GEOTECHNICAL ENGINEERING REPORT

Proposed Remodel
3424 76th Place Southeast
Mercer Island, WA 98040

Project No. 24-472
Rev. July, 2025

Prepared for:

Chris and Harmony Long

PanGEO INCORPORATED

Geotechnical & Earthquake Engineering Consultants

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July 7, 2025
File No. 24-472

Chris and Harmony Long
3424 76th Pl SE
Mercer Island, WA 98040

Subject: **Geotechnical Engineering Report – Revision 1**
Proposed Remodel
3424 76th Place Southeast, Mercer Island, WA 98040

Dear Chris and Harmony,

Please find attached our geotechnical engineering report for the proposed remodel project at the subject site in Mercer Island, Washington. This report documents the subsurface conditions at the site and presents our geotechnical engineering recommendations for the proposed improvements to the existing house foundation.

In summary, the test borings advanced at the property generally encountered up to about 5 to 6 feet of loose fill, overlying loose/stiff to medium stiff silt and silty sand (colluvium/mass wastage deposits) to about 23 feet deep, overlying stiff to very stiff silt (recessional lacustrine deposits).

In our opinion, the proposed additions may be supported by driven small diameter pin piles.

We appreciate the opportunity to be of service. Please call if there are any questions.

Sincerely,

Bryce C. Townsend, P.E.
Senior Geotechnical Engineer
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GEOTECHNICAL ENGINEERING REPORT
PROPOSED REMODEL
3424 76TH PLACE SOUTHEAST
MERCER ISLAND, WASHINGTON

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 remodel to the existing residence at the subject site, in Mercer Island, Washington. This study was performed in general accordance with our mutually agreed scope of services outlined in our proposal dated November 18, 2024, which was subsequently approved by you on November 22, 2024. Our scope of services included reviewing readily available geologic and geotechnical data, drilling two test borings, conducting a site reconnaissance, performing engineering analysis, and developing the conclusions and recommendations presented in this report.

2.0 SITE AND PROJECT DESCRIPTION

The project site is located at 3424 76th Place Southeast on Mercer Island, Washington (see Figure 1 – Vicinity Map). The site is a rectangular property with an area of about 8,400 square feet, spanning about 120 feet from east to west and about 70 feet from north to south. The site is bordered to the north, east, and south by single-family residences, and to the west by 76th Place Southeast. The site is occupied by a one-story house with a basement that daylights to the east and is generally located in the middle of the property.

We understand that you plan to remodel the current house. The remodel includes the construction of a new addition at the basement level on the southeast corner of the house, a deck at the main level on the northeast corner of the house, and a new entryway with an extended roof on the west side of the house. We understand that the existing foundation consisting of spread footings are planned to be reused where there are no new buildings loads. New foundations will be needed for the new deck and additions. We understand that there are no new retaining or basement walls planned as part of the proposed remodel.

The site is located on an east-facing slope. A site-specific topographic survey was not available at the time of this report. However, based on our review of the Mercer Island GIS Map elevation contours, the site has a vertical relief of about 16 to 18 feet with slopes ranging from 15 to 25 percent. There are level yards immediately east and west of the existing house with slopes generally located along the north and south sides of the house (descending from first floor to daylit basement level) and on the east side of the backyard. Site conditions at the time of our site visits are shown on Plates 1 and 2 below.

Plate 1. *View of the front yard of subject property near the proposed new entryway, looking northeast from the driveway.*

Plate 2*. View of the east side of the house near the proposed basement level additions, looking south.*

Based on our review of the Mercer Island GIS Map, the site has three mapped geologic hazards on the property: potential landslide, erosion, and seismic hazard. As a result, the City has requested a Critical Area Study to address the mapped geologic hazards. Considerations regarding the mapped geologic hazards are presented in <u>Section 5.0</u>.

The conclusions and recommendations in this report are based on our understanding of the proposed project, which is in turn based on the project information provided. If the above project description is incorrect, or the project information changes, we should be consulted to review the recommendations contained in this study and make modifications, if needed. In any case, PanGEO should be retained to provide a review of the final design to confirm that our geotechnical recommendations have been correctly interpreted and adequately implemented in the construction documents.

3.0 SUBSURFACE EXPLORATIONS

Two test borings (PG-1 and PG-2) were drilled at the project site on December 6, 2024. The test borings were advanced to about 30½ to 31½ feet below the existing ground surface. The approximate boring locations are shown on the attached Figure 2 – Site and Exploration Plan.

The drill rigs were equipped with 5-inch outside diameter hollow stem augers. Soil samples were obtained from the borings in general at 2½- and 5-foot depth intervals using Standard Penetration Test (SPT) sampling methods in general accordance with ASTM D1586, *Standard Test Method for Penetration Test and Split Barrel Sampling of Soils*, 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 below the tip of the augers using a 140-pound weight falling a distance of 30 inches using an automated hammer. 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.

An engineer from PanGEO was present throughout the field exploration program to observe the drilling, assist in sampling, and to document the soil samples obtained from the borings. The soil samples retrieved from the borings were described using the system outlined on Figure A-1 of Appendix A and the summary boring logs are included as Figures A-2 and A-3.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Site Geology

Based on our review of *The Geologic Map of Mercer Island* (Troost and Wisher, 2006), the subject property is underlain by Pre-Olympia fine-grained glacial deposits (Map Unit Qpogf) with recessional lacustrine deposits (Map Unit Qvrl) mapped immediately east of the site. Pre-Olympia fine-grained glacial deposits are described by Troost and Wisher as hard, silt and clay deposited by proglacial lakes in front of the continental ice sheet. Recessional lacustrine deposits are described as very soft to stiff, laminated silt and clay with localized sand, peat and other organics deposited by slow-glowing water and lakes. The geologic map also indicates possible mass wastage, or landslide debris, at the project site originating from steep slopes west of the site.

Pre-Olympia fine-grained glacial deposits have been glacially overridden and thus typically exhibit low compressibility and high strength characteristics in their undisturbed state. Recessional lacustrine and mass wastage deposits have not been glacially overridden and typically have a softer consistency than the Pre-Olympia deposits.

4.2 Soil Conditions

In summary, the soils observed in our test borings appear consistent with the mapped geology. For simplicity, we have divided the encountered soils into three distinct soil units. A description of the soil units encountered in our test borings is presented below. A generalized subsurface profile of the project site is also shown in Figure 3. The approximate alignment of the subsurface profile across the site is shown in Figure 2. Detailed descriptions of the encountered soils in our test borings can be seen in our boring logs included in Appendix A.

> ***Fill:*** In both PG-1 and PG-2, we encountered a surficial soil unit of very loose to loose, silty sand with gravel and organics. We interpret this soil unit as fill, which extended to about 5 to 6 feet deep in both test borings.
>
> ***Colluvium/Mass Wastage Deposits:*** Below the fill in both borings, we encountered about 6 to 10 feet of loose/medium stiff to stiff silty sand and sandy silt interbeds, overlying loose to medium dense, very silty sand. We interpret these soil units as colluvium/mass wastage deposits based on the disturbed and nonuniform consistency. The colluvium/mass wastage deposits extended to about 23 and 25 feet deep in borings PG-1 and PG-2, respectively.
>
> ***Recessional Lacustrine Deposits:*** Below the colluvium/mass wastage deposits in both borings, we encountered medium stiff to very stiff, massive, low plasticity silt with sand. We interpret this soil unit as the mapped recessional lacustrine deposits. The recessional lacustrine deposits extended to the bottom of both test borings.

Our subsurface descriptions are based on the conditions encountered at the specific locations at the time of our exploration. Soil conditions between our exploration locations may vary from those encountered. The nature and extent of variations between our exploratory locations may not become evident until construction. If variations do appear, PanGEO should be requested to reevaluate the recommendations in this report and to modify or verify them in writing prior to proceeding with earthwork and construction.

4.3 Groundwater Conditions

Perched groundwater was observed in PG-1 from about 25 to 29 feet deep. A continuous groundwater layer was not encountered in PG-2 but interbeds of moist and wet silty sand were observed from about 16 to 25 feet deep. Groundwater levels will vary depending on the season, local subsurface conditions, and other factors. Groundwater levels are normally highest during the winter and early spring (typically October through May).

5.0 GEOLOGIC HAZARDS EVALUATION

As part of our study, we conducted an assessment of potential geologic hazards within the subject site as defined in Mercer Island City Code (MICC) Chapter 19.07.160. The MICC specifies four different geologic hazards a: potential landslide, steep slope, seismic and erosion. Based on our review of the City of Mercer Island GIS Map, there are potential landslide, seismic, and erosion geologic hazard areas mapped on the project site. No other geologic hazards (i.e. steep slope) are mapped at the site. The City's criteria for those various hazard areas and our assessment of the hazard areas with respect to the planned project are provided in the following sections of this report.

5.1 Landslide Hazards

The site is not mapped by the City of Mercer Island as containing a steep slope (40% or greater); however, the site is mapped as a landslide hazard area due to the mapped geology and proximity to historic landslide scarps west of the property.

During our site reconnaissance, we did not observe evidence of past slope instability or ground movement at the site. The west side of the property adjacent to 76th Place Southeast and the backyard east of the basement level are practically flat. About 25 feet from the west side of the existing house, the grade descends about 6 feet at an approximate 20 to 25 percent slope to the adjacent east property. Based on our review of the elevation contours shown on the Mercer Island GIS Map, the backyard for the adjacent east property is practically level.

We understand that the proposed developments on the east side of the house consist of a deck and addition that extend about 14 feet from the existing east perimeter wall, about 10 feet from the edge of the east slope.

Based on our observations, the site appears globally stable and the vertical relief on the east side of the property is minor relative to the overall site. In our opinion, the proposed developments should not adversely affect the overall stability of the site or adjacent properties, provided our recommendations contained in this report are properly incorporated into the project design and construction.

We also performed a quantitative global stability analysis of the site to supplement our visual assessment of the slope stability. Details of our analysis and the results are discussed below:

5.1.1 Slope Stability Analysis

We performed our slope stability analysis using the program SLIDE2 (Slide) published by Rocscience Inc. Slide is a two-dimensional limit equilibrium slope stability analysis program. Our analysis used the Spencer Method to determine potential failure planes.

The analysis was based on the soil profile shown on Figure 3. The soil profile was oriented to be generally perpendicular to the slope and most critical to the slope stability. Our analysis was performed for the post-construction condition for both the static and seismic cases.

5.1.2 Soil Parameters

A summary of the input soil parameters is provided in Table 1 below. The soil strength parameters estimated from our back calculations were consistent with the general estimates provided in USGS Open-File Report 2006-1139 (Laprade et al., 2006) for similar soil conditions, the SPT-N values, and our own judgement and experience with similar soils. Laprade (2006) notes that colluvium/mass wastage deposits can have cohesion values ranging from 400 to 500 psf and a friction angle of 30 degrees. We utilized a cohesion of 200 psf and a friction angle of 24 degrees for conservatism.

Groundwater was modeled at a constant level across the site at an approximate elevation of 100 feet (roughly 16 below grade on the east side of the existing house).

Due to the presence of potentially liquefiable soils within the colluvium/mass wastage deposits (see Section 5.2 for additional discussions regarding liquefaction), we calculated the residual strength for the post-liquefaction condition using the method recommended by Kramer (2007). Using Kramer's Method, we calculated a post-liquefaction undrained shear strength of about 194 psf and a friction angle of zero. These values were only used for colluvium/mass wastage deposits located below the modeled groundwater level.

We also used a surcharge pressure of 200 psf to model the existing house and the proposed additions.

Table 1 – SLIDE Soil Parameters

Soil Type	Display Color	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Fill	Yellow	120	30	0
Colluvium/Mass Wastage	Green	120	24	200
Colluvium – Post-Liquefaction	Purple	120	0	194
Recession Lacustrine Deposits	Orange	120	24	300

5.1.3 PGA for Seismic Slope Stability Analysis

Seismic design parameters for the site were developed in conformance with the 2018 International Building Code (IBC), which specifies a design earthquake having a 2 percent probability of occurrence in 50 years (return interval of 2,475 years), per Section 6.1 below. Per the methodology described in Chapter 20 of ASCE 7-16, a site modified peak ground acceleration (PGA_M) of about 0.66g was obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2008 data) for the project latitude and longitude, based on Site Class E. The lateral seismic coefficient is then taken as one half of the PGA_M. As such, we used a horizontal seismic coefficient 0.33g for our analysis.

5.1.4 Analysis Results

Based on the results from our analysis, we calculated minimum factors of safety for the existing site condition of 1.85 for the static case (see Plate 3 on the following page) and 1.17 for the seismic case (see Plate 4 on the following page). Additionally, for the post-liquefaction condition (i.e. immediately after earthquake while soil is liquefied), we calculated a minimum factor of safety of 1.38 (see Plate 5).

While the Mercer Island Municipal Code does not specify required factors of safety, the industry standard for minimum factors of safety for slope stability analysis are generally 1.5 for the static case and 1.1 for the seismic and post-liquefaction cases. Based on the results from our analysis, the site meets the minimum factors of safety for the static, seismic, and post-liquefaction cases. In our opinion, the proposed developments should not adversely affect the overall stability of the site or adjacent properties, provided our recommendations contained in this report are properly incorporated into the project design and construction.

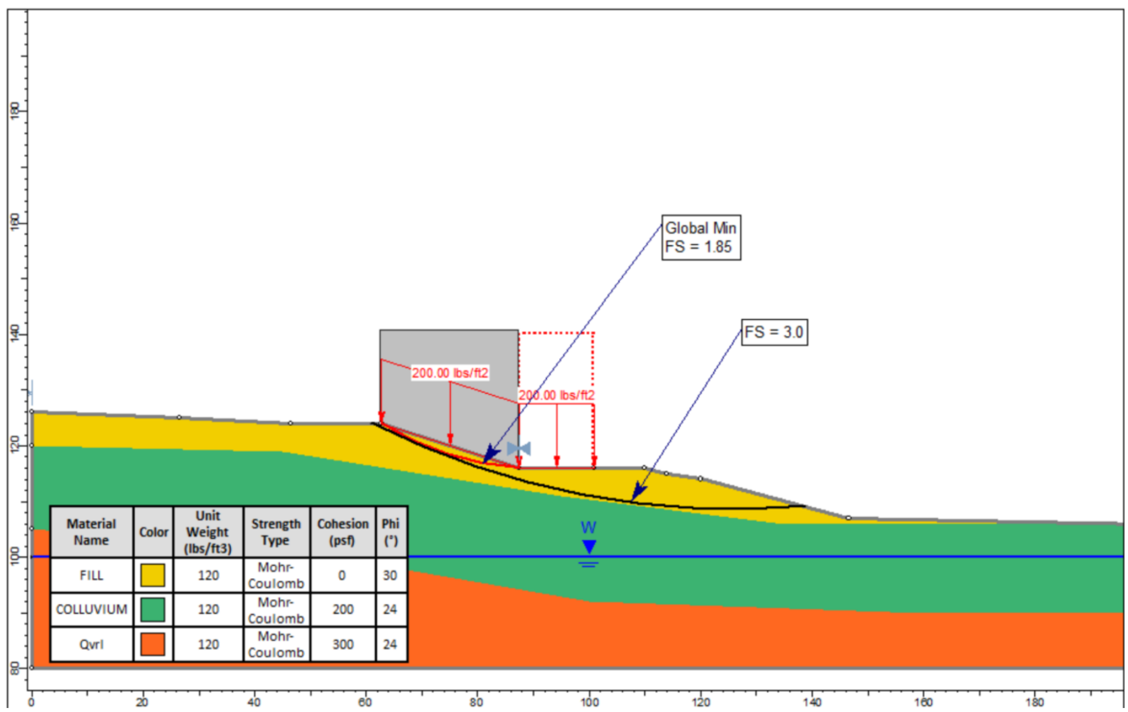


Plate 3. *Static analysis with global minimum FS of about 1.85 below the existing house. FS of 3.0 from the existing house to the bottom of the backyard slope.*

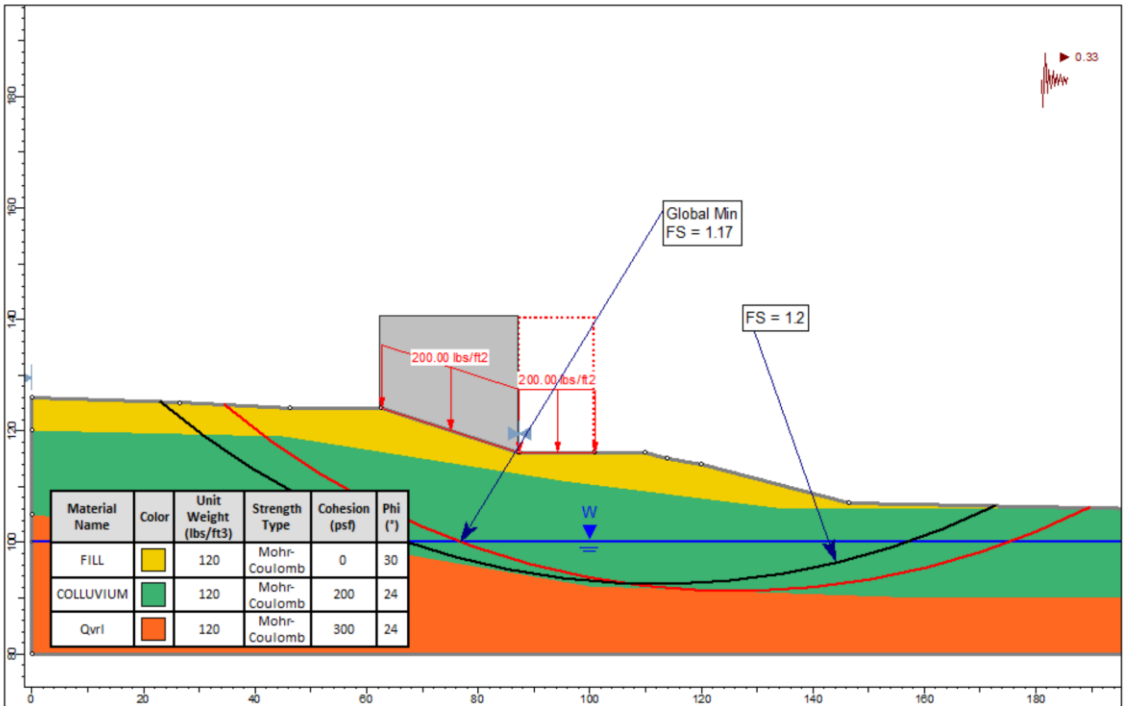


Plate 4. *Seismic analysis with global minimum FS of about 1.17 along slope.*

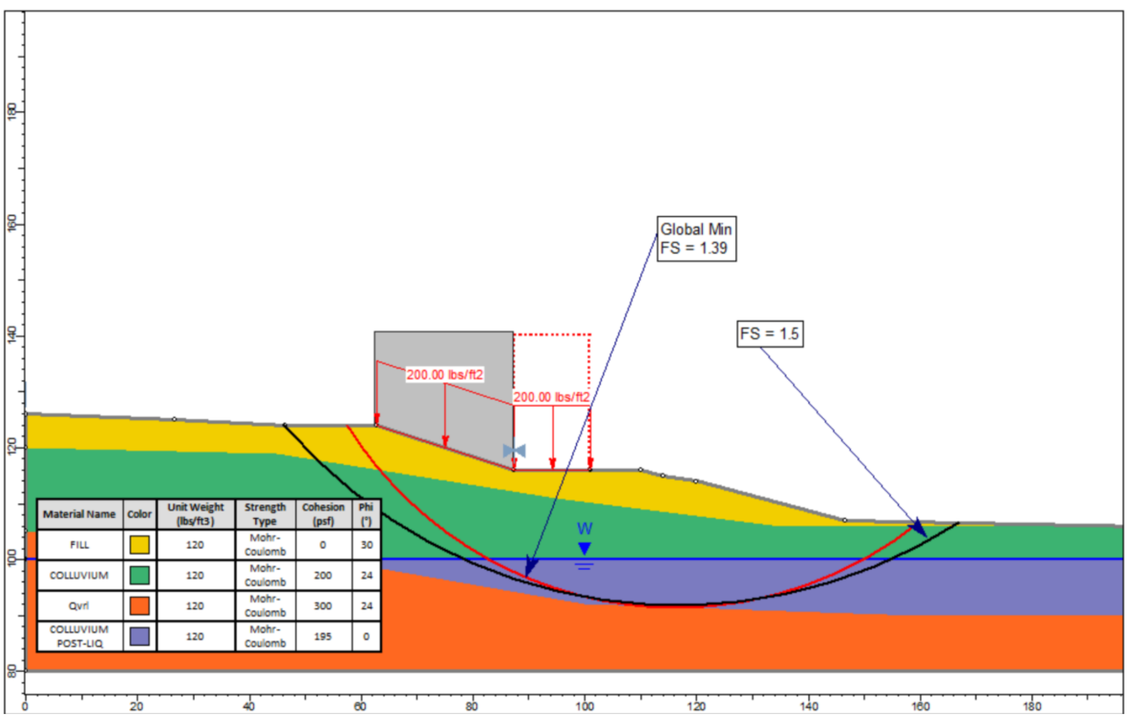


Plate 5. *Post-liquefaction analysis with global minimum FS of about 1.39 along slope.*

5.2 SEISMIC HAZARDS

Based on review of the City of Mercer Island Seismic Hazard Map, the property is mapped as having soil liquefaction potential.

Liquefaction is a process that can occur when soil loses shear strength for short periods of time during a seismic event. Ground shaking of sufficient strength and duration can result 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, with a predominately silt and sand grain size, must be loose to medium dense, and be below the groundwater table.

5.2.1 Liquefaction Induced Settlement

We evaluated the liquefaction potential for boring PG-2 which encountered interbeds of wet silty sand from 16 to 25 feet deep. We did not analyze test boring PG-1 which encountered one sample of wet silt at about 25 feet deep, which is not considered a risk to liquefaction due to the high fines content. The analyses were conducted using the computer liquefaction assessment software program LiqSVs 2.0.2.1, developed by Geologismiki. LiqSVs is a one-dimensional equivalent-linear site response analysis program that utilizes various methods to estimate liquefaction potential and liquefaction induced settlements. The method proposed by Idriss and Boulanger (2014) was used to estimate liquefaction susceptibility and liquefaction-induced settlements.

An earthquake with a magnitude of 7.0 and a site modified peak ground acceleration (PGA_M) of 0.66g was used in our analysis, based on expected ground motion at the project site that has a 2 percent probability of exceedance in a 50-year period (return interval of about 2,475 years), which is consistent with ASCE 7-16. The PGA_M was based on the Maximum Considered Earthquake (MCE) geometric mean peak ground acceleration adjusted for Seismic Site Class E (see Section 6.1).

Our analysis indicates a high risk of soil liquefaction below the groundwater table within the mass wastage deposits during the IBC level seismic event. The lacustrine deposits are not considered susceptible to liquefaction due to its high fines content and plasticity. Using the Idriss and Boulanger Method (2014), the free field ground surface settlement is estimated to be about 2.5 inches of vertical settlement. To mitigate the risk for seismic induced settlement, the proposed developments may be supported by pin piles per our recommendations in Section 6.2.

Plots of the calculated settlement with depth are included in Appendix B.

5.2.2 Lateral Spreading Evaluation

As summarized in Section 5.1 above, we performed a slope stability analysis to assess the risk for lateral spreading due to liquefaction. We calculated a global minimum factor of safety of 1.39 for the post-liquefaction condition, indicating a relatively stable slope condition in the event of liquefaction. As such, it is our opinion that the risk for lateral spreading is low for the site and that additional mitigation measures to address lateral spreading are not needed for this site.

5.3 Erosion Hazards

Based on our review of the Mercer Island GIS Map, portions of the site are mapped as erosion hazard areas. According to the USDA Soil Survey Map, the site is mapped as Kitsap Silt Loam, 15 to 30 percent (Map Unit KpD). Based on the USDA Soil Survey data and soils observed in our explorations, the site soils are anticipated to exhibit moderate erosion potential when disturbed and left unprotected. In our opinion, the erosion hazards at the site can be effectively mitigated with the best management practice during construction and with properly designed and implemented landscaping for permanent erosion control. During construction, the temporary erosion hazard can also be effectively managed with an appropriate erosion and sediment control plan, including but not limited to installing a silt fence at the construction perimeter, placing quarry spalls or hay bales at the disturbed and traffic areas, covering stockpiled soil or cut slopes with plastic sheets, constructing a temporary drainage pond to control surface runoff and sediment trap, placing rocks at the construction entrance, etc.

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

Additional recommendations and considerations for surface drainage and wet weather construction are provided in Section 8.4 and Section 8.5, respectively.

Provided that proper erosion control measures are implemented during construction, in our opinion the risk of off-site soil transport during and after construction of the project is minimal.

5.4 Conclusions and Alteration of Geologic Hazard Areas

Based on the results from our slope stability and liquefaction analyses summarized above, it is our opinion that the proposed developments meets the following standards outlined in MICC 19.07.160.B(2) for development within landslide and seismic geologic hazard areas:

a) Will not adversely impact other critical areas;

b) Will not adversely impact the subject property or adjacent properties;

c) Will mitigate impacts to the geologically hazardous area consistent with best available science to the maximum extent reasonably possible such that the site is determined to be safe; and

d) Includes the landscaping of all disturbed areas outside of building footprints and installation of hardscape prior to final inspection.

6.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

6.1 Seismic Design Considerations

The seismic design of the project may be accomplished using the 2018 or 2021 International Building Code (IBC), which specifies a design earthquake having a 2 percent 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 7-16. According to Chapter 20 of ASCE 7-16, based on the results from our test borings, the site soil should be classified as Site Class E for soft soil.

6.2 Foundations – Small Diameter Pin Piles

Based on the results from our test borings, a deep foundation system consisting of driven, small diameter pipe piles (often referred to as pin piles) may be utilized to support new foundations for the proposed development. Pin piles provide a high level of foundation performance by transferring the loads from the new development into deeper bearing soils and eliminates the need for over-excavations.

Two- to 4-inch diameter pipes are typically utilized for residential projects. Two-inch pin piles are typically installed using portable, handheld equipment and are suited for areas where limited site access exists, or in low-headroom areas (i.e., under roofs or inside basements). Three-inch and 4-inch pin piles are typically installed using small to large hammers (600 to 2,000 lbs) mounted on small to medium-sized excavators.

6.2.1 Pin Pile Capacity

The number of piles required depends on the magnitude of the design load and the pile size. Table 2 on the following page shows our recommended allowable axial capacities in compression for pin piles with an approximate factor of safety of at least 2.0.

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

Table 2 – Pin Pile Capacities

Pile Diameter (in)	Allowable Axial Capacity in Compression (kips)
2	4
3	12
4	20

6.2.2 Pin Pile Specifications

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

1. 2-inch diameter piles should consist of Schedule-80, ASTM A-53 Grade "A" pipe.
2. 3-inch and 4-inch diameter piles should consist of Schedule-40, ASTM A-53 Grade "A" pipe.
3. 2-inch piles shall be driven to refusal with a minimum 90-lb jackhammer. Refusal is defined as no more than 1 inch of penetration for 1 minute of continuous driving.
4. 3-inch piles shall be driven to refusal with a minimum 600-lb hydraulic hammer. We recommend the following refusal criteria based on the size of hammer utilized:

Table 3 – Three-Inch Pile Refusal Criteria

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

The driving criteria recommended in the table above will be verified by a static load test program (see discussion in Item 7).

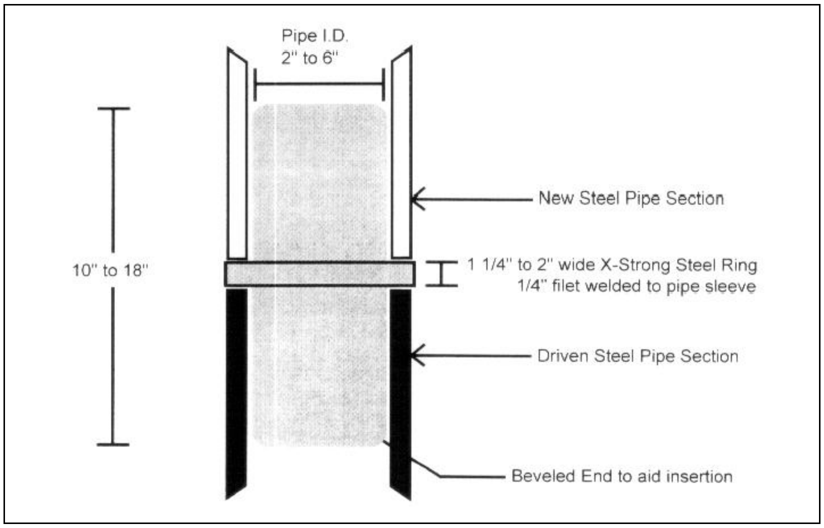
5. 4-inch piles shall be driven to refusal with a minimum 850-lb hydraulic hammer. We recommend the following refusal criteria based on the size of hammer utilized:

Table 4 – Four-Inch Pile Refusal Criteria

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

The driving criteria recommended in the table above will be verified by a static load test program (see discussion in Item 7).

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



7. At least 3 percent (but no more than 5) of the 3-inch and 4-inch pin piles should be load tested. All load tests shall be performed in accordance with the procedure outlined in ASTM D1143. The maximum test load shall be 2 times the design load. The objective of the testing program is to verify the adequacy of the driving criteria, and the efficiency of the hammer used for the project.
8. The geotechnical engineer of record or his/her representative shall provide full-time observation of pile installation and testing.

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

6.2.3 Grade Beam Lateral Resistance

Lateral capacity of vertical pin piles should be ignored in design calculations. Some resistance to lateral loads may be accomplished by battering the piles to a maximum slope of 1(H):4(V) or less. In addition, lateral forces from wind or seismic loading may be resisted by tying the grade beams and pile caps to shoring walls.

Passive soil resistance values for embedded pile caps and grade beams may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf). This value includes a geotechnical factor of safety of at least 1.5 assuming that a properly compacted structural fill will be placed adjacent to the sides of the pile caps and grade beams, and level ground surface. For the seismic condition, the recommended passive pressure may be increased by one third.

Friction below pile caps and grade beams should be ignored.

6.2.4 Estimated Pile Length

The required pile length in order to develop the recommended pile capacity may vary depending on the actual driving conditions encountered. Based on the results from our subsurface explorations, we encountered bearing soils about 25 For planning and cost estimating purposes, we estimate an average pile length of about 40 feet will be needed, assuming about 10 feet of embedment into the stiff lacustrine deposits. We recommend a minimum pile length of 25 feet be specified on the project plans.

6.2.5 Underpinning Exiting House Foundation

We do not recommend additional loads from the proposed additions be added to the existing spread and strip footings for the house. The existing foundations are bearing on colluvium/mass wastage deposits and additional loading on the existing conventional footings could induce settlement. If additional loads must be applied to existing conventional footings, we recommend that the foundations be underpinned with pin piles per our recommendations above. Alternatively, for a higher level of overall performance for the overall house, all existing foundations may be underpinned and eliminate the risk of long-term settlement from static or seismic loading.

6.2.6 Obstructions

Obstructions may be encountered within the upper fill and colluvium/mass wastage deposits. Where possible, the obstructions should be removed to facilitate pile driving. If obstructions cannot be removed, the structural engineer of record should be notified to revise the pile layout to accommodate moving the piles.

6.3 CONCRETE SLAB ON GRADE

Slab on Grade Preparation – A conventual slab-on-grade may be used for the floor of the new addition. However, due to the existing loose soil, some settlement of the floor slab, and associated distress, may occur if a slab-on-grade is constructed on the existing loose fill. The risk of the settlement will depend on how much of the existing loose soil is excavated out from under the footprint of the structure during construction. To reduce the potential of slab settlement and distress, where loose fill is present below the proposed slab, we recommend removing a minimum of 1 foot of existing fill below the slab, heavily recompacting the exposed soils to a dense and unyielding condition, and placing 1 foot of properly compacted structural fill to create a firm surface for the slab. For the subgrade improvement described above, and for slab areas bearing on the native medium dense soils, the floor slab design may be accomplished using a modulus of subgrade reaction of 125 pci.

Capillary Break - We recommend that the slabs be constructed on a minimum 4-inch-thick capillary break. The capillary break should consist of free-draining, clean crushed rock or well-graded gravel compacted to a firm and unyielding condition. The capillary break material should 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. We also recommend that a 10-mil polyethylene vapor barrier be placed below the slab.

7.0 STATEMENT OF RISK

As outlined above, the site is mapped as a geologic hazard area. Per Mercer Island City Code Section 19.07.160.B(3), development within geologic hazard areas and critical slopes may occur if the geotechnical engineer provides a statement of risk with supporting documentation indicating that one of the following conditions can be met:

a. An evaluation of site-specific subsurface conditions demonstrates that the proposed development is not located in a landslide hazard area or seismic hazard area;

b. The landslide hazard area or seismic hazard area will be modified or the development has been designed so that the risk to the site and adjacent property is eliminated or mitigated such that the site is determined to be safe;

c. Construction practices are proposed for the alteration that would render the development as safe as if it were not located in a geologically hazardous area and do not adversely impact adjacent properties; or

d. The development is so minor as not to pose a threat to the public health, safety and welfare.

Based on the results from our slope stability analysis, the site is globally stable and the proposed developments should not adversely affect the site and the neighboring properties. Additionally, the proposed developments will be supported by pin piles to mitigate seismic hazard for liquefaction induced settlement.

As such, it is our opinion that Criteria B applies to the proposed development, provided our recommendations contained within this report are adequately incorporated into the projects plans.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 Temporary Excavations

We anticipate that excavations up to a maximum of about 4 feet deep may be needed for the construction of the new foundations. The excavation is anticipated to encounter very loose sandy soils.

Temporary excavations greater than 4 feet deep should be properly sloped or shored, however, vertical excavations 4 feet deep or less will likely not remain stable, and will slough or collapse, due to the very loose nature of the sandy soils anticipated at the site. 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 less than 4 feet for the footing over-excavation, as well as for other site features, be sloped no steeper than 1H:1V (horizontal:vertical). Cuts deeper than 4 feet, if needed, will likely need to be cut back at a shallower angle of 2H:1V, to maintain stability. 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.

8.2 Material Reuse

In the context of this report, structural fill is defined as compacted fill placed under footings, concrete stairs, landings, slabs, or other load-bearing areas. In our opinion, the on-site soils are not suitable to be reused as structural fill due to the relatively high fines content. Suitable materials for use as structural fill are described in Section 8.3 below.

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.

8.3 Structural Fill Placement and Compaction

For planning purposes, structural fill should consist of imported, well-graded, granular material such as Seattle Type 17 Mineral Aggregate (*Seattle Standards and Specifications*, 2024, Section 9-03.14), WSDOT Gravel Borrow (*WSDOT Standards and Specifications,* 2024, Section 9-03.14(1)), or an approved equivalent. Based on the absence of groundwater anticipated within the proposed excavation, recycled crushed concrete (1¼-inch with fines) may also be used as structural fill. However, the use of recycled crushed concrete should be approved by PanGEO prior to installation.

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.

8.4 Surface Drainage Considerations

Adequate drainage provisions are imperative to improve the performance of the proposed developments and adjacent structures. We recommend both short-term and long-term drainage measures be incorporated into the project design and construction. 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 or to prevent runoff from the construction area leaving the immediate work site. Collected water should be directed to a positive and permanent discharge system.

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 proposed developments and existing structures and into the 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. Collected water from surface runoff should not drain into retaining wall drain systems.

8.5 Wet Weather Construction

It is our opinion that construction of the project can be accomplished during the wet season (October to April). However, performing earthwork activities during the wet season is anticipated to be costlier than during dry weather conditions. The following procedures are best management practices recommended for use in wet weather construction:

- All footing surfaces should be protected against inclement weather unless the footings can be poured immediately after the design subgrade is prepared. It is the contractor's responsibility to protect the footing subgrade from disturbance.
- Earthwork should be performed in small areas to minimize subgrade exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.
- During wet weather, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight based on the portion passing the 0.075-mm sieve. The fines should be non-plastic.
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water.
- Geotextile silt fences should be installed at strategic locations around the site to control erosion and the movement of soil.
- Excavation slopes and soils stockpiled on site should be covered with plastic sheeting.

9.0 ADDITIONAL SERVICES

To confirm that our recommendations are properly incorporated into the design and construction of the proposed project, 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, may also require geotechnical construction inspection services. PanGEO can provide a cost estimate for construction monitoring services after the design has been finalized.

10.0 UNCERTAINTY AND LIMITATIONS

We have prepared this report for Chris and Harmony Long and the project design team. Recommendations contained in this report are based on a site reconnaissance, a subsurface exploration program, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of services.

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

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

This report 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 of 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.

Bryce C. Townsend, P.E.
Senior Geotechnical Engineer

11.0 REFERENCES

ASTM D1557, 2012, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort,* www.astm.org.

ASTM D1586 / D1586-18, 2018, *Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*, www.astm.org.

City of Seattle, 2024, *Standard Specifications for Road, Bridges, and Municipal Construction*. International Code Council, 2018, *International Building Code (IBC), 2018.*

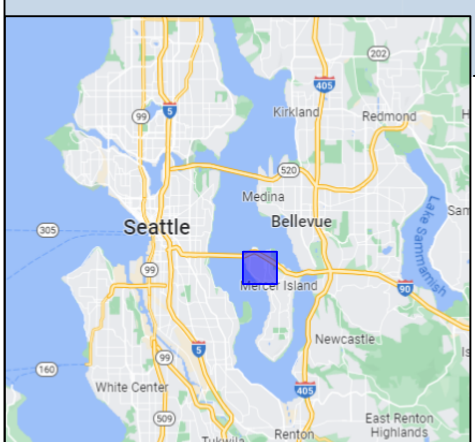
City of Mercer Island, 2025, Mercer Island City Code.

Kramer, S.L., 2007, *Evaluation of Liquefaction Hazards in Washington State,* WSDOT, Report WA-RD 668.1, 312 pp.

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

Washington Administration Code (WAC), 2019, *Part N – Excavation, Trenching, and Shoring.*

WSDOT, 2024, *Standard Specifications for Road, Bridges, and Municipal Construction, M41-10*.



Base Map: King County iMap

N

Not to Scale

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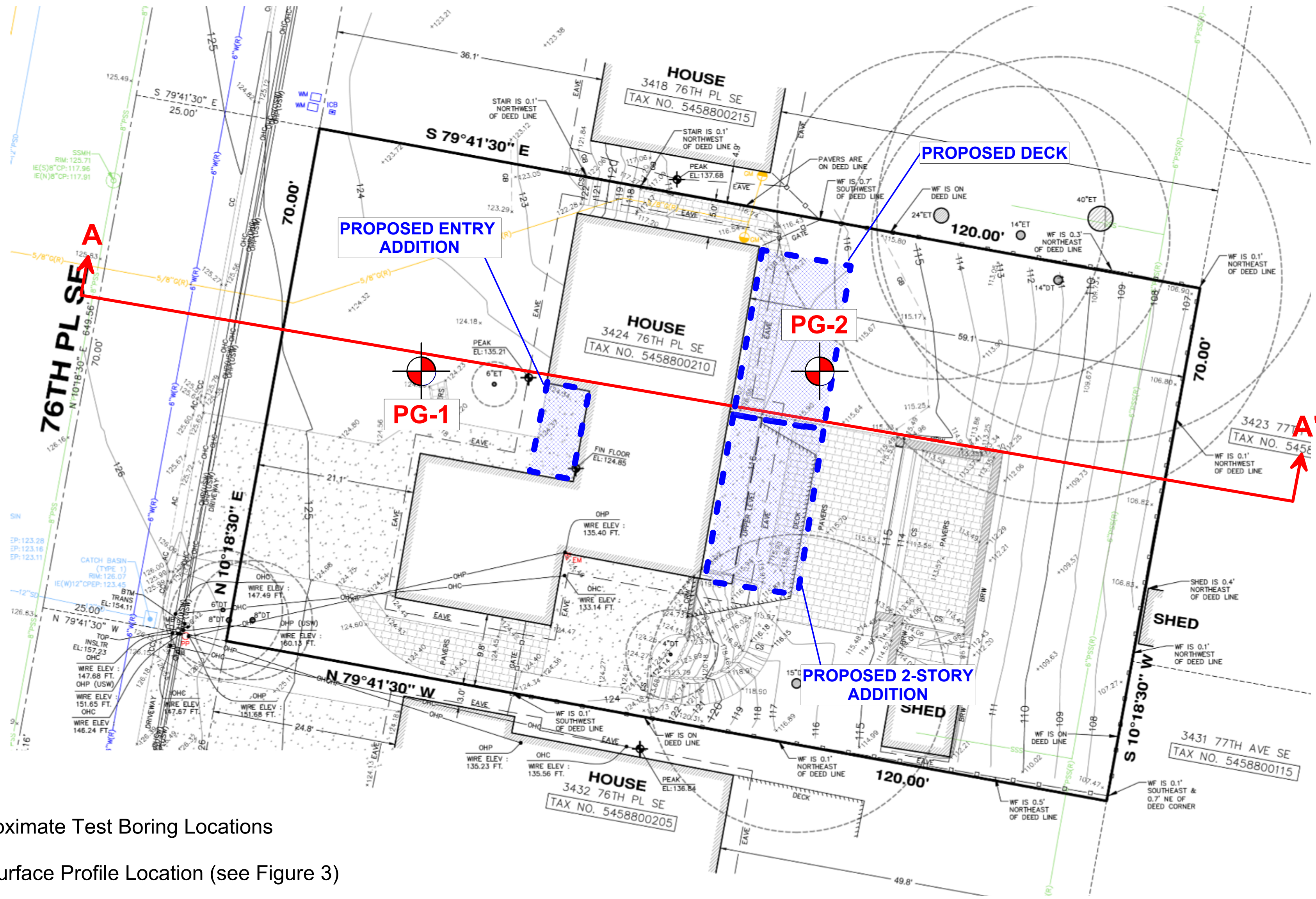
Proposed Remodel
3424 76th Place Southeast
Mercer Island, Washington

VICINITY MAP

Project No.	Figure No.
24-472	1

N

Approx. Scale
1 inch = 20 feet



Legend:

- Approximate Test Boring Locations
- Subsurface Profile Location (see Figure 3)
- Proposed Additions

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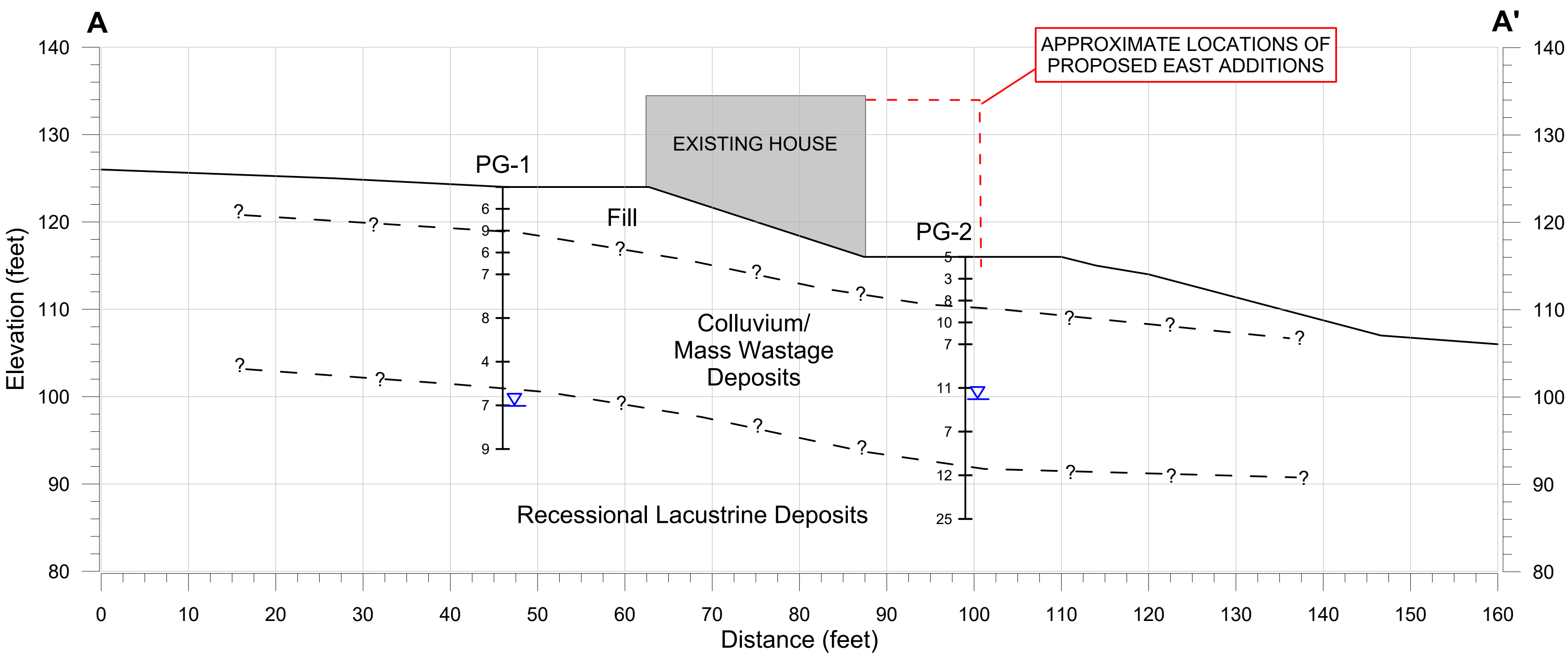
Proposed Remodel
3424 76th Place Southeast
Mercer Island, Washington

SITE AND EXPLORATION PLAN

Project No.	Figure No.
24-472	2



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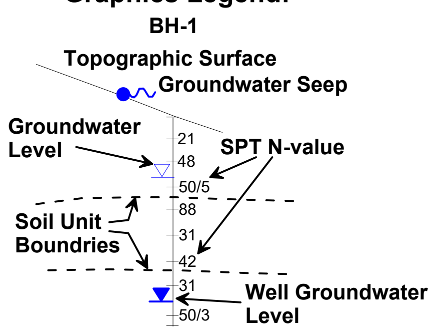

Notes:

1. Ground surface elevations approximated from elevation contours shown on project topographic survey (see Figure 2).
2. Transitions between soil units and in between borings are best estimates and may vary from the actual soil conditions.
3. See Figure 2 for Site Plan view of the site with approximate profile location.

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Proposed Remodel
3424 76th Place Southeast
Mercer Island, Washington

GENERALIZED SUBSURFACE PROFILE
A-A'

Project No. **24-472**

Figure No. **3**

APPENDIX A

SUMMARY BORING LOGS

RELATIVE DENSITY / CONSISTENCY

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

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Notes:

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

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

DESCRIPTIONS OF SOIL STRUCTURES

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

COMPONENT DEFINITIONS

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

TEST SYMBOLS

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

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

SYMBOLS

Sample/In Situ test types and intervals

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

MONITORING WELL

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

MOISTURE CONTENT

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

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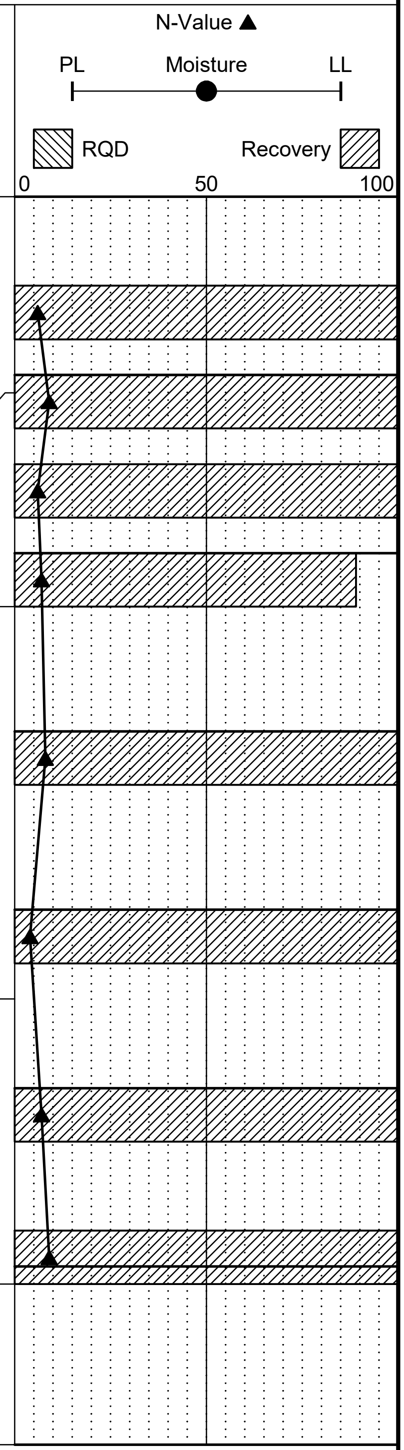
Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	Proposed Remodel	Surface Elevation:	~125 ft
Job Number:	24-472	Top of Casing Elev.:	N/A
Location:	3424 76th Place Southeast, Mercer Island, Washington	Drilling Method:	Portable Acker, hollow stem auger
Coordinates:	Northing: 47.57922, Easting: -122.23685	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						**FILL** Top 4 inches of grass landscaping; Loose, moist, gray-brown, silty SAND with gravel; numerous organic roots.
	S-1	X	1 1 5			
5.0	S-2	X	1 4 5			-- S2: top 6 inches of gray-brown, silty sand, with iron oxide staining. **COLLUVIUM/MASS WASTAGE DEPOSITS** Loose/medium stiff to stiff, moist, gray, silty SAND and sandy SILT; trace gravel and organics, fine sand, iron oxide staining disturbed consistency.
	S-3	X	2 2 4			-- S3: becomes moist to very moist.
10.0	S-4	X	1 2 5			-- S4: Bands of iron oxide staining, becomes silty sand in bottom of sample. **COLLUVIUM/MASS WASTAGE DEPOSITS** Loose, moist to very moist, gray-brown, very silty SAND; trace of gravel, fine sand, iron oxide staining present, disturbed consistency.
15.0	S-5	X	1 3 5			
20.0	S-6	X	1 1 3			-- Becomes very moist to wet. **RECESSIONAL LACUSTRINE DEPOSITS** Medium stiff, wet, gray, SILT with sand; very fine sand, low plasticity, massive.
25.0	S-7	X	1 2 5		▽	
30.0	S-8	X	2 2 7			-- Becomes stiff, moist Boring terminated at about 30.5 feet below the ground surface; Groundwater was not encountered during drilling; Perched groundwater was encountered from about 25 to 29 feet below the ground surface.
35.0						



Completion Depth:	30.5ft
Date Borehole Started:	12/6/24
Date Borehole Completed:	12/6/24
Logged By:	H. Wang
Drilling Company:	CN Drilling

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Elevation is approximated from the Mercer Island GIS elevation contours. Coordinates are approximated based on their relative location to known site features. This information is provided for relative information only and is not a substitution for field survey. **Datum: WGS84/NAVD88.**

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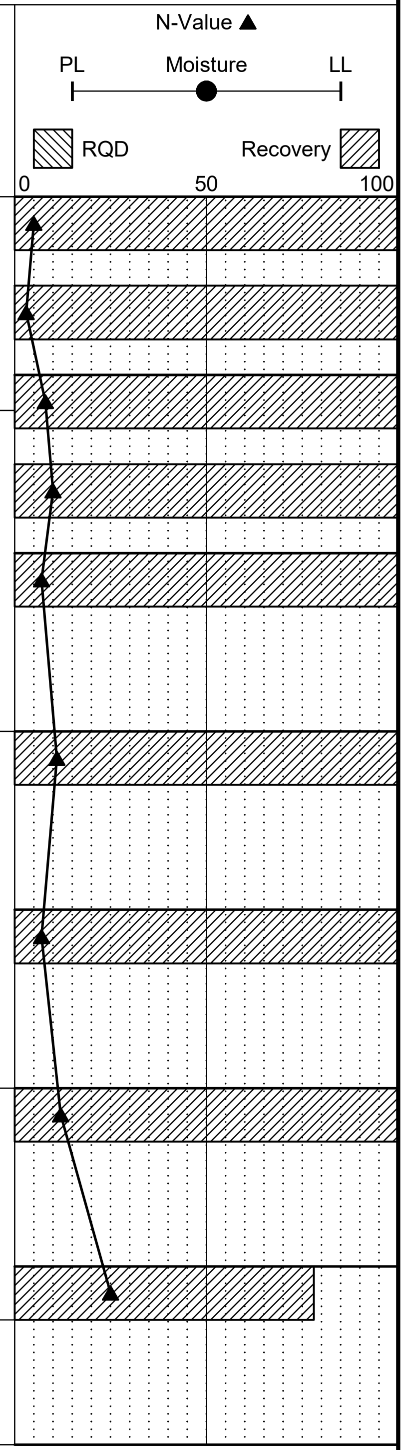
LOG OF TEST BORING PG-1

Figure A-2

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

Project:	Proposed Remodel	Surface Elevation:	~116 ft
Job Number:	24-472	Top of Casing Elev.:	N/A
Location:	3424 76th Place Southeast, Mercer Island, Washington	Drilling Method:	Portable Acker, hollow stem auger
Coordinates:	Northing: 47.57921, Easting: -122.2366	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0	S-1	X	4 2 3			**FILL** Top 4 inches of grass landscaping; Loose, moist, gray-brown, silty SAND with gravel; fine sand, trace organic roots.
	S-2	X	1 1 2			-- S2: Becomes very loose.
5.0	S-3	X	1 1 7			-- S3: Becomes loose.
						COLLUVIUM/MASS WASTAGE DEPOSITS Loose/medium stiff to stiff, moist, brown-gray, silty SAND and sandy SILT; trace of gravel, fine sand, iron oxide staining, disturbed consistency.
	S-4	X	1 4 6			
10.0	S-5	X	1 3 4			-- S5: becomes very moist.
15.0	S-6	X	1 4 7			**COLLUVIUM/MASS WASTAGE DEPOSITS** Medium dense, very moist to wet, gray, very silty SAND; very fine sand, disturbed consistency, interbeds of moist and wet silty sand from 16' to 25'.
20.0	S-7	X	0 2 5			-- Becomes loose.
25.0	S-8	X	2 4 8			**RECESSIONAL LACUSTRINE DEPOSITS** Stiff, very moist, blue-gray, SILT with sand; very fine sand, low plasticity, massive.
30.0	S-9	X	3 10 15			-- Becomes very stiff, moist.
						Boring terminated at about 31.5 feet below the ground surface; Groundwater was not encountered during drilling; continuous groundwater not encountered during drilling, interbeds of moist and wet silty sand observed from 16 to 25 feet deep.
35.0						



Completion Depth:	31.5ft	Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Elevation is approximated from the Mercer Island GIS elevation contours. Coordinates are approximated based on their relative location to known site features. This information is provided for relative information only and is not a substitution for field survey. **Datum: WGS84/NAVD88.**
Date Borehole Started:	12/6/24	
Date Borehole Completed:	12/6/24	
Logged By:	H. Wang	
Drilling Company:	CN Drilling	

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LOG OF TEST BORING PG-2

Figure A-3

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

APPENDIX B

LIQUEFACTION ANALYSIS

PanGEO INCORPORATED
Geotechnical Consultants
3213 Eastlake Ave E, Suite B
Seattle, WA 98102
www.pangeoinc.com

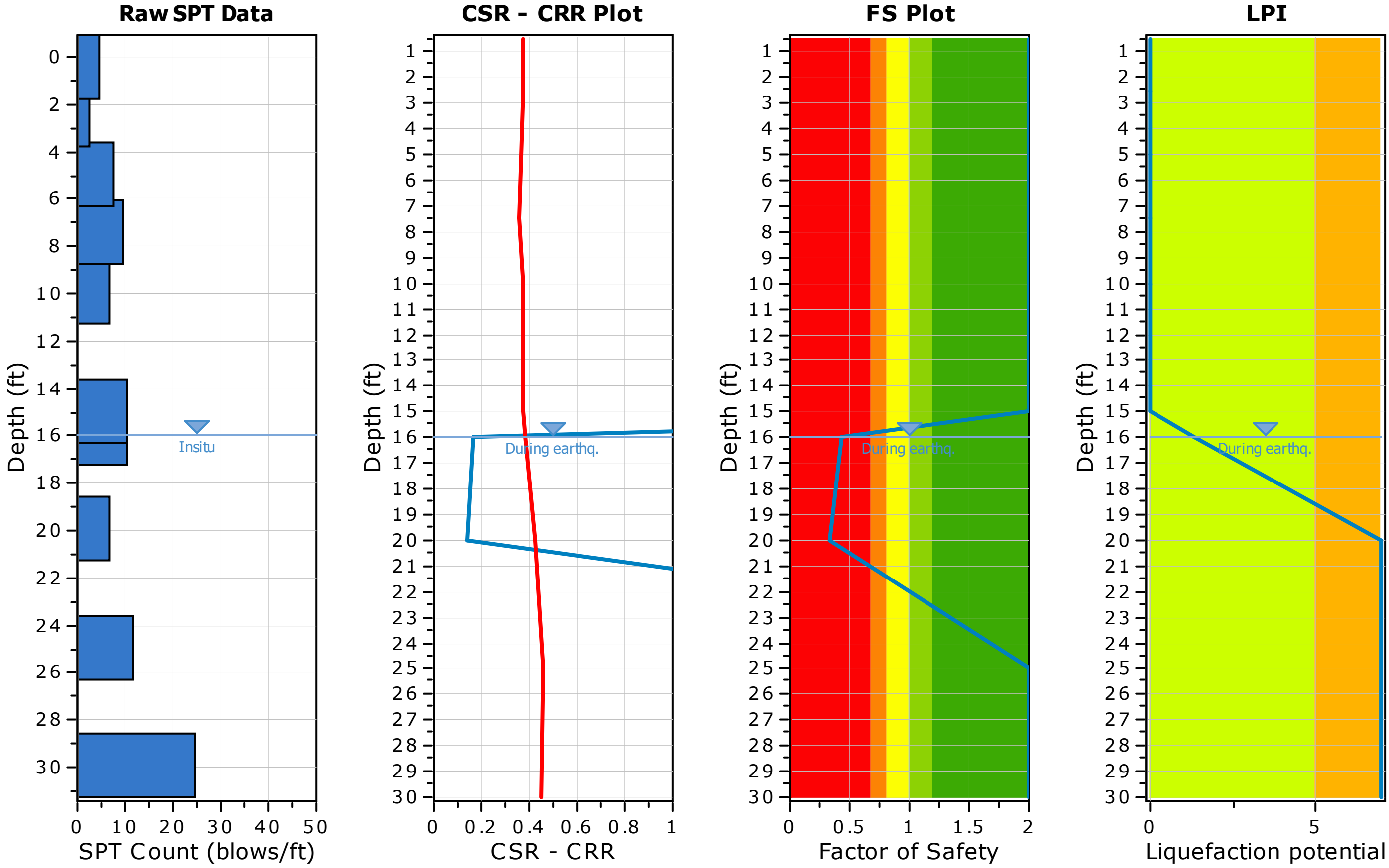
SPT BASED LIQUEFACTION ANALYSIS REPORT

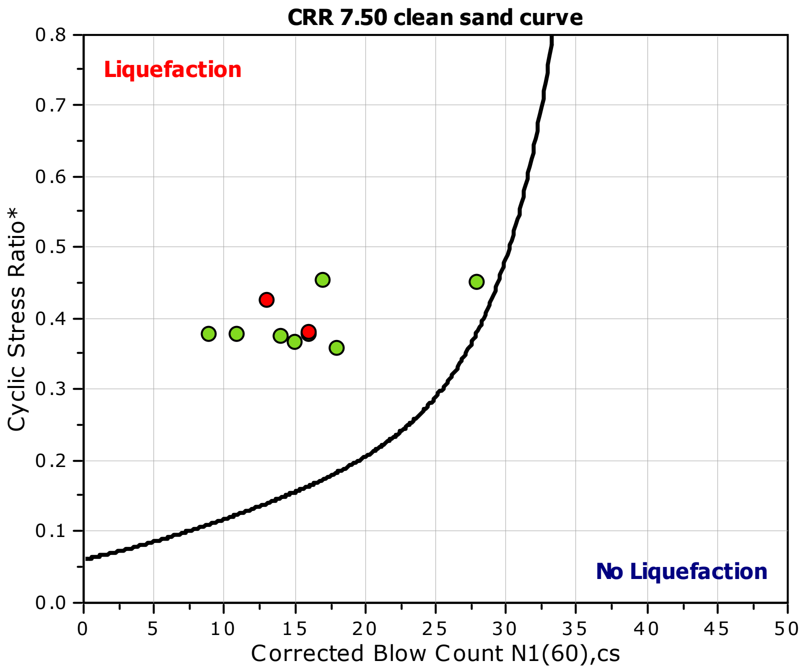
Project title : Long Residence **SPT Name: PG-2**

Location : 3424 76th Pl SE, Mercer Island, WA

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	16.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	16.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.00
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.66 g
Rod length:	3.28 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		

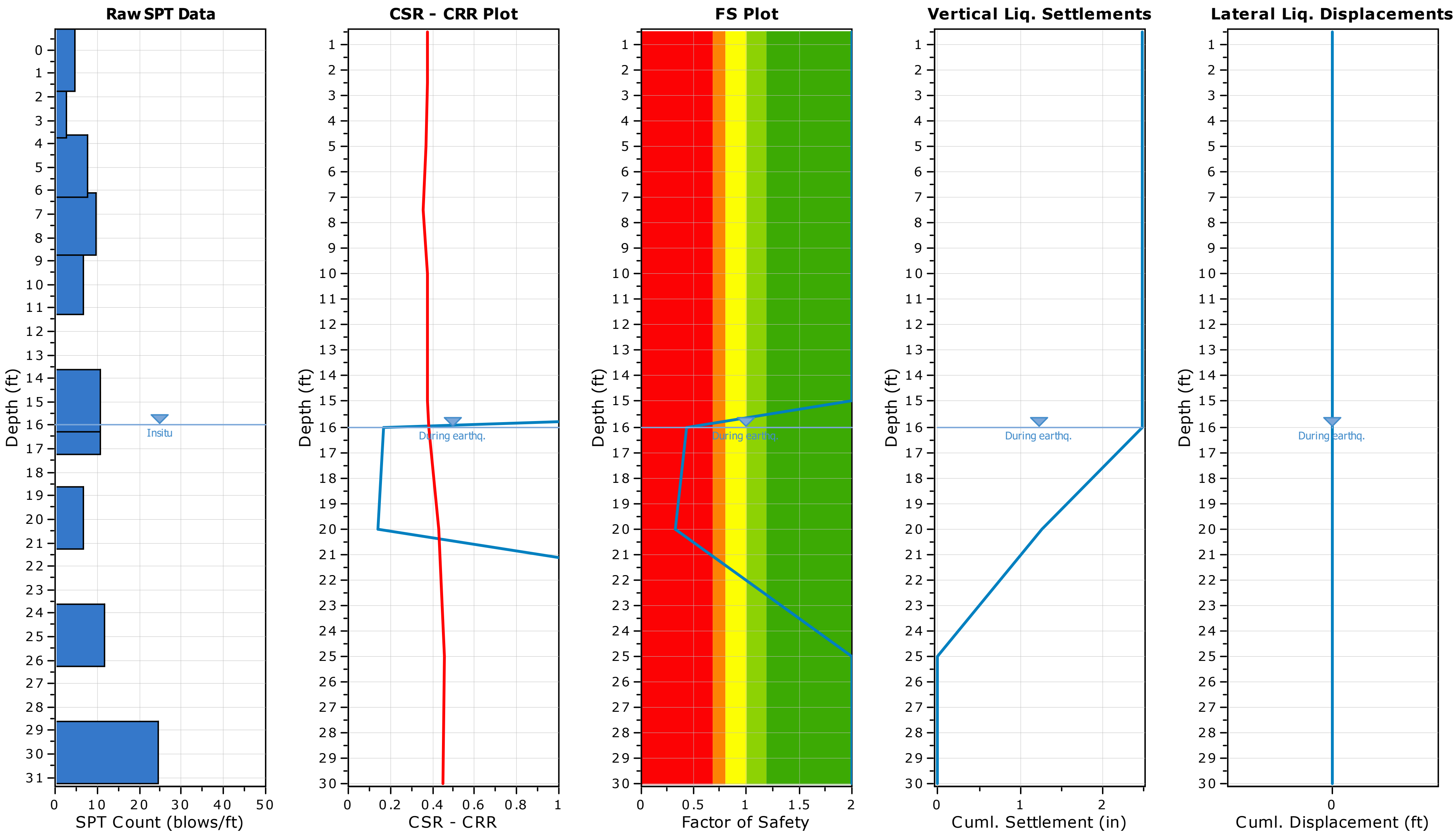




F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
0.50	5	30.00	120.94	2.00	Yes
2.50	3	30.00	120.94	2.50	Yes
5.00	8	30.00	120.94	2.50	Yes
7.50	10	50.00	120.94	2.50	Yes
10.00	7	50.00	120.94	5.00	Yes
15.00	11	45.00	120.94	5.00	Yes
16.00	11	45.00	120.94	5.00	Yes
20.00	7	45.00	120.94	5.00	Yes
25.00	12	75.00	120.94	5.00	No
30.00	25	75.00	120.94	5.00	No

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
0.50	5	120.94	0.03	0.00	0.03	0.59	1.70	1.00	1.00	0.75	1.00	6	30.00	5.36	11	4.000
2.50	3	120.94	0.15	0.00	0.15	0.63	1.70	1.00	1.00	0.75	1.00	4	30.00	5.36	9	4.000
5.00	8	120.94	0.30	0.00	0.30	0.54	1.70	1.00	1.00	0.75	1.00	10	30.00	5.36	15	4.000
7.50	10	120.94	0.45	0.00	0.45	0.51	1.55	1.00	1.00	0.80	1.00	12	50.00	5.61	18	4.000
10.00	7	120.94	0.60	0.00	0.60	0.56	1.37	1.00	1.00	0.85	1.00	8	50.00	5.61	14	4.000
15.00	11	120.94	0.91	0.00	0.91	0.54	1.09	1.00	1.00	0.85	1.00	10	45.00	5.61	16	4.000
16.00	11	120.94	0.97	0.00	0.97	0.54	1.05	1.00	1.00	0.85	1.00	10	45.00	5.61	16	0.165
20.00	7	120.94	1.21	0.12	1.08	0.59	0.99	1.00	1.00	0.95	1.00	7	45.00	5.61	13	0.140
25.00	12	120.94	1.51	0.28	1.23	0.53	0.92	1.00	1.00	0.95	1.00	11	75.00	5.56	17	4.000
30.00	25	120.94	1.81	0.44	1.38	0.42	0.90	1.00	1.00	1.00	1.00	22	75.00	5.56	28	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
0.50	120.94	0.03	0.00	0.03	1.01	1.00	0.431	1.21	11	1.04	0.416	1.10	0.378	2.000

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
2.50	120.94	0.15	0.00	0.15	1.00	1.00	0.429	1.17	9	1.03	0.416	1.10	0.378	2.000
5.00	120.94	0.30	0.00	0.30	0.99	1.00	0.426	1.32	15	1.06	0.403	1.10	0.366	2.000
7.50	120.94	0.45	0.00	0.45	0.98	1.00	0.422	1.42	18	1.07	0.393	1.10	0.357	2.000
10.00	120.94	0.60	0.00	0.60	0.97	1.00	0.418	1.29	14	1.05	0.398	1.06	0.375	2.000
15.00	120.94	0.91	0.00	0.91	0.95	1.00	0.409	1.35	16	1.06	0.385	1.02	0.378	2.000
16.00	120.94	0.97	0.00	0.97	0.95	1.00	0.407	1.35	16	1.06	0.383	1.01	0.379	0.434
20.00	120.94	1.21	0.12	1.08	0.93	1.00	0.445	1.26	13	1.05	0.425	1.00	0.426	0.329
25.00	120.94	1.51	0.28	1.23	0.90	1.00	0.477	1.38	17	1.07	0.447	0.98	0.455	2.000
30.00	120.94	1.81	0.44	1.38	0.88	1.00	0.496	1.88	28	1.16	0.429	0.95	0.451	2.000

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (tsf)
r_d:	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cyclic Stress Ratio
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$:	CSR adjusted for M=7.5
K_{sigma}:	Effective overburden stress factor
CSR^*:	CSR fully adjusted (user FS applied)***
FS:	Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
0.50	2.000	0.00	9.92	2.00	0.00
2.50	2.000	0.00	9.62	2.00	0.00
5.00	2.000	0.00	9.24	2.50	0.00
7.50	2.000	0.00	8.86	2.50	0.00
10.00	2.000	0.00	8.48	2.50	0.00
15.00	2.000	0.00	7.71	5.00	0.00
16.00	0.434	0.57	7.56	1.00	1.30
20.00	0.329	0.67	6.95	4.00	5.69
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00

Overall potential I_L : 7.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} weight factor	ε_{Nc} (%)	Δh (ft)	ΔS (in)
0.50	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.99	0.00	2.00	0.000
2.50	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.96	0.00	2.50	0.000
5.00	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.92	0.00	2.50	0.000
7.50	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.88	0.00	2.50	0.000

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} weight factor	ε_{Nc} (%)	Δh (ft)	ΔS (in)
10.00	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.83	0.00	5.00	0.000
15.00	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.75	0.00	5.00	0.000

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v weight factor	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
16.00	16	24.69	0.71	0.434	24.69	0.73	2.01	5.00	1.207	0.00
20.00	13	34.14	0.83	0.329	34.14	0.67	2.11	5.00	1.269	0.00
25.00	17	0.00	0.00	2.000	0.00	0.00	0.00	5.00	0.000	0.00
30.00	28	0.00	0.00	2.000	0.00	0.00	0.00	5.00	0.000	0.00

Cumulative settlements: 2.476 0.00

Abbreviations

γ_{lim}: Limiting shear strain (%)
F_α/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

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