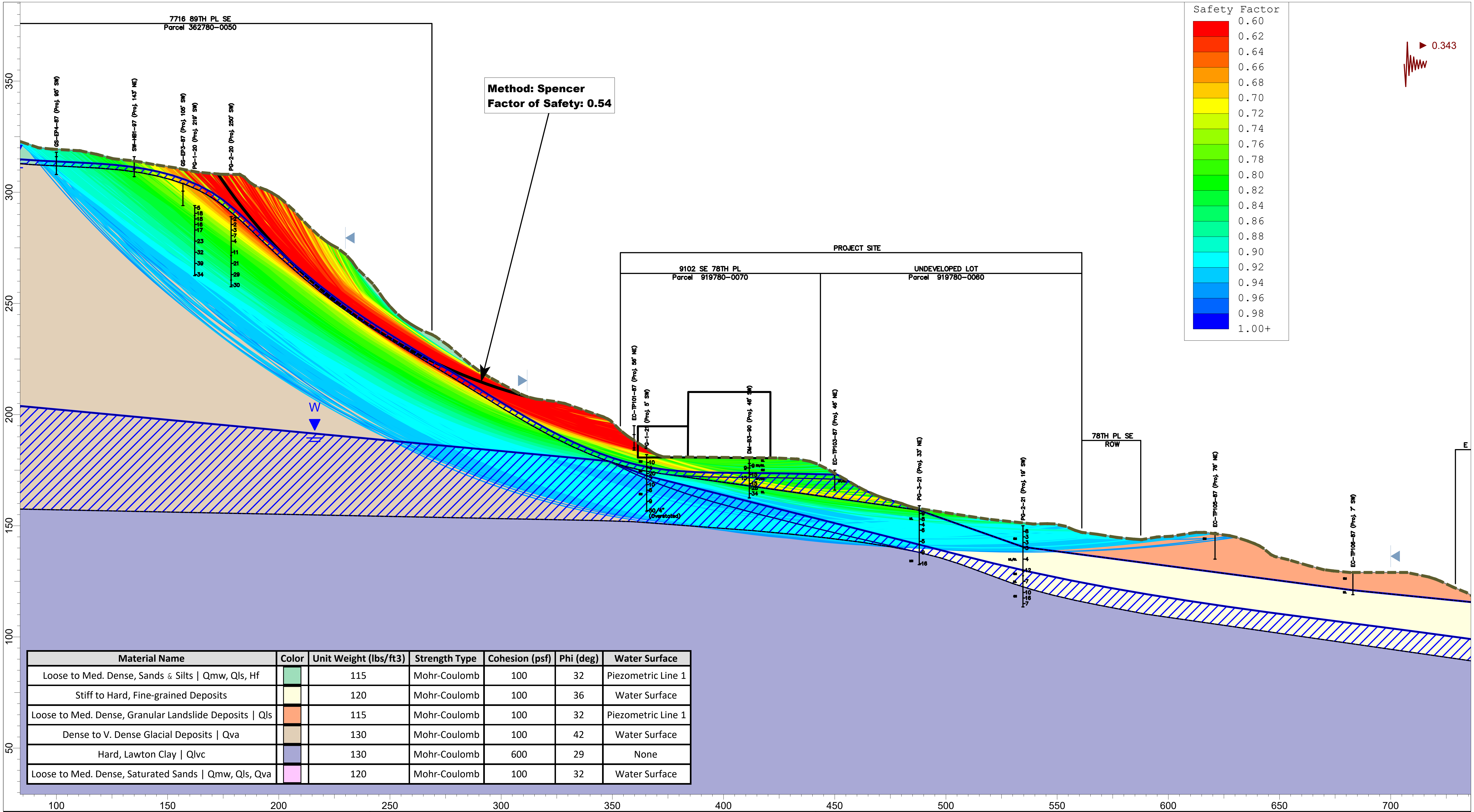
Mercer Island, Washington

Static Stability Analysis

Section A | Existing Condition | F.S. < 1.5

| Scale: | Project No. | Figure No. |
|---|---|---|
| 1:480 | 21-537 | 6 |



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface |
|---|---|---|---|---|---|---|
| Loose to Med. Dense, Sands & Silts \| Qmw, Qls, Hf | 🟩 | 115 | Mohr-Coulomb | 100 | 32 | Piezometric Line 1 |
| Stiff to Hard, Fine-grained Deposits | 🟨 | 120 | Mohr-Coulomb | 100 | 36 | Water Surface |
| Loose to Med. Dense, Granular Landslide Deposits \| Qls | 🟧 | 115 | Mohr-Coulomb | 100 | 32 | Piezometric Line 1 |
| Dense to V. Dense Glacial Deposits \| Qva | 🟫 | 130 | Mohr-Coulomb | 100 | 42 | Water Surface |
| Hard, Lawton Clay \| Qlvc | 🟪 | 130 | Mohr-Coulomb | 600 | 29 | None |
| Loose to Med. Dense, Saturated Sands \| Qmw, Qls, Qva | 🟪 | 120 | Mohr-Coulomb | 100 | 32 | Water Surface |

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Leung Residence
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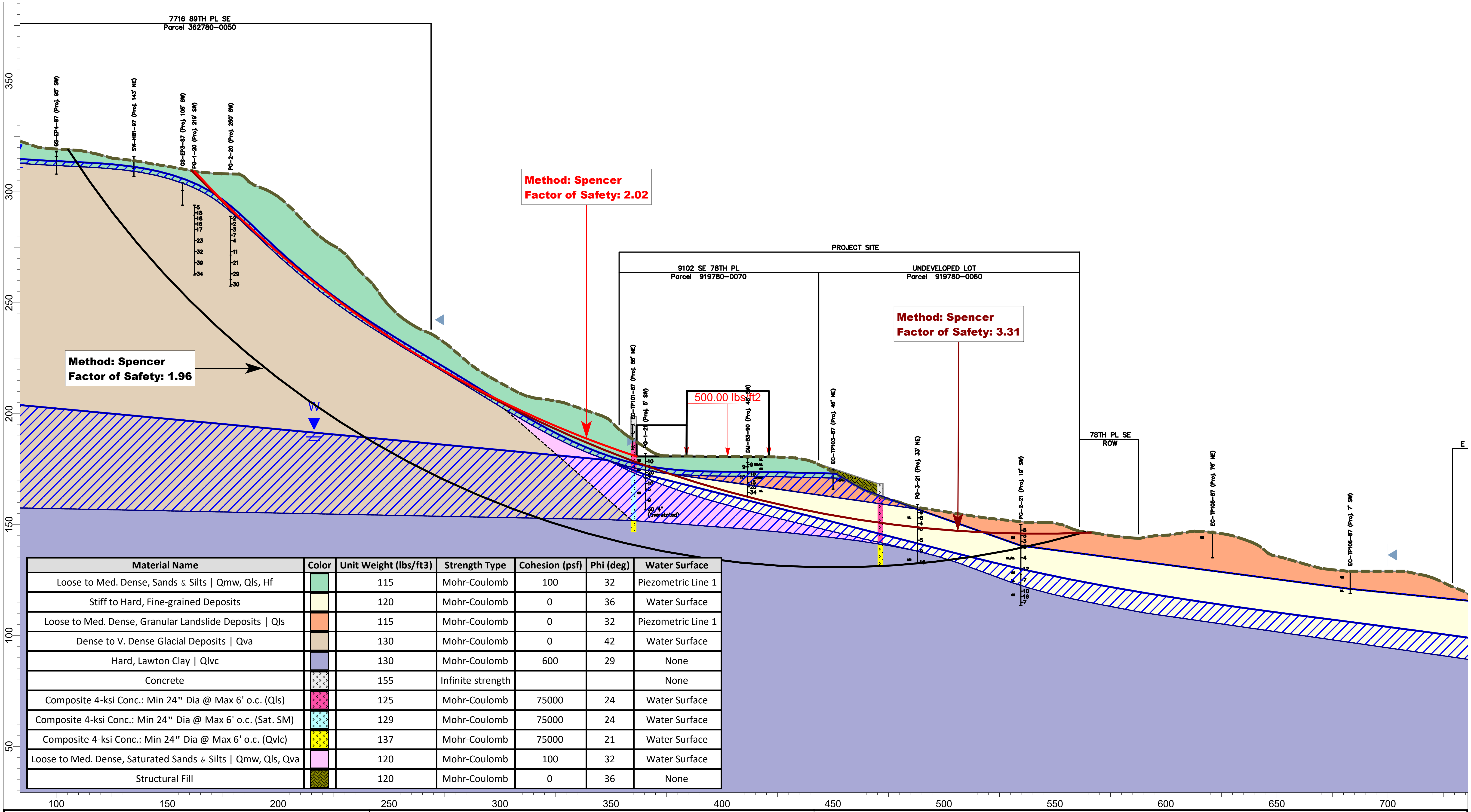
Mercer Island, Washington

**Pseudo-Static (Seismic) Stability Analysis**

**Section A | Existing Condition | F.S. < 1.0**

| Scale: | Project No. | Figure No. |
|---|---|---|
| 1:480 | 21-537 | 7 |



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface |
|---|---|---|---|---|---|---|
| Loose to Med. Dense, Sands & Silts \| Qmw, Qls, Hf | 🟩 | 115 | Mohr-Coulomb | 100 | 32 | Piezometric Line 1 |
| Stiff to Hard, Fine-grained Deposits | ⬜ | 120 | Mohr-Coulomb | 0 | 36 | Water Surface |
| Loose to Med. Dense, Granular Landslide Deposits \| Qls | 🟧 | 115 | Mohr-Coulomb | 0 | 32 | Piezometric Line 1 |
| Dense to V. Dense Glacial Deposits \| Qva | 🟫 | 130 | Mohr-Coulomb | 0 | 42 | Water Surface |
| Hard, Lawton Clay \| Qlvc | 🟪 | 130 | Mohr-Coulomb | 600 | 29 | None |
| Concrete | ▨ | 155 | Infinite strength | | | None |
| Composite 4-ksi Conc.: Min 24" Dia @ Max 6' o.c. (Qls) | 🟥 | 125 | Mohr-Coulomb | 75000 | 24 | Water Surface |
| Composite 4-ksi Conc.: Min 24" Dia @ Max 6' o.c. (Sat. SM) | 🟦 | 129 | Mohr-Coulomb | 75000 | 24 | Water Surface |
| Composite 4-ksi Conc.: Min 24" Dia @ Max 6' o.c. (Qvlc) | 🟨 | 137 | Mohr-Coulomb | 75000 | 21 | Water Surface |
| Loose to Med. Dense, Saturated Sands & Silts \| Qmw, Qls, Qva | 🟪 | 120 | Mohr-Coulomb | 100 | 32 | Water Surface |
| Structural Fill | ▨ | 120 | Mohr-Coulomb | 0 | 36 | None |

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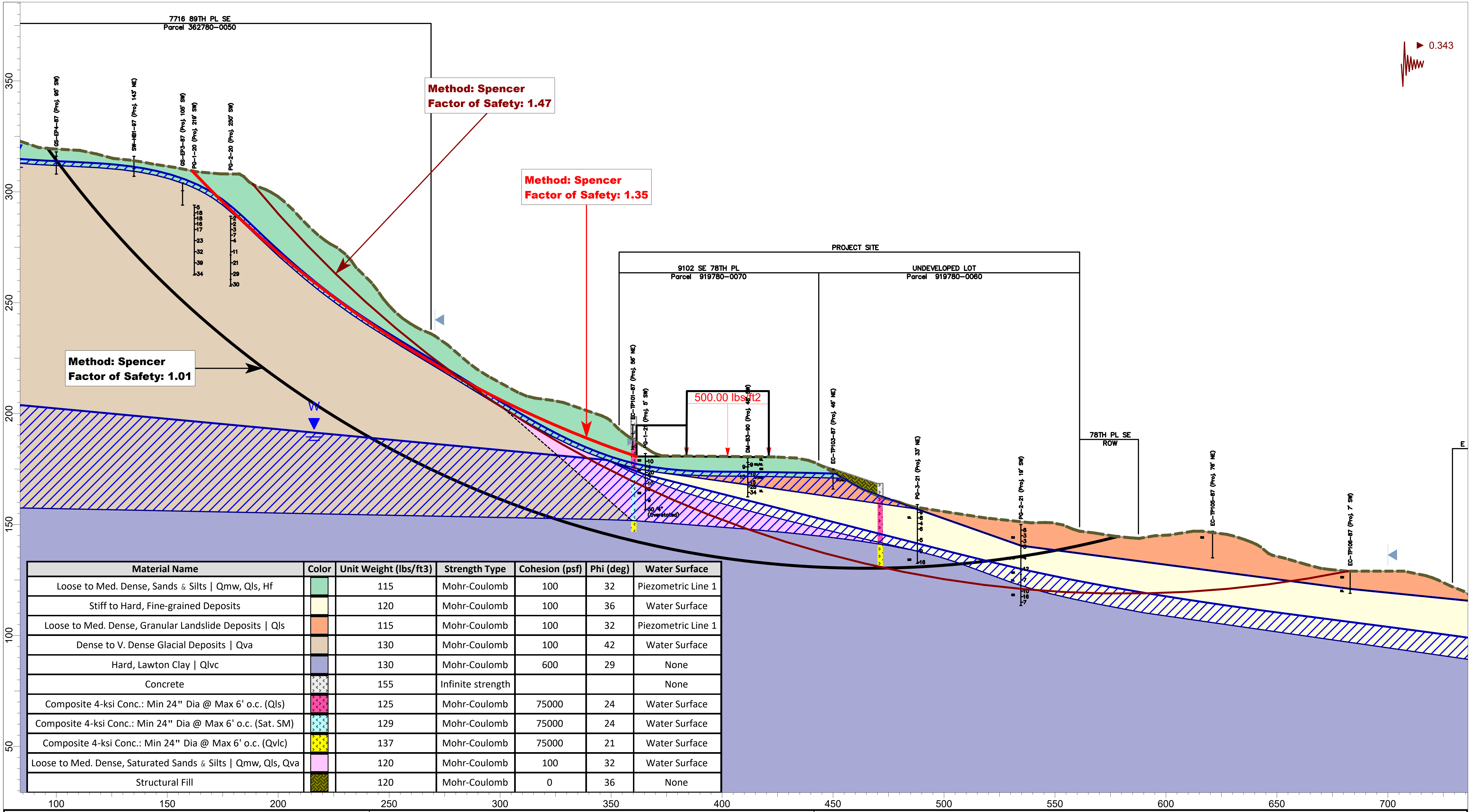
**Leung Residence**
**9102 SE 78th Place**
**Mercer Island, Washington**

## Static Stability Analysis

**Section A | Proposed Condition | Upper Catchment Wall**

| Scale: | Project No. | Figure No. |
|---|---|---|
| 1:480 | 21-537 | 8 |



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface |
|---|---|---|---|---|---|---|
| Loose to Med. Dense, Sands & Silts \| Qmw, Qls, Hf | 🟩 | 115 | Mohr-Coulomb | 100 | 32 | Piezometric Line 1 |
| Stiff to Hard, Fine-grained Deposits | ⬜ | 120 | Mohr-Coulomb | 100 | 36 | Water Surface |
| Loose to Med. Dense, Granular Landslide Deposits \| Qls | 🟧 | 115 | Mohr-Coulomb | 100 | 32 | Piezometric Line 1 |
| Dense to V. Dense Glacial Deposits \| Qva | 🟫 | 130 | Mohr-Coulomb | 100 | 42 | Water Surface |
| Hard, Lawton Clay \| Qlvc | 🟪 | 130 | Mohr-Coulomb | 600 | 29 | None |
| Concrete | ▨ | 155 | Infinite strength | | | None |
| Composite 4-ksi Conc.: Min 24" Dia @ Max 6' o.c. (Qls) | ▨ | 125 | Mohr-Coulomb | 75000 | 24 | Water Surface |
| Composite 4-ksi Conc.: Min 24" Dia @ Max 6' o.c. (Sat. SM) | ▨ | 129 | Mohr-Coulomb | 75000 | 24 | Water Surface |
| Composite 4-ksi Conc.: Min 24" Dia @ Max 6' o.c. (Qvlc) | ▨ | 137 | Mohr-Coulomb | 75000 | 21 | Water Surface |
| Loose to Med. Dense, Saturated Sands & Silts \| Qmw, Qls, Qva | ⬜ | 120 | Mohr-Coulomb | 100 | 32 | Water Surface |
| Structural Fill | ▨ | 120 | Mohr-Coulomb | 0 | 36 | None |

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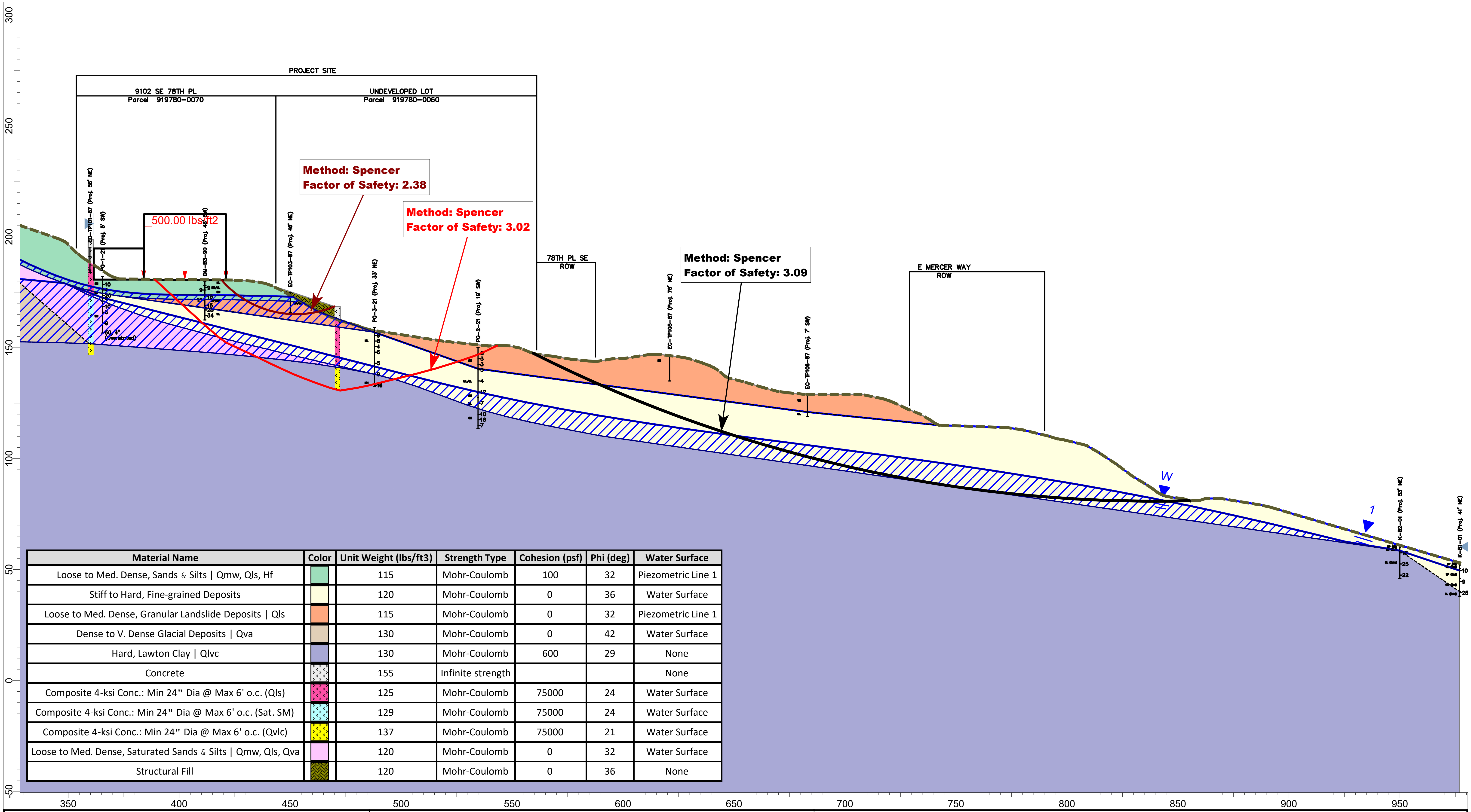
Leung Residence
9102 SE 78th Place
Mercer Island, Washington

## Pseudo-static (Seimsic) Stability Analysis

## Section A | Proposed Condition | Upper Catchment Wall

| Scale: | Project No. | Figure No. |
|---|---|---|
| 1:480 | 21-537 | 9 |



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface |
|---|---|---|---|---|---|---|
| Loose to Med. Dense, Sands & Silts \| Qmw, Qls, Hf | 🟩 | 115 | Mohr-Coulomb | 100 | 32 | Piezometric Line 1 |
| Stiff to Hard, Fine-grained Deposits | ⬜ | 120 | Mohr-Coulomb | 0 | 36 | Water Surface |
| Loose to Med. Dense, Granular Landslide Deposits \| Qls | 🟧 | 115 | Mohr-Coulomb | 0 | 32 | Piezometric Line 1 |
| Dense to V. Dense Glacial Deposits \| Qva | 🟫 | 130 | Mohr-Coulomb | 0 | 42 | Water Surface |
| Hard, Lawton Clay \| Qlvc | 🟪 | 130 | Mohr-Coulomb | 600 | 29 | None |
| Concrete | ▨ | 155 | Infinite strength | | | None |
| Composite 4-ksi Conc.: Min 24" Dia @ Max 6' o.c. (Qls) | 🟥 | 125 | Mohr-Coulomb | 75000 | 24 | Water Surface |
| Composite 4-ksi Conc.: Min 24" Dia @ Max 6' o.c. (Sat. SM) | 🟦 | 129 | Mohr-Coulomb | 75000 | 24 | Water Surface |
| Composite 4-ksi Conc.: Min 24" Dia @ Max 6' o.c. (Qvlc) | 🟨 | 137 | Mohr-Coulomb | 75000 | 21 | Water Surface |
| Loose to Med. Dense, Saturated Sands & Silts \| Qmw, Qls, Qva | ▨ | 120 | Mohr-Coulomb | 0 | 32 | Water Surface |
| Structural Fill | ▨ | 120 | Mohr-Coulomb | 0 | 36 | None |

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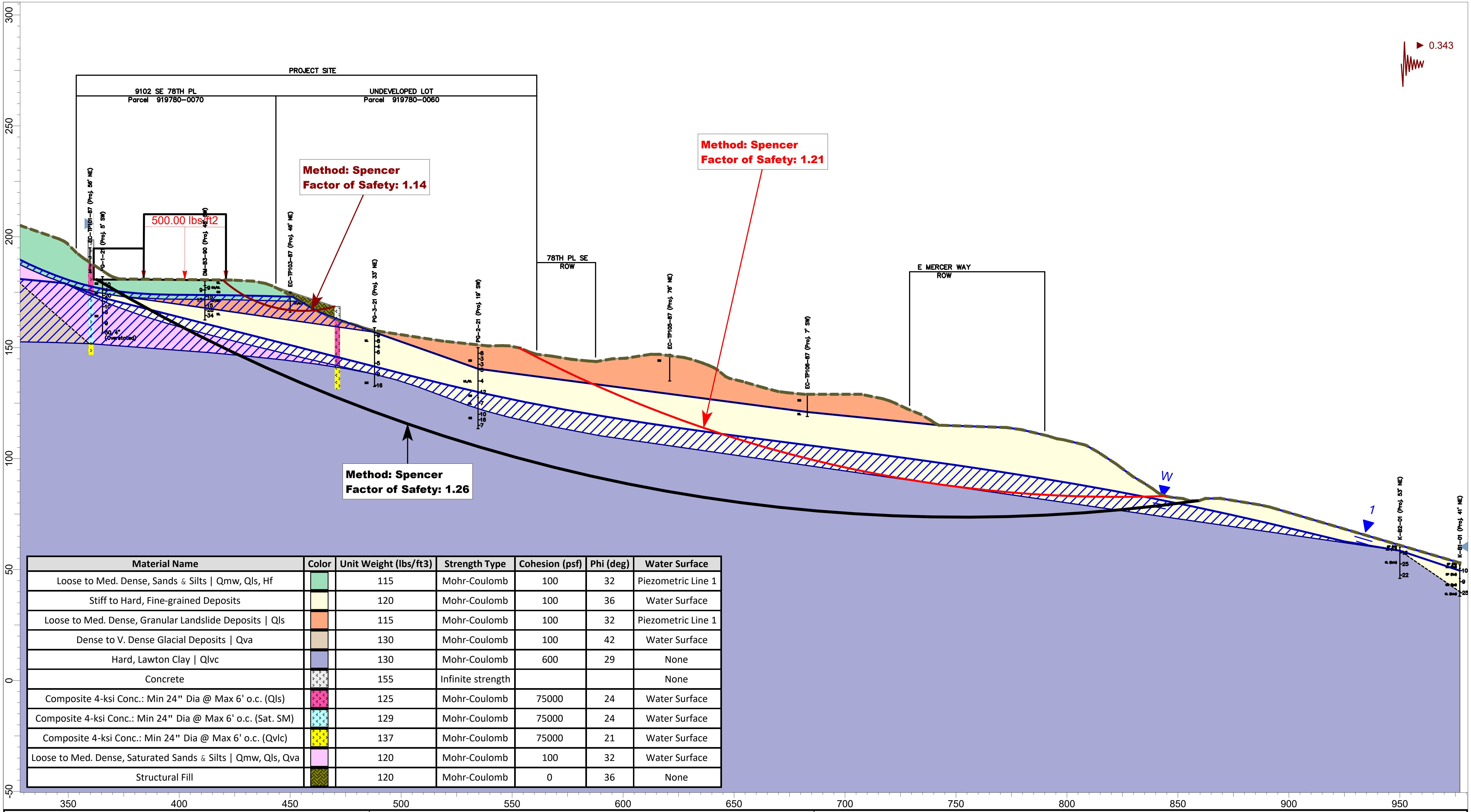
Leung Residence
9102 SE 78th Place
Mercer Island, Washington

**Static Stability Analysis**

**Section A | Proposed Condition | Lower Stabilzation Wall**

| Scale: | Project No. | Figure No. |
|---|---|---|
| 1:480 | 21-537 | 10 |



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface |
|---|---|---|---|---|---|---|
| Loose to Med. Dense, Sands & Silts \| Qmw, Qls, Hf | 🟩 | 115 | Mohr-Coulomb | 100 | 32 | Piezometric Line 1 |
| Stiff to Hard, Fine-grained Deposits | 🟨 | 120 | Mohr-Coulomb | 100 | 36 | Water Surface |
| Loose to Med. Dense, Granular Landslide Deposits \| Qls | 🟧 | 115 | Mohr-Coulomb | 100 | 32 | Piezometric Line 1 |
| Dense to V. Dense Glacial Deposits \| Qva | 🟫 | 130 | Mohr-Coulomb | 100 | 42 | Water Surface |
| Hard, Lawton Clay \| Qlvc | 🟪 | 130 | Mohr-Coulomb | 600 | 29 | None |
| Concrete | ▨ | 155 | Infinite strength | | | None |
| Composite 4-ksi Conc.: Min 24" Dia @ Max 6' o.c. (Qls) | ▨ | 125 | Mohr-Coulomb | 75000 | 24 | Water Surface |
| Composite 4-ksi Conc.: Min 24" Dia @ Max 6' o.c. (Sat. SM) | ▨ | 129 | Mohr-Coulomb | 75000 | 24 | Water Surface |
| Composite 4-ksi Conc.: Min 24" Dia @ Max 6' o.c. (Qvlc) | ▨ | 137 | Mohr-Coulomb | 75000 | 21 | Water Surface |
| Loose to Med. Dense, Saturated Sands & Silts \| Qmw, Qls, Qva | 🟪 | 120 | Mohr-Coulomb | 100 | 32 | Water Surface |
| Structural Fill | ▨ | 120 | Mohr-Coulomb | 0 | 36 | None |

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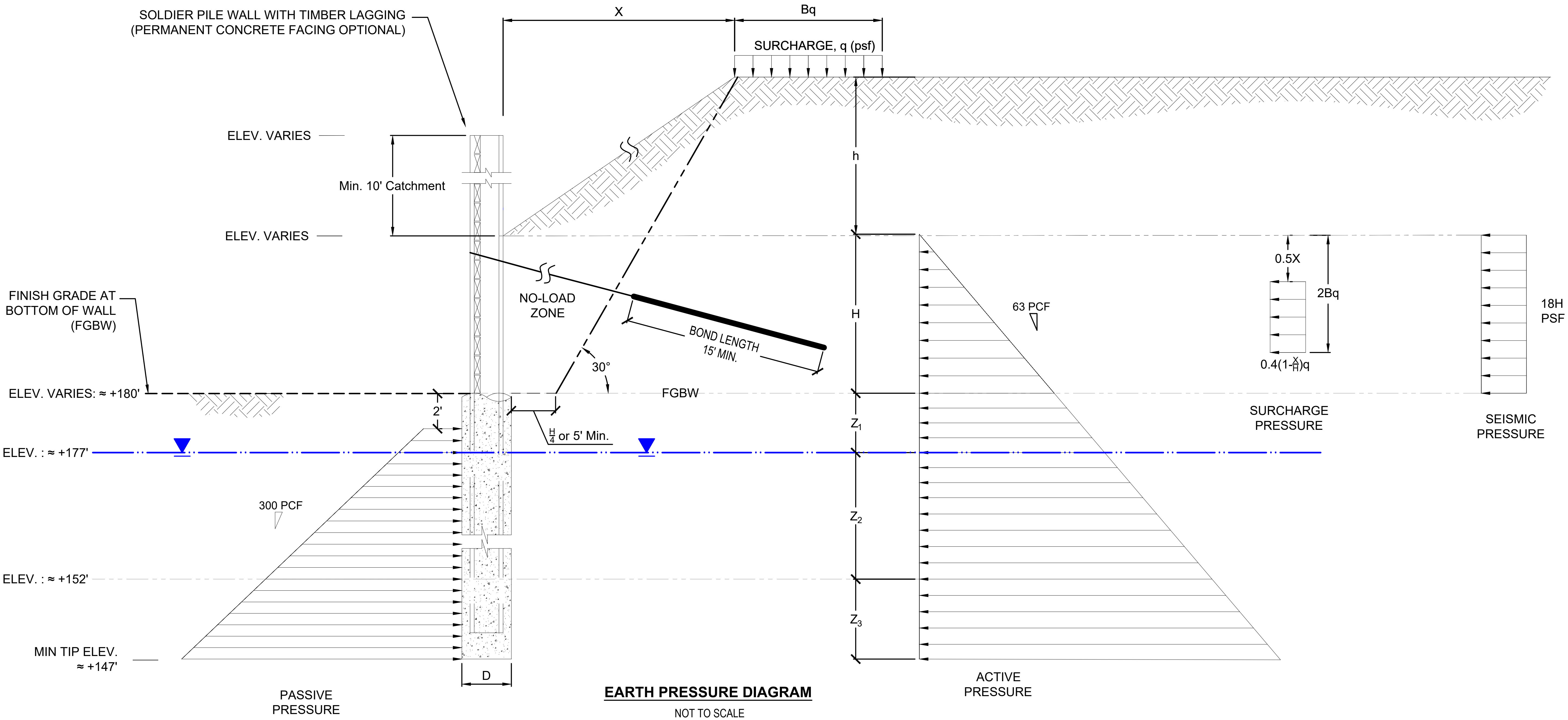
Leung Residence
9102 SE 78th Place
Mercer Island, Washington

**Pseudo-static (Seimsic) Stability Analysis**

**Section A | Proposed Condition | Lower Stabilzation Wall**

| Scale: | Project No. | Figure No. |
|---|---|---|
| 1:480 | 21-537 | 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 tip elevation should be +147', or or deeper as determined by structural analysis.
2. A factor of safety of 1.5 has been applied to the recommended passive earth pressure value. No factor of safety 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. 152 ft)
   End Bearing = 30 ksf
8. For seismic condition, combine the seismic pressure (psf) with the static (active and surcharge) pressures. The passive pressure may be increased by one-third for the seismic condition.
9. Refer to the report text for anchor recommendations and additional discussions.

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Leung Residence
9102 SE 78th Place
Mercer Island, Washington

DESIGN LATERAL PRESSURES
PERMANENT SOLDIER PILE WALL
WITH SINGLE TIEBACK
*STATIC & SEISMIC CONDITIONS*

PROJECT NO. 21-537

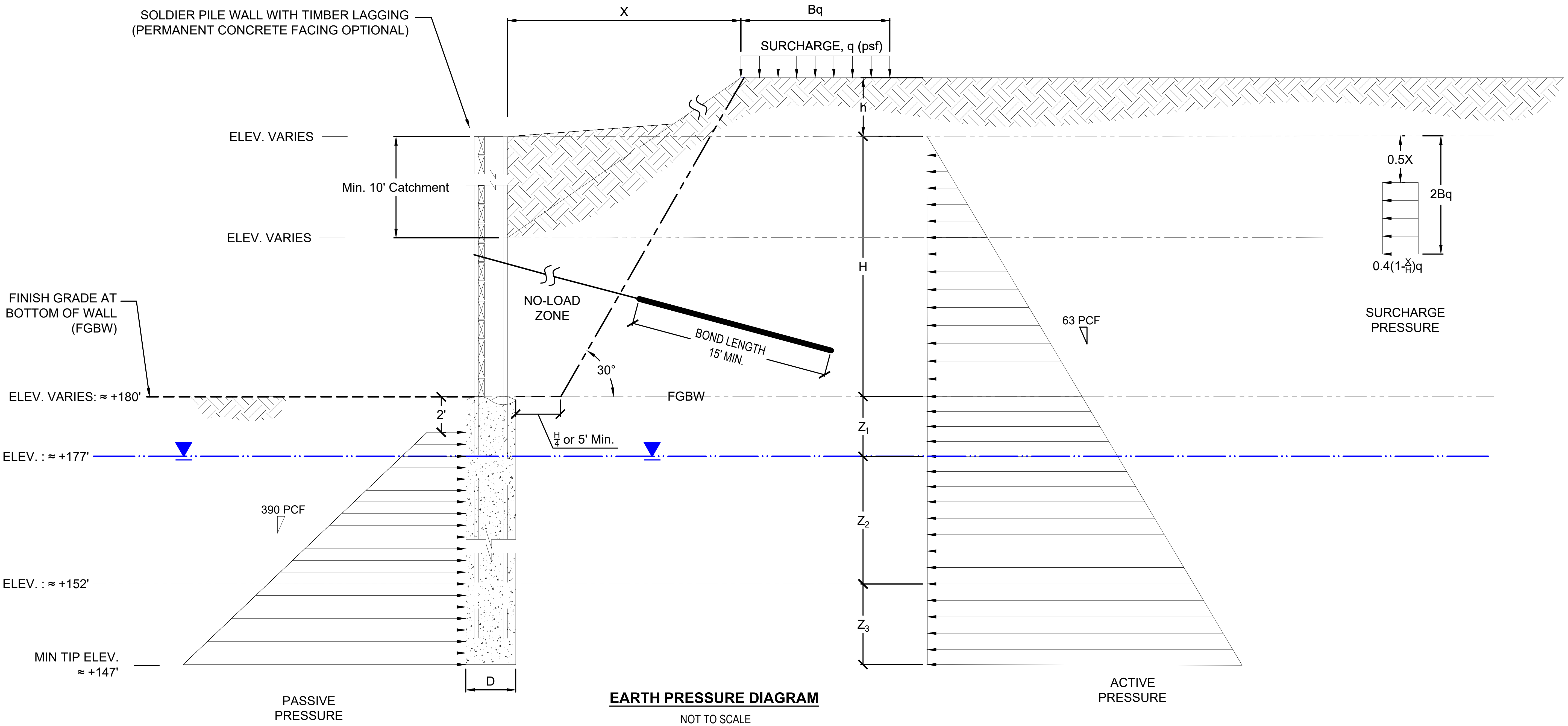
FIGURE NO. 12

Z:\21-537 9102 SE 78th Place, MI\

DATE: 2022.01.18

CHECKED BY: HMX

DRAWN BY: NTW



**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 tip elevation should be +147', or or deeper as determined by structural analysis.
2. A factor of safety of 1.15 has been applied to the recommended passive earth pressure value. No factor of safety 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. 152 ft)
   End Bearing = 30 ksf
8. Refer to the report text for anchor recommendations and additional discussions.

Z:\21-537 9102 SE 78th Place, MI\

DATE: 2022.01.18

CHECKED BY: HMX

DRAWN BY: NTW

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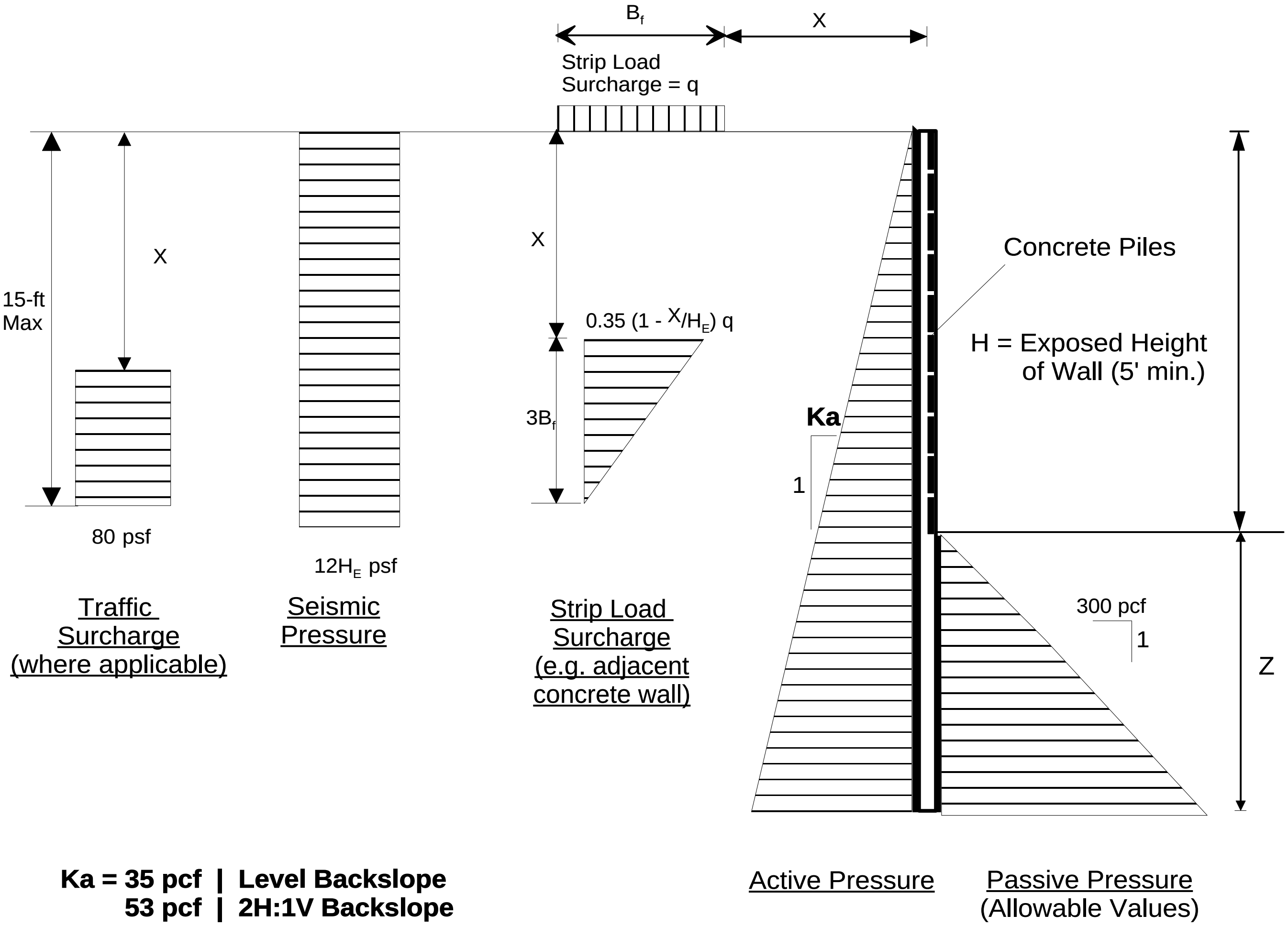
Leung Residence
9102 SE 78th Place
Mercer Island, Washington

DESIGN LATERAL PRESSURES
PERMANENT SOLDIER PILE WALL
WITH SINGLE TIEBACK
*POST-SLIDE CONDITION*

PROJECT NO. 21-537

FIGURE NO. 13



**Ka = 35 pcf | Level Backslope**
**53 pcf | 2H:1V Backslope**

**LEGEND**

H = Cantilevered Height (ft)

Z = Embedment Depth (ft)

Notes:
1. Minumum embedment at Elevation 133' or 10 feet.
2. A factor of safety of 1.5 has been applied to the recommended passive earth pressure value. No factor of safety has been applied to the recommended active earth pressure values.
3. Active and surcharge pressures should be applied over the full width of the pile spacing above the assumed ground surface, and over one pile diameter below the base of the excavation.
4. Seismic pressures should be applied over the full width of the pile spacing.
5. Passive pressure should be applied to two times the diameter of the concrete piles.
6. Refer to report text for additional discussions.

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**Leung Residence**
**9102 SE 78th Place**
**Mercer Island, Washington**

**DESIGN LATERAL EARTH PRESSURES**
**FOR CONCRETE STABILIZATION PILES**
**CANTILEVERED CONDITION**

Project No. **21-537**

Figure No. **14**

# APPENDIX A

## SUBSURFACE EXPLORATIONS BY PANGEO

**Appendix A1 – Current Subsurface Investigation**
***9102 SE 78th Place | PanGEO, Inc, 2021***
Test Borings PG-1 through PG-3

**Appendix A2 – Previous Subsurface Investigation**
***7710 89th Place SE | PanGEO, Inc, 2020***
Test Borings PG-1 and PG-2

## 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**<br>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**<br>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**<br>50%or more passing #200 sieve | Liquid Limit < 50 | ML | SILT |
| | | CL | Lean CLAY |
| | | OL | Organic SILT or CLAY |
| | Liquid Limit > 50 | MH | Elastic SILT |
| | | CH | Fat CLAY |
| | | OH | Organic SILT or CLAY |
| Highly Organic Soils | | PT | PEAT |

**Notes:**

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

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

## DESCRIPTIONS OF SOIL STRUCTURES

**Layered:** Units of material distinguished by color and/or composition from material units above and below
**Laminated:** Layers of soil typically 0.05 to 1mm thick, max. 1 cm
**Lens:** Layer of soil that pinches out laterally
**Interlayered:** Alternating layers of differing soil material
**Pocket:** Erratic, discontinuous deposit of limited extent
**Homogeneous:** Soil with uniform color and composition throughout

**Fissured:** Breaks along defined planes
**Slickensided:** Fracture planes that are polished or glossy
**Blocky:** Angular soil lumps that resist breakdown
**Disrupted:** Soil that is broken and mixed
**Scattered:** Less than one per foot
**Numerous:** More than one per foot
**BCN:** Angle between bedding plane and a plane normal to core axis

## COMPONENT DEFINITIONS

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

## TEST SYMBOLS

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

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

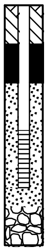
## SYMBOLS

**Sample/In Situ test types and intervals**

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

## MONITORING WELL

| | |
|---|---|
| ∇ | Groundwater Level at time of drilling (ATD) |
| ▼ | Static Groundwater Level |



Cement / Concrete Seal
Bentonite grout / seal
Silica sand backfill
Slotted tip
Slough
Bottom of Boring

## MOISTURE CONTENT

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

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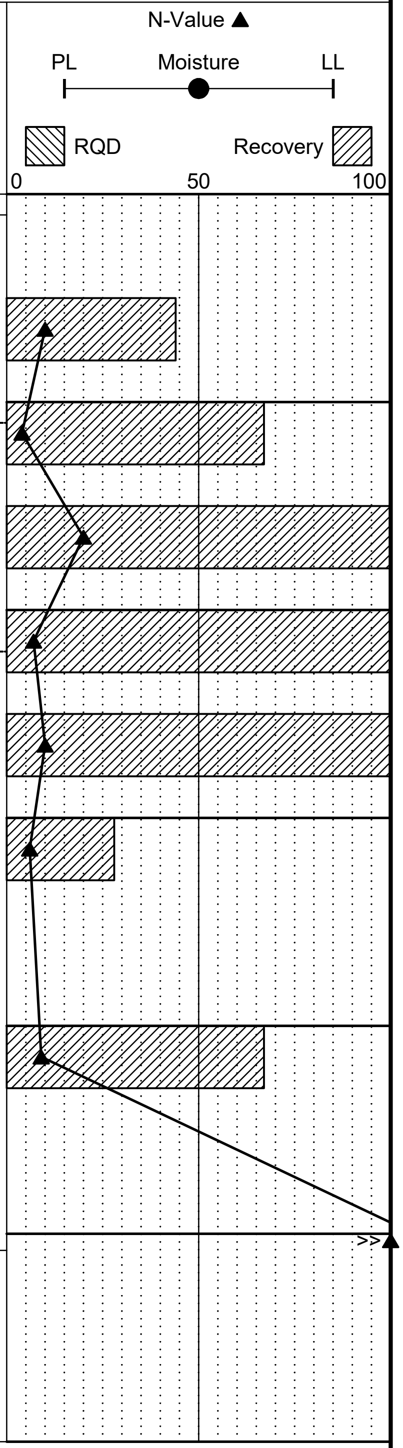
## Terms and Symbols for Boring and Test Pit Logs

**Figure A-1**

Project: Leung Residence
Job Number: 21-537
Location: 9102 SE 78th Place, Mercer Island, WA
Coordinates: Northing: 47.53314, Easting: -122.21884

Surface Elevation: ~182 ft
Top of Casing Elev.: n/a
Drilling Method: HSA
Sampling Method: SPT

| Depth, (ft) | Sample No. | Sample Type | Blows / 6 in. | Other Tests | Symbol | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|
| 0 | | | | | | Approximately 6 inches of topsoil (loose, dark brown silty sand with organics). |
| | | | | | | **DISTURBED NATIVE, COLLUVIUM & LANDSLIDE DEPOSITS**<br>Loose to medium dense, brown, silty, fine- to medium-grained SAND, trace gravel; moist. |
| | S-1 | ☒ | 5<br>5<br>5 | | | |
| 5 | S-2 | ☒ | 4<br>2<br>2 | | | Soft to medium stiff, brown, SILT; moist, low plasticity, minor iron-oxide staining, slightly laminated. |
| | S-3 | ☒ | 6<br>8<br>12 | | | -- Becomes very stiff, occasional sand seams. |
| 10 | S-4 | ☒ | 5<br>4<br>3 | | | -- Becomes medium stiff. |
| | | | | | | Loose, dark brown, silty, fine- to medium-grained SAND; very moist to wet, massive, occasional layers of well-graded gravel. |
| | S-5 | ☒ | 5<br>5<br>5 | | | |
| 15 | S-6 | ☒ | 5<br>3<br>3 | | | -- Becomes water-bearing. |
| 20 | S-7 | ☒ | 7<br>3<br>6 | | | |
| 25 | S-8 | ☒ | 50/4 | | | *Driller reported approximately 3 feet of heaving sand in augers at 25 feet. SPT N-Value overstated; loose to medium dense. No recovery in Sample S-10.* |
| | | | | | | Boring terminated at 25.4 feet below grade in heaving sand conditions.<br>Groundwater encountered about 9.5 feet below grade at the time of drilling. |
| 30 | | | | | | |



Completion Depth: 25.4ft
Date Borehole Started: 12/7/21
Date Borehole Completed: 12/7/21
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 Terrane, dated April 22, 2021. **Vertical datum: NAVD 88.**

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# LOG OF TEST BORING PG-1

**Figure A-2**

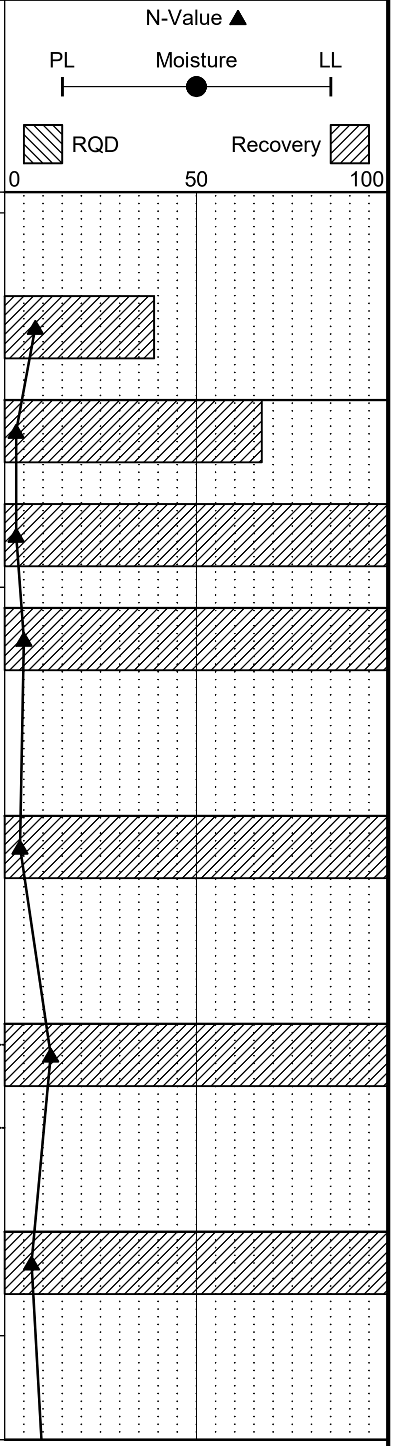
The stratification lines represent approximate boundaries. The transition may be gradual.

Sheet 1 of 1

Project: Leung Residence
Job Number: 21-537
Location: 9102 SE 78th Place, Mercer Island, WA
Coordinates: Northing: 47.53294, Easting: -122.21822

Surface Elevation: ~150 ft
Top of Casing Elev.: n/a
Drilling Method: HSA
Sampling Method: SPT

| Depth, (ft) | Sample No. | Sample Type | Blows / 6 in. | Other Tests | Symbol | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|
| 0 | | | | | | Grass sod over approximately 5 inches of topsoil (loose, dark brown silty sand with organics). |
| | | | | | | **LANDSLIDE DEPOSITS**<br>Loose, brown, silty SAND with organics (roots, wood), gravel; moist, disturbed. |
| | S-1 | ⊠ | 3<br>4<br>4 | | | -- Trace wood debris; decrease in gravel content. |
| 5 | S-2 | ⊠ | 3<br>2<br>1 | | | -- Becomes very loose; occasional thin organic lenses. |
| | S-3 | ⊠ | 3<br>1<br>2 | | | -- Abundant charcoal, wood fiber; occasional gray, silt pockets. |
| 10 | S-4 | ⊠ | 3<br>2<br>3 | | | **FINE-GRAINED DEPOSITS**<br>Medium stiff, blue-gray, clayey, sandy SILT, organics, trace gravel; moist, low to medium plasticity. |
| 15 | S-5 | ⊠ | 3<br>2<br>2 | | | -- Abundant organics; highly disturbed. |
| 20 | S-6 | ⊠ | 4<br>7<br>5 | | ∇ | Medium dense, blue-gray, silty, poorly-graded, medium-grained SAND with gravel; very moist to wet, massive, gravel well-graded. |
| | | | | | | Medium stiff, gray, clayey, sandy SILT with gravel, trace organics; moist. |
| 25 | S-7 | ⊠ | 3<br>3<br>4 | | | |
| | | | | | | **LAWTON CLAY**<br>Stiff, gray, fat CLAY, trace silt; moist, massive. |
| 30 | | | | | | |



Completion Depth: 36.5ft
Date Borehole Started: 12/7/21
Date Borehole Completed: 12/7/21
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 Terrane, dated April 22, 2021. **Vertical datum: NAVD 88.**

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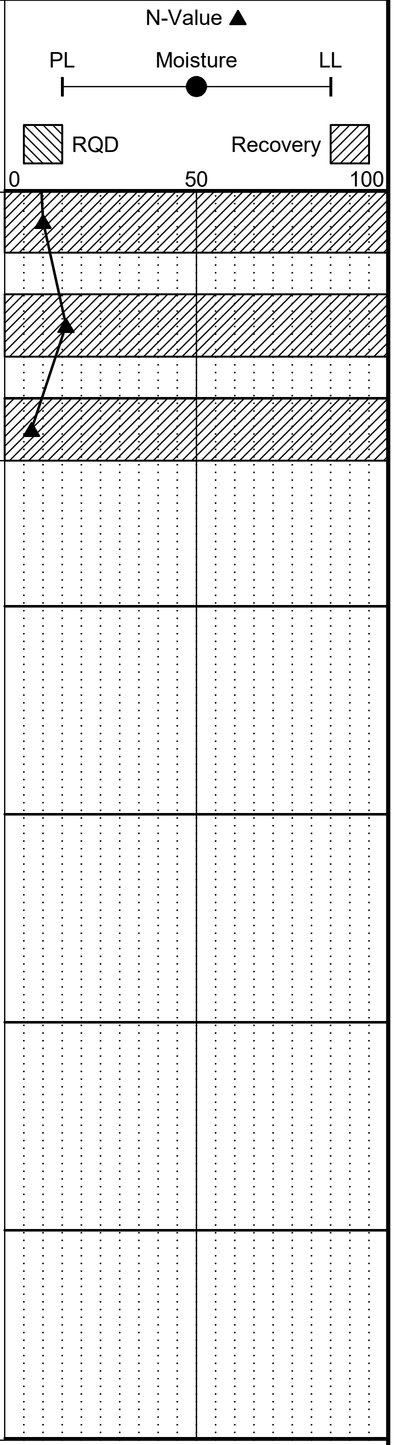
# LOG OF TEST BORING PG-2

**Figure A-3**

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

| | | | |
|---|---|---|---|
| Project: | Leung Residence | Surface Elevation: | ~150 ft |
| Job Number: | 21-537 | Top of Casing Elev.: | n/a |
| Location: | 9102 SE 78th Place, Mercer Island, WA | Drilling Method: | HSA |
| Coordinates: | Northing: 47.53294, Easting: -122.21822 | Sampling Method: | SPT |

| Depth, (ft) | Sample No. | Sample Type | Blows / 6 in. | Other Tests | Symbol | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|
| 30 | S-8 | ☒ | 3<br>4<br>6 | | ▨ | |
| | S-9 | ☒ | 7<br>8<br>8 | | ▨ | -- Becomes very stiff. |
| 35 | S-10 | ☒ | 2<br>3<br>4 | | ▨ | -- Becomes medium stiff; increase in silt content; increase in moisture content. |
| | | | | | | Boring terminated at 36.5 feet below grade.<br>Perched groundwater encountered about 20 feet below grade at the time of drilling. |
| 40 | | | | | | |
| 45 | | | | | | |
| 50 | | | | | | |
| 55 | | | | | | |
| 60 | | | | | | |



| | |
|---|---|
| Completion Depth: | 36.5ft |
| Date Borehole Started: | 12/7/21 |
| Date Borehole Completed: | 12/7/21 |
| 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 Terrane, dated April 22, 2021.**Vertical datum: NAVD 88.**

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# LOG OF TEST BORING PG-2

**Figure A-3**

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

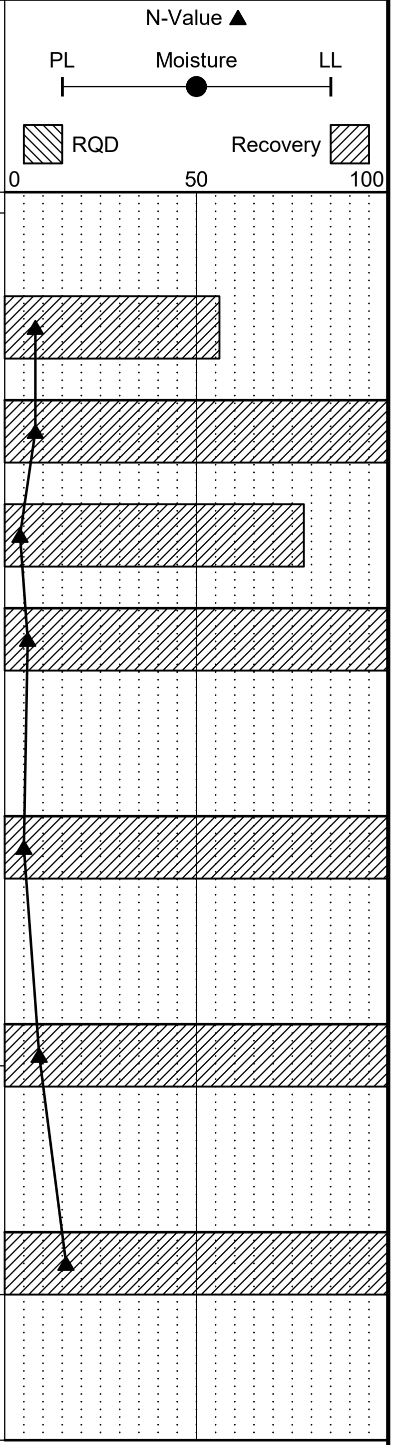
Sheet 2 of 2

**Appdx A1 - PanGEO Boring Logs 2021**

Project: Leung Residence
Job Number: 21-537
Location: 9102 SE 78th Place, Mercer Island, WA
Coordinates: Northing: 47.53288, Easting: -122.21849

Surface Elevation: ~159 ft
Top of Casing Elev.: n/a
Drilling Method: HSA
Sampling Method: SPT

| Depth, (ft) | Sample No. | Sample Type | Blows / 6 in. | Other Tests | Symbol | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|
| 0 | | | | | | Grass sod over approximately 5 inches of topsoil (loose, dark brown silty sand with organics). |
| | | | | | | **FINE-GRAINED DEPOSITS**<br>Stiff, brown, sandy SILT, trace organics; moist, iron-oxide staining throughout. |
| | S-1 | ☒ | 4<br>3<br>5 | | | |
| 5 | S-2 | ☒ | 4<br>4<br>4 | | | -- Becomes medium stiff to stiff; highly weathered. |
| | S-3 | ☒ | 3<br>2<br>2 | | | -- Becomes soft to medium stiff. |
| 10 | S-4 | ☒ | 2<br>3<br>3 | | | -- Increase in moisture at toe of Sample S-4. |
| 15 | S-5 | ☒ | 3<br>2<br>3 | | | -- Moist to very moist. |
| 20 | S-6 | ☒ | 1<br>2<br>7 | | | -- Water in sampler. |
| | | | | | | **LAWTON CLAY**<br>Medium stiff to stiff, gray-brown, elastic SILT; moist, trace iron-oxide staining. |
| 25 | S-7 | ☒ | 5<br>6<br>10 | | | -- Becomes very stiff; massive. |
| | | | | | | Boring terminated at 26.5 feet below grade.<br>Perched groundwater encountered about 17.5 feet below grade at the time of drilling. |
| 30 | | | | | | |



Completion Depth: 26.5ft
Date Borehole Started: 12/7/21
Date Borehole Completed: 12/7/21
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 Terrane, dated April 22, 2021.**Vertical datum: NAVD 88.**

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# LOG OF TEST BORING PG-3

**Figure A-4**

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

Sheet 1 of 1

| | | | |
|---|---|---|---|
| Project: | Preliminary Slope Evaluation and Deck Replacement | Surface Elevation: | ~294 ft |
| Job Number: | 20-236 | Top of Casing Elev.: | n/a |
| Location: | 7710-89th PL SE, Mercer Island, WA | Drilling Method: | HSA |
| Coordinates: | Northing: 47.53393, Easting: -122.21911 | Sampling Method: | SPT |

| Depth, (ft) | Sample No. | Sample Type | Blows / 6 in. | Other Tests | Symbol |
|---|---|---|---|---|---|
| 0 | S-1 | ☒ | 3<br>3<br>2 | | |
| | S-2 | ☒ | 6<br>10<br>8 | | |
| 5 | S-3 | ☒ | 7<br>8<br>10 | | |
| | S-4 | ☒ | 6<br>7<br>9 | | |
| 10 | S-5 | ☒ | 4<br>8<br>9 | | |
| 15 | S-6 | ☒ | 9<br>11<br>12 | | |
| 20 | S-7 | ☒ | 8<br>14<br>18 | | |
| 25 | S-8 | ☒ | 10<br>17<br>22 | | |
| 30 | S-9 | ☒ | 5<br>14<br>20 | | |
| 35 | | | | | |

## MATERIAL DESCRIPTION

**ESU-1**

Loose, brown, silty, fine SAND, scattered gravel, trace organics (rootlets), occasional silt/clay pockets ; moist, disturbed, minor iron-oxide staining.

*-----> Approximately at 2 ft bgs becomes Medium dense, orange brown to gray brown.*

**ESU-2**

Medium dense, orange brown, silty, fine-medium SAND, trace gravel, trace organics (rootlets), minor iron-oxide staining, occasional cobble; moist.

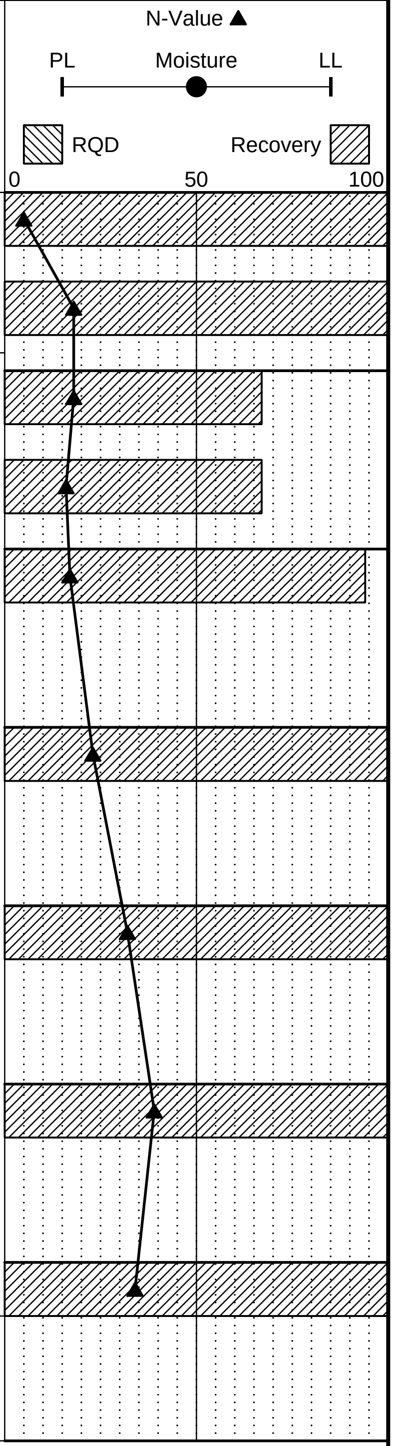
*-----> Becomes brown, slightly silty SAND, massive to slightly laminated.*

*-----> Approximately at 14 ft bgs becomes well-graded SAND.*

*-----> Becomes dense.*

*Boring terminated at approximately 31.5 ft bgs.*

*No Gound water was observed at the time of drilling.*



Completion Depth: 31.5ft
Date Borehole Started: 7/8/20
Date Borehole Completed: 7/8/20
Logged By: S. Harrington
Drilling Company: CN Drilling

Remarks: Limited access Acker hand portable 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 Mercer Island GIS portal. Vertical elevations based on NAVD88 datum. Horizontal datum based on WGS84.

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# LOG OF TEST BORING PG-1

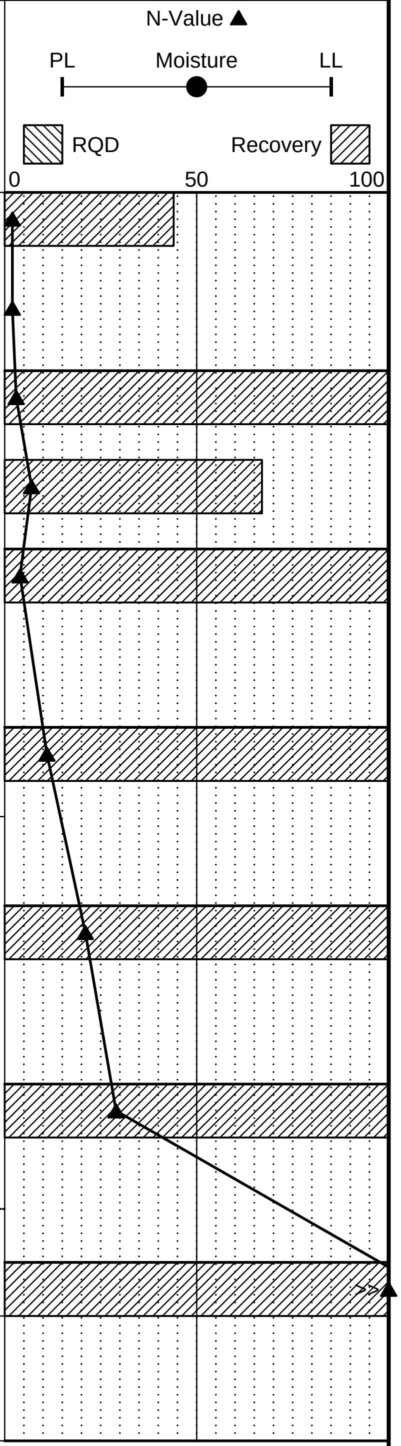
**Figure A-2**

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

Sheet 1 of 1

| | | | |
|---|---|---|---|
| Project: | Preliminary Slope Evaluation and Deck Replacement | Surface Elevation: | ~289 ft |
| Job Number: | 20-236 | Top of Casing Elev.: | n/a |
| Location: | 7710-89th PL SE, Mercer Island, WA | Drilling Method: | HSA |
| Coordinates: | Northing: 47.53398, Easting: -122.21899 | Sampling Method: | SPT |

| Depth, (ft) | Sample No. | Sample Type | Blows / 6 in. | Other Tests | Symbol | MATERIAL DESCRIPTION |
|---|---|---|---|---|---|---|
| 0 | S-1 | ☒ | 1<br>1<br>1 | | | **ESU-1**<br>Very loose, brown, silty, fine SAND, trace gravel, organics (rootlets) ; moist, disturbed. |
| | S-2 | ☒ | 1<br>1<br>1 | | | *-----> No recovery at ~2.5 ft, auger grinding on a rock.* |
| 5 | S-3 | ☒ | 1<br>1<br>2 | | | *-----> Becomes gray brown, occasional silt pockets.* |
| | S-4 | ☒ | 1<br>3<br>4 | | | *-----> Becomes loose; minor iron-oxide staining.* |
| 10 | S-5 | ☒ | 1<br>2<br>2 | | | |
| 15 | S-6 | ☒ | 2<br>5<br>6 | | | *-----> Becomes medium dense.* |
| | | | | | | **ESU-2**<br>Medium dense, gray brown, silty SAND, trace fine gravel; moist, massive. |
| 20 | S-7 | ☒ | 6<br>9<br>12 | | | *-----> Becomes gray approximately at 18 ft bgs.* |
| | | | | | | *-----> Becomes dense, occasional iron-oxide staining bands, slightly laminated, very moist; fine gravel observed in top of sample 9.* |
| 25 | S-8 | ☒ | 7<br>13<br>16 | | | |
| | | | | | ▽ | *-----> Ground water encounterd at ~28 ft bgs.* |
| | | | | | | Very dense, dark gray, silty sandy GRAVEL; wet, fine some iron-oxide staining trace.<br>Driller reported increase in GRAVEL at ~28 ft bgs and a 6" heave at ~30 ft bgs. |
| 30 | S-9 | ☒ | 16<br>50/6 | | | |
| | | | | | | *Boring terminated at approximately 32 ft bgs.*<br>*Ground water encountered at ~28 ft bgs during drilling.* |
| 35 | | | | | | |



| | |
|---|---|
| Completion Depth: | 31.5ft |
| Date Borehole Started: | 7/8/20 |
| Date Borehole Completed: | 7/8/20 |
| Logged By: | S. Harrington |
| Drilling Company: | CN Drilling |

Remarks: Limited access Acker hand portable 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 Mercer Island GIS portal. Vertical elevations based on NAVD88 datum. Horizontal datum based on WGS84.

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# LOG OF TEST BORING PG-2

**Figure A-3**

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

Sheet 1 of 1

# APPENDIX B

## SUBSURFACE EXPLORATIONS BY OTHERS

**Appendix B1 – Previous Subsurface Investigation**
***7625 E Mercer Way | Applied Geotechnology, Inc 1988***
Test Pits TP-1 through TP-3

**Appendix B2 – Previous Subsurface Investigation**
***9060 SE 79th Street | Cascade Geotechnical, 1987***
Test Pits TP-E and TP-F

**Appendix B3 – Previous Subsurface Investigation**
***Beach Lane Development | Dames & Moore, 1989***
Test Borings B-1 through B-3

**Appendix B4 – Previous Subsurface Investigation**
***4298 E Mercer Way | Earth Consultants, 1986 - 1987***
Test Pits TP-101 through and TP-106 (1987)
Test Borings B-1 and B-2 (1987)
Test Pits TP-1 and TP-9 (1986)

**Appendix B5 – Previous Subsurface Investigation**
***9103 SE 78th Place | Earth Consultants, 1995***
Test Pits TP-1 and TP-2

**Appendix B6 – Previous Subsurface Investigation**
***7712 89th Place SE | Geological Services, 1987***
Test Pits TP-1 through TP-4

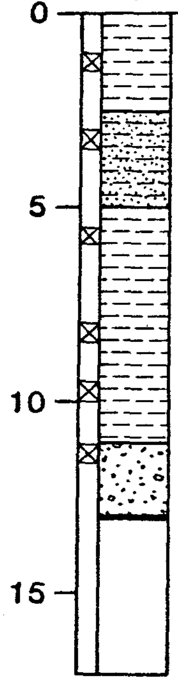
**Appendix B7 – Previous Subsurface Investigation**
***Tarywood Park Stairway | Hart Crowser, 1988 - 1990***
Test Boring HC-1 through HC-3 (1988)
Test Boring HC-101 through HC-104 (1990)

**Appendix B8 – Previous Subsurface Investigation**
***7800 89th Place SE | Shannon & Wilson 1997***
Hand Boring HB-1 through HB-3

Dry Density (pcf) | Moisture Content (%) | Depth (ft) | Sample

**Test Pit Number** 1

**Elevation** 140 feet **Date** 9/9/88



BROWN MEDIUM SANDY SILT (ML) loose to medium dense, dry; trace rounded gravel.

MOTTLED GRAY-BROWN SILTY MEDIUM SAND (SM) dense, dry; slightly cemented.

BLUE-GRAY SANDY SILT (ML) soft to medium stiff, wet to saturated; trace rounded gravels, blocks of clay.

Becomes stiff to very stiff.

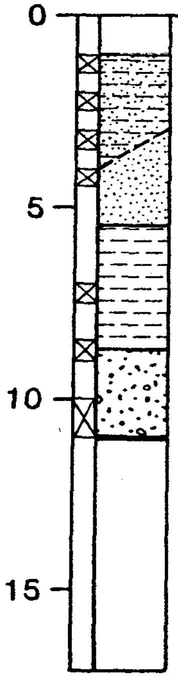
GRAY-BLUE GRAVELLY SAND (SW) medium dense, saturated; trace to some silt.

Test pit terminated at 13.0 feet on 9/9/88.

Groundwater encountered at 11.0 feet.

**Test Pit Number** 2

**Elevation** 138 feet **Date** 9/9/88



Topsoil.

BROWN SILTY FINE SAND (SM) medium dense to dense, moist; trace gravel.

BROWN MEDIUM TO COARSE SAND (SP) dense, wet; with some silt and small gravel.

MOTTLED BLUE-GRAY SILT (ML) soft to medium stiff, saturated; with trace sand lenses, seeps water at 6.0 feet.

BLUE-GRAY GRAVELLY SAND (SW) loose to medium dense, saturated; with rounded gravels to 3-inch diameter and lenses of gray, fine sand.

Test pit terminated at 11.0 feet on 9/9/88.

Groundwater encountered at 11.0 feet.

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## Log of Test Pits 1 and 2

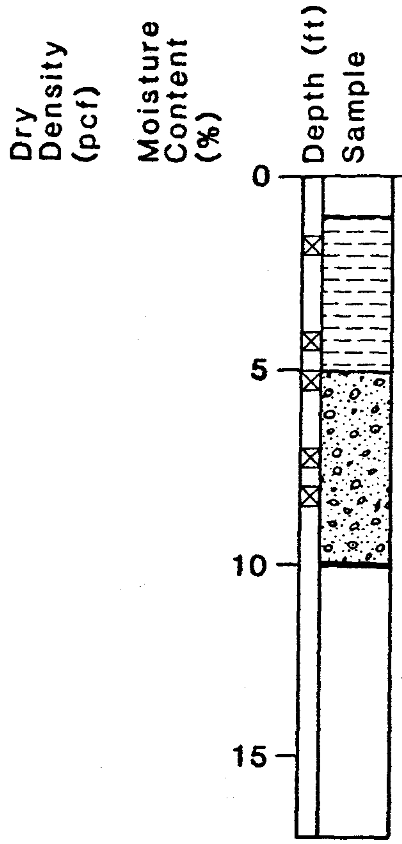
Schroeder Residence
Mercer Island, Washington

PLATE
**2**

| JOB NUMBER | DRAWN | APPROVED | DATE | REVISED | DATE |
|---|---|---|---|---|---|
| 15,336.001 | SAC | | 4 October 88 | | |

**Test Pit Number** 3

**Elevation** 136 feet **Date** 9/9/88



Topsoil.

BROWN SANDY SILT (ML) medium stiff to stiff, dry; with trace gravel to 3-inch diameter.

GRAY-BROWN SANDY GRAVEL (GP) medium dense, moist; with trace silt.

Becomes saturated at 8.5 feet.

Test pit terminated at 10.0 feet on 9/9/88.

Groundwater encountered at 8.5 feet.

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Geotechnical Engineering
Geology & Hydrogeology

**Log of Test Pits 3**

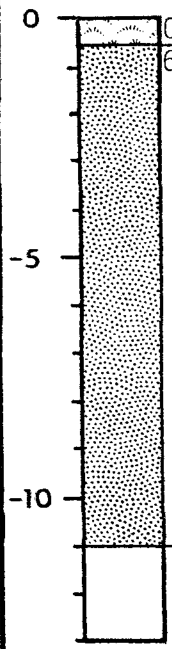
Schroeder Residence
Mercer Island, Washington

PLATE
**3**

| JOB NUMBER | DRAWN | APPROVED | DATE | REVISED | DATE |
|---|---|---|---|---|---|
| 15,336.001 | SAC | | 4 October 88 | | |

T.P. – E

**Soil Description and Classification**



0 - 6" TOPSOIL; DARK BROWN, SANDY DUFF

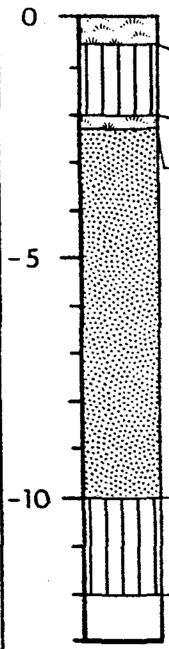
6"-11' SAND; LIGHT BROWN, FINE TO MEDIUM GRAINED, LOOSE TO MEDIUM DENSE, DAMP. (SP)
BECOMES MOTTLED, SILTY TO GRAVELLY, DENSE, BELOW 8'.

T.D. = 11'

Notes: SOME CAVING TO 8'.

T.P. – F

**Soil Description and Classification**



0 - 6" TOPSOIL; DARK BROWN, ORGANIC SILTY SAND.

6"- 2' SANDY SILT; MOTTLED BROWN, VERY STIFF, DAMP. (ML)

2'-28" TOPSOIL; AS ABOVE.

28"-10' SAND; LIGHT BROWN, MEDIUM DENSE, DAMP. (SP)
BECOMES SILTY, MOTTLED, DENSE TO VERY DENSE BELOW 6', THEN CLEANER BELOW 8'.

10'-12' SANDY SILT; MOTTLED BROWN, STIFF, WITH A FEW ROOTS, SMALL VOIDS, DAMP. (ML)

T.D. = 12'

Notes:

T.P. –

**Soil Description and Classification**

0
-5
-10

Notes:

T.P. –

**Soil Description and Classification**

0
-5
-10

Notes:

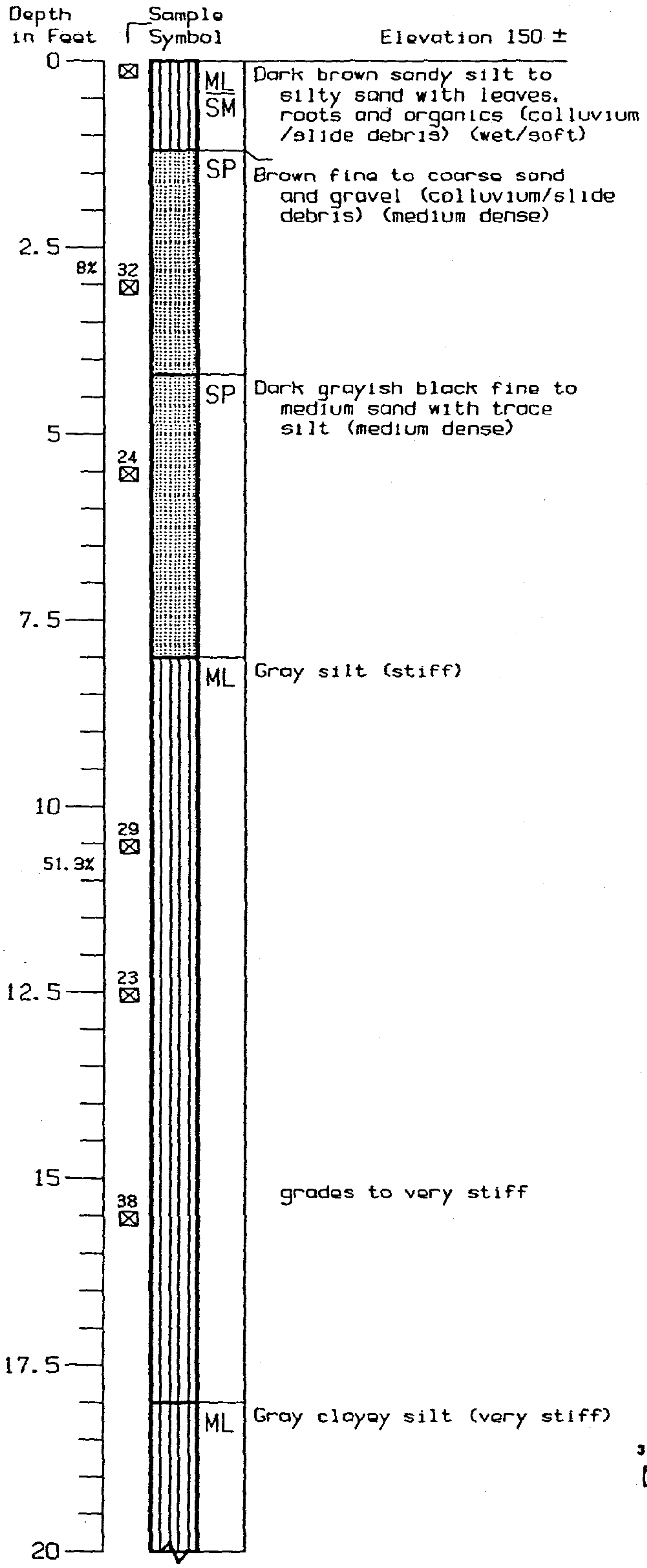
# TEST PIT LOG

CASCADE GEOTECHNICAL
A DIVISION OF
CASCADE TESTING LABORATORY, INC.

## SUMMERWOOD GLEN
## LOT 1

| | | | |
|---|---|---|---|
| Date 07/20/87 | Cert. No. 877 - 12G | Dwn. By HLA | Geo / Eng. SF |

## Boring B-1-89



Dark brown sandy silt to silty sand with leaves, roots and organics (colluvium /slide debris) (wet/soft)

Brown fine to coarse sand and gravel (colluvium/slide debris) (medium dense)

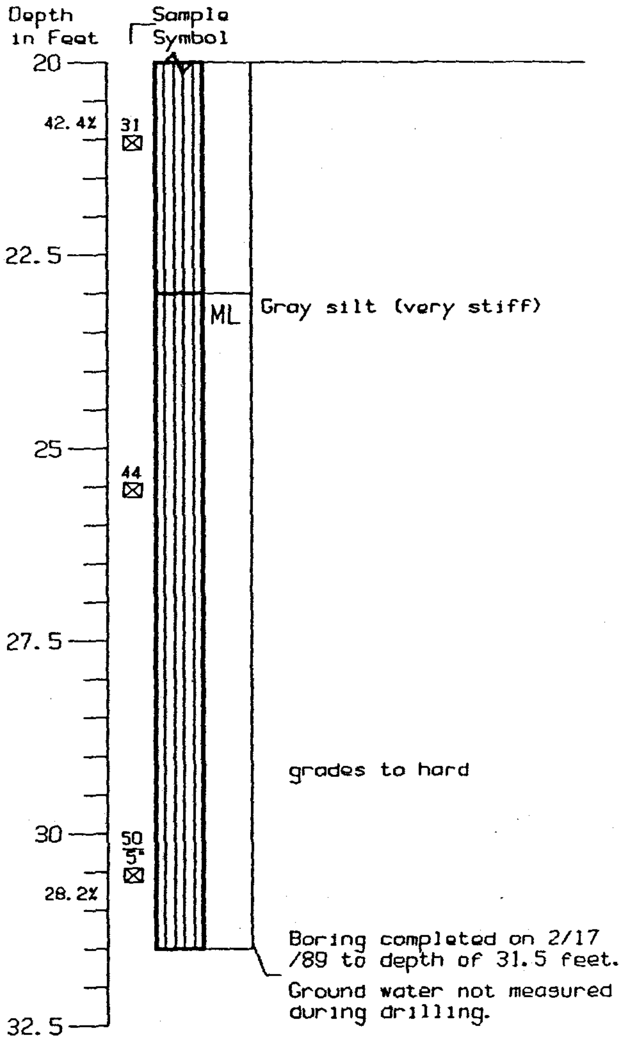
Dark grayish black fine to medium sand with trace silt (medium dense)

Gray silt (stiff)

grades to very stiff

Gray clayey silt (very stiff)

## Boring B-1-89, Cont.



Gray silt (very stiff)

grades to hard

Boring completed on 2/17 /89 to depth of 31.5 feet.

Ground water not measured during drilling.

**Key:**

MOISTURE CONTENT

51.3%

Blows required to drive a split spoon sampler one foot with a hammer weight of 140 lbs and a drop of 30 inches.

32 ⊠ Indicates depth at which split spoon sample was extracted.

**NOTE:**
**The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.**

## Log of Borings

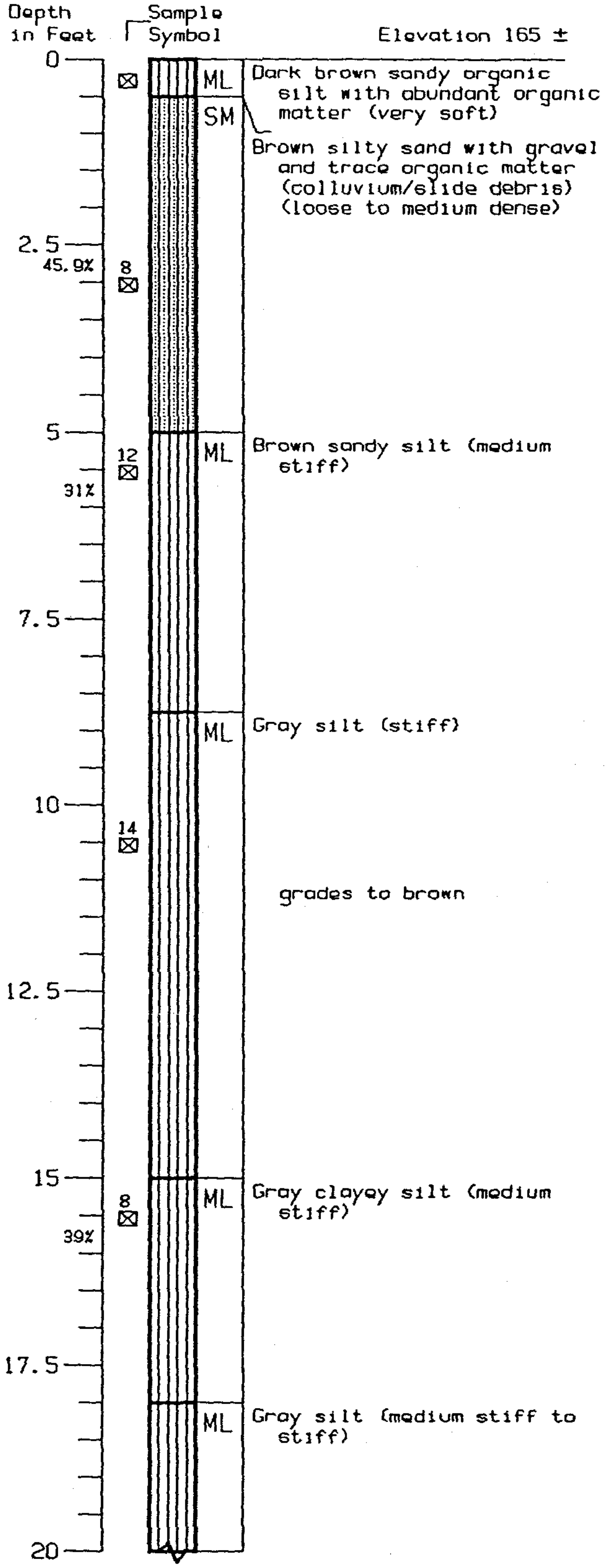
Dames & Moore

Job No. 17263-002-016

Plate 2

**Appdx B3 - Dames & Moore (1989) Beach Lane**

## Boring B-2-89



Dark brown sandy organic silt with abundant organic matter (very soft)

Brown silty sand with gravel and trace organic matter (colluvium/slide debris) (loose to medium dense)

Brown sandy silt (medium stiff)

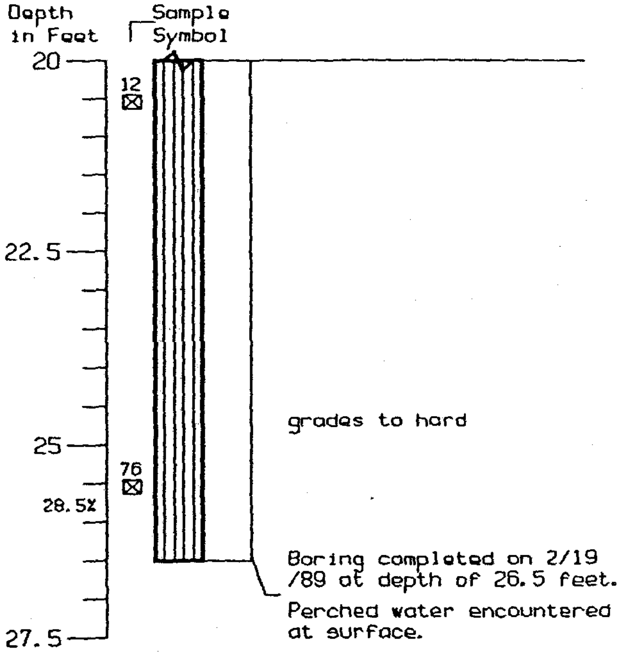
Gray silt (stiff)

grades to brown

Gray clayey silt (medium stiff)

Gray silt (medium stiff to stiff)

## Boring B-2-89, Cont.



grades to hard

Boring completed on 2/19/89 at depth of 26.5 feet.

Perched water encountered at surface.

Log of Borings

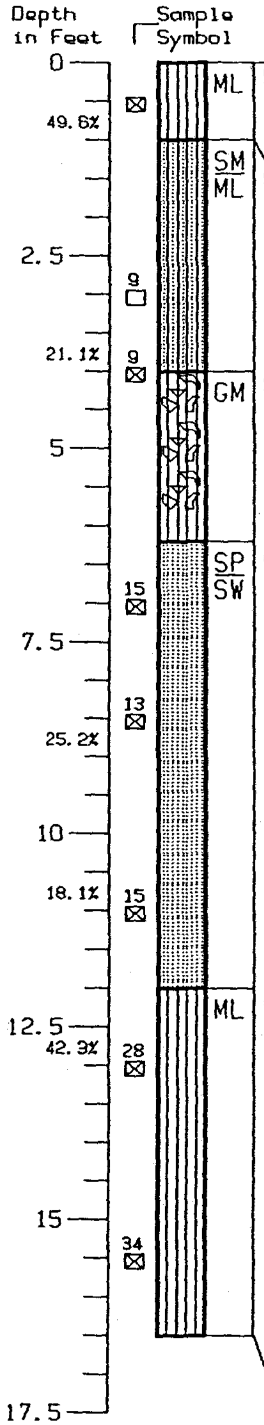
Dames & Moore

Job No. 17263-002-016

Plate 3

# Boring B-3-89

Elevation 180 ±



Dark brown organic silt with sand and abundant roots (colluvium/slide debris) (very soft)

Light brownish gray silty fine sand to sandy silt (colluvium/slide debris) (loose and medium dense)

Brown to gray-brown silty gravel (colluvium/slide debris) (loose to medium dense)

Black fine to coarse sand with gravel and trace silt (loose to medium dense)

Gray silt with some sand (very stiff)

Boring completed on 2/21/89 at depth of 16.5 feet.

Ground water not measured during drilling.

Log of Borings

Dames & Moore

# TEST PIT NO. 101 ✓

**Logged By** FC

**Date** 8-12-87

**Elev.** 200'±

| Depth (ft.) | USCS | Soil Description | W (%) |
|---|---|---|---|
| 0 | | (6" topsoil) | |
| | | | 6 |
| | sp<br>sm | Tan SAND, fine to medium grained, dry to moist, loose | |
| 5 | | - becomes medium dense below 4' | 8 |
| 10 | | - becomes very wet below 10' | ◁<br>17 |
| 15 | | Test pit terminated at 11 feet below existing grade.<br>Groundwater seepage encountered at 10 feet during excavation. | |

**Logged By** FC

**Date** 8-12-87

# TEST PIT NO. 102 ✓

**Elev.** 190'±

| Depth (ft.) | USCS | Soil Description | W (%) |
|---|---|---|---|
| 0 | | (6" topsoil) | 5 |
| | SP | Tan SAND, fine to medium grained, dry to moist, loose | |
| 5 | | - becomes medium dense below 5' | 11 |
| | | | 6 |
| 10 | | | 20 |
| 15 | | Test pit terminated at 11.5 feet below existing grade.<br>No groundwater seepage encountered during excavation. | |

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## TEST PIT LOGS

KINCAID PROPERTY
MERCER ISLAND, WASHINGTON

**Proj. No.** 3085-2 **Date** Aug'87 **Plate** 4

# TEST PIT NO. 103

**Logged By** FC

**Date** 8-12-87

**Elev.** 175'±

| Depth (ft.) | USCS | Soil Description | W (%) |
|---|---|---|---|
| 0 | pt | Black PEAT, wet, very soft | 595 |
| 5 | GP<br>GM | Tan sandy GRAVEL with a trace of silt, gravel to 3", fine to coarse SAND, saturated, loose<br>- pit walls unstable below 3' | ◁<br>16<br>10 |

Test pit terminated at 9 feet below existing grade due to heavy caving. Groundwater seepage encountered at 2 feet during excavation.

10

15

**Logged By** FC

**Date** 8-12-87

# TEST PIT NO. 104

**Elev.** 150'±

| Depth (ft.) | USCS | Soil Description | W (%) |
|---|---|---|---|
| 0<br>5<br>10 | gm<br>pt | Tan to gray silty sandy GRAVEL with abundant scattered organics lenses of peat and wood fragments, wet, loose<br>(colluvium - slide debris)<br>- logs to 8" diameter at 9' | 9<br>22<br>222<br>22◁<br>28 |

Test pit terminated at 11 feet below existing grade. Groundwater seepage encountered at 9 feet during excavation.

15

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**TEST PIT LOGS**
KINCAID PROPERTY
MERCER ISLAND, WASHINGTON

| **Proj. No.** 3085-2 | **Date** Aug'87 | **Plate** 5 |
|---|---|---|

# TEST PIT NO. 105 ✓

**Logged By** FC

**Date** 8-12-87

**Elev.** 143'±

| Depth (ft.) | USCS | Soil Description | W (%) |
|---|---|---|---|
| 0 | | (6" topsoil) | |
| | sm | Tan mottled silty SAND with small rhombic clay fragments, fine to medium sand, trace of gravel, occasional organics, slightly moist, loose | 9<br>9 |
| 5 | | - very moist and medium dense below 6' (colluvium - slide debris | 13<br>28 |
| 10 | | - becomes gray at 10' | 23 |
| 15 | | Test pit terminated at 12 feet below existing grade. No groundwater seepage encountered during excavation. | |

**Logged By** FC

**Date** 8-12-87

# TEST PIT NO. 106 ✓

**Elev.** 125'±

| Depth (ft.) | USCS | Soil Description | W (%) |
|---|---|---|---|
| 0 | | (6" topsoil) | 12 |
| | ml<br>sm | Tan silty SAND to sandy SILT, trace of gravel, slightly moist, medium dense, no caving | |
| 5 | | | 6 |
| | sp | Brown orange gravelly SAND, fine to medium SAND, gravel to 3.5', slightly moist, medium dense | 3 |
| 10 | | Test pit terminated at 10 feet below existing grade. No groundwater seepage encountered during excavation. | |
| 15 | | | |

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## TEST PIT LOGS
KINCAID PROPERTY
MERCER ISLAND, WASHINGTON

**Proj. No.** 3085-2 | **Date** Aug'87 | **Plate** 6

# BORING NO. 1

Logged By FC
Date 1-7-87

ELEV. 163±

| Graph | US CS | Soil Description | Depth (ft.) | Sample | (N) Blows Ft. | W (%) |
|---|---|---|---|---|---|---|
| | sm | Tan to gray silty SAND with gravel, wet to saturated, loose | 5 | I | 5 | 24 |
| | SP | Tan SAND, fine to medium grained, in layers with fine gravel, saturated, medium dense | 10<br>15 | I<br>I<br>I | 17<br>14<br>23 | 24<br>17<br>16 |
| ▨ | ch | Gray CLAY, plastic, fissured, wet, hard | | I | 63 | 32 |
| | sm | Gray silty SAND, non-plastic dilatent, saturated, dense | 20 | I | 32 | 24 |
| ▨ | CH | Gray CLAY, plastic, fissured, wet, hard | 25 | I<br>I | 71<br>63/0 | 36<br>33 |
| | ml | Gray SILT, non-plastic in layers with clay, wet, hard | 30<br>35 | I<br>I<br>I | 61<br>82<br>75/11 | 25<br>25<br>26 |

Boring terminated at 39 feet below existing grade. Groundwater encountered at surface during drilling. (springs) Boring backfilled with cuttings.

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BORING LOG
MORRIS PROPERTY
MERCER ISLAND, WASHINGTON

| Proj. No. 3085-2 | Date Jan. '87 | Plate 7 |
|---|---|---|

Appdx B4 - Earth Consultants (1986-87) 4298 E Mercer Way

# BORING NO. 2

Logged By FC

Date 1-8-87

ELEV. 143±

| Graph | US CS | Soil Description | Depth (ft.) | Sample | (N) Blows Ft. | W (%) |
|---|---|---|---|---|---|---|
| | SM | Tan silty SAND With gravel and frequent organic fragments, slightly plastic, wet to saturated, loose to medium dense | | I | 15 | 23 |
| | | | 5 | I | 8 | 23 |
| | | | | I | 7 | 50 |
| | | | 10 | I | 20 | |
| | | | | I | 13 | 29 |
| | sp | Gray SAND with gravel, in layers, saturated, medium dense to dense | 15 | I | 12 | 16 |
| | | | | I | 42 | 13 |
| | sp-gp | | 20 | I | 34 | 19 |
| | | | | I | 47 | 15 |
| | sp | | 25 | I | 46 | 19 |
| | cl | Gray silty CLAY with rhombic clay fragments, slightly plastic, wet, very stiff | | I | 18 | 41 |
| | ch | Gray CLAY with hard rhombic clay fragments in the groundmass, plastic, wet, very stiff | 30 | I | 26 | 39 |
| | ml | Gray SILT, non-plastic, in layers with clay, very stiff to hard | 35 | I | 28 | 31 |
| | | | | I | 54 | 30 |
| | CH | Gray CLAY, plastic, varved bedding, wet, hard | 40 | T | 78 | 36 |

Boring terminated at 44.5 feet below existing grade. Groundwater encountered at 10 feet during drilling. Boring backfilled with cuttings.

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BORING LOG

MORRIS PROPERTY
MERCER ISLAND, WASHINGTON

| Proj. No. 3085-2 | Date Jan. '87 | Plate 8 |
|---|---|---|

# TEST PIT NO. 1

Logged By FC

Date 1-22-86

Elev. 140±

| Depth (ft.) | | USCS | Soil Description | W (%) |
|---|---|---|---|---|
| 0 | | ML | (6" light brown silty TOPSOIL)<br>Tan SILT with scattered gravel and clay fragments, frequent wood and organics, fragmental texture, moist, medium dense (colluvium/slide debris) | 9<br>14<br>40 |
| 5 | | | | |
| 10 | ░ | sm | Gray silty SAND with abundant wood fragments including small logs, occasional sand lenses, saturated, medium dense (colluvium/slide debris) | 44<br>28<br>8 |
| 15 | | | Test pit terminated at 14 feet below existing grade.<br>No groundwater seepage encountered during excavation. | |
| 20 | | | | |

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TEST PIT LOGS
MORRIS PROPERTY
MERCER ISLAND, WASHINGTON

8

Proj. No. 3085-2 | Date Jan.'87 | Plate 9

# TEST PIT NO. 2 ✓

Logged By FC

Date 10-22-86

Elev. 145±

| Depth (ft.) | USCS | Soil Description | W (%) | |
|---|---|---|---|---|
| 0 | | (6" tan silty TOPSOIL) | | |
| | Fill | Tan SILT with gravel and occasional organics, moist, stiff (fill) | 8 | |
| | | | 32 | |
| | | Thin brown TOPSOIL at 4' | | |
| 5 | ml | Tan clayey SILT, slightly plastic, with sandy zones and abundant wood fragments, moist, stiff | | 2.0tsf |
| | | | 65 | |
| | ml | Becomes gray, wet and soft to medium stiff below 7' | | .25-1tsf |
| 10 | | (colluvium/slide debris) | 27 | |
| | | Small log at 13' | | |
| 15 | | | 17 | |

Test pit terminated at 15 feet below existing grade.
No groundwater seepage encountered during excavation.

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**TEST PIT LOGS**
MORRIS PROPERTY
MERCER ISLAND, WASHINGTON

Proj. No. 3085-2 | Date Jan. '87 | Plate 10

# TEST PIT NO. 3

Logged By FC
Date 10-22-86

Elev. 143±

| Depth (ft.) | USCS | Soil Description | W (%) | |
|---|---|---|---|---|
| 0 | sm | Gray silty SAND, fine grained, saturated, loose (alluvium) | | |
| | pt | Brown PEAT with occasional wood fragments and small logs, wet, very soft | 218 | |
| | sp | Gray SAND, fine to medium grained, saturated, loose (alluvium) | | |
| 5 | | Test pit terminated at 3.5 feet below existing grade.<br>Water encountered at surface during excavation. | | |
| 10 | | | | |
| 15 | | | | |

Logged By FC
Date 10-22-86

# TEST PIT NO. 4

Elev. 143±

| | | | | |
|---|---|---|---|---|
| 0 | | (6" tan silty TOPSOIL) | | |
| | ml | Tan to light gray SILT with fragments of clay and clayey silty fragmental texture, moist, medium dense to stiff | 15 | |
| 5 | | | | 1.0tsf |
| | | | 26 | |
| 10 | | | 27 | |
| | | Test pit terminated at 11 feet below existing grade.<br>No groundwater seepage encountered during excavation. | | |
| 15 | | | | |

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TEST PIT LOGS
MORRIS PROPERTY
MERCER ISLAND, WASHINGTON

Proj. No. 3085-2 | Date Jan. '87 | Plate 11

# TEST PIT NO. 5

Logged By FC

Date 10-23-86

Elev. 142±

| Depth (ft.) | USCS | Soil Description | W (%) |
|---|---|---|---|
| 0 | pt | Brown PEAT, fibrous, numerous wood fragments, occasional small logs, wet, very soft | 639 |
| 5 | sp | Gray SAND, fine to coarse grained with gravel, saturated, loose (alluvium) | 13 |
| 10<br>15 | | Test pit terminated at 5 feet below existing grade. Very heavy groundwater seepage encountered at 3 feet during excavation. Standing water on surface. | |

Logged By FC

Date 10-23-86

# TEST PIT NO. 6

Elev. 136±

| Depth (ft.) | USCS | Soil Description | W (%) |
|---|---|---|---|
| 0<br>5<br>10 | Slide Debris | Tan to gray SILT containing fragments of clay, sand lenses, pods of peat, logs, and scattered gravel, very wet, loose<br>(slide debris) | 40<br>202<br>39<br>50 |
| 15 | | Test pit terminated at 12 feet below existing grade. Abundant groundwater seepage encountered throughout excavation. | |

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TEST PIT LOGS
MORRIS PROPERTY
MERCER ISLAND, WASHINGTON

Proj. No. 3085-2 | Date Jan. '87 | Plate 12

# TEST PIT NO. 7

Logged By FC

Date 10-23-86

Elev. 165±

| Depth (ft.) | USCS | Soil Description | W (%) |
|---|---|---|---|
| 0 | pt | Brown PEAT, fibrous, wet, very soft | 65 |
| | sm | Gray silty SAND with woody organics. saturated, loose | 65 |
| 5 | sm-gm<br>GP-GM | Tan gravelly SAND with silt, contains pockets of clay and organics, saturated, loose (slide debris) | 39<br>10<br>54<br>17 |
| 10 | | Test pit terminated at 10 feet below existing grade.<br>Water encountered at surface during excavation. | |
| 15 | | | |

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TEST PIT LOGS
MORRIS PROPERTY
MERCER ISLAND, WASHINGTON

Proj. No. 3085-2 | Date Jan. '87 | Plate 13

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# TEST PIT NO. 8

Logged By FC

Date 10-23-86

Elev. 172±

| Depth (ft.) | USCS | Soil Description | W (%) |
|---|---|---|---|
| 0<br>5 | sp-<br>gp<br>GP-<br>GM<br>sp-<br>gp | Tan gravelly SAND, fine to coarse grained, gravel to 4", in pods or lenses, slightly moist, loose to medium dense (colluvium/slide debris) | 7<br>3<br>3 |
| 10 | ml | Gray SILT, fragmental texture with sand and gravel lenses, loose, wet (slide debris) | 39 |
| | sp | Tan gravelly SAND with silt or clay lenses, saturated, loose (colluvium/slide debris) | 7 |
| 15 | | Test pit terminated at 14 feet below existing grade. Groundwater seepage encountered at 12 feet during excavation. | |
| 20 | | | |

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TEST PIT LOGS
MORRIS PROPERTY
MERCER ISLAND, WASHINGTON

Proj. No. 3085-2 | Date Jan.'87 | Plate 14

# TEST PIT NO. 9

Logged By FC

Date 10-23-86

Elev. 161±

| Depth (ft.) | USCS | Soil Description | W (%) |
|---|---|---|---|
| 0<br>5<br>10 | SP-SM | Tan gravelly SAND to sandy GRAVEL, with silt, in lenses, moist to saturated, loose to medium dense (colluvium/slide debris) | 8<br>36<br>11<br>18 |
| 15 | | Test pit terminated at 11 feet below existing grade. Groundwater seepage encountered at 4 feet during excavation. | |

Earth Consultants Inc.
GEOTECHNICAL ENGINEERING & GEOLOGY

TEST PIT LOGS
MORRIS PROPERTY
MERCER ISLAND, WASHINGTON

Proj. No. 3085-2 | Date Jan. '87 | Plate 15

# Test Pit Log

| | | | |
|---|---|---|---|
| Project Name:<br>Lot 4, Waterside | | | Sheet 1 of 1 |
| Job No.<br>6945 | Logged by:<br>SD | Date:<br>6/22/95 | Test Pit No.:<br>**TP-1** |
| Excavation Contactor:<br>By Owner | | | Ground Surface Elevation: |
| Notes: | | | |

| | W (%) | Graphic Symbol | Depth Ft. | Sample | USCS Symbol | Surface Conditions: Depth of topsoil and duff 8" |
|---|---|---|---|---|---|---|
| | | ▨ | | | SM | (8" Topsoil and Duff)<br>Brown silty medium SAND with gravel, medium dense, moist |
| | | ▨ | 1 | | | |
| | | ▨ | 2 | | | |
| | | ▨ | 3 | | | |
| | | ▨ | 4 | | | |
| | 3.8 | ▨ | 5 | | | |
| | | ▨ | 6 | | | |
| | | | 7 | | | Test pit terminated at 7.0 feet below existing grade. No groundwater seepage encountered during excavation. |

TPL 6945 7/6/95

Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

**Test Pit Log**
Lot 4, Waterside
Mercer Island, Washington

| | | | | | |
|---|---|---|---|---|---|
| Proj. No. 6945 | Dwn. GLS | Date July '95 | Checked SD | Date 7/6/95 | Plate A2 |

# Test Pit Log

Project Name: Lot 4, Waterside

Sheet 1 of 1

Job No.: 6945

Logged by: SD

Date: 6/22/95

Test Pit No.: **TP-2**

Excavation Contactor: By Owner

Ground Surface Elevation:

Notes:

| | W (%) | Graphic Symbol | Depth Ft. | Sample | USCS Symbol | Surface Conditions: Depth of topsoil and duff 6" |
|---|---|---|---|---|---|---|
| | | ░ | 1 | | SM | (6" Topsoil and Duff)<br>*Brown silty medium SAND with gravel, Medium dense, moist* |
| | | ▥ | 2 | | ML | *Grayish tan SILT, medium dense, moist* |
| | | ▥ | 3 | | | |
| | 26.5 | ▥ | 4 | | | |
| | | ▥ | 5 | | | |
| | | ▥ | 6 | | | |
| | | ▥ | 7 | | | |
| | | | | | | Test pit terminated at 7.0 feet below existing grade. No groundwater seepage encountered during excavation. |

TPL 6945 7/6/95

**Earth Consultants Inc.**
Geotechnical Engineers, Geologists & Environmental Scientists

## Test Pit Log
Lot 4, Waterside
Mercer Island, Washington

| | | | | | |
|---|---|---|---|---|---|
| Proj. No. 6945 | Dwn. GLS | Date July '95 | Checked SD | Date 7/6/95 | Plate A3 |

# EXPLORATORY PIT LOG

EP-1 Elevation 292 feet

| | |
|---|---|
| 0.0' - 0.9' | Duff, roots and loose, sandy topsoil, damp. |
| 0.9' - 2.2' | Loose, brown, silty fine to medium sand with gravels and some cobbles to 8 inches, damp, root penetration to 2 feet. |
| 2.2' - 4.5' | Loose to medium dense, grey, gravelly mixed sand, damp, well sorted, recessional/fluvial sediments. |
| | ------- contact dips 15 degrees to the east ------- |
| 4.5' - 6.5' | Loose to medium dense, tan brown, silty fine sand, damp, vague bedding. |
| 6.5' -11.0' | Medium dense, grey brown, slightly silty sand gravel, damp. |

T.D. @ 11.3', 4-30-87

EP-2 Elevation 291 feet

| | |
|---|---|
| 0.0' - 2.2' | Duff, roots and loose, dark brown, sandy topsoil, damp. |
| 2.2' - 6.5' | Loose to medium dense, grey brown, silty sandy fine gravel, damp, seams of glacial drift-like materials @ 5 feet, becomes medium dense beyond about 5 feet. |
| 6.5' -11.5' | Loose to medium dense, silty fine sand, damp, some gravel, well sorted, fluvial sediments. |

T.D. @ 11.5', 4-30-87

874-8 June 1987
Eleanor Hill Property
EPL-1 and EPL-2

**Appdx B1 - Geological Services (1987) 7712 89th Place SE**

**Appdx B6 - Geological Services (1987) 7712 89th Place S.E.**

# EXPLORATORY PIT LOG (cont'd)

## EP-3 Elevation 294 feet

| | |
|---|---|
| 0.0' - 1.2' | Duff, roots and loose, dark brown, sandy topsoil, damp. |
| 1.2' - 3.5' | Medium dense, reddish brown (to grey brown beyond 2.5 feet), gravel, silty fine to medium sand, damp, moderately well sorted. |
| 3.5' -10.0' | Loose to medium dense, grey, gravelly medium sand, damp, clean and well sorted, occasional glacial drift-like seams, well bedded, medium dense beyond about 6 feet, sand coarsens with depth trending to sandy gravel. |

T.D. @ 10.0', 4-30-87

## EP-4 Elevation 308 feet

| | |
|---|---|
| 0.0' - 0.9' | Duff, roots and loose, tan brown, silty topsoil, damp. |
| 0.9' - 2.0' | Logs, tan brown, silty fine to medium sand with scatt gravels and cobbles, damp. |
| 2.0' -10.0' | Medium dense, grey, gravelly, medium sand, damp, well sorted and clean, some bedding, gravel content increases beyond about 6.5 feet. |

T.D. @ 10.0', 4-30-87

874-8 June 1987
Eleanor Hill Property
EPL-3 and EPL-4

# Boring Log HC-1

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet 193

(Very loose), wet, brown, slightly gravelly, silty SAND with numerous small roots and scattered silt lumps. (COLLUVIUM)

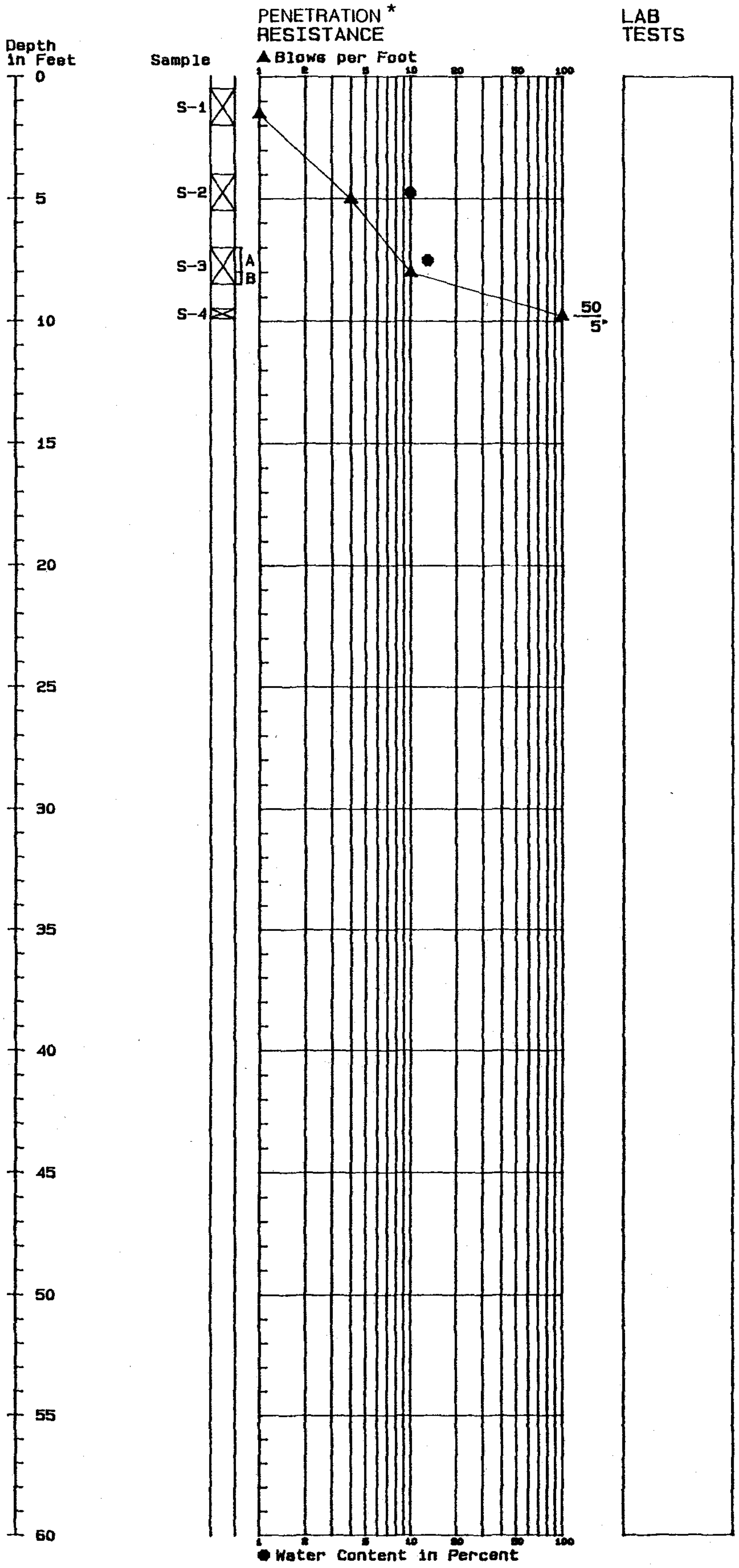
(Very loose), moist, brown, clean to slightly silty, fine to medium SAND with scattered gravel, silt lumps and organics. (COLLUVIUM)

(Loose), moist, brown, slightly silty, sandy GRAVEL with scattered organics. (COLLUVIUM)

(Very dense), moist, brown, sandy GRAVEL.

Bottom of Boring at 9.9 Feet.
Completed 12/7/88.

*Penetration Resistance based on:
Hammer Weight: 40 lbs.
Drop: 18 inches
I.D. of Sampler: 1-3/8 inches
O.D. of Sampler: 2 inches



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

J-2289 December 1988

HART-CROWSER & associates, inc.

Figure A-2

# Boring Log HC-2

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet 218

(Very loose), wet, brown, silty, fine to medium SAND with scattered gravel, silt lumps, and organics. (COLLUVIUM)

(Very stiff to hard), moist, gray, clayey to slightly fine sandy SILT with scattered small roots and brown, oxidized zones.

Thin seams of fine sand.

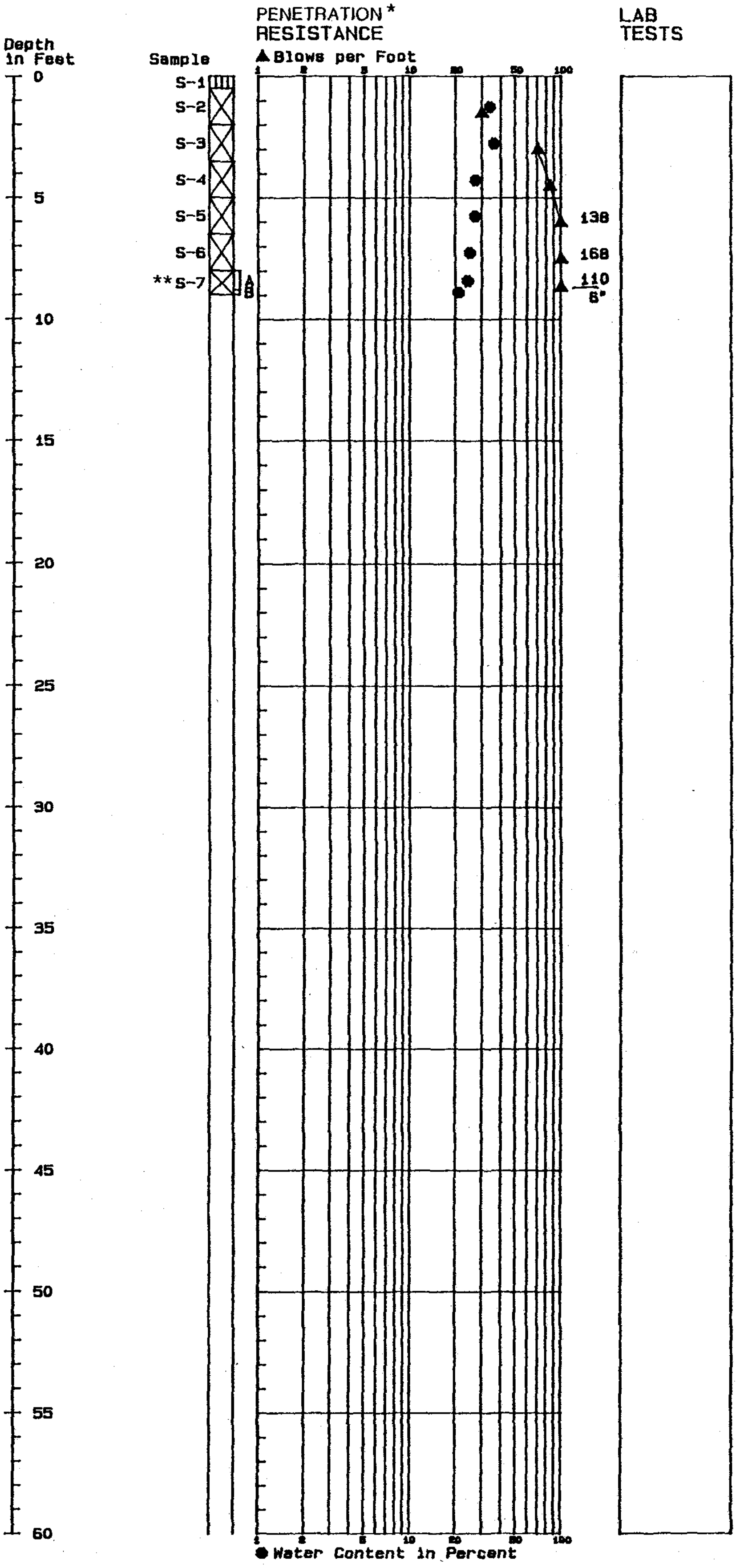
(Hard), moist, red-brown (oxidized), fine sandy SILT.

Bottom of Boring at 9.0 Feet.
Completed 12/8/88.

*Penetration Resistance based on:

Hammer Weight: 40 lbs.
Drop: 18 inches
I.D. of Sampler: 1-3/8 inches
O.D. of Sampler: 2 inches

**I.D. of Sampler: 1 inch
O.D. of Sampler: 1-3/8 inches



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

J-2289 December 1988
HART-CROWSER & associates, inc.
Figure A-3

# Boring Log HC-3

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet 195

(Loose), wet, brown, slightly gravelly, silty SAND with numerous small roots, organics, and silt lumps. (COLLUVIUM)

(Very loose to loose), moist, brown, clean to slightly silty, fine to medium SAND and GRAVEL with scattered organics. (COLLUVIUM)

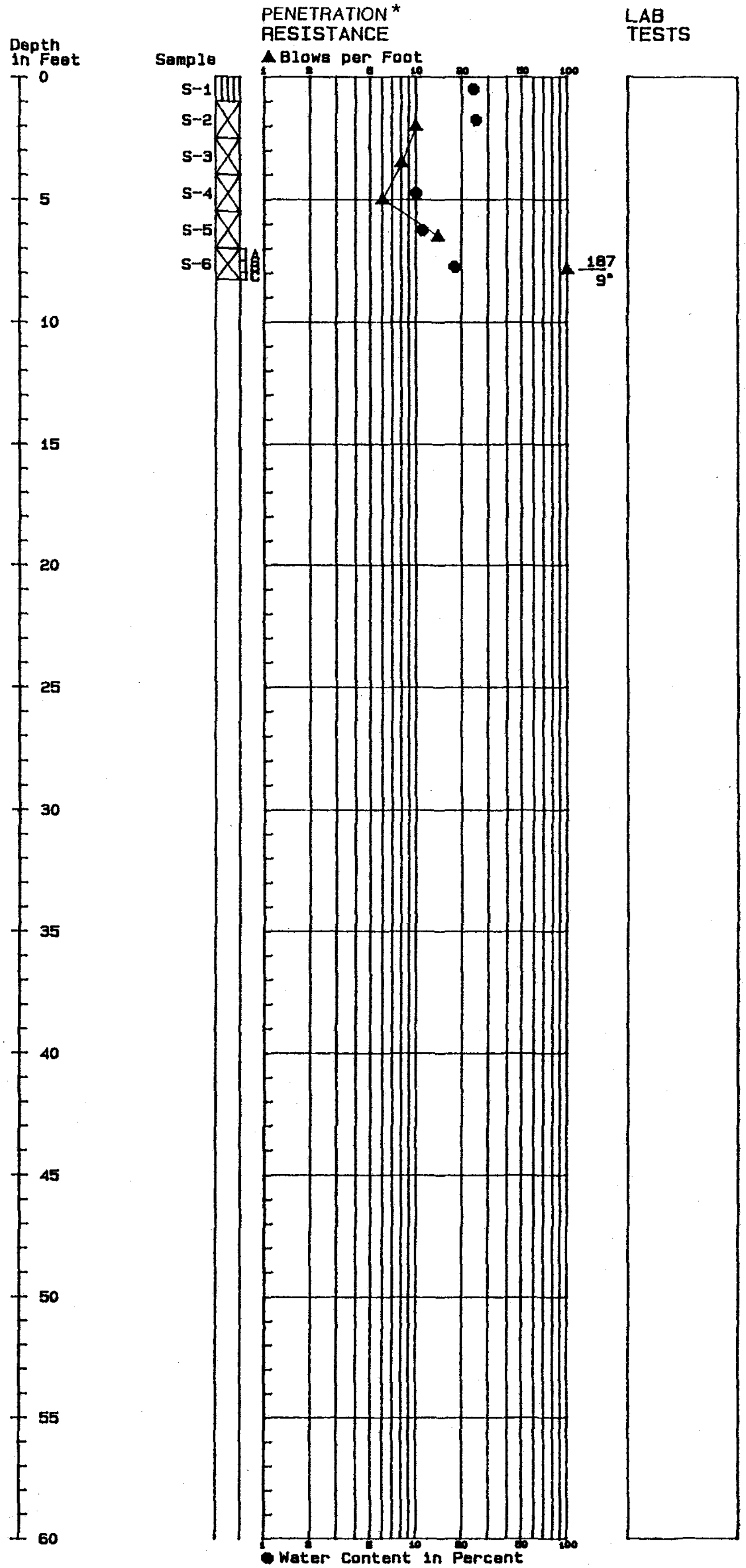
(Very dense), moist, brown, silty, fine to medium SAND.

(Very dense), moist, brown, sandy GRAVEL.

Bottom of Boring at 8.3 Feet.
Completed 12/8/88.

*Penetration Resistance based on:

Hammer Weight: 40 lbs.
Drop: 18 inches
I.D. of Sampler: 1-3/8 inches
O.D. of Sampler: 2 inches



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

J-2289 December 1988

HART-CROWSER & associates, inc.

Figure A-4

# Boring Log HC-101

Soil Descriptions

Ground Surface Elevation in Feet 180

(Loose), wet, brown, silty, gravelly SAND. (COLLUVIUM)

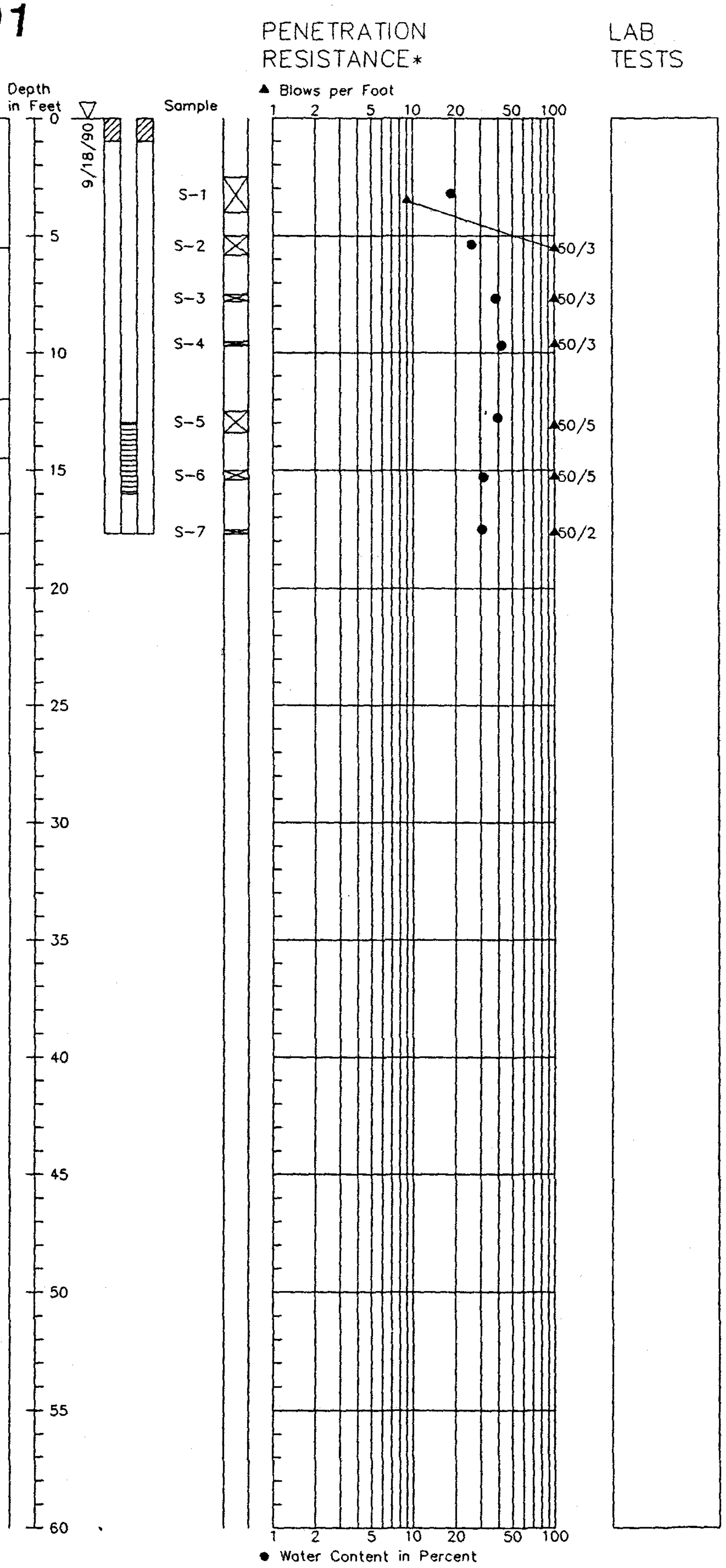
(Medium stiff), wet, gray SILT to fine sandy SILT. (COLLUVIUM)

(Medium stiff), wet, gray, fractured, clayey SILT. (COLLUVIUM)

(Hard), moist, gray, clayey SILT.

Bottom of Boring at 17.7 Feet.
Completed 9/4/90.

*Penetration Resistance based on:
Hammer Weight: 40 lbs.
Drop: 18 inches
I.D. of Sampler: 1-3/8 inches
O.D. of Sampler: 2 inches



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER
J-2289-01 9/90
Figure A-5 1/1

# Boring Log HC-102

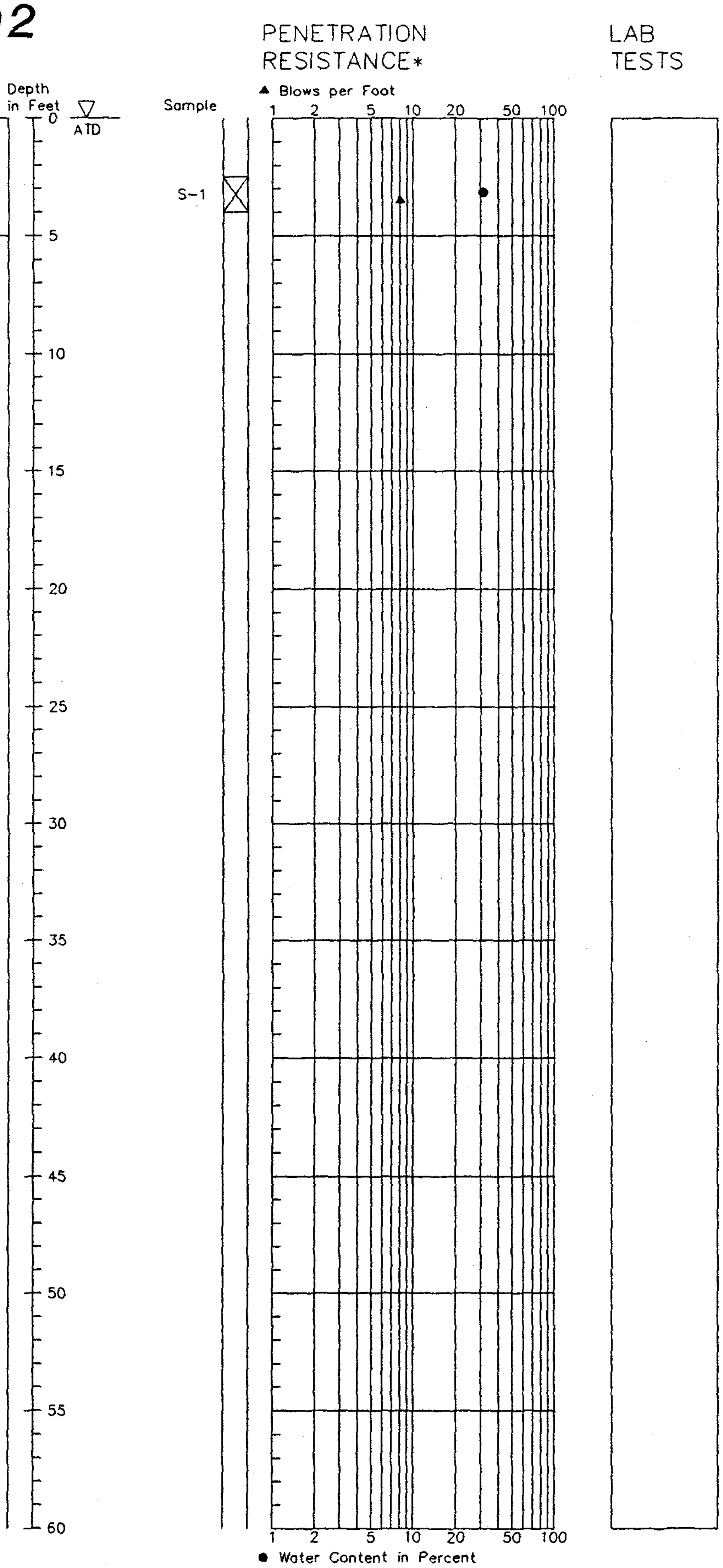
## Soil Descriptions

Ground Surface Elevation in Feet 168

(Soft), wet, mottled gray and orange, fine sandy SILT with scattered organics. (COLLUVIUM)

Bottom of Boring at 5.0 Feet.
Completed 9/5/90.

*Penetration Resistance based on:
Hammer Weight: 40 lbs.
Drop: 18 inches
I.D. of Sampler: 1-3/8 inches
O.D. of Sampler: 2 inches



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER
J-2289-01 9/90
Figure A-6 1/1

# Boring Log HC-103

Soil Descriptions

Ground Surface Elevation in Feet 159

Loose, moist to wet, gray and red-brown, silty to clean, gravelly SAND with organics. (COLLUVIUM)

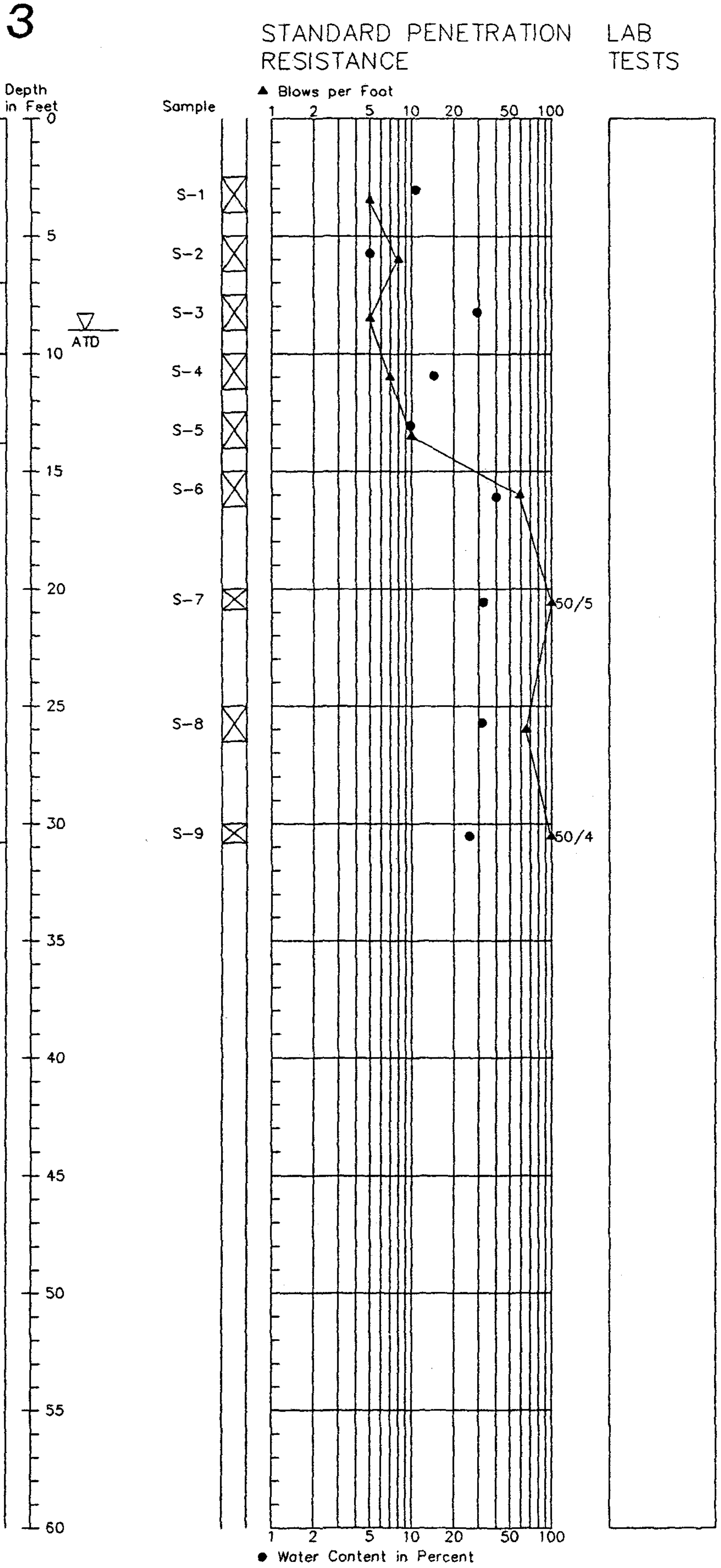
Medium stiff, wet, gray, fine sandy SILT with layers of sand. (COLLUVIUM)

Loose, wet, gray, gravelly SAND to sandy GRAVEL. (COLLUVIUM)

Hard, moist, gray SILT to fine sandy SILT.

Bottom of Boring at 30.8 Feet.
Completed 8/29/90.

2.75-inch outside diameter slope inclinometer casing installed to elevation 130.5 feet.



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

**HARTCROWSER**

**J-2289-01** **9/90**
**Figure A-7** **1/1**

# Boring Log HC-104

## Soil Descriptions

Ground Surface Elevation in Feet 158

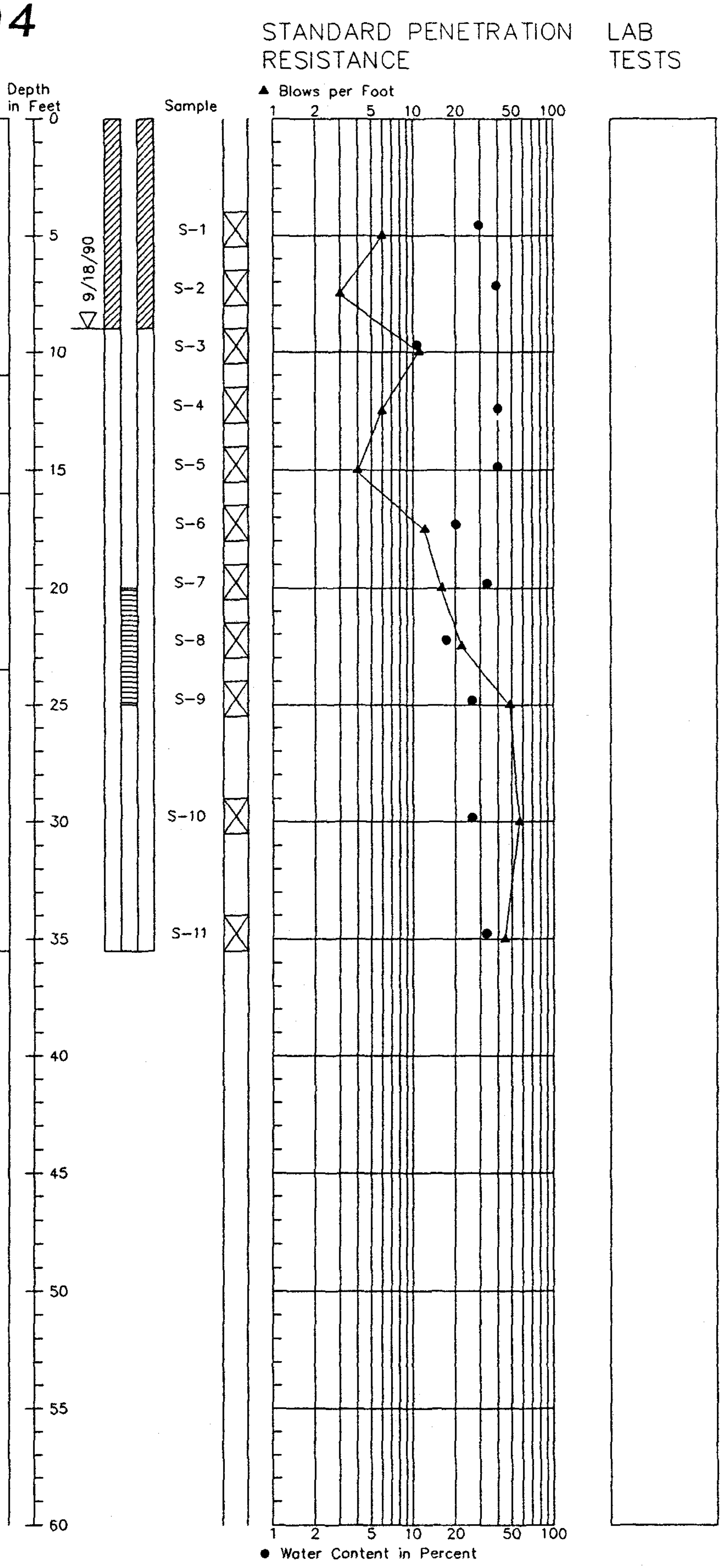
Very loose to medium dense, wet, gray and brown, slightly gravelly, silty SAND to sandy GRAVEL with organics. (COLLUVIUM)

Medium stiff to soft, wet, gray, fractured, clayey SILT. (COLLUVIUM)

Medium dense, wet, gray, clayey, gravelly SAND with organics and layers of silt and sand. (COLLUVIUM)

Hard, wet, gray, fine sandy SILT.

Bottom of Boring at 35.5 Feet.
Completed 9/4/90.



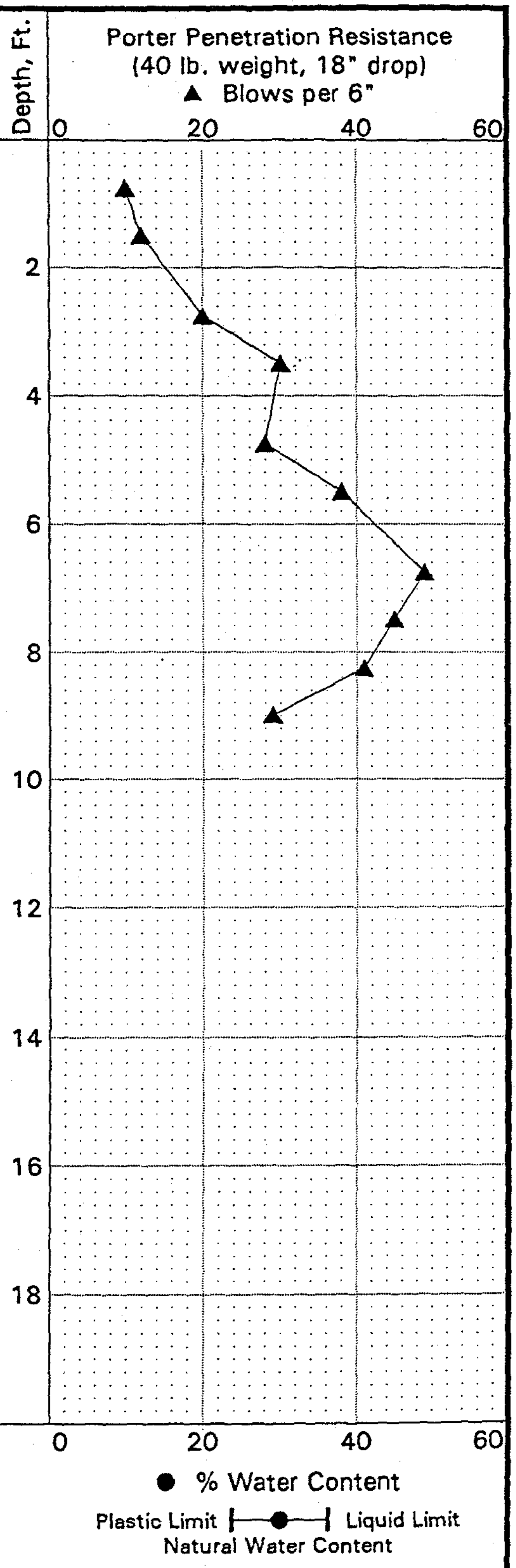
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER
J-2289-01 9/90
Figure A-8 1/1

**Appdx B7 - Hart Crowser (1988-90) Tarywood Park Stairway**

HANDBRNG 7/22/97

| MATERIAL DESCRIPTION<br>Surface Elevation: Approx. Feet | Depth, Ft. | Symbol | Samples | Ground Water |
|---|---|---|---|---|
| Medium dense, brown, medium and coarse SAND, trace of silt, occasional fine gravel; moist; SP. | | | 1 | |
| Dense, brown, gravelly, medium to coarse SAND; moist; SP. | 2.0 | | 2<br>3 | |
| Dense, brown, medium to coarse SAND; moist to wet; occasional fine gravel; SP. | 5.5 | | 4<br>5 | |
| BOTTOM OF BORING<br>COMPLETED 7/10/97 | 9.0 | | | |



## LEGEND

- * Sample Not Recovered
- ⊻ Water Level
- ⌶ 1.5" O.D. Split Spoon Sample
- ◫ Surface Seal
- ⊠ Annular Sealant
- ☷ Piezometer Screen
- ▨ Grout

## NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. The Porter penetration resistance in blows per 6 inches correlates approximately to the Standard Penetration Resistance in blows per foot.
5. Refer to KEY for explanation of 'Soil Log' symbols.
6. USC letter symbol based on visual classification.

Pemberton Wall Evaluation
Mercer Island, Washington

# LOG OF HAND BORING HB-1

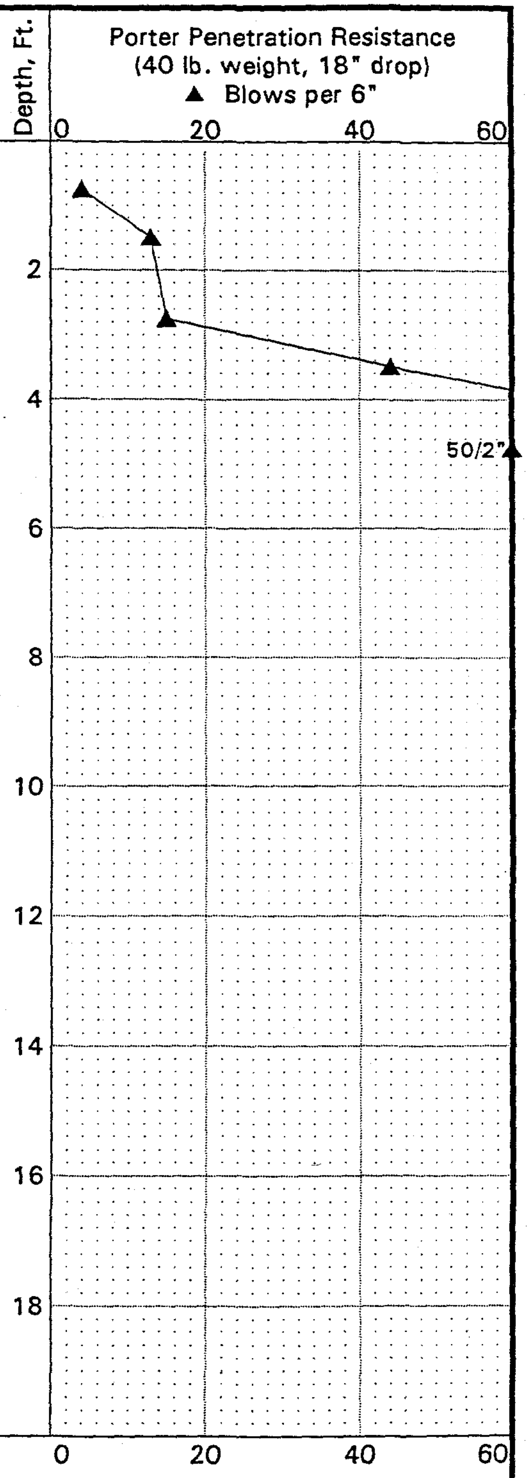
July 1997 W-7882-01

**SHANNON & WILSON, INC.**
Geotechnical and Environmental Consultants

**FIG. A-2**

HANDBRNG 7/22/97

| MATERIAL DESCRIPTION<br>Surface Elevation: Approx. Feet | Depth, Ft. | Symbol | Samples | Ground Water |
|---|---|---|---|---|
| Medium dense, brown, slightly gravelly SAND, trace of silt; moist; SW. | | | 1 | |
| Dense, brown, gravelly, medium to coarse SAND; moist; SP. | 2.0 | | 2<br>3 | |
| BOTTOM OF BORING<br>COMPLETED 7/10/97 | 5.5 | | | |



● % Water Content

Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

## LEGEND

- * Sample Not Recovered
- Water Level
- 1.5" O.D. Split Spoon Sample
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout

## NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. The Porter penetration resistance in blows per 6 inches correlates approximately to the Standard Penetration Resistance in blows per foot.
5. Refer to KEY for explanation of 'Soil Log' symbols.
6. USC letter symbol based on visual classification.

Pemberton Wall Evaluation
Mercer Island, Washington

# LOG OF HAND BORING HB-2

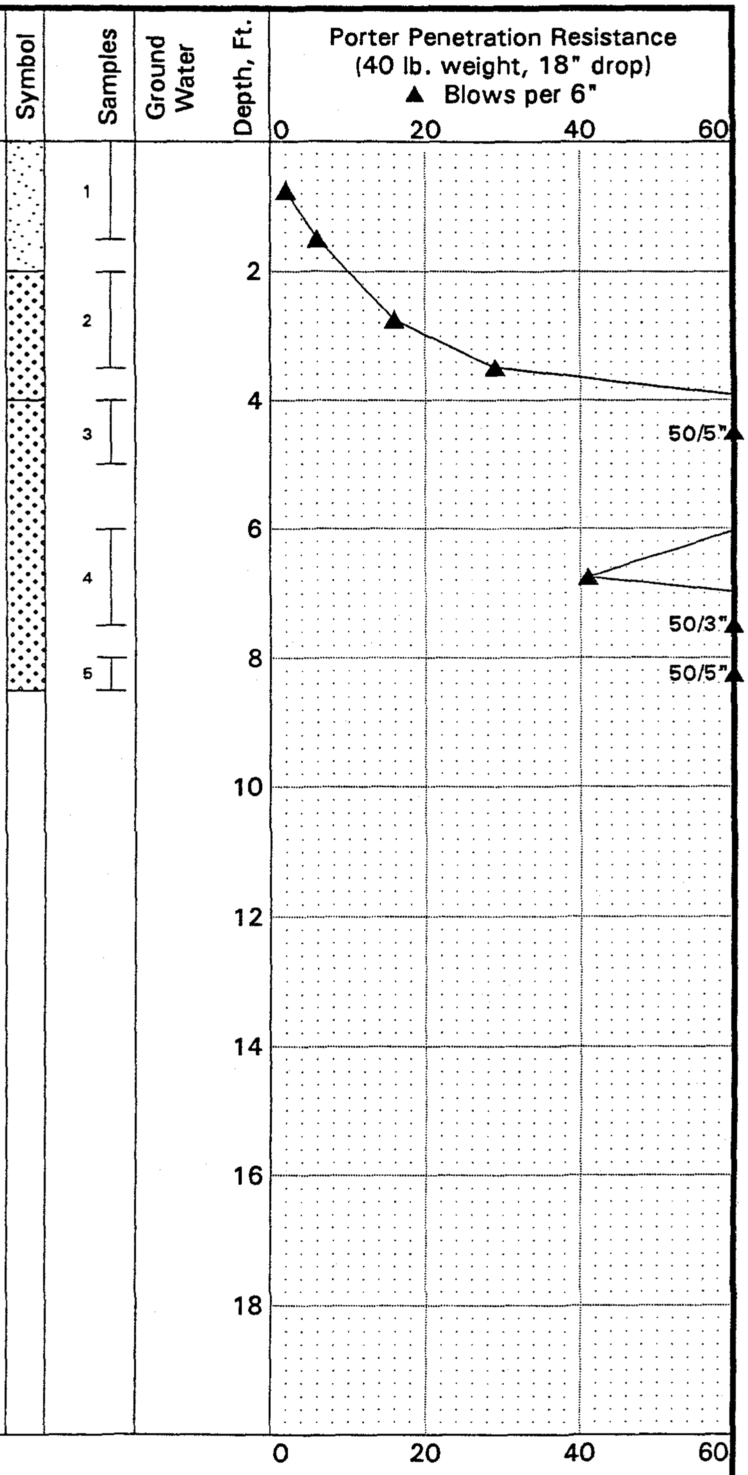
July 1997 W-7882-01

**SHANNON & WILSON, INC.**
Geotechnical and Environmental Consultants

**FIG. A-3**

HANDBRNG 7/22/97

| MATERIAL DESCRIPTION<br>Surface Elevation: Approx. Feet | Depth, Ft. |
|---|---|
| Loose, brown, gravelly, medium to coarse SAND; moist; SP. | 2.0 |
| Medium dense, brown, slightly gravelly, fine to medium SAND, trace of silt; moist; SW. | 4.0 |
| Very dense, light brown, fine to medium SAND; moist; occasional traces of fine gravel and silt; SW. | 8.5 |
| BOTTOM OF BORING<br>COMPLETED 7/10/97 | |



## LEGEND

- * Sample Not Recovered
- ⊻ Water Level
- ⌶ 1.5" O.D. Split Spoon Sample
- ◫ Surface Seal
- ⊠ Annular Sealant
- ▤ Piezometer Screen
- ▨ Grout

## NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. The Porter penetration resistance in blows per 6 inches correlates approximately to the Standard Penetration Resistance in blows per foot.
5. Refer to KEY for explanation of 'Soil Log' symbols.
6. USC letter symbol based on visual classification.

Pemberton Wall Evaluation
Mercer Island, Washington

# LOG OF HAND BORING HB-3

July 1997 W-7882-01

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**FIG. A-4**