GEOTECHNICAL REPORT
Proposed Additions
4836 East Mercer Way
Mercer Island, WA

PROJECT NO. 25-195
June 2025

Prepared for:

Jennifer & Greg Rosenwald

PanGEO INCORPORATED

Geotechnical & Earthquake
Engineering Consultants

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June 18, 2025
File No. 25-195

Jennifer & Greg Rosenwald
4836 East Mercer Way
Mercer Island, WA 98040

Subject: **Geotechnical Report**
Proposed Additions
4836 East Mercer Way, Mercer Island, Washington

Dear Jennifer and Greg,

Please find attached our geotechnical report for the proposed additions to the single-family residence in Mercer Island, Washington. This report documents the subsurface conditions at the site and presents our geotechnical engineering recommendations for the proposed project.

Soil Conditions – In summary, the proposed addition areas are underlain by marginal soils consisting of loose to medium dense silty sand, with groundwater encountered between 5 to 10 feet below ground surface. Some settlement due to soil liquefaction is possible during the large, code-level earthquake. The soils at the lake-side of the house found a greater thickness of poor soils, with the potential to settle under static loads and a greater risk of more settlement from liquefaction.

Foundation Recommendations – Based on the current design and the results of our test borings, we recommend that the garage and hallway footings be supported by shallow strip foundations so that they perform similarly to the shallow foundations of the existing structure, without significant differential settlements. To improve the bearing capacity of the new footings, we recommend that the new footings bear on a minimum of 18-inch-thick layer of crushed rock and be tied into the existing house and garage footings. For the two pad footings for the patio roof addition, we recommend that they be supported on small-diameter driven pipe piles, 2- to 4-inches in diameter. We recommend that at least two of the pin piles under each footing be battered to resist lateral loads on the column footing.

Critical Area Considerations – Provided that the recommendations presented in this report are incorporated into the project plans and construction of the project, in our opinion the proposed project is feasible from the geotechnical standpoint, and will not adversely affect the mapped critical areas at the site.

We appreciate the opportunity to work on this project. Please call if there are any questions.

Sincerely,

Jon C. Rehkopf, P.E.
Principal Geotechnical Engineer
jrehkopf@pangeoinc.com

Encl.: Geotechnical Engineering Report

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6836 EAST MERCER WAY
MERCER ISLAND, WASHINGTON

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering study that was undertaken to support the design of the proposed additions for the single-family residence (SFR) at 6836 East Mercer Way (Parcel 1924059001) on Mercer Island, Washington. This study was performed in general accordance with our mutually agreed scope of services outlined in our proposal dated May 5, 2025, which was subsequently authorized by you on May 28, 2025. Our scope of services included reviewing readily available geologic and geotechnical data, performing engineering analyses, and preparing this geotechnical engineering report.

2.0 SITE AND PROJECT DESCRIPTION

The project site is an approximately 18,403 square-foot (0.42 acre) waterfront lot located at 4836 East Mercer Way on the east side of Mercer Island, Washington (See Figure 1, Vicinity Map). The site is roughly rectangular in shape and is bordered by existing single-family residences to the south and west, by the Appleton Lane ROW and a shared moorage parcel to the north, and Lake Washington to the east.

The site is currently occupied by a two-story single-family residence (SFR) at the approximate center of the site, with a detached two-car garage located at the center-south of the site. The existing structures are adjacent to a concrete driveway and patio, with grass lawn and landscaping to the west and the east (See Figure 2, Site and Exploration Plan).

Based on our review of the project site survey dated September 3, 2024, the existing site grade generally descends from west to east, with an average gradient of about 5 to 10 percent from about Elevation 36 feet (NAVD88) to Elevation 19 feet, for a total vertical relief of about 17 feet between the west and east property lines.

The property is mapped by the City of Mercer Island within or adjacent to several geological hazards, including an erosion hazard area, potential landslide area, and seismic hazard area. As such, the development will need to consider these hazards, which are addressed in *Section 5.0* of this report.

We understand that the scope of the project includes a new at-grade enclosed connecting hallway at the south side of the property between the existing detached garage and the SFR, along with a second-floor addition to the garage, interior renovations, and exterior improvements to the east patio. These proposed improvements are indicated on the attached Figure 2. We understand that these improvements will require new footings for the south hallway addition, northeast covered patio, and garage. We anticipate the foundation excavations to be no deeper than 5 feet below existing grade.

Plates 1 to 4, following, show the existing site conditions at the proposed improvement areas.

Plate 1: View of existing detached garage, looking southeast.

Plate 2: View of proposed covered deck post area, looking northwest.

Plate 3: View of proposed south side hallway area, looking west from southeast corner of existing house.

Plate 4: View of proposed south side hallway area, looking east from southeast corner of existing garage.

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

PanGEO completed three test borings (PG-1, PG-2 and PG-3) at the subject site on June 4, 2025. The borings were advanced to between 21½ and 36 feet below the existing ground surface using a limited-access mini track-mounted drill rig equipped with 5-inch diameter hollow stem augers. The approximate boring locations are shown on the attached Figure 2.

Soil samples were obtained from the borings at 2½-foot and 5-foot depth intervals in general accordance with Standard Penetration Test (SPT) sampling methods (ASTM test method D-1586) in which the samples were obtained using a 2-inch outside diameter split-spoon sampler. The sampler was driven 18-inches into the soil using a 140-pound weight freely falling a distance of 30 inches below the bottom of the auger. 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.

A geologist from PanGEO was present on a full-time basis to observe the drilling, assist in sampling, and to describe and document the soil samples obtained from the borings. The soils were logged in general accordance with the system summarized on *Figure A-1, Terms and Symbols for Boring and Test Pit Logs*. Summary boring logs are included as Figures A-2 to A-4 in Appendix A. The stratigraphic contacts indicated on the boring logs represent the approximate depth to the boundaries between soils units. Actual transitions between soil units may be more gradual or occur at different elevations. The descriptions of groundwater conditions and depth are likewise approximate.

4.0 SUBSURFACE CONDITIONS

4.1 Site Geology

According to *The Geologic Map of Mercer Island (Troost and Wisher, 2006)*, the subject parcel is underlain by pre-Olympia non-glacial deposits (Qpon) and Lake Deposits (Ql), with an overlay of Mass-Wastage Deposits (Qmw) mapped over the surface of the site. These geologic units are described below, in order from youngest to oldest:

- **Mass-wastage Deposits (Qmw) –** Mass-wastage deposits are surficial soils transported downslope in mass by gravity (landslides, colluvial soil movement, and other gravitational processes). Mass-wastage deposits typically consist of intermixed, very loose to medium dense, coarse-grained deposits and soft to stiff fine-grained deposits with voiding. This geologic unit typically exhibits moderate to high compressibility and low to moderate strength characteristics due to the highly variable composition and the nature in which this unit was deposited.

- **Lake Deposits (Ql)** – Lake deposits as mapped are described as lake-bottom sediments exposed by the lowering of Lake Washington in 1916. Lake deposits soils typically consist of silt and clay with layers of sand, peat, and other organic sediments, deposited in slow-moving water. This geologic unit is typically very soft to medium stiff or very loose to medium dense, and predominantly fine-grained and horizontally bedded.

- **Pre-Olympia Non-Glacial Deposits (Qpon)** – This geologic unit is described by Troost and Wisher (2006) as generally consisting of very dense and hard, sand, gravel, silt, clay and organics of non-glacial origin. The unit may contain tephra beds, paleosols, and iron oxidized layers. These pre-Olympia deposits also typically exhibit low compressibility and high strength characteristics in an undisturbed state.

4.2 Soil Conditions

The test borings advanced at the project site generally encountered soils consistent with the mapped geologic stratigraphy, although we did not encounter significant mass-wasting deposits. Rather, the disturbed surficial materials we found at the site were either interpreted to be fill, or deposits attributable to alluvial processes.

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.

Fill (Hf) – Fill soils were encountered in all three borings below a shallow layer of topsoil and sod, to depths of about 2½ to 2¾ feet below ground surface. This soil material generally consisted of loose to medium dense silty sand with gravel and sandy silt, with organic fragments. We characterized this material as fill based on the relative density, disturbed texture, and presence of organics.

Lake Deposits (Ql) – Directly below the fill in all three explorations, lake deposit soils were encountered to depths below ground surface of about 10 feet in PG-1 and PG-2, and 7½ feet in PG-3. This soil material generally consisted of interlayered, loose to medium dense silty sand and stiff to very stiff sandy silt, with gravel. We characterized this material as lake deposits based on the relative density, bedded structure, and constitutive components. Lake deposits at the site were likely exposed during the lowering of Lake Washington, and subsequently regraded and disturbed during the original construction of the property. Groundwater was also encountered within or at the base of this soil unit.

Pre-Olympia Non-Glacial Deposits (Qpon) – Directly below the lake deposits in all three explorations, pre-Olympia non-glacial deposits were encountered to termination depths below ground surface of 21½ feet in PG-1 and PG-3, and 36 feet in PG-2. This soil unit generally consisted of interlayered medium dense to very dense silty sand and stiff silt. We characterized this material as pre-Olympia non-glacial deposits based on the relative density, bedded structure, and constitutive components.

Our subsurface descriptions are based on the conditions encountered and observed at the time of our exploration. Soil conditions between 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

Groundwater was encountered in all explorations at depths of about 10, 5, and 7½ feet below the existing ground surface in PG-1 (at Elevation 26 feet), PG-2 (Elevation 26 feet), and PG-3 (Elevation 31 feet), respectively.

Please note that there will be fluctuations in seepage and groundwater levels, depending on the season, amount of rainfall, surface water runoff, local subsurface conditions, water levels in Lake Washington, and other factors. Generally, the groundwater levels are higher and seepage rates are greater in the wetter winter months (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 Chapter 19.07.160, Geologically Hazardous Areas. Mercer Island City Code identifies three different types of Geologic Hazards: Erosion Hazards, Potential Landslide Hazards, and Seismic Hazards. The City’s criteria for the various hazard areas and our assessment of the hazard areas with respect to the planned improvements are provided in the following sections of this report. Plate 5, following, shows the geologic hazards mapped at the site:

Plate 5: Mercer Island GIS map showing erosion hazard (orange overlay), seismic hazard (pink overlay), and landslide hazard areas (blue stripe overlay) relative to the project site (red rectangle). No steep slopes were mapped in the immediate project vicinity.

5.1 Erosion Hazards

The site is not mapped as a potential erosion hazard area in accordance with the City of Mercer Island's Geologic Hazards Map, but erosion hazard area is mapped directly upslope to the west. Based on the Web Soil Survey data, the mapped soils present on the site (Kitsap Silt Loam, 2 to 8 percent slopes, KpB) have an Erosion Factor K of 0.37 to sheet and rill erosion. Factor K values range between 0.02 and 0.69, with the higher number indicating higher vulnerability. As such, we interpret the site soils to have a moderate susceptibility to erosion.

Conclusions: In our opinion, the erosion hazards at the site can be effectively mitigated with best management practices during construction and with properly designed and implemented landscaping for permanent erosion control. During construction, the temporary erosion hazard can 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 high traffic areas, covering stockpiled soil or cut slopes with plastic sheets, constructing a temporary drainage pond, if needed, to control surface runoff and trap sediment, and by maintaining a stabilized construction entrance.

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

5.2 Potential Landslide Hazards

The subject site is mapped within a potential landslide hazard area according to the City of Mercer Island's Geologic Hazards Map. The map indicates that steep slopes are not present at the site.

The City of Mercer Island GIS mapping identifies a few landslide indicators within about 1,000 feet of the subject site. Several are above East Mercer Way, to the west and southwest of the project site. These slides are mapped within steep slope and erosion hazard areas. These indicators are likely in response to inadequate drainage and pavement settlement. The map also shows landslide scarps to the west of the site, the closest of which is over 1,000 feet away.

Site Reconnaissance and Observations: We conducted a reconnaissance visit to review the condition of the sloping areas of the site, and areas adjacent to the site, and identify indications of potential historical slope instability.

The site grade gently descends from west to east, and approximately situated downslope of a descending ridge created by drainage bowl areas to the north and south of the site, west of East Mercer Way. Each drainage is defined by landslide scarps mapped to the west. See Plate 6, following, showing the drainage locations relative to the site:

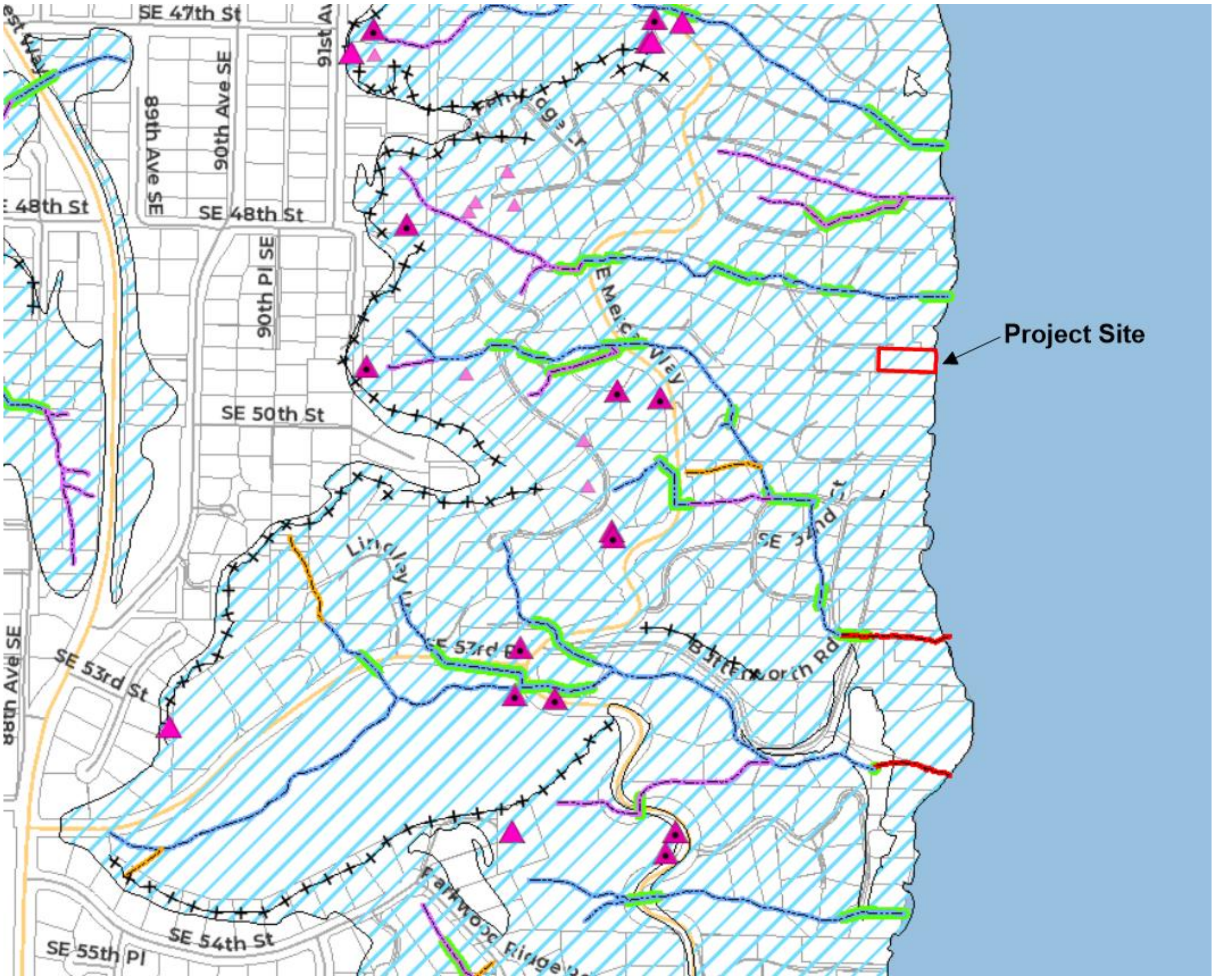


Plate 6: Mercer Island GIS map showing drainage channels (various colors), previous known landslides (pink triangles), landslide scarps (black hashes), and landslide hazard areas (blue stripe overlay) relative to the project site (red rectangle).

During our site visits, we did not observe evidence of recent or current slope instability such as slide scarps or tension cracks within the subject property. In addition, no recent or historical slides have been mapped on or directly adjacent to the subject property.

Conclusions: Based on our reconnaissance, review of existing data, our understanding of subsurface conditions at the site, and lack of significant topographic relief at the site, in our opinion neither shallow nor large, deep-seated-type slope failures are likely to occur at the site. It is also our opinion that the proposed improvements will not adversely affect the stability of the site, or the stability of adjacent sites.

5.3 Seismic Hazards

Based on our review of the City of Mercer Island's Geologic Hazards Maps, the project site is mapped in a seismic hazard area. The City of Mercer Island Code defines seismic hazard areas as those areas subject to risk of damage due to earthquake-induced ground shaking, slope failure, soil liquefaction or surface faulting.

Based on our subsurface explorations, the site is underlain by loose to medium dense sand and non-plastic silt, with a relatively shallow static groundwater table. Based on these conditions, in our opinion, the liquefaction potential of the soils at the site is high, and design considerations related to soil liquefaction are necessary for this project.

Seismic design considerations for liquefaction are discussed as follows under Section 6.1, following.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 Seismic Design Considerations

6.1.1 Site Class

We understand that the project will be designed in accordance with the 2021 editions of the International Building Code (IBC), and ASCE 7-16, 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 7-16.

According to Chapter 20 of ASCE 7-16, the site soil should be classified as Site Class F because of its liquefaction potential (see discussions in Section 6.1.2 of this report). Section 20.3.1 of ASCE 7-16 indicates that for Site Class F a site-specific ground 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 7-16 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 the limited building height, we assume that the fundamental periods of the proposed addition are less than 0.5 second. As such, based on the results of our test borings, the site coefficients may be determined based on a Site Class D.

6.1.2 Liquefaction Analysis and Results

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

We evaluated the liquefaction potential at the site using an earthquake having a 2-percent probability of occurrence in 50 years (return interval of 2,475 years) in accordance with the 2021 IBC. The analysis utilized the data collected from PG-1 through PG-3 and the Liquefaction Assessment Software *LiqSVS* by GeoLogismiki. A peak ground acceleration associated with the IBC level earthquake of 0.67g and magnitude of 7.5 was used to determine the risk of liquefaction, using the method suggested by Boulanger & Idriss (2014).

The results of our analyses indicate that there is a high risk of soil liquefaction between a depth of about 7 to 15 feet below grade in PG-1 and PG-3, and between about 9 and 25 feet below grade in PG-2. These ranges are where the test borings encountered saturated, loose to medium dense sands. Below these loose layers, liquefaction risk is low due to an increase in soil density to dense and very dense.

For the large, design-level earthquake, our analysis also estimated total settlement of the ground surface due to liquefaction of site soils on the order of about 1 to 1½ inches at PG-1 and PG-3, and about 3 inches at PG-2. We anticipate that differential settlements due to varying degrees of soil liquefaction may be on the order of 1 to 2 inches across the footprint of the improvement areas.

If liquefaction occurs at the site, it will likely result in the settlement of the existing house and garage foundations, which will be connected to the proposed additions. Because the upper 5 to 7½ feet of soil at the site is not saturated and therefore not expected to liquefy, and based on the historic performance of the existing house, it is our opinion that a significant loss of bearing capacity due to liquefaction is not anticipated. It is also our opinion that the potential damage associated with differential settlements of the house foundation has a low risk of resulting in a life safety issue for the occupants and would not significantly impede entrance or egress from the structures following an earthquake.

6.1.3 Liquefaction Mitigation

Based on the results of our analyses and our understanding of the site, to reduce the potential of differential settlements between isolated spread footings, we recommend that the new south hallway addition and the garage addition be supported on continuous strip footings structurally connected to the existing structure. Because the house and detached garage are also supported on footings, the existing and proposed foundations should perform similarly during a seismic event, and the potential for differential settlement between the current and planned structures should be tolerable.

However, PG-2, advanced at the location of the proposed patio roof columns, encountered a greater thickness of poor soils at the surface, with greater potential to settle under static loads but also a greater potential for more liquefaction-induced settlement. A deep foundation type such as small diameter driven pipe piles (pin piles) should be used to support the patio roof columns.

6.2 FOUNDATION RECOMMENDATIONS

6.2.1 Conventional Footings (Connecting Hallway and Garage Addition)

As discussed above, it is our opinion that conventional footings are appropriate for the proposed connecting hallway and the garage addition. To improve the bearing capacity of the proposed footings, we recommend that the new footings bear on a minimum 18-inch-thick layer of crushed rock and be tied into the existing house and garage footings. For planning purposes, our recommendations assume that all the new footings will require over-excavation of unsuitable soils and replacement with crushed rock, or similarly suitable structural fill approved by PanGEO.

> ***Over-excavation and Replacement*** – The over-excavation should extend to, at minimum, 18 inches below the design footing subgrade and should be backfilled with properly compacted 1¼-inch minus crushed rock, 1¼-inch minus recycled crushed concrete, or

approved equivalent. The over-excavation and backfill should extend at least one foot horizontally beyond the edge of the new footings. The bottom of the footing excavation should be compacted to a dense and unyielding condition before placing the crushed rock.

Allowable Bearing Pressure – A maximum allowable soil bearing pressure of 2,000 pounds per square foot may be used to size footings constructed as discussed above. For allowable stress design, the recommended allowable bearing pressure may be increased by one-third for transient loading conditions such as those due to wind. Due to soil liquefaction potential at the site, the one-third increase is not appropriate for seismic conditions.

Minimum Footing Embedment – For frost protection considerations, footings should be placed at least 18 inches below adjacent finished grade.

Lateral Resistance – Lateral loads acting on footings may be resisted by passive earth pressure developed against the embedded portion of the footings and by frictional resistance at the base of the footings.

- An allowable frictional coefficient of 0.4 may be used to evaluate sliding resistance.
- An allowable passive soil resistance may be calculated using an equivalent fluid pressure of 300 pcf, assuming the footings are backfilled with structural fill and level ground surface. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches of soil should be neglected.

The above values include a geotechnical factor of safety of 1.5.

Footing Drains – Footing drains should be installed around the perimeter of the building, at or just below the bottom of the footings. 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.

Footing Subgrade Preparation – Footing subgrades should be carefully prepared and should not contain loose, soft, or disturbed soils. The adequacy of footing subgrades should be verified by a representative of PanGEO prior to placing forms or rebar. The footing subgrades should be in a dense and unyielding condition prior to pouring concrete.

Please note that the site soils are moderately to highly moisture sensitive and can become disturbed and softened when exposed to moisture and construction traffic. Protection of the foundation bearing soils should be the responsibility of the contractor.

Foundation Performance – Total and differential settlements are anticipated to be within tolerable limits for footings designed and constructed as discussed above. Footing settlement under static loading conditions is estimated to be about ¾ inch, and differential settlement across the structure should be about ½ inch or less. Most settlement will be realized during construction as the dead loads are applied.

6.2.2 Driven Pin Piles (Patio Cover Columns)

Due to poor soil conditions and the risk for foundation settlement under static conditions, it is our opinion that the new foundation elements for the proposed patio cover should be supported on piles, such as small diameter steel pipe piles. Small diameter driven pipe piles, also known as pin piles, are utilized to transfer the structure loads through the weak and marginal soils to the underlying competent bearing layer. Pin piles of 2- to 4-inches in diameter are typically utilized for light weight structures such as the proposed patio cover. Two-inch pin piles can be driven with hand equipment, but 3- and 4-inch diameter pin piles are typically installed using small to large hammers (600 to 2,000 lbs) mounted on small to medium-sized excavators.

Pin Pile Sizes – We have provided recommendations for 2-, 3- and 4-inch diameter pipe piles. The structural engineer should evaluate the pile sizing and spacing based on the design loads and pile capacities.

Pin Pile Capacity – The following allowable axial compression capacities can be used per pile assuming a factor of safety of at least 2.0:

- 3 tons (6 kips) per 2-inch diameter pile
- 6 tons (12 kips) per 3-inch diameter pile
- 10 tons (20 kips) per 4-inch diameter pile

Penetration resistance required to achieve the capacities will be determined based on the hammer used to install the pile. The tensile capacity of pin piles should be ignored in design calculations.

***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 or a 140-lb Rhino hammer. Refusal is defined as no more than 1 inch of penetration for 1 minute of continuous driving.

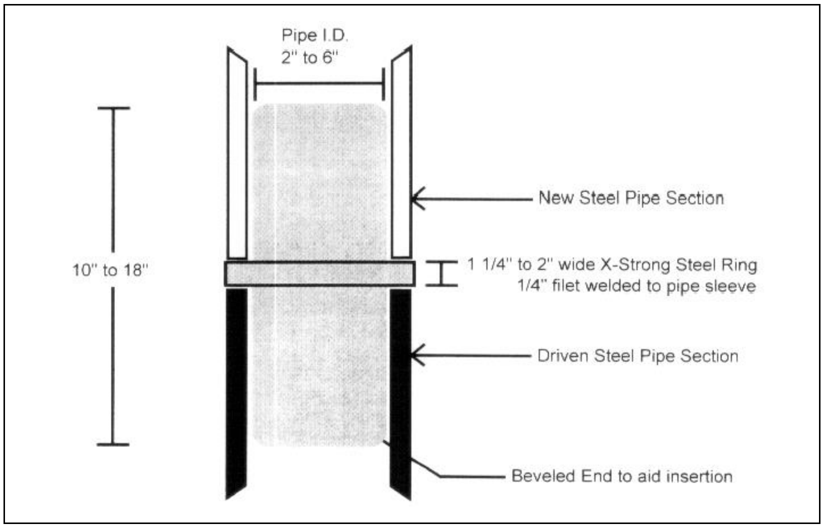
4. 3-inch piles shall be driven to refusal with a minimum 600-lb hydraulic hammer. We recommend the following refusal criteria based on the size of hammer utilized:

Hammer Size	Blow 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

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:

Hammer Size	Blow 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

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



7. At least 3% (but no more than 5) of the 3-inch, and 4-inch diameter pin piles should be load tested to verify the driving criteria listed above. The load tests should be performed prior to installed production piles. If more than one size of pipe pile is used, each pipe size should be subject to separate testing. Contractors may elect to use a different hammer system and driving criteria, provided that the driving criteria for the selected hammer can be verified with the load test program.

8. All load tests shall be performed in accordance with the procedure outlined in ASTM D1143 - *Standard Test Methods for Deep Foundations Under Static Axial Compressive Load*. 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.

9. The geotechnical engineer of record or his/her representative shall provide full time observation of pile installation and testing.

Installation Monitoring – As it is not possible to observe the completed pile below the ground, judgment and experience must be used as the basis for determining the acceptability of a pile. Therefore, all piles should be installed under the full-time observation of a representative of PanGEO. This will allow us to fully evaluate the contractor's operation, collect and interpret the installation data, and verify bearing stratum elevations.

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. Furthermore, we will also understand the implications of variations from normal procedures with respect to the design criteria. The contractor's equipment and procedures should be reviewed by PanGEO before the start of construction.

Lateral Resistance – The lateral capacity of pin pipes 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 3(H):12(V) or steeper]. Friction at the base of pile-supported footings and grade beam should be ignored in the design calculations.

Passive resistance values may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf), assuming level ground surface in front of the footings. This value includes a safety factor of about 1.5 assuming that properly compacted granular fill will be placed adjacent to the pile caps and grade beams, and level ground surface.

Estimated Pile Length – The required pile length in order to develop the recommended pile capacity is expected to vary across the footprint of the structure, depending on the actual driving conditions encountered. For planning and cost-estimating purposes, we anticipate that a 10-foot penetration into the dense native soils be expected. Based on the soil conditions encountered in our test borings during drilling, we estimate that average pile lengths may range between about 30 to 40 feet. Minimum installed pile lengths should be no shorter than 10 feet.

Pin Pile Performance – It is our experience that the driven pipe pile foundations should provide adequate support with total settlements on the order of ¼ to ½ inch.

Obstructions – Obstructions may be encountered during pile installation. 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 as needed.

6.3 FLOOR SLABS

6.3.1 Concrete Slab-on-grade

A slab-on-grade may be used for the floor slab of the proposed hallway addition and patio renovation. However, loose/soft soils may be present below the slab elevation in some areas of the house footprint. In these areas there is a potential for some slab settlement to occur over the design life of the structure, which can result in cracks and uneven floor surfaces.

To reduce the potential of slab settlement and distress in these areas, we recommend that the floor slab subgrade (below the base of the capillary break material, as outlined below) be over-excavated by at least 12 inches, and existing fill or native subgrade soil recompacted to a firm and unyielding condition. Any soft/loose and pumping native subgrade soil observed during compaction should be removed and replaced with granular structural fill. If more than two feet of unsuitable soils are present, to improve subgrade, we recommend that a layer of geogrid reinforcement be placed over the native subgrade prior to placement and compaction of structural fill. The geogrid should be overlapped a minimum of 12-inches. We also recommend that construction joints be incorporated into the floor slab to control cracking.

6.3.2 Structural Slab

If a higher level of slab performance is desired than described above for slab-on-grades, a structural slab can be designed to span between foundations. If a structural slab is utilized, the existing loose/soft soils below the slab may be left in place without re-compaction or replacement. A capillary break and vapor barrier should be placed below the slab, as described below. We recommend a structural slab when the floor surface will receive settlement sensitive floor coverings, such as tile, or if the exposed floor will be polished concrete.

6.3.3 Capillary Break

We recommend that at least 4 inches of capillary break be placed below interior slabs. The capillary break material should consist of free-draining, clean (less than 3 percent fines) crushed rock compacted to a firm and unyielding condition. The capillary break material should have no more than 10 percent and 5 percent by weight of material passing the U.S. Standard No. 4 and No. 100 sieves, respectively. We also recommend that a 10-mil polyethylene vapor barrier be placed below the slab.

6.4 RETAINING WALL DESIGN PARAMETERS

Retaining walls, if needed, shall be designed to resist the lateral earth pressures exerted by the soils behind the walls. Adequate drainage provisions should also be provided behind the new walls to intercept and remove groundwater or surface water that may accumulate behind the walls.

Our geotechnical recommendations for the design and construction of retaining and below grade walls are presented below:

6.4.1 Concrete Wall Foundation Support and Lateral Resistance

The footing recommendations outlined in Section 6.2.1 of this report are also applicable for the walls. For walls with fore slopes, with a maximum height of 5 feet, we recommend that the footing be embedded a minimum of 2 feet below the finished grade in front of the wall.

6.4.2 Lateral Earth Pressures

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 50 pcf should be used for level backfills behind the walls.

Walls with a maximum 2H:1V backslope should be designed for an active and at rest earth pressure of 50 and 65 pcf, respectively. The recommended lateral pressures assume that the backfill behind the wall consists of free draining and properly compacted fill with adequate drainage provisions to prevent the development of hydrostatic pressure.

Permanent walls should be designed for an additional uniform lateral pressure of 10H psf for seismic loading, where H corresponds to the buried depth of the wall in feet. The recommended lateral pressures assume that the backfill behind the wall consists of a free draining and properly compacted fill with adequate drainage provisions.

6.4.3 Wall Surcharge

Surcharge loads, where present, should also be included in the design of basement or retaining walls. We recommend 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 of the wall height.

6.4.4 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.

Under no circumstances should roof downspout drain lines be connected to the perforated footing/wall drain systems for basement walls. 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.

6.4.5 Wall Backfill

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

Wall backfill should be moisture conditioned to 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. If density tests will be performed, the test results should demonstrate 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.

6.4.6 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 foundation 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.

6.5 Permanent Surface Drainage

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 pavements and structures, and into appropriate outlets. Adequate surface gradients should be incorporated into the grading design such that surface runoff is directed away from structures and steep slope.

Under no circumstances should collected surface water or downspout drains be allowed to discharge onto open slopes or behind walls. Furthermore, it is important to note that roof downspouts should be tightlined to a suitable outlet, and not discharged into the wall or perimeter footing drain system.

6.6 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 clearing, grubbing, and excavations to the design subgrade. All stripped surface materials should be properly disposed of off-site.

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 be over-excavated and replaced with compacted structural fill.

7.2 Material Reuse

The soils at the site are moisture sensitive and will become disturbed / soft when exposed to inclement weather conditions. In our opinion, the on-site soils are not suitable to be reused as structural fill. In the context of this report, structural fill is defined as compacted fill placed under footings, pavements, concrete stairs, landings, and slabs, or other load-bearing areas. Material for use as structural fill is described in the following section.

The on-site soil may potentially be used as general fill in the non-structural and landscaping areas. If use of the on-site soil is planned, the excavated soil should be stockpiled and protected with plastic sheeting to prevent softening from rainfall in the wet season.

7.3 Structural Fill Placement and Compaction

For planning purposes, structural fill should consist of imported, well-graded, granular material such as Seattle Type 17 Mineral Aggregate (*COS Standards and Specifications, 2023, Section 9-03.14*), WSDOT Gravel Borrow (*WSDOT Standards and Specifications, 2025, Section 9-03.14(1)*), or an approved equivalent.

Structural fill should be moisture conditioned near its optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and systematically compacted to a dense and relatively unyielding condition. The adequacy of the compaction should be verified by PanGEO. If density tests are performed, the test results should indicate at least 95 percent of the maximum dry density, as determined using test method ASTM D1557 (modified proctor). For utility backfill or backfill within 5 feet of retaining walls, the backfill should be compacted to at least 90 percent of the maximum dry density.

The procedure to achieve proper density of a compacted fill depends on the size and type of compacting equipment, the number of passes, thickness of the lifts being compacted, and certain soil properties. We recommend that structural fill supporting foundations be compacted with jumping jack compactors at a minimum. If the excavation to be backfilled is constricted and limits the use of heavy equipment, smaller equipment can be used, but the lift thickness will need to be reduced to achieve the required relative compaction. PanGEO can provide additional recommendations regarding structural fill and compaction during construction.

Generally, loosely compacted soils are a result of poor construction technique or improper moisture content. Soils with high fines contents are particularly susceptible to becoming too wet and coarse-grained materials easily become too dry, for proper compaction. Silty or clayey soils with a moisture content too high for adequate compaction should be dried as necessary, or moisture conditioned by mixing with drier materials, or other methods.

7.4 Temporary Excavations

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. All temporary excavations deeper than a total of 4 feet should be sloped or shored. Temporary excavations less than 4 feet along the property lines should also be sloped or shored.

For planning purposes, we recommend that temporary excavations be sloped no steeper than 1H:1V (Horizontal:Vertical). If temporary excavations are not in the fill, but in the dense native soil, steeper excavations may be feasible, based on PanGEO's field observations and the configuration of the excavations.

The temporary excavations and cut slopes should be re-evaluated in the field during construction based on actual observed soil conditions. If groundwater seepage is encountered the temporary slope will likely need to be cut to shallower angles to maintain stability, or require shoring. During wet weather, runoff water should be prevented from entering excavations and the exposed slopes should be covered with plastic sheets.

7.5 Temporary Erosion and Drainage Considerations

We recommend that the exposed temporary slopes be covered with plastic sheeting.

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

We recommend that the contractor should be prepared to provide temporary groundwater control methods, especially if excavation is conducted in the wet season. If present, we anticipate that the groundwater can likely be controlled with sumps and pumps.

7.6 Wet Earthwork Recommendations

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

- All surfaces of the foundation subgrade should be protected against inclement weather. It is the contractor's responsibility to protect the footing subgrade from disturbance. One option is to place a 2- to 3-inch-thick layer of lean-mix concrete or 3 to 4 inches of clean crushed rock on the footing subgrade as soon as the subgrade is exposed.

- 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

We anticipate the City of Mercer Island will require a plan review and geotechnical special inspections to confirm that our recommendations are properly incorporated into the design and construction of the proposed development. Specifically, we anticipate that the following construction support services may be needed:

- Review final project plans and specifications;
- Verify implementation of erosion control measures;

- Observe the stability of open cut slopes;
- Monitor pin pile installation and testing;
- Confirm the adequacy of the compaction of structural backfill;
- Observe installation of subsurface drainage provisions, and;
- Other consultation as may be required during construction.

Modifications to our recommendations presented in this report may be necessary, based on the actual conditions encountered during construction.

9.0 STATEMENT OF RISK

Per the Mercer Island City Code, development within geologic hazard areas requires a statement of risk. The statement of risk shall meet one of the following criteria:

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

b. Construction practices are proposed for the alteration that would render the development as safe as if it were not located in a geologic hazard area;

c. The alteration is so minor as not to pose a threat to the public health, safety and welfare; or

d. An evaluation of site-specific subsurface conditions demonstrates that the proposed development is not located in a geologic hazard area.

Based on our understanding of the proposed project, it is our opinion that criteria (a) and (c) above, are applicable. PanGEO will be available to review the final design plans to confirm our statement of risk prior to construction.

10.0 LIMITATIONS

We have prepared this report for use by Jennifer & Greg Rosenwald 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 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.

June 18, 2025

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11.0 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 (COS), 2023, *Standard Specifications for Road, Bridges, and Municipal Construction*., Seattle, Washington

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Troost, K. G. and Wisher, A.P., 2006, *Geologic Map of Mercer Island,* Geomap NW, University of Washington and the City of Mercer Island.

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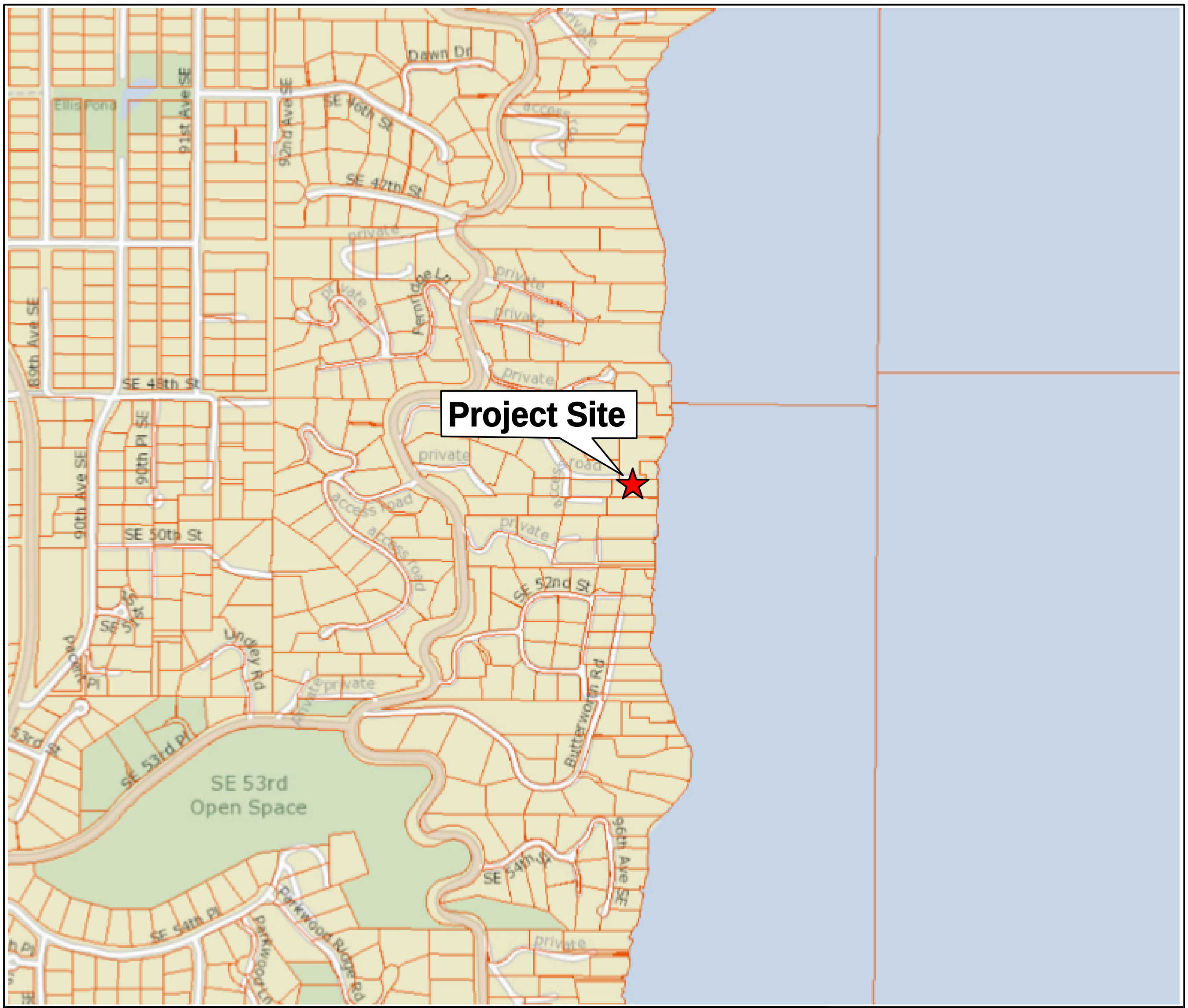


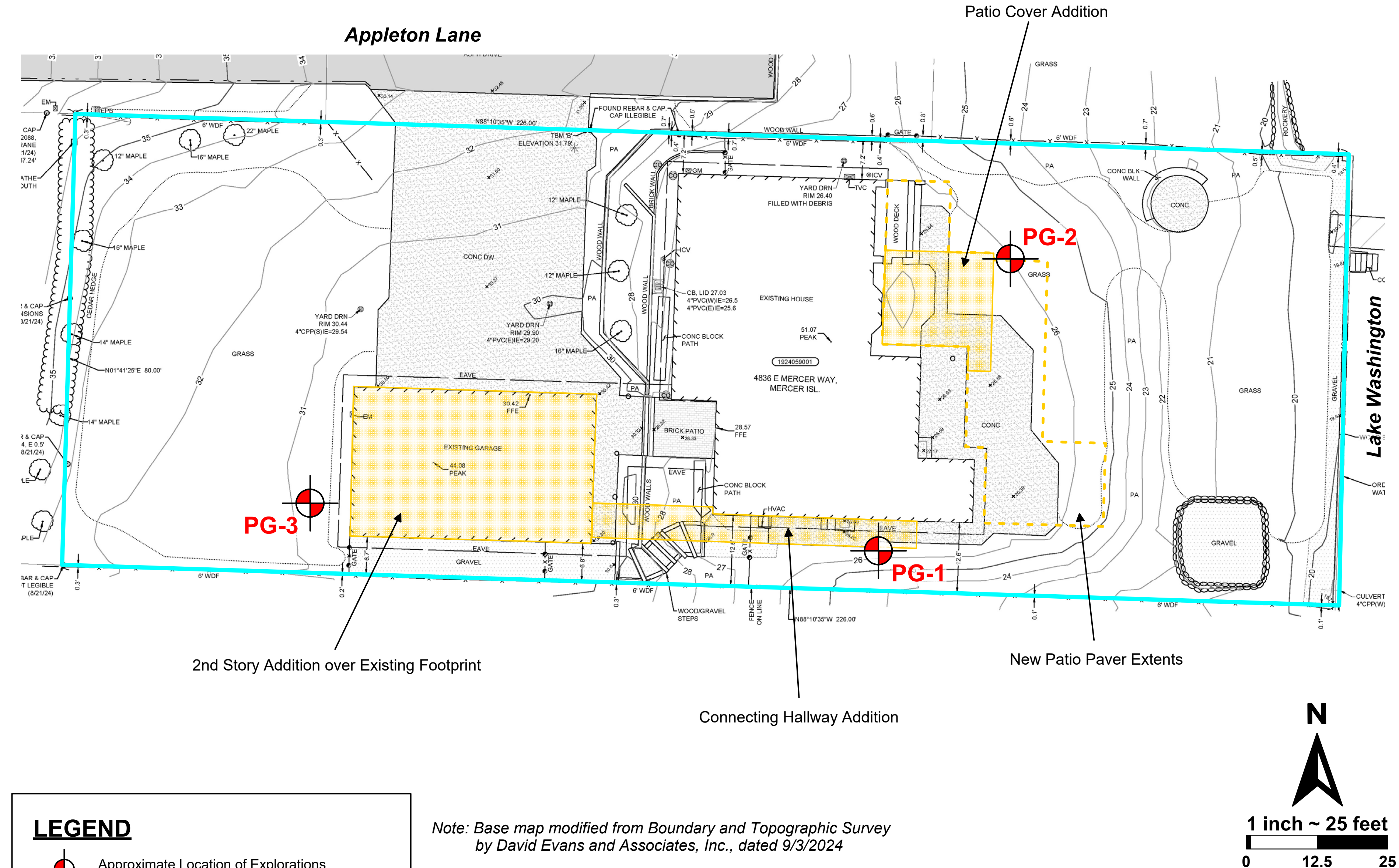
Base Map: King County iMap

N

NOT TO SCALE

PanGEO INCORPORATED	**Proposed Additions** **4836 East Mercer Way** **Mercer Island, Washington**	**VICINITY MAP**	
		Project No. **25-195**	Figure No. **1**

Fig 1 - Vicinity Map.gpj 6/12/25 (2:56:42) ADO



LEGEND

- Approximate Location of Explorations (PanGEO, 2025)
- Approximate Site Boundary
- Approximate Extent of Proposed Addition Areas
- Approximate Extent of Proposed Improvements

Note: Base map modified from Boundary and Topographic Survey by David Evans and Associates, Inc., dated 9/3/2024

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Proposed Additions
4836 East Mercer Way
Mercer Island, Washington

SITE AND EXPLORATION PLAN

Project No.	25-195	Figure No.	2

Fig 2 - Site and Exploration Plan.gpj 6/17/25 (2:05:15) ADO

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

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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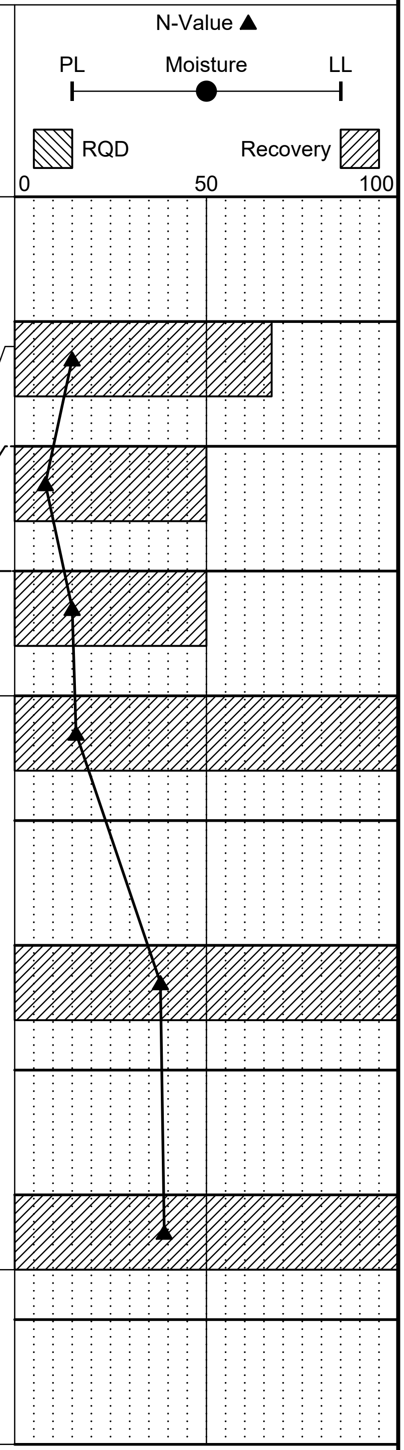
Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	Proposed Additions	Surface Elevation:	~26 ft
Job Number:	25-195	Top of Casing Elev.:	N/A
Location:	4836 East Mercer Way, Mercer Island	Drilling Method:	HSA, Mini Track Rig
Coordinates:	Northing: 47.55886, Easting: 122.20977	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						**(FILL / TOPSOIL)** Ivy groundcover at surface. Potholed about 2 feet to clear irrigation. Loose, olive-brown to brown, silty SAND with gravel, moist. Organic fragments throughout.
2.5	S-1		7 9 6			- becomes medium dense, with trace organics and detritus. Some coarse sand.
						(LAKE DEPOSITS) Stiff to very stiff, oxidation-mottled light bluish-gray, fine sandy SILT, moist.
5.0	S-2		4 4 4			Loose, oxidation-mottled olive-brown and light bluish-gray, interbedded silty fine SAND and fine sandy SILT, moist to very moist. Trace coarse sand and gravel throughout. Bedding disturbed.
7.5	S-3		4 6 9			Medium dense, oxidized olive-brown, silty fine to medium SAND, moist to very moist. Some gravel and rootlets throughout. Indistinct structure.
10.0	S-4		4 4 12			**(PRE-OLYMPIA NON-GLACIAL DEPOSITS)** Medium dense, oxidized olive-brown with olive-gray and gray, finely bedded and thinly interlayered, fine to medium SAND, SAND with silt, silty SAND, and SILT, very moist to wet. Sand lenses wet.
12.5						
15.0	S-5		9 18 20			- becomes dense, olive-gray to gray. Predominantly SAND and SAND with silt, wet. Some fine lenses of olive-brown laminated SILT. Dilatant.
17.5						
20.0	S-6		7 15 24			- same as above, layers becoming thicker.
						Boring terminated at about 21.5 feet below existing grade and abandoned with bentonite. Groundwater encountered at about 10 feet below ground surface during drilling.
22.5						
25.0						



Completion Depth:	21.5ft
Date Borehole Started:	6/4/25
Date Borehole Completed:	6/4/25
Logged By:	A. Ong
Drilling Company:	Geologic Drill Partners, Inc.

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is pro

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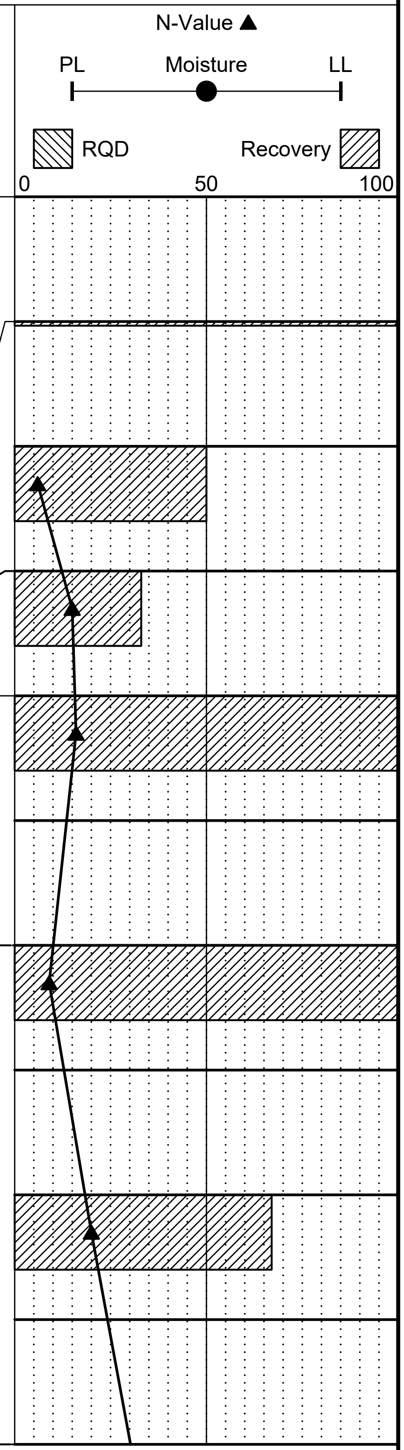
LOG OF TEST BORING PG-1

Figure A-2

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

Project:	Proposed Additions	Surface Elevation:	~26 ft
Job Number:	25-195	Top of Casing Elev.:	N/A
Location:	4836 East Mercer Way, Mercer Island	Drilling Method:	HSA, Mini Track Rig
Coordinates:	Northing: 47.55899, Easting: 122.20965	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						**(FILL / TOPSOIL)** Grass lawn at surface. Potholed about 12 inches, exposing about 9 inches of sod, topsoil, and fill material. Loose, brown to olive-brown, silty SAND with trace gravel, moist. Organic fragments throughout.
2.5	S-1					Loose, oxidation-mottled olive-brown and light bluish-gray, SILT with fine sand, moist.
						(LAKE DEPOSITS) - drillers encountered obstruction while sampling at 2.5 feet, postholed deeper and drilled past large rock. Cuttings blackish-brown to oxidized bluish-green, sandy SILT and silty SAND.
5.0	S-2		2 3 3			Loose, oxidized olive-brown with black and gray patches, silty SAND with silt lenses and trace gravel, wet. Bedded but jumbled in sections. One layer of blackened organic-rich silt. - gravelly drilling at about 6 feet.
7.5	S-3		4 10 5			Medium dense, olive-brown to olive-gray, silty fine to medium SAND, wet. Coarse sand and gravel throughout. Coarsely bedded, with oxidized lenses.
10.0	S-4		6 9 7			**(PRE-OLYMPIA NON-GLACIAL DEPOSITS)** Medium dense, olive-brown, thinly-bedded, silty fine SAND, wet. Some laminated silt at top of sample, and throughout in fine interbeds. - driller added slurry after sample.
12.5						
15.0	S-5		3 4 5			Loose and stiff, oxidized olive-brown with olive-gray and gray, finely bedded and interlayered, fine to medium SAND with silt, wet, and laminated SILT, very moist.
17.5						
20.0	S-6		5 8 12			- becomes medium dense, predominantly SAND, wet. Interbeds of silt thinner. Grades to trace silty, fine to medium SAND.
22.5						
25.0						



		Remarks
Completion Depth:	36.0ft	Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is pro
Date Borehole Started:	6/4/25	
Date Borehole Completed:	6/4/25	
Logged By:	A. Ong	
Drilling Company:	Geologic Drill Partners, Inc.	

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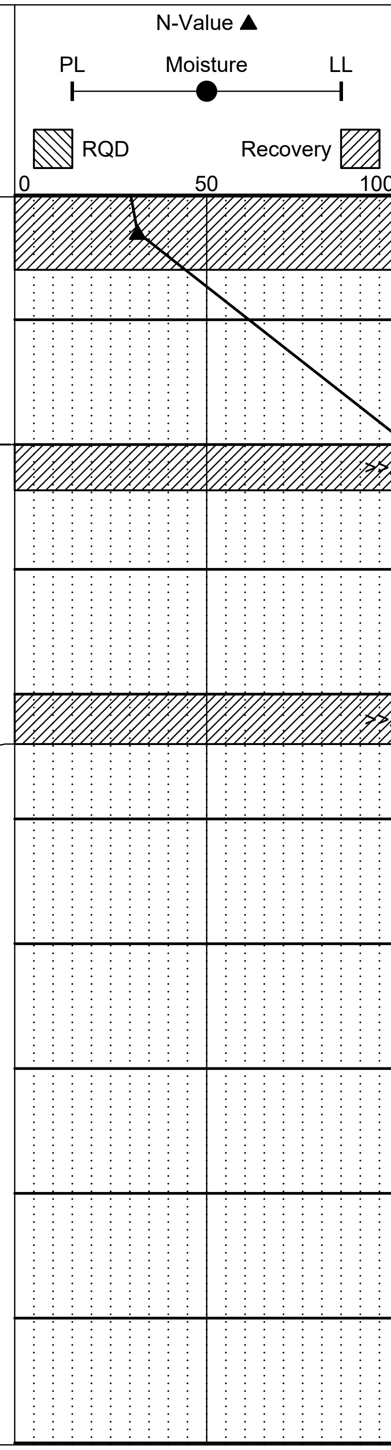
LOG OF TEST BORING PG-2

Figure A-3

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

Project:	Proposed Additions	Surface Elevation:	~26 ft
Job Number:	25-195	Top of Casing Elev.:	N/A
Location:	4836 East Mercer Way, Mercer Island	Drilling Method:	HSA, Mini Track Rig
Coordinates:	Northing: 47.55899, Easting: 122.20965	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
25.0	S-7		6 16 16			Dense, oxidized olive-brown to olive-gray to gray, bedded silty SAND, wet. Mottled oxidation throughout and in bands. Trace gravel and coarse sand. Some lenses of very silty SAND and sandy SILT.
27.5						
30.0	S-8		22 50/5			Very dense, olive-brown to oxidized reddish-brown and black, fine to medium SAND with lenses of laminated silty fine SAND, wet. Grades to gravelly SAND at bottom of sample. - gravelly drilling below sample.
32.5						
35.0	S-9		24 50/6			Very dense, olive-gray, medium to coarse SAND, wet. Trace silt. Sand subangular/subrounded.
						Boring terminated at about 36 feet below existing grade and abandoned with bentonite. Groundwater encountered at about 5 feet below ground surface during drilling.
37.5						
40.0						
42.5						
45.0						
47.5						
50.0						



Completion Depth:	36.0ft
Date Borehole Started:	6/4/25
Date Borehole Completed:	6/4/25
Logged By:	A. Ong
Drilling Company:	Geologic Drill Partners, Inc.

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is pro

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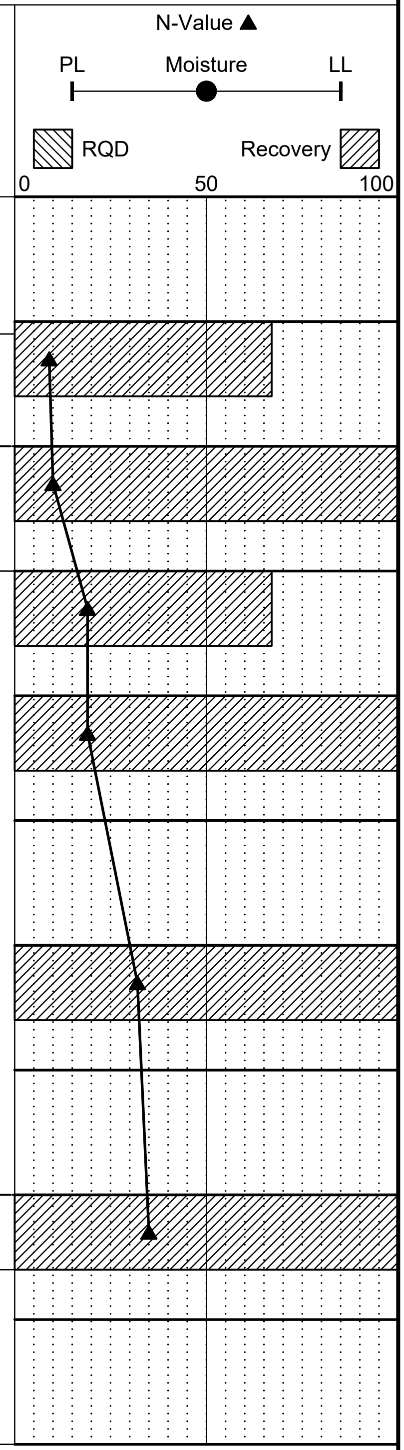
LOG OF TEST BORING PG-2

Figure A-3

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

Project:	Proposed Additions	Surface Elevation:	~31 ft
Job Number:	25-195	Top of Casing Elev.:	N/A
Location:	4836 East Mercer Way, Mercer Island	Drilling Method:	HSA, Mini Track Rig
Coordinates:	Northing: 47.55886, Easting: 122.21014	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						**(FILL / TOPSOIL)** Grass lawn at surface. Medium stiff and loose, brown, fine sandy SILT and silty SAND, dry.
2.5	S-1	X	4 5 4			**(LAKE DEPOSITS)** Loose, oxidation-mottled olive-brown and olive-gray, very silty SAND, very moist. Some charcoal fragments, trace coarse sand.
5.0	S-2	X	4 5 5			Medium dense, oxidation-mottled olive-brown, bedded silty fine to medium SAND, very moist to wet. Trace gravel and coarse sand. Oxidation in bands.
7.5	S-3	X	7 8 11			**(PRE-OLYMPIA NON-GLACIAL DEPOSITS)** Medium dense, olive-gray to oxidized olive-brown, bedded silty fine to medium SAND, wet. Oxidized in bands. Trace coarse sand and fine gravel.
10.0	S-4	X	8 9 10			- same as above, alternating layers of bedded olive-gray SAND with silt and oxidized olive-brown silty SAND, wet. Trace coarse sand and fine gravel. - driller added slurry after sample.
12.5						
15.0	S-5	X	11 16 16			- becomes dense, olive-gray, indistinctly bedded, predominantly SAND.
17.5						
20.0	S-6	X	7 13 22			Dense, oxidized olive-brown and olive-gray, finely bedded and interlayered fine SAND with laminated SILT, wet.
22.5						Boring terminated at about 21.5 feet below existing grade and abandoned with bentonite. Groundwater encountered at about 7.5 feet below ground surface during drilling.
25.0						



Completion Depth:	21.5ft
Date Borehole Started:	6/4/25
Date Borehole Completed:	6/4/25
Logged By:	A. Ong
Drilling Company:	Geologic Drill Partners, Inc.

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is pro

PanGEO INCORPORATED

LOG OF TEST BORING PG-3

Figure A-4

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

APPENDIX B

LIQUEFACTION ANALYSIS OUTPUT

PanGEO Inc.
3213 Eastlake Ave E, Suite B
Seattle, WA 98102-7127
https://www.pangeoinc.com/

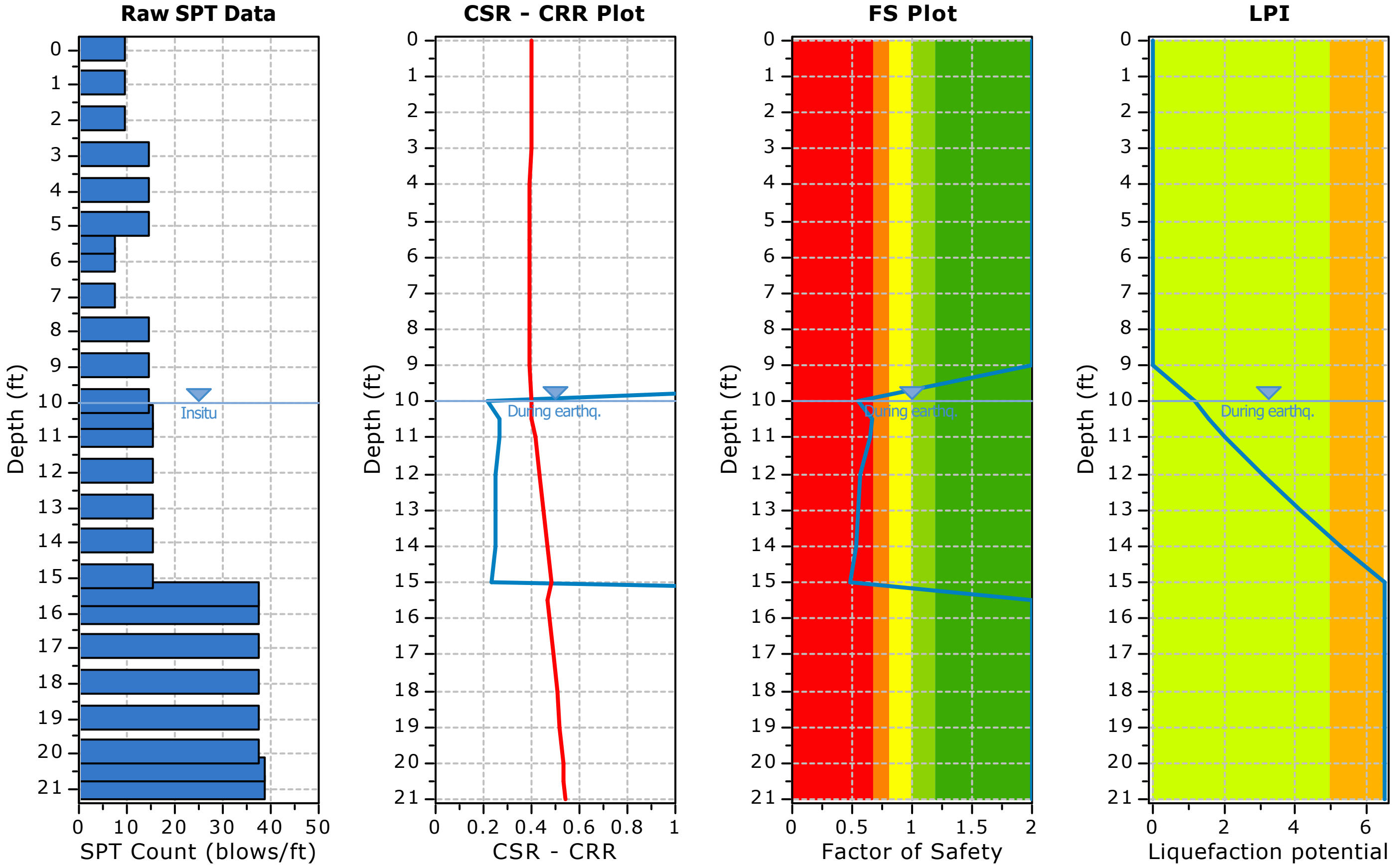
SPT BASED LIQUEFACTION ANALYSIS REPORT

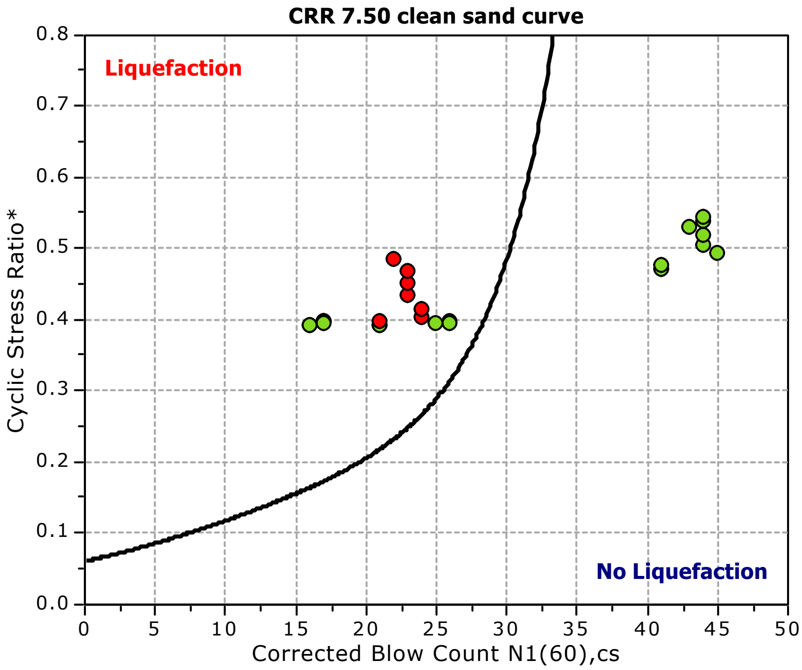
Project title : Proposed Additions **SPT Name: PG-1**

Location : 4836 East Mercer Way, Mercer Island

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	10.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	10.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.50
Borehole diameter:	150mm	Peak ground acceleration:	0.67 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		





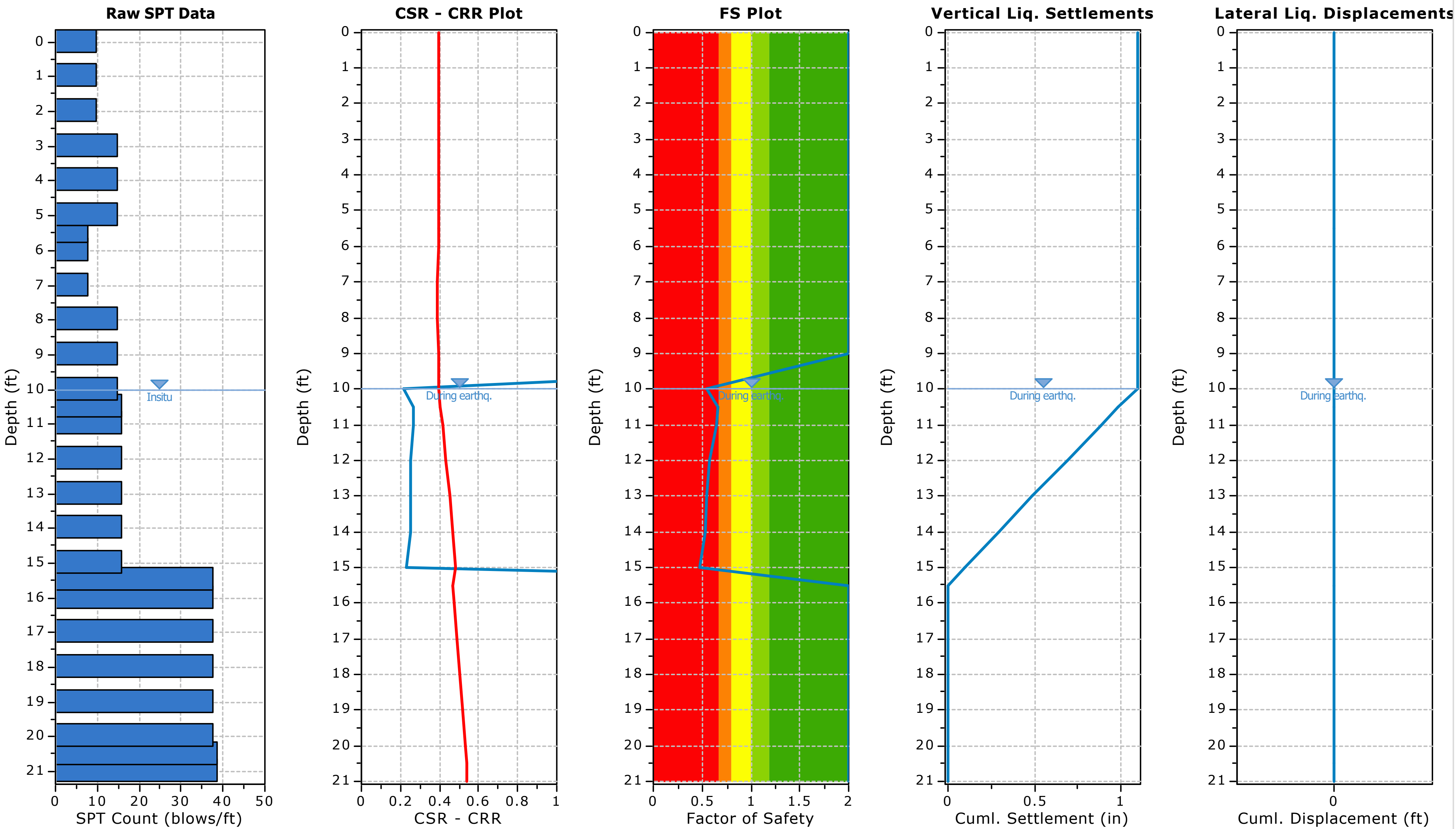
F.S. color scheme

- 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
0.01	10	20.00	120.00	1.00	Yes
1.00	10	20.00	120.00	1.00	Yes
2.00	10	20.00	120.00	1.00	Yes
3.00	15	80.00	120.00	1.00	Yes
4.00	15	80.00	120.00	1.00	Yes
5.00	15	80.00	120.00	0.50	Yes
5.50	8	50.00	120.00	0.50	Yes
6.00	8	50.00	120.00	1.00	Yes
7.00	8	50.00	120.00	1.00	Yes
8.00	15	20.00	120.00	1.00	Yes
9.00	15	20.00	120.00	1.00	Yes
10.00	15	20.00	120.00	0.50	Yes
10.50	16	40.00	125.00	0.50	Yes
11.00	16	40.00	125.00	1.00	Yes
12.00	16	40.00	125.00	1.00	Yes
13.00	16	40.00	125.00	1.00	Yes
14.00	16	40.00	125.00	1.00	Yes
15.00	16	40.00	125.00	0.50	Yes
15.50	38	20.00	125.00	0.50	Yes
16.00	38	20.00	125.00	1.00	Yes
17.00	38	20.00	125.00	1.00	Yes
18.00	38	20.00	125.00	1.00	Yes
19.00	38	20.00	125.00	1.00	Yes
20.00	38	20.00	125.00	0.50	Yes
20.50	39	20.00	125.00	0.50	Yes
21.00	39	20.00	125.00	0.50	Yes

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 (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
0.01	10	120.00	0.00	0.00	0.00	0.43	1.70	1.00	1.05	0.75	1.00	13	20.00	4.48	17	4.000
1.00	10	120.00	0.06	0.00	0.06	0.43	1.70	1.00	1.05	0.75	1.00	13	20.00	4.48	17	4.000
2.00	10	120.00	0.12	0.00	0.12	0.43	1.70	1.00	1.05	0.75	1.00	13	20.00	4.48	17	4.000
3.00	15	120.00	0.18	0.00	0.18	0.37	1.70	1.00	1.05	0.75	1.00	20	80.00	5.54	26	4.000
4.00	15	120.00	0.24	0.00	0.24	0.37	1.70	1.00	1.05	0.75	1.00	20	80.00	5.54	26	4.000
5.00	15	120.00	0.30	0.00	0.30	0.38	1.61	1.00	1.05	0.75	1.00	19	80.00	5.54	25	4.000
5.50	8	120.00	0.33	0.00	0.33	0.44	1.67	1.00	1.05	0.75	1.00	11	50.00	5.61	17	4.000
6.00	8	120.00	0.36	0.00	0.36	0.45	1.62	1.00	1.05	0.75	1.00	10	50.00	5.61	16	4.000
7.00	8	120.00	0.42	0.00	0.42	0.45	1.52	1.00	1.05	0.80	1.00	10	50.00	5.61	16	4.000
8.00	15	120.00	0.48	0.00	0.48	0.41	1.38	1.00	1.05	0.80	1.00	17	20.00	4.48	21	4.000
9.00	15	120.00	0.54	0.00	0.54	0.42	1.33	1.00	1.05	0.80	1.00	17	20.00	4.48	21	4.000

:: 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 (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
10.00	15	120.00	0.60	0.00	0.60	0.42	1.27	1.00	1.05	0.85	1.00	17	20.00	4.48	21	0.219
10.50	16	125.00	0.63	0.02	0.62	0.40	1.24	1.00	1.05	0.85	1.00	18	40.00	5.58	24	0.268
11.00	16	125.00	0.66	0.03	0.63	0.40	1.23	1.00	1.05	0.85	1.00	18	40.00	5.58	24	0.268
12.00	16	125.00	0.72	0.06	0.66	0.41	1.21	1.00	1.05	0.85	1.00	17	40.00	5.58	23	0.249
13.00	16	125.00	0.79	0.09	0.69	0.41	1.19	1.00	1.05	0.85	1.00	17	40.00	5.58	23	0.249
14.00	16	125.00	0.85	0.12	0.73	0.41	1.17	1.00	1.05	0.85	1.00	17	40.00	5.58	23	0.249
15.00	16	125.00	0.91	0.16	0.76	0.42	1.15	1.00	1.05	0.85	1.00	16	40.00	5.58	22	0.233
15.50	38	125.00	0.94	0.17	0.77	0.29	1.09	1.00	1.05	0.85	1.00	37	20.00	4.48	41	4.000
16.00	38	125.00	0.97	0.19	0.79	0.29	1.09	1.00	1.05	0.85	1.00	37	20.00	4.48	41	4.000
17.00	38	125.00	1.04	0.22	0.82	0.27	1.07	1.00	1.05	0.95	1.00	41	20.00	4.48	45	4.000
18.00	38	125.00	1.10	0.25	0.85	0.27	1.06	1.00	1.05	0.95	1.00	40	20.00	4.48	44	4.000
19.00	38	125.00	1.16	0.28	0.88	0.27	1.05	1.00	1.05	0.95	1.00	40	20.00	4.48	44	4.000
20.00	38	125.00	1.23	0.31	0.91	0.27	1.04	1.00	1.05	0.95	1.00	39	20.00	4.48	43	4.000
20.50	39	125.00	1.26	0.33	0.93	0.27	1.04	1.00	1.05	0.95	1.00	40	20.00	4.48	44	4.000
21.00	39	125.00	1.29	0.34	0.94	0.27	1.03	1.00	1.05	0.95	1.00	40	20.00	4.48	44	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
0.01	120.00	0.00	0.00	0.00	1.01	1.00	0.438	1.38	17	1.00	0.438	1.10	0.398	2.000
1.00	120.00	0.06	0.00	0.06	1.00	1.00	0.437	1.38	17	1.00	0.437	1.10	0.398	2.000
2.00	120.00	0.12	0.00	0.12	1.00	1.00	0.436	1.38	17	1.00	0.436	1.10	0.397	2.000
3.00	120.00	0.18	0.00	0.18	1.00	1.00	0.435	1.77	26	1.00	0.435	1.10	0.396	2.000
4.00	120.00	0.24	0.00	0.24	1.00	1.00	0.434	1.77	26	1.00	0.434	1.10	0.395	2.000
5.00	120.00	0.30	0.00	0.30	1.00	1.00	0.433	1.72	25	1.00	0.433	1.10	0.394	2.000
5.50	120.00	0.33	0.00	0.33	0.99	1.00	0.433	1.38	17	1.00	0.433	1.10	0.393	2.000
6.00	120.00	0.36	0.00	0.36	0.99	1.00	0.432	1.35	16	1.00	0.432	1.10	0.393	2.000
7.00	120.00	0.42	0.00	0.42	0.99	1.00	0.431	1.35	16	1.00	0.431	1.10	0.392	2.000
8.00	120.00	0.48	0.00	0.48	0.99	1.00	0.430	1.53	21	1.00	0.430	1.10	0.391	2.000
9.00	120.00	0.54	0.00	0.54	0.98	1.00	0.429	1.53	21	1.00	0.429	1.09	0.392	2.000
10.00	120.00	0.60	0.00	0.60	0.98	1.00	0.427	1.53	21	1.00	0.427	1.08	0.396	0.552
10.50	125.00	0.63	0.02	0.62	0.98	1.00	0.438	1.67	24	1.00	0.438	1.08	0.403	0.665
11.00	125.00	0.66	0.03	0.63	0.98	1.00	0.447	1.67	24	1.00	0.447	1.08	0.414	0.648
12.00	125.00	0.72	0.06	0.66	0.98	1.00	0.465	1.62	23	1.00	0.465	1.07	0.434	0.574
13.00	125.00	0.79	0.09	0.69	0.97	1.00	0.480	1.62	23	1.00	0.481	1.06	0.452	0.552
14.00	125.00	0.85	0.12	0.73	0.97	1.00	0.495	1.62	23	1.00	0.495	1.06	0.468	0.533

:: 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
15.00	125.00	0.91	0.16	0.76	0.97	1.00	0.507	1.58	22	1.00	0.507	1.05	0.484	0.482
15.50	125.00	0.94	0.17	0.77	0.96	1.00	0.513	2.20	41	1.00	0.513	1.09	0.469	2.000
16.00	125.00	0.97	0.19	0.79	0.96	1.00	0.519	2.20	41	1.00	0.519	1.09	0.477	2.000
17.00	125.00	1.04	0.22	0.82	0.96	1.00	0.529	2.20	45	1.00	0.529	1.08	0.492	2.000
18.00	125.00	1.10	0.25	0.85	0.96	1.00	0.538	2.20	44	1.00	0.538	1.06	0.506	2.000
19.00	125.00	1.16	0.28	0.88	0.95	1.00	0.546	2.20	44	1.00	0.546	1.05	0.519	2.000
20.00	125.00	1.23	0.31	0.91	0.95	1.00	0.554	2.20	43	1.00	0.554	1.04	0.531	2.000
20.50	125.00	1.26	0.33	0.93	0.95	1.00	0.557	2.20	44	1.00	0.557	1.04	0.537	2.000
21.00	125.00	1.29	0.34	0.94	0.94	1.00	0.561	2.20	44	1.00	0.561	1.03	0.542	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
0.01	2.000	0.00	10.00	0.99	0.00
1.00	2.000	0.00	9.85	0.99	0.00
2.00	2.000	0.00	9.70	1.00	0.00
3.00	2.000	0.00	9.54	1.00	0.00
4.00	2.000	0.00	9.39	1.00	0.00
5.00	2.000	0.00	9.24	1.00	0.00
5.50	2.000	0.00	9.16	0.50	0.00
6.00	2.000	0.00	9.09	0.50	0.00
7.00	2.000	0.00	8.93	1.00	0.00
8.00	2.000	0.00	8.78	1.00	0.00
9.00	2.000	0.00	8.63	1.00	0.00
10.00	0.552	0.45	8.48	1.00	1.16
10.50	0.665	0.34	8.40	0.50	0.43
11.00	0.648	0.35	8.32	0.50	0.45
12.00	0.574	0.43	8.17	1.00	1.06
13.00	0.552	0.45	8.02	1.00	1.10
14.00	0.533	0.47	7.87	1.00	1.12
15.00	0.482	0.52	7.71	1.00	1.22
15.50	2.000	0.00	7.64	0.50	0.00
16.00	2.000	0.00	7.56	0.50	0.00
17.00	2.000	0.00	7.41	1.00	0.00
18.00	2.000	0.00	7.26	1.00	0.00
19.00	2.000	0.00	7.10	1.00	0.00
20.00	2.000	0.00	6.95	1.00	0.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
20.50	2.000	0.00	6.88	0.50	0.00
21.00	2.000	0.00	6.80	0.50	0.00

Overall potential I_L : 6.53

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)
0.01	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	1.00	0.000
1.00	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.98	0.00	1.00	0.000
2.00	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.97	0.00	1.00	0.000
3.00	20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.95	0.00	1.00	0.000
4.00	20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.94	0.00	1.00	0.000
5.00	19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.92	0.00	0.50	0.000
5.50	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.92	0.00	0.50	0.000
6.00	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.91	0.00	1.00	0.000
7.00	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.89	0.00	1.00	0.000
8.00	17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.88	0.00	1.00	0.000
9.00	17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.86	0.00	1.00	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)
10.00	21	14.20	0.46	0.552	14.20	0.85	1.87	0.50	0.112	0.00
10.50	24	10.02	0.29	0.665	8.88	0.84	1.65	0.50	0.099	0.00
11.00	24	10.02	0.29	0.648	9.41	0.83	1.64	1.00	0.196	0.00
12.00	23	11.27	0.35	0.574	11.27	0.82	1.67	1.00	0.200	0.00
13.00	23	11.27	0.35	0.552	11.27	0.80	1.64	1.00	0.196	0.00
14.00	23	11.27	0.35	0.533	11.27	0.78	1.60	1.00	0.193	0.00
15.00	22	12.67	0.41	0.482	12.67	0.77	1.64	0.50	0.098	0.00
15.50	41	0.70	-0.88	2.000	0.00	0.76	0.00	0.50	0.000	0.00
16.00	41	0.70	-0.88	2.000	0.00	0.75	0.00	1.00	0.000	0.00
17.00	45	0.25	-1.19	2.000	0.00	0.74	0.00	1.00	0.000	0.00

:: 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)
18.00	44	0.34	-1.11	2.000	0.00	0.72	0.00	1.00	0.000	0.00
19.00	44	0.34	-1.11	2.000	0.00	0.71	0.00	1.00	0.000	0.00
20.00	43	0.44	-1.03	2.000	0.00	0.69	0.00	0.50	0.000	0.00
20.50	44	0.34	-1.11	2.000	0.00	0.68	0.00	0.50	0.000	0.00
21.00	44	0.34	-1.11	2.000	0.00	0.68	0.00	0.50	0.000	0.00

Cumulative settlements: 1.095 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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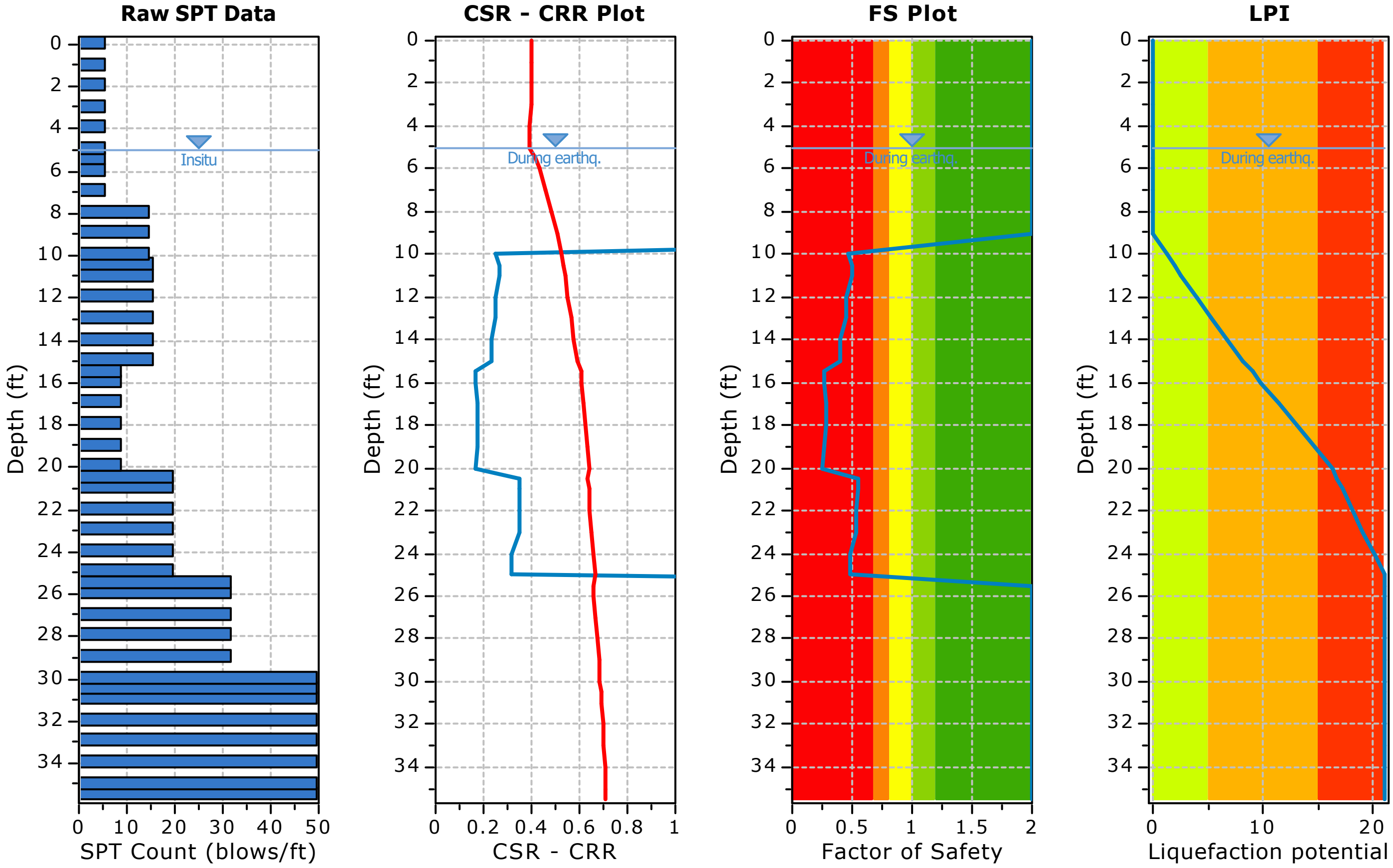
SPT BASED LIQUEFACTION ANALYSIS REPORT

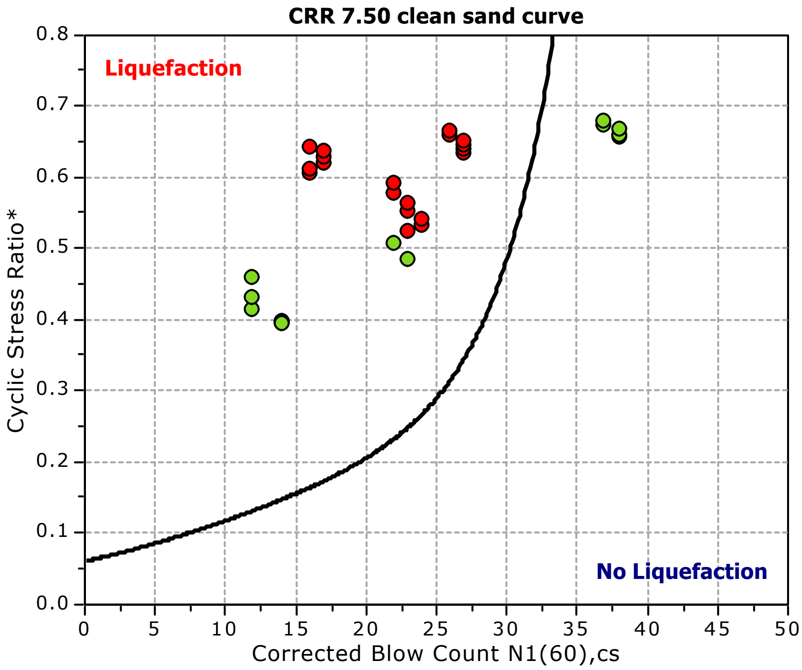
Project title : Proposed Additions **SPT Name: PG-2**

Location : 4836 East Mercer Way, Mercer Island

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	5.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	5.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.50
Borehole diameter:	150mm	Peak ground acceleration:	0.67 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		





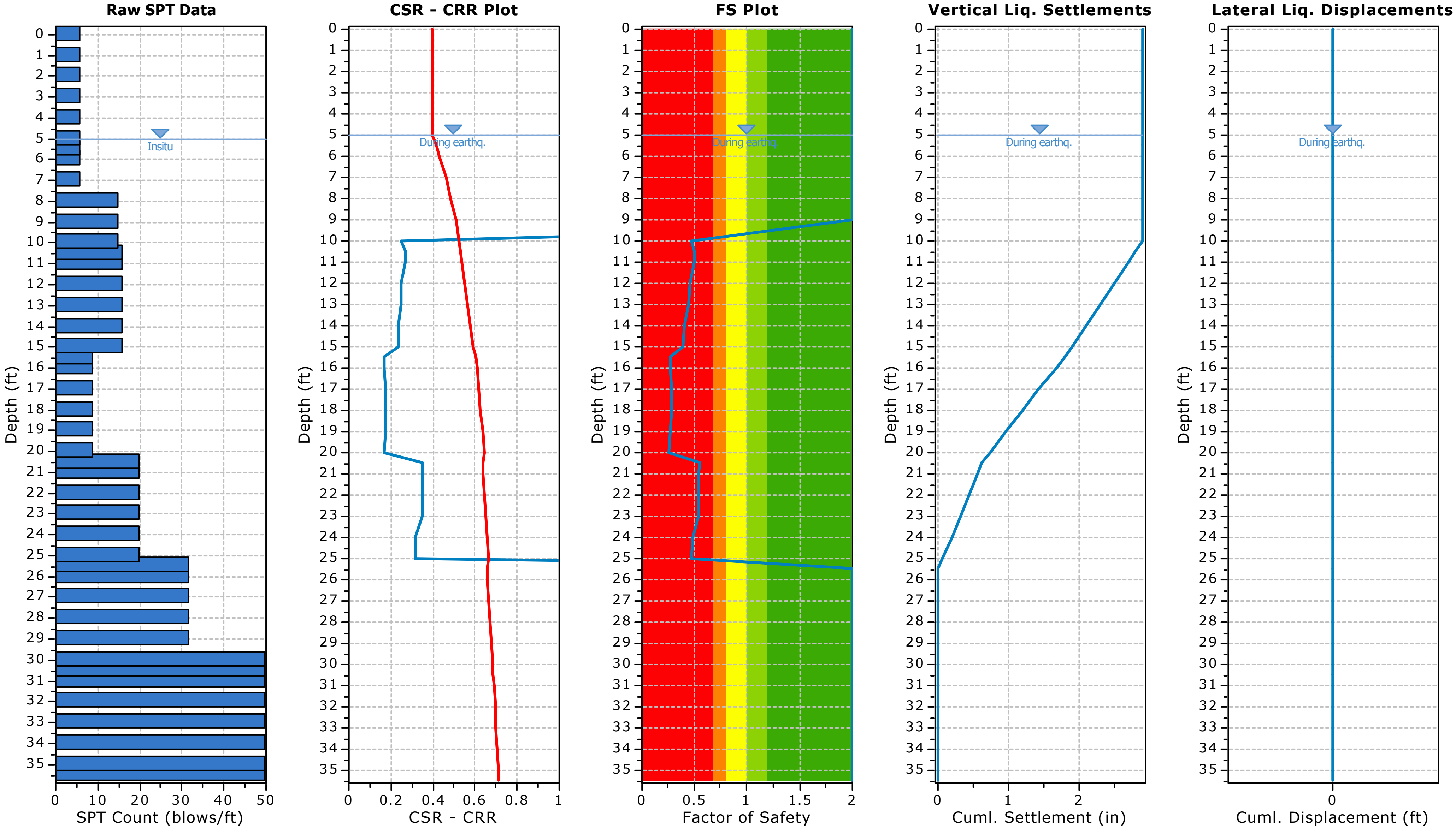
F.S. color scheme

- 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
0.01	6	80.00	120.00	1.00	No
1.00	6	80.00	120.00	1.00	No
2.00	6	80.00	120.00	1.00	No
3.00	6	50.00	120.00	1.00	No
4.00	6	50.00	120.00	1.00	No
5.00	6	50.00	120.00	0.50	No
5.50	6	20.00	120.00	0.50	No
6.00	6	20.00	120.00	1.00	No
7.00	6	20.00	120.00	1.00	No
8.00	15	20.00	120.00	1.00	No
9.00	15	20.00	120.00	1.00	No
10.00	15	20.00	120.00	0.50	Yes
10.50	16	20.00	125.00	0.50	Yes
11.00	16	20.00	125.00	1.00	Yes
12.00	16	20.00	125.00	1.00	Yes
13.00	16	20.00	125.00	1.00	Yes
14.00	16	20.00	125.00	1.00	Yes
15.00	16	20.00	125.00	0.50	Yes
15.50	9	60.00	125.00	0.50	Yes
16.00	9	60.00	125.00	1.00	Yes
17.00	9	60.00	125.00	1.00	Yes
18.00	9	60.00	125.00	1.00	Yes
19.00	9	60.00	125.00	1.00	Yes
20.00	9	60.00	125.00	0.50	Yes
20.50	20	30.00	125.00	0.50	Yes
21.00	20	30.00	125.00	1.00	Yes
22.00	20	30.00	125.00	1.00	Yes
23.00	20	30.00	125.00	1.00	Yes
24.00	20	30.00	125.00	1.00	Yes
25.00	20	30.00	125.00	0.50	Yes
25.50	32	30.00	130.00	0.50	Yes
26.00	32	30.00	130.00	1.00	Yes
27.00	32	30.00	130.00	1.00	Yes
28.00	32	30.00	130.00	1.00	Yes
29.00	32	30.00	130.00	1.00	Yes
30.00	50	20.00	130.00	0.50	No
30.50	50	20.00	130.00	0.50	No
31.00	50	20.00	130.00	1.00	No
32.00	50	20.00	130.00	1.00	No
33.00	50	20.00	130.00	1.00	No
34.00	50	20.00	130.00	1.00	No
35.00	50	20.00	130.00	0.50	No
35.50	50	10.00	130.00	0.50	No

:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy

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 (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
0.01	6	120.00	0.00	0.00	0.00	0.46	1.70	1.00	1.05	0.75	1.00	8	80.00	5.54	14	4.000
1.00	6	120.00	0.06	0.00	0.06	0.46	1.70	1.00	1.05	0.75	1.00	8	80.00	5.54	14	4.000
2.00	6	120.00	0.12	0.00	0.12	0.46	1.70	1.00	1.05	0.75	1.00	8	80.00	5.54	14	4.000
3.00	6	120.00	0.18	0.00	0.18	0.46	1.70	1.00	1.05	0.75	1.00	8	50.00	5.61	14	4.000
4.00	6	120.00	0.24	0.00	0.24	0.46	1.70	1.00	1.05	0.75	1.00	8	50.00	5.61	14	4.000
5.00	6	120.00	0.30	0.00	0.30	0.46	1.70	1.00	1.05	0.75	1.00	8	50.00	5.61	14	4.000
5.50	6	120.00	0.33	0.02	0.31	0.48	1.70	1.00	1.05	0.75	1.00	8	20.00	4.48	12	4.000
6.00	6	120.00	0.36	0.03	0.33	0.48	1.70	1.00	1.05	0.75	1.00	8	20.00	4.48	12	4.000
7.00	6	120.00	0.42	0.06	0.36	0.48	1.68	1.00	1.05	0.80	1.00	8	20.00	4.48	12	4.000
8.00	15	120.00	0.48	0.09	0.39	0.40	1.49	1.00	1.05	0.80	1.00	19	20.00	4.48	23	4.000
9.00	15	120.00	0.54	0.12	0.42	0.40	1.46	1.00	1.05	0.80	1.00	18	20.00	4.48	22	4.000
10.00	15	120.00	0.60	0.16	0.44	0.40	1.41	1.00	1.05	0.85	1.00	19	20.00	4.48	23	0.249
10.50	16	125.00	0.63	0.17	0.46	0.39	1.39	1.00	1.05	0.85	1.00	20	20.00	4.48	24	0.268
11.00	16	125.00	0.66	0.19	0.48	0.39	1.37	1.00	1.05	0.85	1.00	20	20.00	4.48	24	0.268
12.00	16	125.00	0.72	0.22	0.51	0.40	1.34	1.00	1.05	0.85	1.00	19	20.00	4.48	23	0.249
13.00	16	125.00	0.79	0.25	0.54	0.40	1.31	1.00	1.05	0.85	1.00	19	20.00	4.48	23	0.249
14.00	16	125.00	0.85	0.28	0.57	0.41	1.29	1.00	1.05	0.85	1.00	18	20.00	4.48	22	0.233
15.00	16	125.00	0.91	0.31	0.60	0.41	1.26	1.00	1.05	0.85	1.00	18	20.00	4.48	22	0.233
15.50	9	125.00	0.94	0.33	0.62	0.46	1.28	1.00	1.05	0.85	1.00	10	60.00	5.60	16	0.165
16.00	9	125.00	0.97	0.34	0.63	0.46	1.27	1.00	1.05	0.85	1.00	10	60.00	5.60	16	0.165
17.00	9	125.00	1.04	0.37	0.66	0.46	1.24	1.00	1.05	0.95	1.00	11	60.00	5.60	17	0.174
18.00	9	125.00	1.10	0.41	0.69	0.46	1.21	1.00	1.05	0.95	1.00	11	60.00	5.60	17	0.174
19.00	9	125.00	1.16	0.44	0.73	0.46	1.19	1.00	1.05	0.95	1.00	11	60.00	5.60	17	0.174
20.00	9	125.00	1.23	0.47	0.76	0.47	1.17	1.00	1.05	0.95	1.00	10	60.00	5.60	16	0.165
20.50	20	125.00	1.26	0.48	0.77	0.37	1.12	1.00	1.05	0.95	1.00	22	30.00	5.36	27	0.347
21.00	20	125.00	1.29	0.50	0.79	0.38	1.12	1.00	1.05	0.95	1.00	22	30.00	5.36	27	0.347
22.00	20	125.00	1.35	0.53	0.82	0.38	1.10	1.00	1.05	0.95	1.00	22	30.00	5.36	27	0.347
23.00	20	125.00	1.41	0.56	0.85	0.38	1.09	1.00	1.05	0.95	1.00	22	30.00	5.36	27	0.347
24.00	20	125.00	1.48	0.59	0.88	0.38	1.07	1.00	1.05	0.95	1.00	21	30.00	5.36	26	0.316
25.00	20	125.00	1.54	0.62	0.91	0.39	1.06	1.00	1.05	0.95	1.00	21	30.00	5.36	26	0.316
25.50	32	130.00	1.57	0.64	0.93	0.31	1.04	1.00	1.05	0.95	1.00	33	30.00	5.36	38	4.000
26.00	32	130.00	1.60	0.66	0.95	0.31	1.03	1.00	1.05	0.95	1.00	33	30.00	5.36	38	4.000
27.00	32	130.00	1.67	0.69	0.98	0.31	1.02	1.00	1.05	0.95	1.00	33	30.00	5.36	38	4.000
28.00	32	130.00	1.73	0.72	1.01	0.31	1.01	1.00	1.05	0.95	1.00	32	30.00	5.36	37	4.000
29.00	32	130.00	1.80	0.75	1.05	0.31	1.00	1.00	1.05	0.95	1.00	32	30.00	5.36	37	4.000
30.00	50	130.00	1.86	0.78	1.08	0.26	0.99	1.00	1.05	1.00	1.00	52	20.00	4.48	56	4.000
30.50	50	130.00	1.90	0.80	1.10	0.26	0.99	1.00	1.05	1.00	1.00	52	20.00	4.48	56	4.000

:: 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 (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
31.00	50	130.00	1.93	0.81	1.12	0.26	0.99	1.00	1.05	1.00	1.00	52	20.00	4.48	56	4.000
32.00	50	130.00	1.99	0.84	1.15	0.26	0.98	1.00	1.05	1.00	1.00	51	20.00	4.48	55	4.000
33.00	50	130.00	2.06	0.87	1.18	0.26	0.97	1.00	1.05	1.00	1.00	51	20.00	4.48	55	4.000
34.00	50	130.00	2.12	0.90	1.22	0.26	0.96	1.00	1.05	1.00	1.00	51	20.00	4.48	55	4.000
35.00	50	130.00	2.19	0.94	1.25	0.26	0.96	1.00	1.05	1.00	1.00	50	20.00	4.48	54	4.000
35.50	50	130.00	2.22	0.95	1.27	0.26	0.95	1.00	1.05	1.00	1.00	50	10.00	1.15	51	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
0.01	120.00	0.00	0.00	0.00	1.01	1.00	0.438	1.29	14	1.00	0.438	1.10	0.398	2.000
1.00	120.00	0.06	0.00	0.06	1.00	1.00	0.437	1.29	14	1.00	0.437	1.10	0.398	2.000
2.00	120.00	0.12	0.00	0.12	1.00	1.00	0.436	1.29	14	1.00	0.436	1.10	0.397	2.000
3.00	120.00	0.18	0.00	0.18	1.00	1.00	0.435	1.29	14	1.00	0.435	1.10	0.396	2.000
4.00	120.00	0.24	0.00	0.24	1.00	1.00	0.434	1.29	14	1.00	0.434	1.10	0.395	2.000
5.00	120.00	0.30	0.00	0.30	1.00	1.00	0.433	1.29	14	1.00	0.433	1.10	0.394	2.000
5.50	120.00	0.33	0.02	0.31	0.99	1.00	0.454	1.24	12	1.00	0.454	1.10	0.413	2.000
6.00	120.00	0.36	0.03	0.33	0.99	1.00	0.473	1.24	12	1.00	0.473	1.10	0.430	2.000
7.00	120.00	0.42	0.06	0.36	0.99	1.00	0.506	1.24	12	1.00	0.506	1.10	0.460	2.000
8.00	120.00	0.48	0.09	0.39	0.99	1.00	0.534	1.62	23	1.00	0.534	1.10	0.485	2.000
9.00	120.00	0.54	0.12	0.42	0.98	1.00	0.558	1.58	22	1.00	0.558	1.10	0.507	2.000
10.00	120.00	0.60	0.16	0.44	0.98	1.00	0.578	1.62	23	1.00	0.578	1.10	0.525	0.475
10.50	125.00	0.63	0.17	0.46	0.98	1.00	0.586	1.67	24	1.00	0.586	1.10	0.533	0.503
11.00	125.00	0.66	0.19	0.48	0.98	1.00	0.594	1.67	24	1.00	0.594	1.10	0.540	0.497
12.00	125.00	0.72	0.22	0.51	0.98	1.00	0.608	1.62	23	1.00	0.608	1.10	0.553	0.451
13.00	125.00	0.79	0.25	0.54	0.97	1.00	0.620	1.62	23	1.00	0.620	1.10	0.564	0.443
14.00	125.00	0.85	0.28	0.57	0.97	1.00	0.630	1.58	22	1.00	0.630	1.09	0.578	0.403
15.00	125.00	0.91	0.31	0.60	0.97	1.00	0.639	1.58	22	1.00	0.639	1.08	0.591	0.394
15.50	125.00	0.94	0.33	0.62	0.96	1.00	0.643	1.35	16	1.00	0.643	1.06	0.605	0.272
16.00	125.00	0.97	0.34	0.63	0.96	1.00	0.647	1.35	16	1.00	0.647	1.06	0.611	0.270
17.00	125.00	1.04	0.37	0.66	0.96	1.00	0.653	1.38	17	1.00	0.653	1.06	0.619	0.281
18.00	125.00	1.10	0.41	0.69	0.96	1.00	0.659	1.38	17	1.00	0.659	1.05	0.627	0.277
19.00	125.00	1.16	0.44	0.73	0.95	1.00	0.664	1.38	17	1.00	0.664	1.04	0.635	0.274
20.00	125.00	1.23	0.47	0.76	0.95	1.00	0.668	1.35	16	1.00	0.668	1.04	0.643	0.256
20.50	125.00	1.26	0.48	0.77	0.95	1.00	0.670	1.82	27	1.00	0.670	1.06	0.635	0.546
21.00	125.00	1.29	0.50	0.79	0.94	1.00	0.672	1.82	27	1.00	0.672	1.05	0.638	0.543

:: 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
22.00	125.00	1.35	0.53	0.82	0.94	1.00	0.675	1.82	27	1.00	0.675	1.05	0.645	0.537
23.00	125.00	1.41	0.56	0.85	0.94	1.00	0.677	1.82	27	1.00	0.677	1.04	0.652	0.531
24.00	125.00	1.48	0.59	0.88	0.93	1.00	0.679	1.77	26	1.00	0.679	1.03	0.659	0.479
25.00	125.00	1.54	0.62	0.91	0.93	1.00	0.681	1.77	26	1.00	0.681	1.02	0.664	0.475
25.50	130.00	1.57	0.64	0.93	0.93	1.00	0.681	2.20	38	1.00	0.681	1.04	0.656	2.000
26.00	130.00	1.60	0.66	0.95	0.92	1.00	0.681	2.20	38	1.00	0.681	1.03	0.660	2.000
27.00	130.00	1.67	0.69	0.98	0.92	1.00	0.681	2.20	38	1.00	0.681	1.02	0.667	2.000
28.00	130.00	1.73	0.72	1.01	0.92	1.00	0.681	2.20	37	1.00	0.681	1.01	0.673	2.000
29.00	130.00	1.80	0.75	1.05	0.91	1.00	0.681	2.20	37	1.00	0.681	1.00	0.679	2.000
30.00	130.00	1.86	0.78	1.08	0.91	1.00	0.680	2.20	56	1.00	0.680	0.99	0.685	2.000
30.50	130.00	1.90	0.80	1.10	0.91	1.00	0.680	2.20	56	1.00	0.680	0.99	0.688	2.000
31.00	130.00	1.93	0.81	1.12	0.90	1.00	0.680	2.20	56	1.00	0.680	0.98	0.691	2.000
32.00	130.00	1.99	0.84	1.15	0.90	1.00	0.679	2.20	55	1.00	0.679	0.98	0.696	2.000
33.00	130.00	2.06	0.87	1.18	0.90	1.00	0.678	2.20	55	1.00	0.678	0.97	0.701	2.000
34.00	130.00	2.12	0.90	1.22	0.89	1.00	0.676	2.20	55	1.00	0.676	0.96	0.706	2.000
35.00	130.00	2.19	0.94	1.25	0.89	1.00	0.675	2.20	54	1.00	0.675	0.95	0.710	2.000
35.50	130.00	2.22	0.95	1.27	0.88	1.00	0.674	2.20	51	1.00	0.674	0.95	0.712	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
0.01	2.000	0.00	10.00	0.99	0.00
1.00	2.000	0.00	9.85	0.99	0.00
2.00	2.000	0.00	9.70	1.00	0.00
3.00	2.000	0.00	9.54	1.00	0.00
4.00	2.000	0.00	9.39	1.00	0.00
5.00	2.000	0.00	9.24	1.00	0.00
5.50	2.000	0.00	9.16	0.50	0.00
6.00	2.000	0.00	9.09	0.50	0.00
7.00	2.000	0.00	8.93	1.00	0.00
8.00	2.000	0.00	8.78	1.00	0.00
9.00	2.000	0.00	8.63	1.00	0.00
10.00	0.475	0.53	8.48	1.00	1.36
10.50	0.503	0.50	8.40	0.50	0.64
11.00	0.497	0.50	8.32	0.50	0.64
12.00	0.451	0.55	8.17	1.00	1.37
13.00	0.443	0.56	8.02	1.00	1.36

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
14.00	0.403	0.60	7.87	1.00	1.43
15.00	0.394	0.61	7.71	1.00	1.42
15.50	0.272	0.73	7.64	0.50	0.85
16.00	0.270	0.73	7.56	0.50	0.84
17.00	0.281	0.72	7.41	1.00	1.62
18.00	0.277	0.72	7.26	1.00	1.60
19.00	0.274	0.73	7.10	1.00	1.57
20.00	0.256	0.74	6.95	1.00	1.58
20.50	0.546	0.45	6.88	0.50	0.48
21.00	0.543	0.46	6.80	0.50	0.47
22.00	0.537	0.46	6.65	1.00	0.94
23.00	0.531	0.47	6.49	1.00	0.93
24.00	0.479	0.52	6.34	1.00	1.01
25.00	0.475	0.52	6.19	1.00	0.99
25.50	2.000	0.00	6.11	0.50	0.00
26.00	2.000	0.00	6.04	0.50	0.00
27.00	2.000	0.00	5.89	1.00	0.00
28.00	2.000	0.00	5.73	1.00	0.00
29.00	2.000	0.00	5.58	1.00	0.00
30.00	2.000	0.00	5.43	1.00	0.00
30.50	2.000	0.00	5.35	0.50	0.00
31.00	2.000	0.00	5.28	0.50	0.00
32.00	2.000	0.00	5.12	1.00	0.00
33.00	2.000	0.00	4.97	1.00	0.00
34.00	2.000	0.00	4.82	1.00	0.00
35.00	2.000	0.00	4.67	1.00	0.00
35.50	2.000	0.00	4.59	0.50	0.00

Overall potential I_L : 21.09

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)
0.01	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
1.00	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
2.00	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
3.00	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
4.00	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	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)

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)	$S_{v\text{-}1D}$ (in)	LDI (ft)
5.00	14	0.00	0.00	2.000	0.00	0.00	0.00	0.50	0.000	0.00
5.50	12	0.00	0.00	2.000	0.00	0.00	0.00	0.50	0.000	0.00
6.00	12	0.00	0.00	2.000	0.00	0.00	0.00	1.00	0.000	0.00
7.00	12	0.00	0.00	2.000	0.00	0.00	0.00	1.00	0.000	0.00
8.00	23	0.00	0.00	2.000	0.00	0.00	0.00	1.00	0.000	0.00
9.00	22	0.00	0.00	2.000	0.00	0.00	0.00	1.00	0.000	0.00
10.00	23	11.27	0.35	0.475	11.27	0.85	1.73	0.50	0.104	0.00
10.50	24	10.02	0.29	0.503	10.02	0.84	1.65	0.50	0.099	0.00
11.00	24	10.02	0.29	0.497	10.02	0.83	1.64	1.00	0.196	0.00
12.00	23	11.27	0.35	0.451	11.27	0.82	1.67	1.00	0.200	0.00
13.00	23	11.27	0.35	0.443	11.27	0.80	1.64	1.00	0.196	0.00
14.00	22	12.67	0.41	0.403	12.67	0.78	1.67	1.00	0.200	0.00
15.00	22	12.67	0.41	0.394	12.67	0.77	1.64	0.50	0.098	0.00
15.50	16	24.69	0.71	0.272	24.69	0.76	2.09	0.50	0.125	0.00
16.00	16	24.69	0.71	0.270	24.69	0.75	2.07	1.00	0.248	0.00
17.00	17	22.15	0.67	0.281	22.15	0.74	1.94	1.00	0.232	0.00
18.00	17	22.15	0.67	0.277	22.15	0.72	1.90	1.00	0.227	0.00
19.00	17	22.15	0.67	0.274	22.15	0.71	1.85	1.00	0.223	0.00
20.00	16	24.69	0.71	0.256	24.69	0.69	1.90	0.50	0.114	0.00
20.50	27	6.92	0.11	0.546	6.92	0.68	1.04	0.50	0.063	0.00
21.00	27	6.92	0.11	0.543	6.92	0.68	1.03	1.00	0.124	0.00
22.00	27	6.92	0.11	0.537	6.92	0.66	1.01	1.00	0.121	0.00
23.00	27	6.92	0.11	0.531	6.92	0.65	0.99	1.00	0.118	0.00
24.00	26	7.85	0.17	0.479	7.85	0.63	1.13	1.00	0.136	0.00
25.00	26	7.85	0.17	0.475	7.85	0.62	1.10	0.50	0.066	0.00
25.50	38	1.30	-0.65	2.000	0.00	0.61	0.00	0.50	0.000	0.00
26.00	38	1.30	-0.65	2.000	0.00	0.60	0.00	1.00	0.000	0.00
27.00	38	1.30	-0.65	2.000	0.00	0.58	0.00	1.00	0.000	0.00
28.00	37	1.56	-0.58	2.000	0.00	0.57	0.00	1.00	0.000	0.00
29.00	37	1.56	-0.58	2.000	0.00	0.55	0.00	1.00	0.000	0.00
30.00	56	0.00	0.00	2.000	0.00	0.00	0.00	0.50	0.000	0.00
30.50	56	0.00	0.00	2.000	0.00	0.00	0.00	0.50	0.000	0.00
31.00	56	0.00	0.00	2.000	0.00	0.00	0.00	1.00	0.000	0.00

:: 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)
32.00	55	0.00	0.00	2.000	0.00	0.00	0.00	1.00	0.000	0.00
33.00	55	0.00	0.00	2.000	0.00	0.00	0.00	1.00	0.000	0.00
34.00	55	0.00	0.00	2.000	0.00	0.00	0.00	1.00	0.000	0.00
35.00	54	0.00	0.00	2.000	0.00	0.00	0.00	0.50	0.000	0.00
35.50	51	0.00	0.00	2.000	0.00	0.00	0.00	0.50	0.000	0.00

Cumulative settlements: 2.891 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)

PanGEO
GEOTECHNICAL CONSULTANTS

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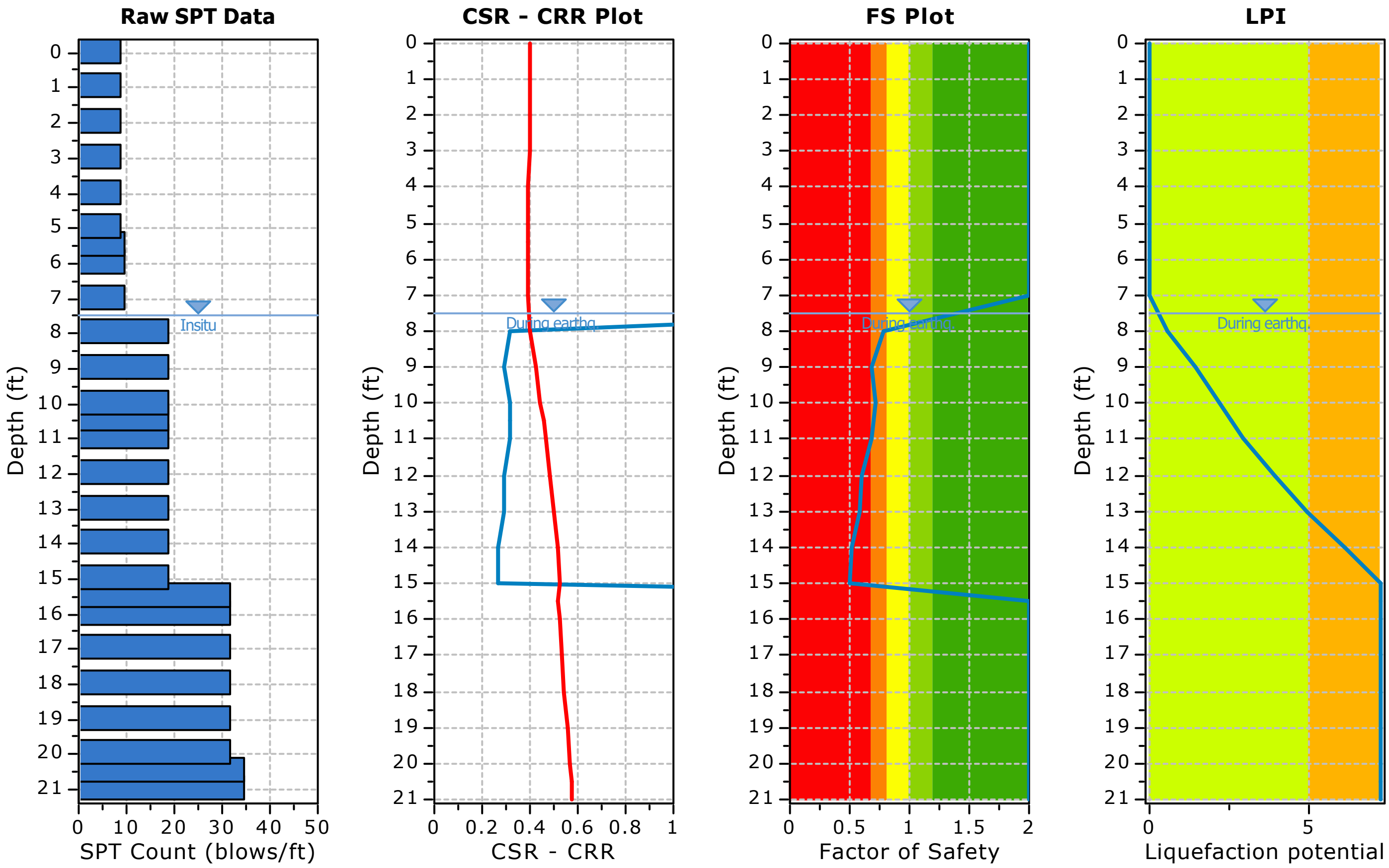
SPT BASED LIQUEFACTION ANALYSIS REPORT

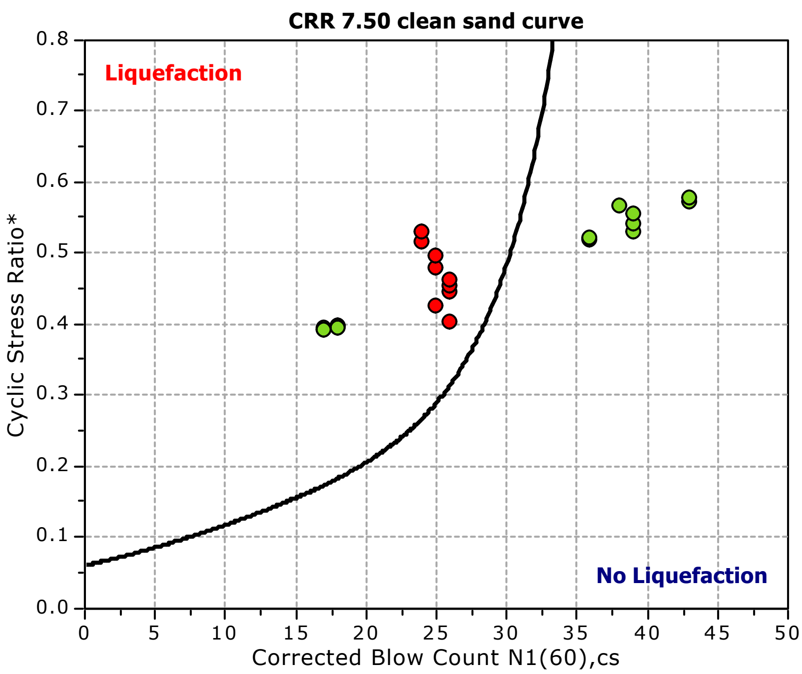
Project title : Proposed Additions — **SPT Name: PG-3**

Location : 4836 East Mercer Way, Mercer Island

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	7.50 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	7.50 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.50
Borehole diameter:	150mm	Peak ground acceleration:	0.67 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		

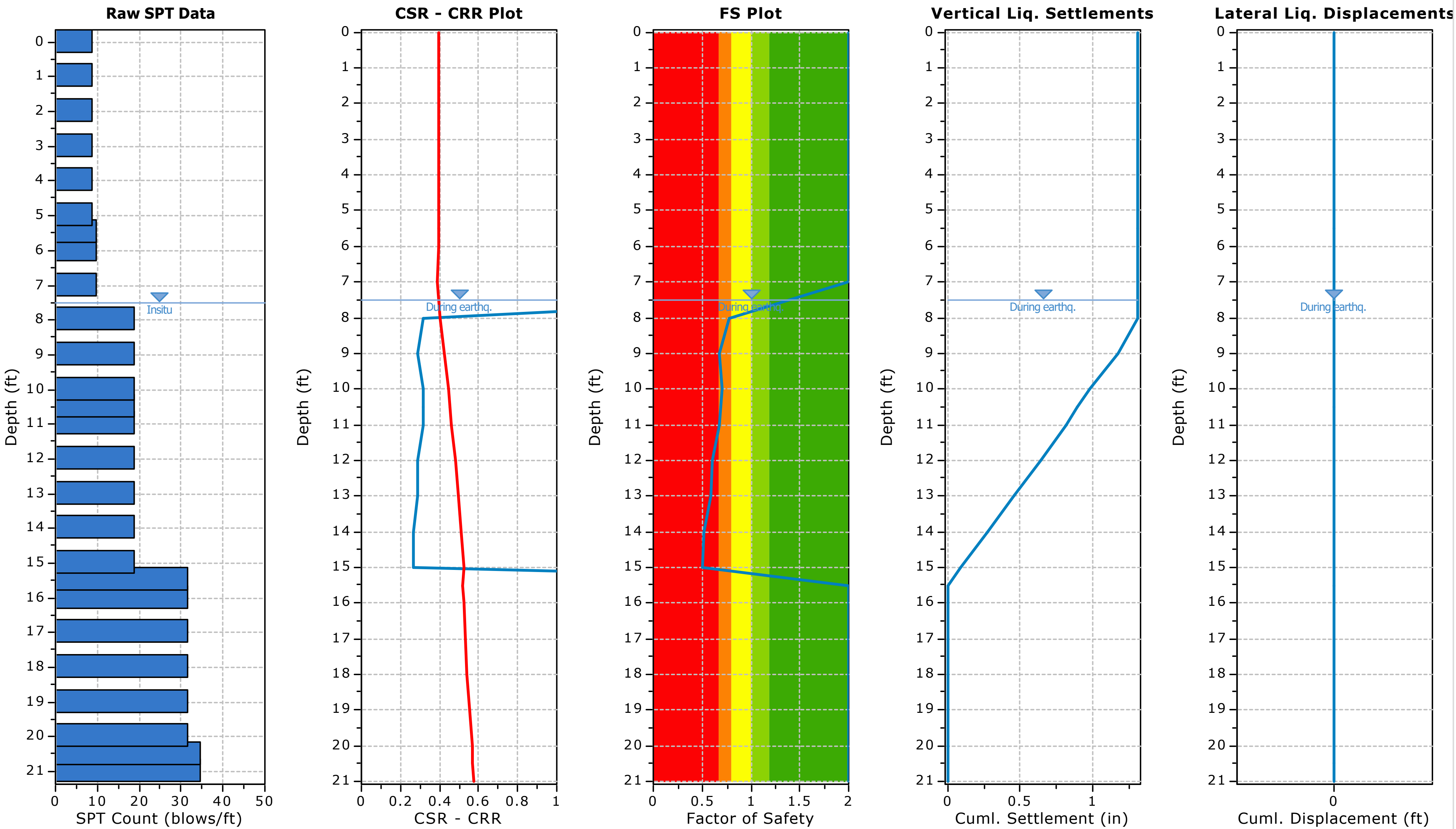




F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

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
0.01	9	60.00	120.00	1.00	Yes
1.00	9	60.00	120.00	1.00	Yes
2.00	9	60.00	120.00	1.00	Yes
3.00	9	40.00	120.00	1.00	Yes
4.00	9	40.00	120.00	1.00	Yes
5.00	9	40.00	120.00	0.50	Yes
5.50	10	20.00	120.00	0.50	Yes
6.00	10	20.00	120.00	1.00	Yes
7.00	10	20.00	120.00	1.00	Yes
8.00	19	20.00	125.00	1.00	Yes
9.00	19	20.00	125.00	1.00	Yes
10.00	19	20.00	125.00	0.50	Yes
10.50	19	20.00	125.00	0.50	Yes
11.00	19	20.00	125.00	1.00	Yes
12.00	19	20.00	125.00	1.00	Yes
13.00	19	20.00	125.00	1.00	Yes
14.00	19	20.00	125.00	1.00	Yes
15.00	19	20.00	125.00	0.50	Yes
15.50	32	20.00	125.00	0.50	Yes
16.00	32	20.00	125.00	1.00	Yes
17.00	32	20.00	125.00	1.00	Yes
18.00	32	20.00	125.00	1.00	Yes
19.00	32	20.00	125.00	1.00	Yes
20.00	32	20.00	125.00	0.50	Yes
20.50	35	50.00	125.00	0.50	Yes
21.00	35	50.00	125.00	0.50	Yes

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 (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
0.01	9	120.00	0.00	0.00	0.00	0.43	1.70	1.00	1.05	0.75	1.00	12	60.00	5.60	18	4.000
1.00	9	120.00	0.06	0.00	0.06	0.43	1.70	1.00	1.05	0.75	1.00	12	60.00	5.60	18	4.000
2.00	9	120.00	0.12	0.00	0.12	0.43	1.70	1.00	1.05	0.75	1.00	12	60.00	5.60	18	4.000
3.00	9	120.00	0.18	0.00	0.18	0.43	1.70	1.00	1.05	0.75	1.00	12	40.00	5.58	18	4.000
4.00	9	120.00	0.24	0.00	0.24	0.43	1.70	1.00	1.05	0.75	1.00	12	40.00	5.58	18	4.000
5.00	9	120.00	0.30	0.00	0.30	0.43	1.70	1.00	1.05	0.75	1.00	12	40.00	5.58	18	4.000
5.50	10	120.00	0.33	0.00	0.33	0.44	1.66	1.00	1.05	0.75	1.00	13	20.00	4.48	17	4.000
6.00	10	120.00	0.36	0.00	0.36	0.44	1.61	1.00	1.05	0.75	1.00	13	20.00	4.48	17	4.000
7.00	10	120.00	0.42	0.00	0.42	0.45	1.51	1.00	1.05	0.80	1.00	13	20.00	4.48	17	4.000
8.00	19	125.00	0.48	0.02	0.47	0.38	1.36	1.00	1.05	0.80	1.00	22	20.00	4.48	26	0.316
9.00	19	125.00	0.55	0.05	0.50	0.38	1.33	1.00	1.05	0.80	1.00	21	20.00	4.48	25	0.290

:: 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 (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
10.00	19	125.00	0.61	0.08	0.53	0.38	1.30	1.00	1.05	0.85	1.00	22	20.00	4.48	26	0.316
10.50	19	125.00	0.64	0.09	0.55	0.38	1.29	1.00	1.05	0.85	1.00	22	20.00	4.48	26	0.316
11.00	19	125.00	0.67	0.11	0.56	0.38	1.27	1.00	1.05	0.85	1.00	22	20.00	4.48	26	0.316
12.00	19	125.00	0.73	0.14	0.59	0.39	1.25	1.00	1.05	0.85	1.00	21	20.00	4.48	25	0.290
13.00	19	125.00	0.80	0.17	0.62	0.39	1.23	1.00	1.05	0.85	1.00	21	20.00	4.48	25	0.290
14.00	19	125.00	0.86	0.20	0.65	0.39	1.21	1.00	1.05	0.85	1.00	20	20.00	4.48	24	0.268
15.00	19	125.00	0.92	0.23	0.69	0.40	1.19	1.00	1.05	0.85	1.00	20	20.00	4.48	24	0.268
15.50	32	125.00	0.95	0.25	0.70	0.31	1.14	1.00	1.05	0.85	1.00	32	20.00	4.48	36	4.000
16.00	32	125.00	0.98	0.27	0.72	0.31	1.13	1.00	1.05	0.85	1.00	32	20.00	4.48	36	4.000
17.00	32	125.00	1.04	0.30	0.75	0.30	1.11	1.00	1.05	0.95	1.00	35	20.00	4.48	39	4.000
18.00	32	125.00	1.11	0.33	0.78	0.30	1.10	1.00	1.05	0.95	1.00	35	20.00	4.48	39	4.000
19.00	32	125.00	1.17	0.36	0.81	0.30	1.08	1.00	1.05	0.95	1.00	35	20.00	4.48	39	4.000
20.00	32	125.00	1.23	0.39	0.84	0.30	1.07	1.00	1.05	0.95	1.00	34	20.00	4.48	38	4.000
20.50	35	125.00	1.26	0.41	0.86	0.28	1.06	1.00	1.05	0.95	1.00	37	50.00	5.61	43	4.000
21.00	35	125.00	1.29	0.42	0.87	0.28	1.06	1.00	1.05	0.95	1.00	37	50.00	5.61	43	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
0.01	120.00	0.00	0.00	0.00	1.01	1.00	0.438	1.42	18	1.00	0.438	1.10	0.398	2.000
1.00	120.00	0.06	0.00	0.06	1.00	1.00	0.437	1.42	18	1.00	0.437	1.10	0.398	2.000
2.00	120.00	0.12	0.00	0.12	1.00	1.00	0.436	1.42	18	1.00	0.436	1.10	0.397	2.000
3.00	120.00	0.18	0.00	0.18	1.00	1.00	0.435	1.42	18	1.00	0.435	1.10	0.396	2.000
4.00	120.00	0.24	0.00	0.24	1.00	1.00	0.434	1.42	18	1.00	0.434	1.10	0.395	2.000
5.00	120.00	0.30	0.00	0.30	1.00	1.00	0.433	1.42	18	1.00	0.433	1.10	0.394	2.000
5.50	120.00	0.33	0.00	0.33	0.99	1.00	0.433	1.38	17	1.00	0.433	1.10	0.393	2.000
6.00	120.00	0.36	0.00	0.36	0.99	1.00	0.432	1.38	17	1.00	0.432	1.10	0.393	2.000
7.00	120.00	0.42	0.00	0.42	0.99	1.00	0.431	1.38	17	1.00	0.431	1.10	0.392	2.000
8.00	125.00	0.48	0.02	0.47	0.99	1.00	0.444	1.77	26	1.00	0.444	1.10	0.404	0.782
9.00	125.00	0.55	0.05	0.50	0.98	1.00	0.469	1.72	25	1.00	0.469	1.10	0.426	0.680
10.00	125.00	0.61	0.08	0.53	0.98	1.00	0.490	1.77	26	1.00	0.490	1.10	0.446	0.708
10.50	125.00	0.64	0.09	0.55	0.98	1.00	0.500	1.77	26	1.00	0.500	1.10	0.455	0.695
11.00	125.00	0.67	0.11	0.56	0.98	1.00	0.509	1.77	26	1.00	0.509	1.10	0.463	0.682
12.00	125.00	0.73	0.14	0.59	0.98	1.00	0.525	1.72	25	1.00	0.525	1.09	0.480	0.604
13.00	125.00	0.80	0.17	0.62	0.97	1.00	0.540	1.72	25	1.00	0.540	1.09	0.497	0.583
14.00	125.00	0.86	0.20	0.65	0.97	1.00	0.553	1.67	24	1.00	0.553	1.07	0.514	0.522

:: 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	
15.00	125.00	0.92	0.23	0.69	0.97	1.00	0.564	1.67	24	1.00	0.564	1.07	0.528	0.508	●
15.50	125.00	0.95	0.25	0.70	0.96	1.00	0.569	2.20	36	1.00	0.569	1.10	0.517	2.000	●
16.00	125.00	0.98	0.27	0.72	0.96	1.00	0.574	2.20	36	1.00	0.574	1.10	0.522	2.000	●
17.00	125.00	1.04	0.30	0.75	0.96	1.00	0.583	2.20	39	1.00	0.583	1.10	0.530	2.000	●
18.00	125.00	1.11	0.33	0.78	0.96	1.00	0.591	2.20	39	1.00	0.591	1.09	0.542	2.000	●
19.00	125.00	1.17	0.36	0.81	0.95	1.00	0.598	2.20	39	1.00	0.598	1.08	0.554	2.000	●
20.00	125.00	1.23	0.39	0.84	0.95	1.00	0.604	2.20	38	1.00	0.604	1.07	0.566	2.000	●
20.50	125.00	1.26	0.41	0.86	0.95	1.00	0.607	2.20	43	1.00	0.607	1.06	0.571	2.000	●
21.00	125.00	1.29	0.42	0.87	0.94	1.00	0.609	2.20	43	1.00	0.609	1.06	0.577	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
0.01	2.000	0.00	10.00	0.99	0.00
1.00	2.000	0.00	9.85	0.99	0.00
2.00	2.000	0.00	9.70	1.00	0.00
3.00	2.000	0.00	9.54	1.00	0.00
4.00	2.000	0.00	9.39	1.00	0.00
5.00	2.000	0.00	9.24	1.00	0.00
5.50	2.000	0.00	9.16	0.50	0.00
6.00	2.000	0.00	9.09	0.50	0.00
7.00	2.000	0.00	8.93	1.00	0.00
8.00	0.782	0.22	8.78	1.00	0.58
9.00	0.680	0.32	8.63	1.00	0.84
10.00	0.708	0.29	8.48	1.00	0.75
10.50	0.695	0.31	8.40	0.50	0.39
11.00	0.682	0.32	8.32	0.50	0.40
12.00	0.604	0.40	8.17	1.00	0.99
13.00	0.583	0.42	8.02	1.00	1.02
14.00	0.522	0.48	7.87	1.00	1.15
15.00	0.508	0.49	7.71	1.00	1.16
15.50	2.000	0.00	7.64	0.50	0.00
16.00	2.000	0.00	7.56	0.50	0.00
17.00	2.000	0.00	7.41	1.00	0.00
18.00	2.000	0.00	7.26	1.00	0.00
19.00	2.000	0.00	7.10	1.00	0.00
20.00	2.000	0.00	6.95	1.00	0.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
20.50	2.000	0.00	6.88	0.50	0.00
21.00	2.000	0.00	6.80	0.50	0.00

Overall potential I_L : 7.28

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)
0.01	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	1.00	0.000
1.00	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.98	0.00	1.00	0.000
2.00	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.97	0.00	1.00	0.000
3.00	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.95	0.00	1.00	0.000
4.00	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.94	0.00	1.00	0.000
5.00	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.92	0.00	0.50	0.000
5.50	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.92	0.00	0.50	0.000
6.00	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.91	0.00	1.00	0.000
7.00	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.89	0.00	1.00	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)
8.00	26	7.85	0.17	0.782	5.78	0.88	1.16	1.00	0.139	0.00
9.00	25	8.88	0.23	0.680	7.91	0.86	1.62	1.00	0.194	0.00
10.00	26	7.85	0.17	0.708	6.97	0.85	1.35	0.50	0.081	0.00
10.50	26	7.85	0.17	0.695	7.23	0.84	1.39	0.50	0.083	0.00
11.00	26	7.85	0.17	0.682	7.47	0.83	1.42	1.00	0.170	0.00
12.00	25	8.88	0.23	0.604	8.88	0.82	1.55	1.00	0.186	0.00
13.00	25	8.88	0.23	0.583	8.88	0.80	1.52	1.00	0.182	0.00
14.00	24	10.02	0.29	0.522	10.02	0.78	1.54	1.00	0.185	0.00
15.00	24	10.02	0.29	0.508	10.02	0.77	1.51	0.50	0.091	0.00
15.50	36	1.86	-0.51	2.000	0.00	0.76	0.00	0.50	0.000	0.00
16.00	36	1.86	-0.51	2.000	0.00	0.75	0.00	1.00	0.000	0.00
17.00	39	1.07	-0.73	2.000	0.00	0.74	0.00	1.00	0.000	0.00

:: 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)
18.00	39	1.07	-0.73	2.000	0.00	0.72	0.00	1.00	0.000	0.00
19.00	39	1.07	-0.73	2.000	0.00	0.71	0.00	1.00	0.000	0.00
20.00	38	1.30	-0.65	2.000	0.00	0.69	0.00	0.50	0.000	0.00
20.50	43	0.44	-1.03	2.000	0.00	0.68	0.00	0.50	0.000	0.00
21.00	43	0.44	-1.03	2.000	0.00	0.68	0.00	0.50	0.000	0.00

Cumulative settlements: 1.311 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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