

GEOTECHNICAL REPORT
Proposed Addition and Renovation
8555 85th Avenue Southeast
Mercer Island, WA

PROJECT NO. 25-237
February 2026

Prepared for:

Nabil Iskander and Engi Attia



*Geotechnical & Earthquake
Engineering Consultants*

February 24, 2026
File No. 25-237

Nabil Iskander and Engi Attia
8555 85th Avenue Southeast
Mercer Island, WA 98040

**Subject: Geotechnical Report
Proposed Addition and Renovations
8555 85th Avenue Southeast, Mercer Island, WA**

Dear Nabil and Engi,

Attached please find our geotechnical report for the proposed addition project at the above referenced site in Mercer Island, Washington. This report documents the subsurface conditions at the site and presents our geotechnical design recommendations for the project per the requirements of Critical Area Review 1.

Soil Conditions – In summary, the proposed addition area near the southwest corner of the house is underlain by loose to medium dense silty sand, with groundwater encountered at about 8 feet below ground surface. Some settlement may occur due to soil liquefaction during a large earthquake consistent with the design codes.

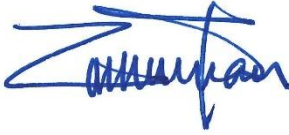
Foundation Recommendations – The proposed addition may be supported on conventional strip footings. Where isolated pad footings will be needed, the pad footings should be tied together with grade beams. The new footings should bear on at least 18 inches of properly compacted crushed rock. For the new deck column footings, unless the column footings can be tied together with grade beam, the deck columns should be supported on piles such as 2-inch diameter pin piles.

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 foundation should perform adequately and will not adversely affect the stability of the site.

Geotechnical Report
8555 85th Avenue Southeast, Mercer Island, WA
February 24, 2026

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

Sincerely,



Siew L. Tan, P.E.
Principal Geotechnical Engineer

Encl: Geotechnical Report

TABLE OF CONTENTS

1.0 INTRODUCTION	1
2.0 PROJECT AND SITE DESCRIPTION	1
3.0 SUBSURFACE EXPLORATIONS.....	3
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS.....	3
4.1 SITE GEOLOGY	3
4.2 SOIL CONDITIONS	4
4.3 GROUNDWATER	5
5.0 GEOLOGIC HAZARDS ASSESSMENT	5
5.1 LANDSLIDE HAZARDS AND STEEP SLOPES	5
<i>5.1.1 Quantitative Slope Stability Analysis.....</i>	<i>5</i>
<i>5.1.2 Results and Conclusions</i>	<i>6</i>
5.2 EROSION HAZARDS	6
5.3 SEISMIC HAZARDS	7
<i>5.3.1 Liquefaction Considerations.....</i>	<i>7</i>
<i>5.3.2 Faults</i>	<i>8</i>
6.0 GEOTECHNICAL DESIGN RECOMMENDATIONS	8
6.1 SEISMIC SITE CLASS.....	8
6.2 FOUNDATIONS RECOMMENDATIONS	9
<i>6.2.1 Conventional Footings (New Addition/Enclosure).....</i>	<i>9</i>
<i>6.2.2 Driven Pin Piles (Deck Columns).....</i>	<i>10</i>
6.3 FLOOR SLABS.....	12
6.4 PERMANENT SURFACE DRAINAGE CONSIDERATIONS.....	12

7.0 CONSTRUCTION CONSIDERATIONS	13
7.1 TEMPORARY EXCAVATION.....	13
7.2 MATERIAL REUSE	13
7.3 STRUCTURAL FILL PLACEMENT AND COMPACTION	14
7.4 GENERAL TEMPORARY EROSION AND DRAINAGE CONSIDERATIONS	14
7.5 WET WEATHER EARTHWORK	14
8.0 STATEMENT OF RISK.....	16
9.0 LIMITATIONS.....	16
10.0 REFERENCES.....	18

LIST OF ATTACHMENTS

Figure 1	Vicinity Map
Figure 2	Site and Exploration Plan
Figure 3	Static Global Stability Analysis
Figure 4	Pseudo-Static Global Stability Analysis

APPENDIX A SUMMARY DYNAMIC CONE PENETRATION TEST LOGS

Figure A-1	Terms and Symbols for Boring and Test Pit Logs
Figure A-2	Log of Test Boring PG-1
Figure A-3	Log of Test Boring PG-2

GEOTECHNICAL REPORT
PROPOSED ADDITION AND RENOVATION
8555 - 85TH AVENUE SOUTHEAST, 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 deck replacement and addition project in Mercer Island, Washington. Our study was performed in general accordance with our mutually agreed-upon scope of work as outlined in our proposal dated December 4, 2025, and approved by you on December 9, 2025. Our service scope included reviewing readily available geologic and geotechnical data in the site vicinity, conducting a site reconnaissance, completing two test borings, performing geotechnical analyses, and preparation of this report.

2.0 PROJECT AND SITE DESCRIPTION

The project site is located at 8555 85th Avenue SE in Mercer Island, Washington (See Figure 1 – Vicinity Map). The subject property is oriented diagonally and is roughly square in shape, has an area of about 9,967 square feet. It is bordered by existing single-family residences to the north, west, and south, and 85th Avenue Southeast to the east (see Figure 2 – Site and Exploration Plan).

The site is currently occupied by a one-story single-family residence with a daylight basement. The house is located at the approximate middle of the lot. The basement level daylights to the driveway and front yard at the south and east sides of the property. A wraparound deck at the main level extends around the front-facing southeast side of the house, and covers a concrete patio area at the exterior of the basement level.

Site grades generally slope downwards from the northwest corner to the southeast corner, for an approximately 23 feet of elevation relief over about 146 linear feet (average ~16% slope gradient). The basement is benched into a southeast-facing slope (~38% at steepest) that comprises the northwest half of the site. An approximately 3-foot-tall retaining wall is present at the west end of the covered carport, in line with the building line of the house.

During our site visit, we did not observe signs of settlement or distress in the existing house and deck foundations, nor within the existing concrete patio or asphalt driveway. No signs of tilting or slumping were present at the west upslope are of the property that would indicate backyard instability next to the house.

We understand that you plan to enclose the existing concrete patio located at the southwest corner of the house to increase the footprint of the basement and the overhead main floor. The existing condition of the subject patio is shown in Plate 1, below. The south portion of the existing deck is to be reconstructed, with two new deck posts. The remainder of the planned improvements will be interior remodeling. No significant grading activities anticipated for the project.



Plate 1: View of the proposed area of constriction southwest corner of the house, looking north.

Based on our review of the Mercer Island GIS Map, the site has three mapped geologic hazards on the property: potential landslide, erosion, and seismic hazards. As a result, the City has requested a Critical Area Study to address the mapped geologic hazards. Based on our understanding of the project, the City is requiring a Critical Area Review 1 for the project per the Mercer Island City Code (MICC) Section 19.07.090. As such, our geotechnical evaluation of the site is limited in scope to the Critical Area Review 1 requirements for the deck replacement. Considerations regarding the critical areas are provided in Section 5.0 below.

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

geotechnical recommendations have been correctly interpreted and adequately implemented in the construction documents.

3.0 SUBSURFACE EXPLORATIONS

PanGEO drilled two test borings (PG-1 and PG-2) at the project site on January 19, 2026. The approximate test boring locations are indicated on Figure 2. The borings were drilled to depths of about 22½ feet deep in PG-1 and 19 feet deep in PG-2, before reaching practical drilling refusal at each boring location.

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

A geologist from PanGEO was present during the field exploration to observe the drilling, assist in sampling, and to describe and document the soil samples obtained from the borings. The soil samples were described and field classified in general accordance with the symbols and terms outlined in Appendix A, Figure A-1, and the summary boring logs are included as Figures A-2 and A-3.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 SITE GEOLOGY

Based on our review of the *Geologic Map of Mercer Island, Washington* (Troost and Wisher, 2006), the site is mapped as Lawton Clay (Geologic Map Unit Qv1c). Lawton clay is described by Troost and Wisher as very stiff to hard, laminated to massive, silt, clayey silt, and silty clay, deposited in proglacial lakes and subsequently glacially overridden.

4.2 SOIL CONDITIONS

The following is a generalized description of the soils encountered in our test borings. For a more detailed description of the subsurface conditions encountered, please refer to our test boring logs provided in Appendix A.

Soil Unit 1 (Fill): Fill was encountered in both borings from the surface to about 5 feet deep. Fill soils generally consisted of medium dense, silty sand with variable angular grave.

Soil Unit 2 (Colluvium): Colluvium was encountered directly below the fill to depths of about 7½ feet below ground surface (bgs) in PG-1 and 10 feet bgs in PG-2. The colluvium generally consisted of loose to medium dense, and medium stiff silt and sand, characterized by relatively low blowcounts and disturbed textures.

Soil Unit 3 (Advance Outwash): Directly below the colluvium in PG-1, located near the southwest corner of the house, medium dense to dense sand with silt was encountered to the boring termination depth of about 22½ feet bgs. We interpret this sand as advance outwash. Groundwater was encountered within this unit at about 8 feet bgs. This soil unit was not encountered in boring PG-2.

Soil Unit 4 (Lawton Clay): Directly below the colluvium in PG-2, located on the upslope side of the house, medium stiff to hard, lean clay was encountered to the termination depth of the test boring at about 19 feet bgs. We interpret this soil unit as Lawton Clay mapped at the site.

Our subsurface descriptions are based on the conditions encountered at the time of our exploration. It should be noted that the stratigraphic contacts indicated on the test logs represent the approximate depth to boundaries between soil units. Actual transitions between soil units may be more gradual or occur at different elevations. Soil and rock conditions between our exploration locations may vary from those encountered. The nature and extent of variations between our exploratory locations may not become evident until construction. If variations do appear, PanGEO should be requested to reevaluate the recommendations in this report and to modify or verify them in writing prior to proceeding with earthwork and construction.

4.3 GROUNDWATER

At the time of drilling, groundwater was observed at about 8 feet bgs in PG-1 but was not encountered in PG-2. It should be noted that groundwater levels and seepage rates are not static. There will likely be fluctuations in the groundwater level depending on the season, amount of rainfall, surface water runoff, and other factors. Groundwater levels are normally highest during the winter and early spring (typically October through May).

5.0 GEOLOGIC HAZARDS ASSESSMENT

5.1 LANDSLIDE HAZARDS AND STEEP SLOPES

Based on our review of the Mercer Island GIS map, the site is located within a Landslide Hazard Areas. Our evaluation of the potential landslide risk is limited to within the property boundaries.

The primary sloped area on the site is located behind the house. Based on our onsite observations, the slope appears covered with vegetation consisting of well-maintained landscaping such as mature trees and low shrubs. We did not observe areas of exposed ground or scarps that would indicate recent instability or slope movement around the house. We also did not observe signs of site instability such as settlement or distress of foundation elements including concrete patio and driveways.

In summary, based on our field observations, it is our opinion that the existing house appears to be globally stable in its present condition. The proposed renovation will enclose the existing basement level concrete patio, within a flat area and with very minor grading anticipated. No soil excavations or disturbances are anticipated in the slope located behind the house. Hence, the proposed project will have no impact to site slope stability.

5.1.1 Quantitative Slope Stability Analysis

To verify the global stability of the site within its property limits, we performed a quantitative slope stability analysis. The analysis was performed using the program SLIDE2 (Slide) published by Rocscience Inc., which is a two-dimensional limit equilibrium slope stability analysis program. A generalized subsurface profile A-A' was developed along the alignment shown in Figure 2 to support our slope stability analysis.

The seismic coefficient for pseudo-static analysis was developed based on the 2021 International Building Code (IBC) and ASCE 7-16, which specifies a design earthquake having a 2 percent probability of occurrence in 50 years (return interval of 2,475 years). Per the methodology described in Chapter 20 of ASCE 7-16, a site modified peak ground acceleration ($PGAM$) of 0.689g was obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2014 data) for the project site, based on Site Class D for stiff soil. The lateral seismic coefficient is then taken as one half of the $PGAM$, consistent with the current state of practice. As such, we used a horizontal seismic coefficient 0.345g for our pseudo-static analysis.

5.1.2 Results and Conclusions

The results of our analysis are summarized in Figures 3 and 4, for static and seismic conditions. In summary, a minimum factor of safety of 2.37 and 1.13 against slope instability was calculated for the static and seismic conditions, respectively. The calculated factors of safety exceed the generally accepted minimum factors of safety of 1.5 for the static condition and 1.1 for the seismic condition. Thus, it is our opinion that the site is adequate stable, and no mitigation measures are needed to improve the site stability.

5.2 EROSION HAZARDS

The site is mapped within a potential erosion hazard area according to the Mercer Island GIS map. Based on the results of our subsurface explorations, the site soils are highly sensitive to moisture and can be prone to erosion when exposed to water or wet weather.

In our opinion, the erosion hazards at the site can be effectively mitigated by maintaining construction activities within the recommended buffer and by using Best Management Practices (BMPs) during construction and with properly designed and implemented landscaping for permanent erosion control. Recommendations for surface drainage and erosion control are provided in [Section 7.4](#). Recommendations for wet-weather earthwork BMPs are provided in [Section 7.5](#). The project stormwater management program shall meet the standards provided in [Section 15.09](#) of the Mercer Island Municipal Code.

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

5.3 SEISMIC HAZARDS

Based on review of the City of Mercer Island GIS Map, the site is mapped 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. The seismic slope hazards are discussed in Section 5.1 above. This section (5.3) discusses the liquefaction hazards at the site.

5.3.1 Liquefaction Considerations

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.

In our test borings, subsurface conditions that may be prone to soil liquefaction were only encountered in boring PG-1, but not PG-2. We evaluated the liquefaction potential in the area of PG-1, which is located in the area of proposed construction, 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 Liquefaction Assessment Software *LiqSVS* by GeoLogismiki was used in our evaluation. A peak ground acceleration associated with the IBC level earthquake of 0.689g and magnitude of 7.0 was used to determine the risk of liquefaction, using the method proposed by Boulanger & Idriss (2014).

The results of our analyses indicate that, at the location PG-1, there is a high risk of soil liquefaction between a depth of about 8 to 21 feet below grade. Below 21 feet, the risk for soil liquefaction is low due to an increase in soil density to dense. We also estimated about 3 inches of free field, liquefaction-induced total settlement; liquefaction induced settlement in the proposed area of construction should be no more than about 1 to 2 inches.

5.3.2 Faults

Where fault lines are present below structures, faulting can potentially displace the structures. Based on our review of the geologic mapping of the site vicinity, the closest Class A seismic source to the project is the Seattle Fault Zone. The Seattle Fault Zone consists of east-west-trending subparallel thrust faults; the subject site is located approximately ½ mile south of the southern fault line passing through Mercer Island. According to the USGS Quaternary Fault Database (Fault No. 570), this fault has been active within the last approximately 1,100 years (Johnson et al., 2016). Based on the distance to the fault, in our opinion, the potential for ground rupture at the subject site during a future earthquake is negligible.

6.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

6.1 SEISMIC 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 (Stiff Soil).

6.2 FOUNDATIONS RECOMMENDATIONS

6.2.1 Conventional Footings (New Addition/Enclosure)

As discussed above, the site soils in the southwest corner of the house where the carport will be enclosed are considered prone to soil liquefaction, and the as much 1 to 2 inches of differential settlement could occur within the footprint of the new addition. To reduce the potential of significant differential settlement between adjacent columns, we recommend that the new addition be supported on continuous strip footings. If isolated pad footings will be used, grade beams should be installed to tie the pad footings together. 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.

Over-excavation and Replacement – We recommend that the new footings bear on at least 18 inches of properly compacted 1¼-inch minus crushed rock, or 1¼-inch minus recycled crushed concrete. 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 – For at-grade buildings, footing drains are optional. If the project team plans to install footing drain, the footing drain 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. Additional settlement could occur during a strong seismic event as discussed above.

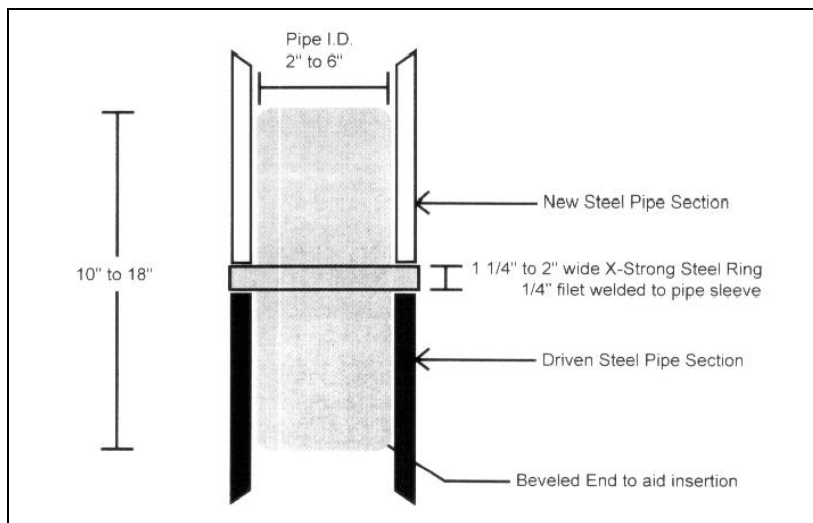
6.2.2 Driven Pin Piles (Deck Columns)

The new deck footings may be supported on footings, provided that the footing can be tied together with grade beam. Otherwise, the deck columns should be supported on piles, such as 2-inch diameter driven pin piles. Two-inch diameter pin piles can be driven with hand equipment.

Pin Pile Capacity – Each 2-inch pin pile can be designed for an allowable compression load of 3 tons.

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. 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.
3. 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.



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

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 20 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.

6.3 FLOOR SLABS

A slab-on-grade floor may be used for the proposed addition. Any loose or soft soils should be removed from below the floor slab and backfilled with properly compacted structural fill.

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 PERMANENT SURFACE DRAINAGE CONSIDERATIONS

Permanent control of surface water and roof runoff should be incorporated in the final grading design. In addition to these sources, irrigation and rainwater infiltrating into landscape and planter areas adjacent to paved areas or building foundations should also be controlled. All collected runoff should be directed into conduits that carry the water away from the pavement or structures and into storm drain systems or other appropriate outlets. Adequate surface gradients should be incorporated into the grading design such that surface runoff is directed away from structures and away from the slopes on the west side of the property.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 TEMPORARY EXCAVATION

Based on our understanding of the project, we anticipate that temporary excavations of less than 4 feet will be needed for the subgrade improvement and new footings for the addition and deck posts. As such, it is our opinion that unsupported slope cut excavations are feasible at the site. Based on the soil conditions at the site, for planning purposes, it is our opinion that any excavations deeper than 4 feet should be sloped to maximum gradient of 1H:1V.

All temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. In general, temporary excavations deeper than a total of 4 feet should be sloped or shored. However, excavations less than 4 feet deep, if located along or near property lines, will also need to be sloped or supported if sufficient space is not available to lay back the excavations without encroaching into neighboring properties.

The temporary excavations and cut slopes should be re-evaluated in the field during construction based on actual observed soil conditions and may need to be flattened in the wet seasons and should be covered with plastic sheets. The cut slopes should be covered with plastic sheets in the rainy season. We also recommend that heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a distance equal to 1/3 the slope height from the top of any excavation.

7.2 MATERIAL REUSE

In the context of this report, structural fill is defined as compacted fill placed under footings, concrete stairs and landings, and slabs, or other load-bearing areas. Based on the results from our explorations, it is our opinion that the on-site soils are not suitable for reuse as structural fill. Structural fill should consist of those described in [Section 7.3](#).

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

7.3 STRUCTURAL FILL PLACEMENT AND COMPACTION

Structural fill below footings should consist of 1¼ inch minus crushed rock or crushed recycled concrete.

Outside of the footings, structural fill should consist of imported, well-graded, granular material, such as WSDOT Gravel Borrow (Section 9.03.14 (1) of the 2025 WSDOT Standard Specifications), Seattle Mineral Aggregate Type 17 (*Seattle Standards and Specifications, 2023*, Section 9-03.14), Crushed Surfacing Base Course (CSBC), or another approved equivalent.

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

Generally, inadequately compacted soils are a result of poor construction technique or improper moisture content. Soil with high fines content is particularly susceptible to becoming too wet and coarse-grained materials easily become too dry, for proper compaction. Depending on the type of compaction equipment used and depending on the type of fill material, it may be necessary to decrease the thickness of each lift in order to achieve adequate compaction. PanGEO can provide additional recommendations regarding structural fill and compaction during construction.

7.4 GENERAL TEMPORARY EROSION AND DRAINAGE CONSIDERATIONS

Surface runoff during construction 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 collect runoff and prevent water from entering excavations or to prevent runoff from the construction area from leaving the immediate work site. Temporary erosion control may require the use of hay bales or silt fence on the downhill side of the construction area to prevent silty water from leaving the site and potential storm water detention to trap sand and silt before the water is discharged to a suitable outlet. All collected water should be directed under control to a positive and permanent discharge system.

7.5 WET WEATHER EARTHWORK

It is our opinion that construction of the project can be accomplished during the wet season (November to April). However, performing earthwork activities during wet seasons may be more

costly than during dry weather conditions. Based on the anticipated soil conditions and topography in the proposed construction area, it is our opinion that potential for erosion at the site can be adequately mitigated by employing sediment control best management practices (BMPs). Additional information and details of the BMPs discussed in this section can be found in the Washington State Department of Ecology's *Stormwater Management Manual for Western Washington, Volume II* (<http://www.ecy.wa.gov/programs/wq/stormwater/manual.html>). Sediment control BMPs should be installed/constructed and functional prior to land disturbing activities. Additionally the project stormwater management program shall meet the standards provided in Section 15.09 of the Mercer Island Municipal Code.

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

- All footing surfaces should be protected against inclement weather unless the footings can be poured immediately after the subgrade is exposed. It is the contractor's responsibility to protect the footing subgrade from disturbance. If needed, one option is to place 2 to 3 inches of lean-mix concrete or 4 to 6 inches of crushed surfacing base course (CSBC) on the exposed foundation 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.
- Where practical, maintain vegetation buffers around cleared areas (BMP C101).
- 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 STATEMENT OF RISK

We understand that the site is mapped with geologic hazard areas, specifically potential slide, seismic, and erosion geologic hazards. Per Mercer Island City Code Section 19.07.160.B(3), development within geologic hazard areas and critical slopes may occur if the geotechnical engineer provides a statement of risk with supporting documentation indicating that one of the following conditions can be met:

- a. An evaluation of site-specific subsurface conditions demonstrates that the proposed development is not located in a geologic hazard area; or
- b. 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; or
- c. Development practices are proposed for the alteration that would render the development as safe as if it were not located in a geologic hazard area; or
- d. The alteration is so minor as not to pose a threat to the public health, safety, and welfare.

It is our opinion that the proposed development meets criterion (b).

9.0 LIMITATIONS

We have prepared this report for use by Nabil Iskander and Engi Attia. 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 has been prepared for planning and design purposes for specific application to the proposed project in accordance with the generally accepted standards of local practice at the time this report was written. No warranty, express or implied, is made.

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

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

We appreciate the opportunity to be of service.

Sincerely,



Amanda D. Ong, E.I.T., G.I.T.
Staff Geotechnical Engineer / Geologist

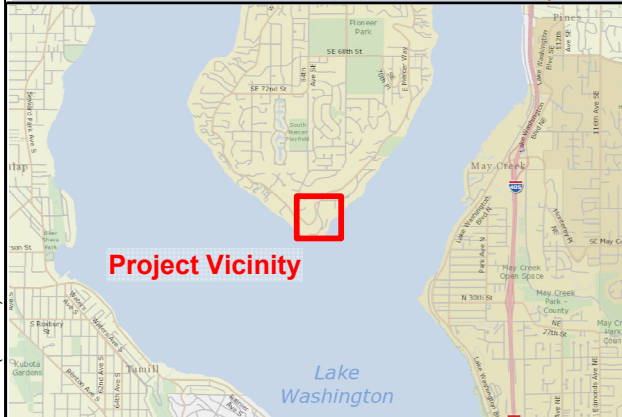
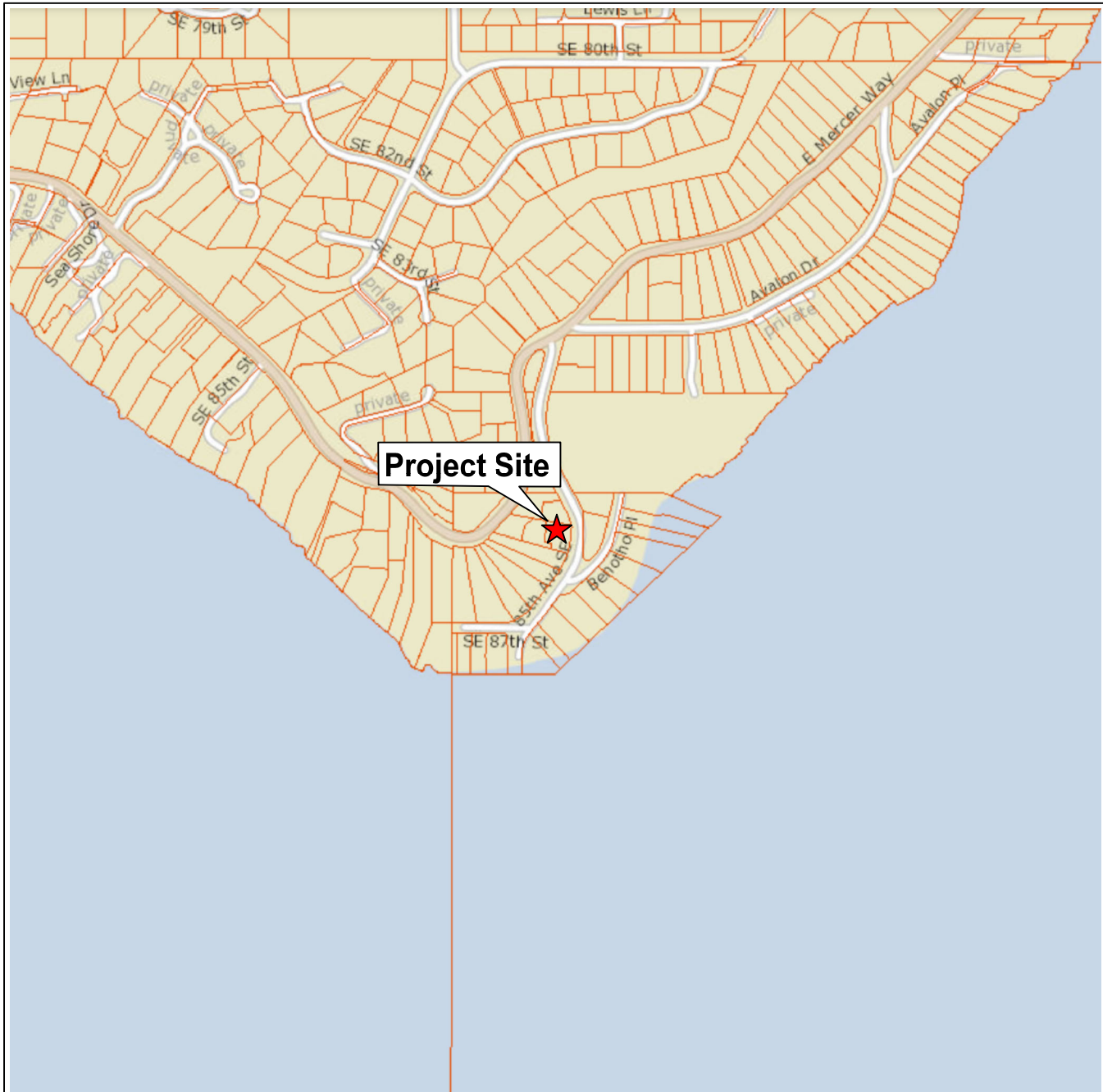


February 24, 2026

Siew L. Tan, P.E.
Principal Geotechnical Engineer

10.0 REFERENCES

- ASCE, 2016, *ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*.
- 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 / D1586-18, 2018, *Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*, www.astm.org.
- International Code Council (IBC), 2021, *International Building Code 2021*. Country Club Hills, IL: International Code Council, Inc.
- Johnson, S.Y., Blakely, R.J., Brocher, T.M., Haller, K.M., Barnett, E.A., Bucknam, R.C., Haeussler, P.J., Pratt, T.L., Nelson, A.R., Sherrod, B.L., Wells, R.E., Lidke, D.J., Harding, D.J., and Kelsey, H.M., 2016, *Fault Number 570, Seattle Fault Zone*, U.S. Geological Survey – Quaternary Fault and Fold Database of the United States, <https://earthquakes.usgs.gov/hazards/qfaults>.
- Triggs, J.F., Simpson, P.D., 1991, *A Portable Dynamic Penetrometer for Geotechnical Investigations*.
- Troost, K.G., Wisner, A. P., 2006, *Geologic Map of Mercer Island*, GeoMapNW, scale 1:12,000.
- USGS, *U.S. Quaternary Faults*, accessed May 12, 2025, via <https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=5a6038b3a1684561a9b0aadf88412fcf>.
- Washington Administration Code (WAC), 2022, *Chapter 296-155. Safety Standards for Construction Work, Part N – Excavation, Trenching, and Shoring*.
- WSDOT, 2025, *Standard Specifications for Road, Bridge and Municipal Construction*, M 41-10, Washington State Department of Transportation.



Base Map: King County iMap

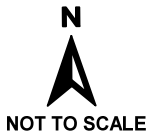


Fig 1 - Vicinity Map.gpl 2/19/26 (1:48:24) ADO



**Proposed Addition and Remodel
8555 85th Avenue Southwest
Mercer Island, Washington**





VICINITY MAP

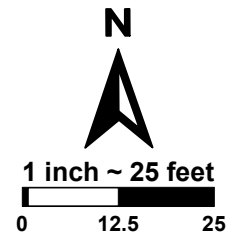
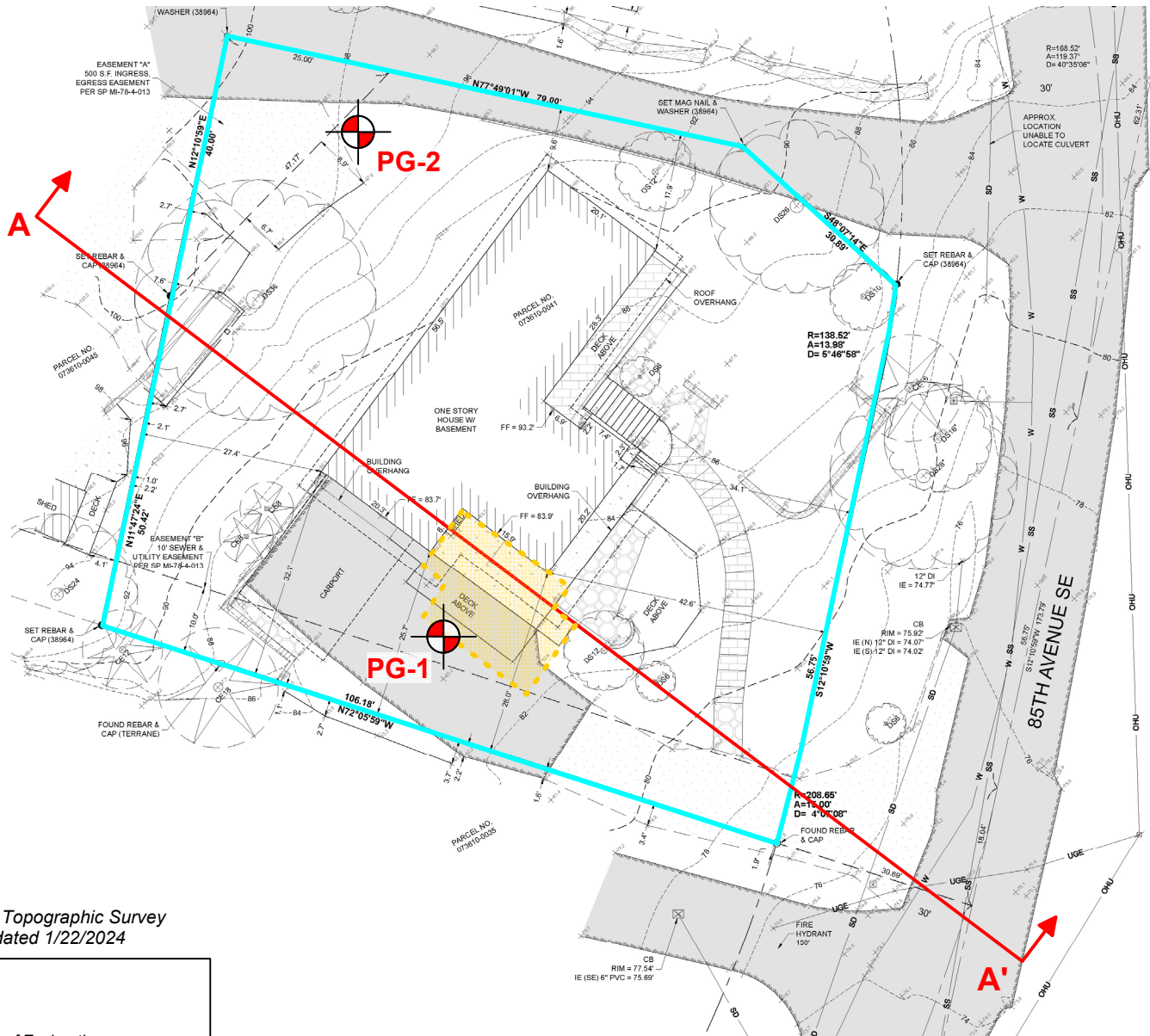
Project No. **25-237**


Figure No. **1**

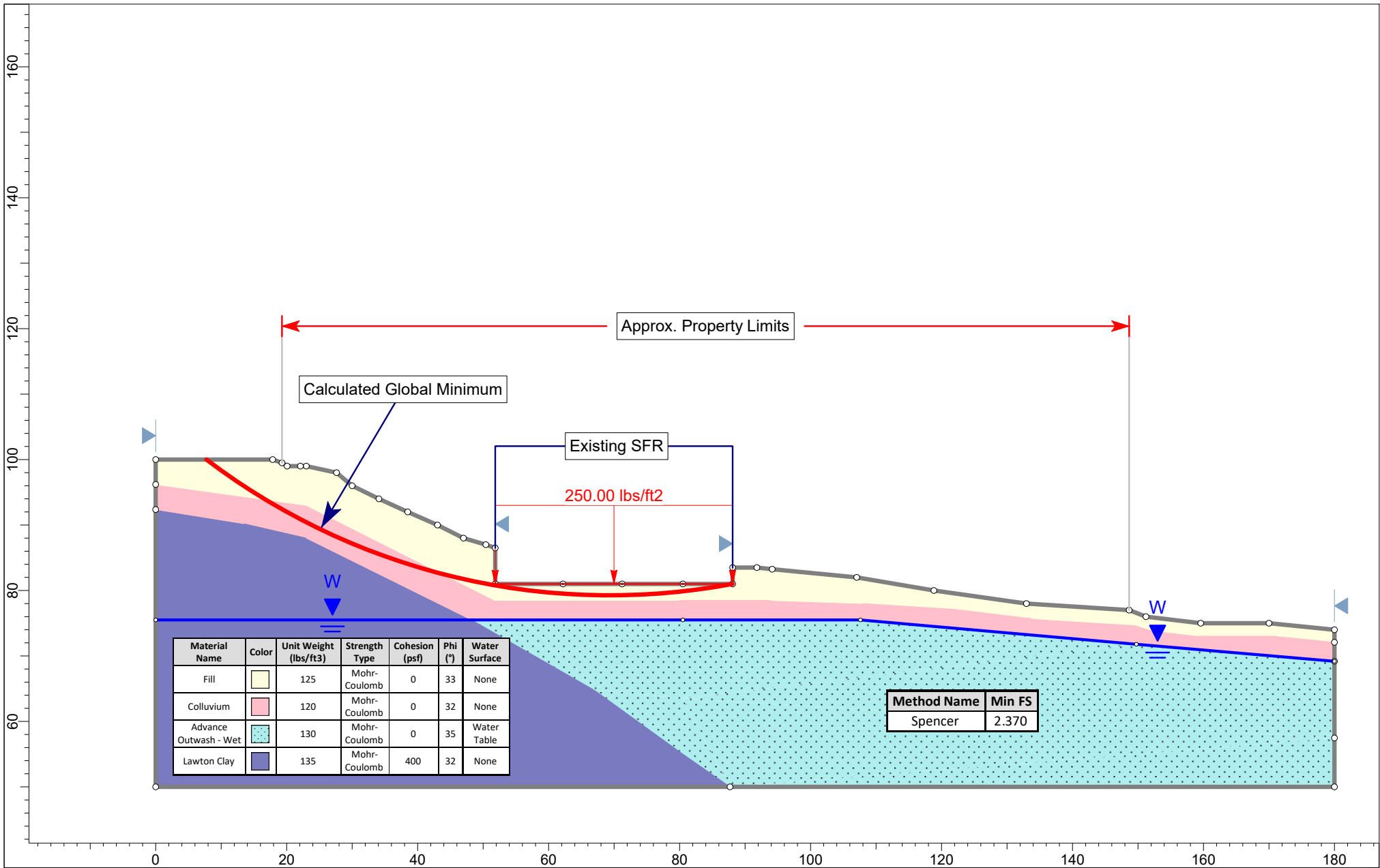
Note: Base map modified from Topographic Survey by Site Surveying, Inc., dated 1/22/2024

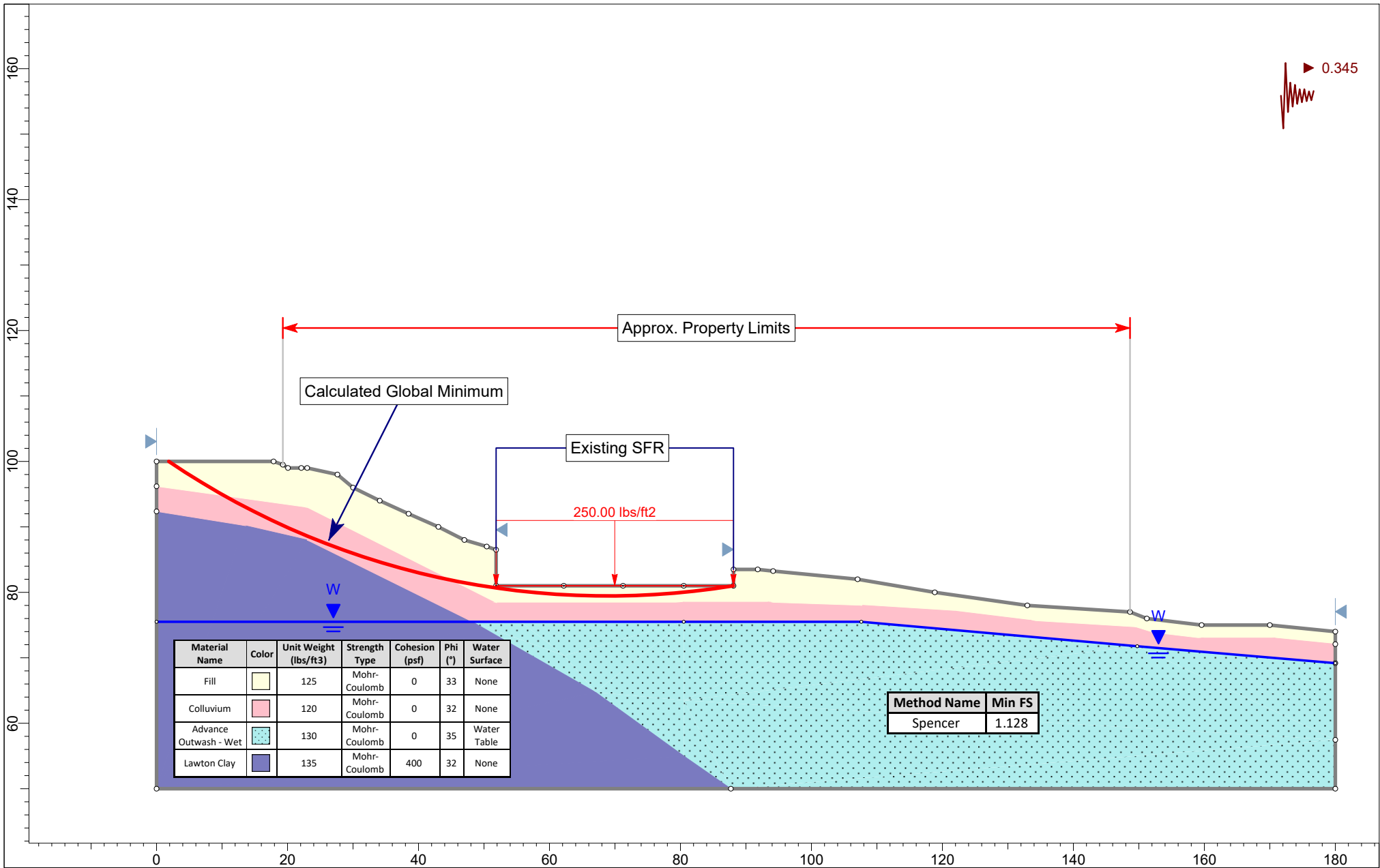
LEGEND

-  Approximate Location of Explorations (PanGEO, 2026)
-  Approximate Site Boundary
-  Approximate Extent of Cross-Section Alignment
-  Approximate Extent of Proposed Improvements



	<p>Proposed Addition and Remodel 8555 85th Avenue Southwest Mercer Island, Washington</p>		<p>SITE AND EXPLORATION PLAN</p>	
	<p>Project No. 25-237</p>	<p>Figure No. 2</p>		





APPENDIX A

SUMMARY EXPLORATION 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
	GRAVEL (>12% fines)		GP: Poorly-graded 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)		GM: Silty GRAVEL
	SAND (>12% fines)		GC: Clayey GRAVEL
			SW: Well-graded SAND
			SP: Poorly-graded SAND
Silt and Clay 50% or more passing #200 sieve	Liquid Limit < 50		SM: Silty SAND
			SC: Clayey SAND
			ML: SILT
	Liquid Limit > 50		CL: Lean CLAY
			OL: Organic SILT or CLAY
			MH: Elastic SILT
			CH: Fat CLAY
Highly Organic Soils			OH: Organic SILT or CLAY
			PT: PEAT

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

DESCRIPTIONS OF SOIL STRUCTURES

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

COMPONENT DEFINITIONS

COMPONENT	SIZE / SIEVE RANGE	COMPONENT	SIZE / SIEVE RANGE
Boulder:	> 12 inches	Sand	
Cobbles:	3 to 12 inches	Coarse Sand:	#4 to #10 sieve (4.5 to 2.0 mm)
Gravel	3 to 3/4 inches	Medium Sand:	#10 to #40 sieve (2.0 to 0.42 mm)
		Fine Sand:	#40 to #200 sieve (0.42 to 0.074 mm)
Coarse Gravel:	3 to 3/4 inches	Silt	0.074 to 0.002 mm
Fine Gravel:	3/4 inches to #4 sieve	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 Split Spoon (300-lb hammer, 30" drop)
	Non-standard penetration test (see boring log for details)
	Thin wall (Shelby) tube
	Grab
	Rock core
	Vane Shear

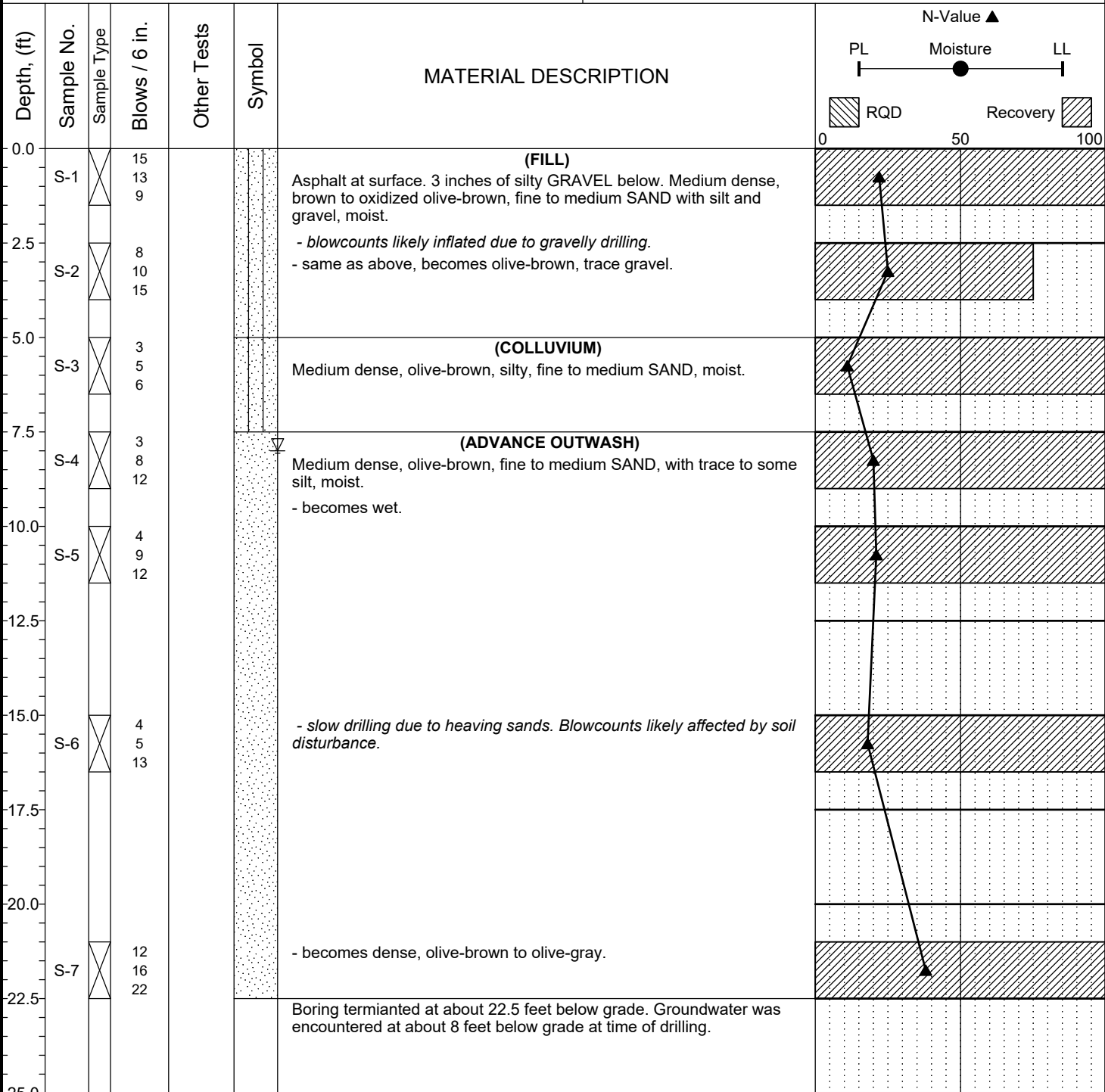
MONITORING WELL

	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

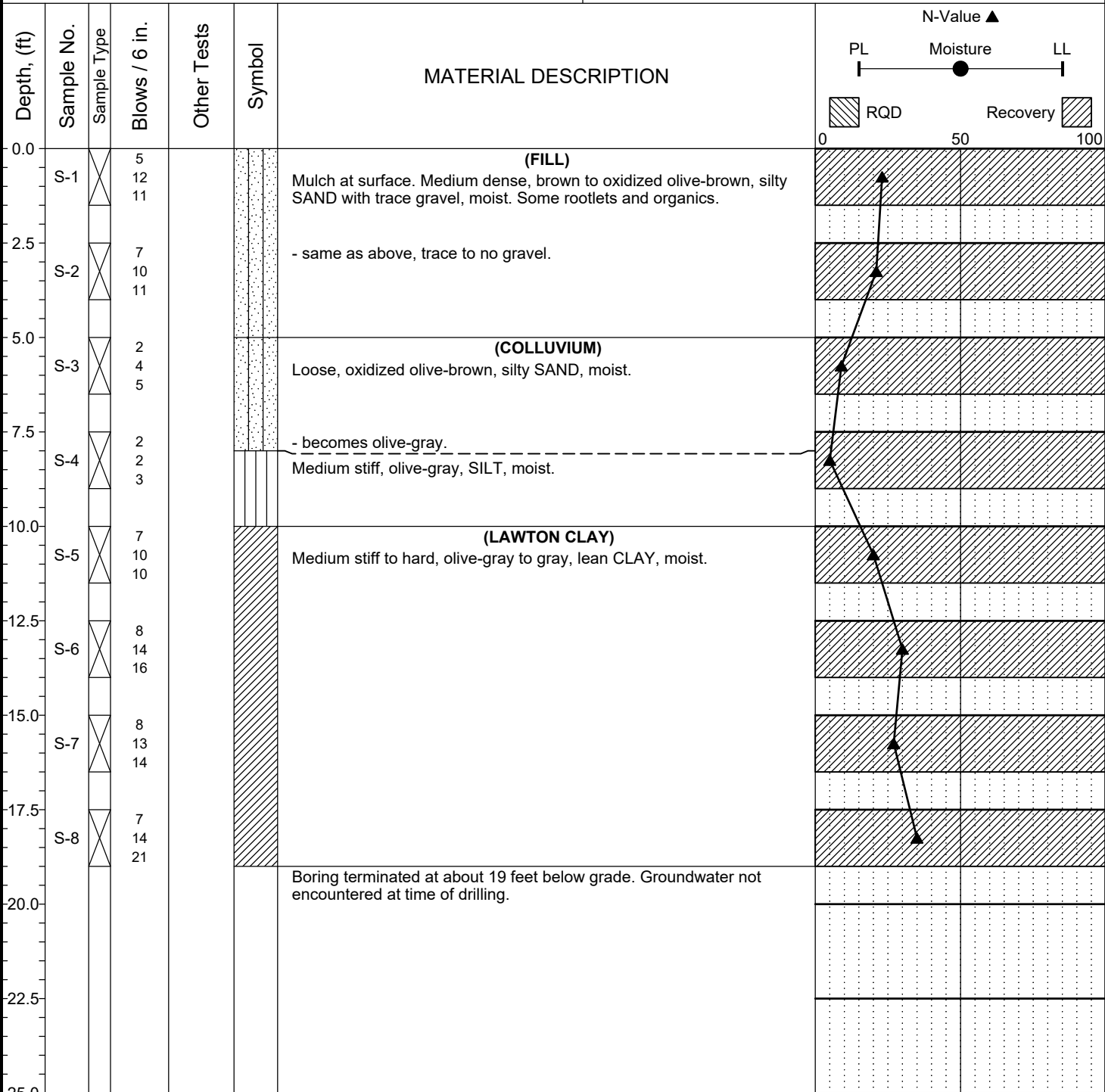
Dry	Dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

Project:	Proposed Addition	Surface Elevation:	~82 ft
Job Number:	25-327	Top of Casing Elev.:	N/A
Location:	8555 85th Avenue Southeast, Mercer Island	Drilling Method:	HSA
Coordinates:	Northing: 47.52582, Easting: -122.22552	Sampling Method:	SPT, Rope and Cathead



Completion Depth:	22.5ft	Remarks: Soil boundaries are approximate. Boring drilled with a portable acker rig. Standard penetration test (SPT) sampler driven with a 140-pound safety hammer operated with a rope and cathead mechanism. Surface elevations estimated from client-provided survey.
Date Borehole Started:	1/19/26	
Date Borehole Completed:	1/19/26	
Logged By:	S. Paquet	
Drilling Company:	CN Drilling, Inc.	

Project:	Proposed Addition	Surface Elevation:	~96 ft
Job Number:	25-327	Top of Casing Elev.:	N/A
Location:	8555 85th Avenue Southeast, Mercer Island	Drilling Method:	HSA
Coordinates:	Northing: 47.68061, Easting: -122.31605	Sampling Method:	SPT, Rope and Cathead



Completion Depth:	19.0ft	Remarks: Soil boundaries are approximate. Boring drilled with a portable acker rig. Standard penetration test (SPT) sampler driven with a 140-pound safety hammer operated with a rope and cathead mechanism. Surface elevations estimated from client-provided survey.
Date Borehole Started:	1/19/26	
Date Borehole Completed:	1/19/26	
Logged By:	S. Paquet	
Drilling Company:	CN Drilling, Inc.	

SPT BASED LIQUEFACTION ANALYSIS REPORT

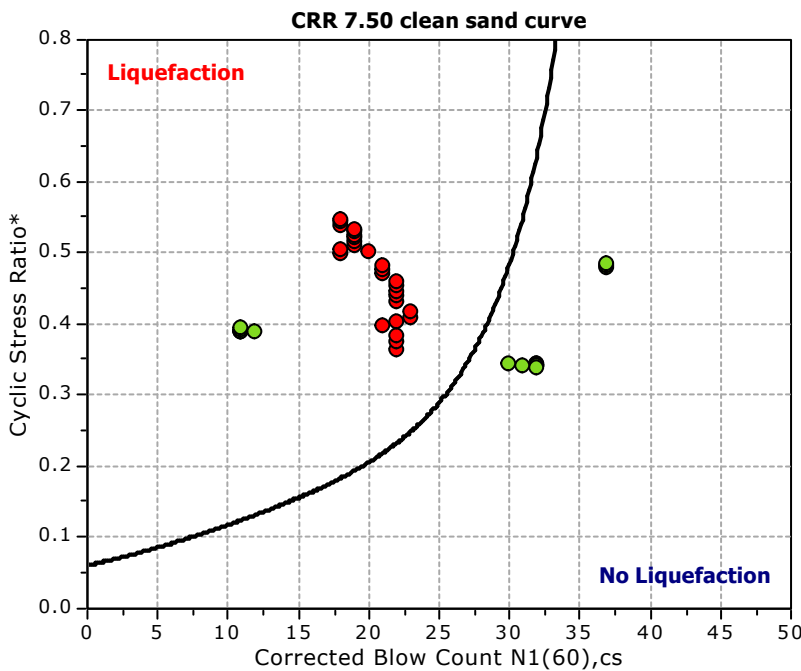
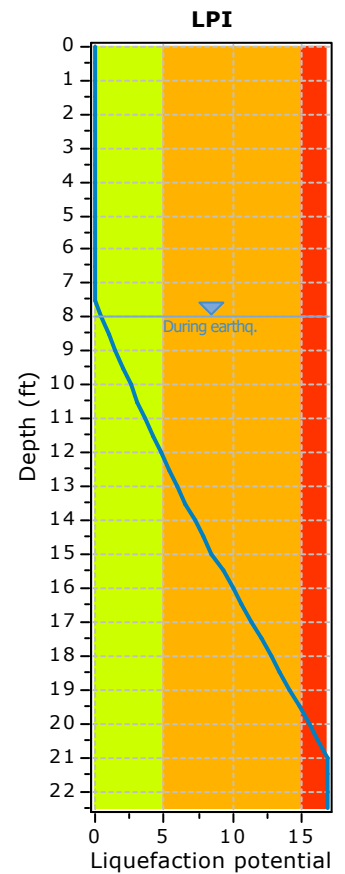
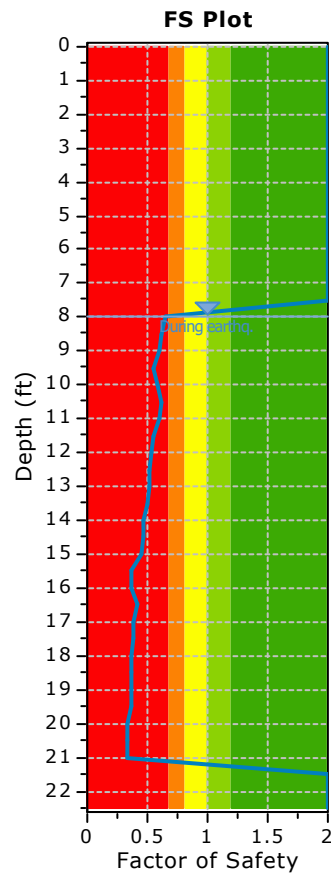
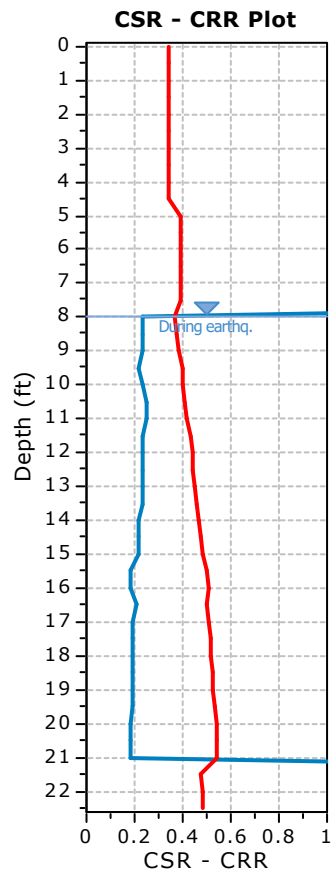
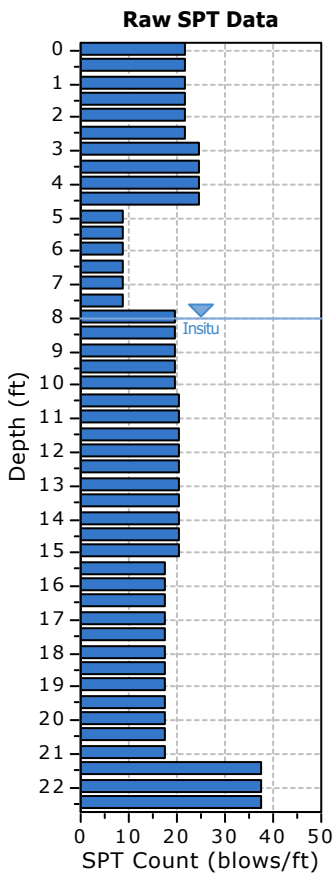
Project title : 25-237

SPT Name: PG-1

Location : 8555 85th Avenue Southeast, Mercer Island

:: Input parameters and analysis properties ::

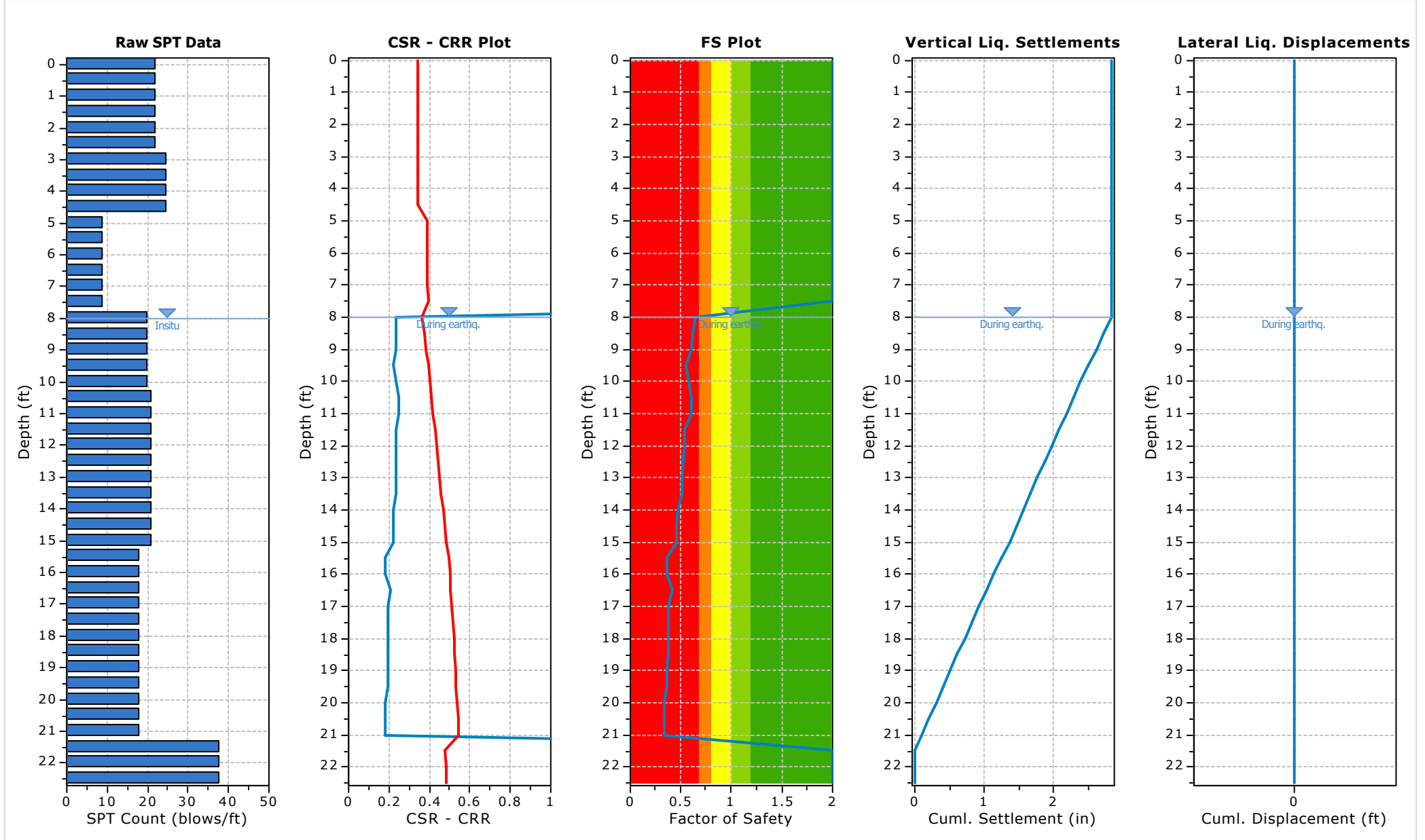
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	8.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	8.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w :	7.00
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.69 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		



- F.S. color scheme**
- Red: Almost certain it will liquefy
 - Orange: Very likely to liquefy
 - Yellow: Liquefaction and no liq. are equally likely
 - Light Green: Unlike to liquefy
 - Dark Green: Almost certain it will not liquefy

- LPI color scheme**
- Red: Very high risk
 - Orange: High risk
 - Yellow: 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	22	20.00	125.00	0.49	No
0.50	22	20.00	125.00	0.50	No
1.00	22	20.00	125.00	0.50	No
1.50	22	20.00	125.00	0.50	No
2.00	22	20.00	125.00	0.50	No
2.50	22	20.00	125.00	0.50	No
3.00	25	5.00	125.00	0.50	No
3.50	25	5.00	125.00	0.50	No
4.00	25	5.00	125.00	0.50	No
4.50	25	5.00	125.00	0.50	No
5.00	9	5.00	120.00	0.50	No
5.50	9	5.00	120.00	0.50	No
6.00	9	5.00	120.00	0.50	No
6.50	9	5.00	120.00	0.50	No
7.00	9	5.00	120.00	0.50	No
7.50	9	5.00	120.00	0.50	No
8.00	20	5.00	130.00	0.50	Yes
8.50	20	5.00	130.00	0.50	Yes
9.00	20	5.00	130.00	0.50	Yes
9.50	20	5.00	130.00	0.50	Yes
10.00	20	5.00	130.00	0.50	Yes
10.50	21	5.00	130.00	0.50	Yes
11.00	21	5.00	130.00	0.50	Yes
11.50	21	5.00	130.00	0.50	Yes
12.00	21	5.00	130.00	0.50	Yes
12.50	21	5.00	130.00	0.50	Yes
13.00	21	5.00	130.00	0.50	Yes
13.50	21	5.00	130.00	0.50	Yes
14.00	21	5.00	130.00	0.50	Yes
14.50	21	5.00	130.00	0.50	Yes
15.00	21	5.00	130.00	0.50	Yes
15.50	18	5.00	130.00	0.50	Yes
16.00	18	5.00	130.00	0.50	Yes
16.50	18	5.00	130.00	0.50	Yes
17.00	18	5.00	130.00	0.50	Yes
17.50	18	5.00	130.00	0.50	Yes
18.00	18	5.00	130.00	0.50	Yes
18.50	18	5.00	130.00	0.50	Yes
19.00	18	5.00	130.00	0.50	Yes
19.50	18	5.00	130.00	0.50	Yes
20.00	18	5.00	130.00	0.50	Yes
20.50	18	5.00	130.00	0.50	Yes
21.00	18	5.00	130.00	0.50	Yes
21.50	38	5.00	130.00	0.50	Yes
22.00	38	5.00	130.00	0.50	Yes
22.50	38	5.00	130.00	0.50	Yes

:: 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)	α_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	22	125.00	0.00	0.00	0.00	0.33	1.70	1.00	1.00	0.75	1.00	28	20.00	4.48	32	4.000
0.50	22	125.00	0.03	0.00	0.03	0.33	1.70	1.00	1.00	0.75	1.00	28	20.00	4.48	32	4.000
1.00	22	125.00	0.06	0.00	0.06	0.33	1.70	1.00	1.00	0.75	1.00	28	20.00	4.48	32	4.000
1.50	22	125.00	0.09	0.00	0.09	0.33	1.70	1.00	1.00	0.75	1.00	28	20.00	4.48	32	4.000
2.00	22	125.00	0.13	0.00	0.13	0.33	1.70	1.00	1.00	0.75	1.00	28	20.00	4.48	32	4.000
2.50	22	125.00	0.16	0.00	0.16	0.33	1.70	1.00	1.00	0.75	1.00	28	20.00	4.48	32	4.000
3.00	25	125.00	0.19	0.00	0.19	0.35	1.70	1.00	1.00	0.75	1.00	32	5.00	0.00	32	4.000
3.50	25	125.00	0.22	0.00	0.22	0.35	1.70	1.00	1.00	0.75	1.00	32	5.00	0.00	32	4.000
4.00	25	125.00	0.25	0.00	0.25	0.35	1.67	1.00	1.00	0.75	1.00	31	5.00	0.00	31	4.000
4.50	25	125.00	0.28	0.00	0.28	0.36	1.61	1.00	1.00	0.75	1.00	30	5.00	0.00	30	4.000
5.00	9	120.00	0.31	0.00	0.31	0.52	1.70	1.00	1.00	0.75	1.00	11	5.00	0.00	11	4.000
5.50	9	120.00	0.34	0.00	0.34	0.52	1.70	1.00	1.00	0.75	1.00	11	5.00	0.00	11	4.000
6.00	9	120.00	0.37	0.00	0.37	0.52	1.70	1.00	1.00	0.75	1.00	11	5.00	0.00	11	4.000
6.50	9	120.00	0.40	0.00	0.40	0.53	1.67	1.00	1.00	0.75	1.00	11	5.00	0.00	11	4.000
7.00	9	120.00	0.43	0.00	0.43	0.52	1.60	1.00	1.00	0.80	1.00	12	5.00	0.00	12	4.000
7.50	9	120.00	0.46	0.00	0.46	0.53	1.55	1.00	1.00	0.80	1.00	11	5.00	0.00	11	4.000
8.00	20	130.00	0.49	0.00	0.49	0.42	1.38	1.00	1.00	0.80	1.00	22	5.00	0.00	22	0.233
8.50	20	130.00	0.53	0.02	0.51	0.43	1.36	1.00	1.00	0.80	1.00	22	5.00	0.00	22	0.233
9.00	20	130.00	0.56	0.03	0.53	0.43	1.35	1.00	1.00	0.80	1.00	22	5.00	0.00	22	0.233
9.50	20	130.00	0.59	0.05	0.54	0.43	1.33	1.00	1.00	0.80	1.00	21	5.00	0.00	21	0.219
10.00	20	130.00	0.62	0.06	0.56	0.42	1.31	1.00	1.00	0.85	1.00	22	5.00	0.00	22	0.233
10.50	21	130.00	0.66	0.08	0.58	0.42	1.29	1.00	1.00	0.85	1.00	23	5.00	0.00	23	0.249
11.00	21	130.00	0.69	0.09	0.60	0.42	1.27	1.00	1.00	0.85	1.00	23	5.00	0.00	23	0.249
11.50	21	130.00	0.72	0.11	0.61	0.42	1.26	1.00	1.00	0.85	1.00	22	5.00	0.00	22	0.233
12.00	21	130.00	0.75	0.12	0.63	0.42	1.25	1.00	1.00	0.85	1.00	22	5.00	0.00	22	0.233
12.50	21	130.00	0.79	0.14	0.65	0.42	1.23	1.00	1.00	0.85	1.00	22	5.00	0.00	22	0.233
13.00	21	130.00	0.82	0.16	0.66	0.43	1.22	1.00	1.00	0.85	1.00	22	5.00	0.00	22	0.233
13.50	21	130.00	0.85	0.17	0.68	0.43	1.21	1.00	1.00	0.85	1.00	22	5.00	0.00	22	0.233
14.00	21	130.00	0.88	0.19	0.70	0.43	1.20	1.00	1.00	0.85	1.00	21	5.00	0.00	21	0.219
14.50	21	130.00	0.92	0.20	0.71	0.43	1.19	1.00	1.00	0.85	1.00	21	5.00	0.00	21	0.219
15.00	21	130.00	0.95	0.22	0.73	0.43	1.17	1.00	1.00	0.85	1.00	21	5.00	0.00	21	0.219
15.50	18	130.00	0.98	0.23	0.75	0.46	1.17	1.00	1.00	0.85	1.00	18	5.00	0.00	18	0.184
16.00	18	130.00	1.01	0.25	0.76	0.46	1.16	1.00	1.00	0.85	1.00	18	5.00	0.00	18	0.184
16.50	18	130.00	1.05	0.27	0.78	0.44	1.14	1.00	1.00	0.95	1.00	20	5.00	0.00	20	0.206
17.00	18	130.00	1.08	0.28	0.80	0.45	1.13	1.00	1.00	0.95	1.00	19	5.00	0.00	19	0.194
17.50	18	130.00	1.11	0.30	0.81	0.45	1.12	1.00	1.00	0.95	1.00	19	5.00	0.00	19	0.194
18.00	18	130.00	1.14	0.31	0.83	0.45	1.11	1.00	1.00	0.95	1.00	19	5.00	0.00	19	0.194

:: Cyclic Resistance Ratio (CRR) calculation data ::																
Depth (ft)	SPT Field Value	Unit Weight (pcf)	α_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}
18.50	18	130.00	1.18	0.33	0.85	0.45	1.10	1.00	1.00	0.95	1.00	19	5.00	0.00	19	0.194
19.00	18	130.00	1.21	0.34	0.87	0.45	1.09	1.00	1.00	0.95	1.00	19	5.00	0.00	19	0.194
19.50	18	130.00	1.24	0.36	0.88	0.45	1.09	1.00	1.00	0.95	1.00	19	5.00	0.00	19	0.194
20.00	18	130.00	1.27	0.37	0.90	0.45	1.08	1.00	1.00	0.95	1.00	18	5.00	0.00	18	0.184
20.50	18	130.00	1.31	0.39	0.92	0.46	1.07	1.00	1.00	0.95	1.00	18	5.00	0.00	18	0.184
21.00	18	130.00	1.34	0.41	0.93	0.46	1.06	1.00	1.00	0.95	1.00	18	5.00	0.00	18	0.184
21.50	38	130.00	1.37	0.42	0.95	0.31	1.03	1.00	1.00	0.95	1.00	37	5.00	0.00	37	4.000
22.00	38	130.00	1.40	0.44	0.97	0.32	1.03	1.00	1.00	0.95	1.00	37	5.00	0.00	37	4.000
22.50	38	130.00	1.44	0.45	0.98	0.32	1.02	1.00	1.00	0.95	1.00	37	5.00	0.00	37	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 correction 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
- $N_{1(60)cs}$: Corrected $N_{1(60)}$ 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)	$\alpha_{v,eq}$ (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	125.00	0.00	0.00	0.00	1.01	1.00	0.452	2.12	32	1.20	0.377	1.10	0.343	2.000	●	
0.50	125.00	0.03	0.00	0.03	1.01	1.00	0.451	2.12	32	1.20	0.376	1.10	0.342	2.000	●	
1.00	125.00	0.06	0.00	0.06	1.00	1.00	0.450	2.12	32	1.20	0.376	1.10	0.342	2.000	●	
1.50	125.00	0.09	0.00	0.09	1.00	1.00	0.450	2.12	32	1.20	0.375	1.10	0.341	2.000	●	
2.00	125.00	0.13	0.00	0.13	1.00	1.00	0.449	2.12	32	1.20	0.375	1.10	0.341	2.000	●	
2.50	125.00	0.16	0.00	0.16	1.00	1.00	0.448	2.12	32	1.20	0.374	1.10	0.340	2.000	●	
3.00	125.00	0.19	0.00	0.19	1.00	1.00	0.448	2.12	32	1.20	0.374	1.10	0.340	2.000	●	
3.50	125.00	0.22	0.00	0.22	1.00	1.00	0.447	2.12	32	1.20	0.373	1.10	0.339	2.000	●	
4.00	125.00	0.25	0.00	0.25	1.00	1.00	0.446	2.06	31	1.19	0.376	1.10	0.342	2.000	●	
4.50	125.00	0.28	0.00	0.28	0.99	1.00	0.446	2.00	30	1.18	0.379	1.10	0.345	2.000	●	
5.00	120.00	0.31	0.00	0.31	0.99	1.00	0.445	1.21	11	1.04	0.429	1.10	0.390	2.000	●	
5.50	120.00	0.34	0.00	0.34	0.99	1.00	0.444	1.21	11	1.04	0.428	1.10	0.389	2.000	●	
6.00	120.00	0.37	0.00	0.37	0.99	1.00	0.443	1.21	11	1.04	0.427	1.10	0.389	2.000	●	
6.50	120.00	0.40	0.00	0.40	0.99	1.00	0.443	1.21	11	1.04	0.427	1.09	0.390	2.000	●	
7.00	120.00	0.43	0.00	0.43	0.98	1.00	0.442	1.24	12	1.04	0.424	1.09	0.389	2.000	●	
7.50	120.00	0.46	0.00	0.46	0.98	1.00	0.441	1.21	11	1.04	0.425	1.08	0.394	2.000	●	
8.00	130.00	0.49	0.00	0.49	0.98	1.00	0.440	1.58	22	1.10	0.399	1.10	0.363	0.642	●	
8.50	130.00	0.53	0.02	0.51	0.98	1.00	0.453	1.58	22	1.10	0.411	1.10	0.373	0.624	●	
9.00	130.00	0.56	0.03	0.53	0.98	1.00	0.464	1.58	22	1.10	0.421	1.10	0.383	0.608	●	
9.50	130.00	0.59	0.05	0.54	0.98	1.00	0.475	1.53	21	1.09	0.434	1.09	0.398	0.550	●	
10.00	130.00	0.62	0.06	0.56	0.97	1.00	0.485	1.58	22	1.10	0.440	1.09	0.404	0.578	●	
10.50	130.00	0.66	0.08	0.58	0.97	1.00	0.495	1.62	23	1.11	0.446	1.09	0.409	0.610	●	
11.00	130.00	0.69	0.09	0.60	0.97	1.00	0.503	1.62	23	1.11	0.453	1.09	0.417	0.597	●	

:: 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_{σ}	CSR*	FS	
11.50	130.00	0.72	0.11	0.61	0.97	1.00	0.511	1.58	22	1.10	0.464	1.08	0.430	0.542	●
12.00	130.00	0.75	0.12	0.63	0.97	1.00	0.519	1.58	22	1.10	0.471	1.07	0.438	0.532	●
12.50	130.00	0.79	0.14	0.65	0.96	1.00	0.526	1.58	22	1.10	0.477	1.07	0.446	0.523	●
13.00	130.00	0.82	0.16	0.66	0.96	1.00	0.533	1.58	22	1.10	0.483	1.07	0.453	0.515	●
13.50	130.00	0.85	0.17	0.68	0.96	1.00	0.539	1.58	22	1.10	0.489	1.06	0.460	0.507	●
14.00	130.00	0.88	0.19	0.70	0.96	1.00	0.545	1.53	21	1.09	0.498	1.06	0.470	0.465	●
14.50	130.00	0.92	0.20	0.71	0.95	1.00	0.550	1.53	21	1.09	0.503	1.05	0.477	0.459	●
15.00	130.00	0.95	0.22	0.73	0.95	1.00	0.555	1.53	21	1.09	0.507	1.05	0.482	0.453	●
15.50	130.00	0.98	0.23	0.75	0.95	1.00	0.560	1.42	18	1.07	0.522	1.04	0.500	0.367	●
16.00	130.00	1.01	0.25	0.76	0.95	1.00	0.564	1.42	18	1.07	0.526	1.04	0.505	0.364	●
16.50	130.00	1.05	0.27	0.78	0.95	1.00	0.568	1.49	20	1.09	0.523	1.04	0.503	0.410	●
17.00	130.00	1.08	0.28	0.80	0.94	1.00	0.572	1.45	19	1.08	0.530	1.04	0.511	0.380	●
17.50	130.00	1.11	0.30	0.81	0.94	1.00	0.576	1.45	19	1.08	0.533	1.03	0.516	0.377	●
18.00	130.00	1.14	0.31	0.83	0.94	1.00	0.579	1.45	19	1.08	0.536	1.03	0.520	0.374	●
18.50	130.00	1.18	0.33	0.85	0.94	1.00	0.582	1.45	19	1.08	0.539	1.03	0.524	0.371	●
19.00	130.00	1.21	0.34	0.87	0.93	1.00	0.585	1.45	19	1.08	0.542	1.03	0.528	0.368	●
19.50	130.00	1.24	0.36	0.88	0.93	1.00	0.588	1.45	19	1.08	0.544	1.02	0.532	0.365	●
20.00	130.00	1.27	0.37	0.90	0.93	1.00	0.590	1.42	18	1.07	0.550	1.02	0.539	0.341	●
20.50	130.00	1.31	0.39	0.92	0.93	1.00	0.593	1.42	18	1.07	0.552	1.02	0.543	0.339	●
21.00	130.00	1.34	0.41	0.93	0.92	1.00	0.595	1.42	18	1.07	0.554	1.02	0.546	0.337	●
21.50	130.00	1.37	0.42	0.95	0.92	1.00	0.597	2.20	37	1.21	0.493	1.03	0.477	2.000	●
22.00	130.00	1.40	0.44	0.97	0.92	1.00	0.599	2.20	37	1.21	0.494	1.03	0.481	2.000	●
22.50	130.00	1.44	0.45	0.98	0.92	1.00	0.600	2.20	37	1.21	0.496	1.02	0.485	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_{σ} : 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	0.49	0.00
0.50	2.000	0.00	9.92	0.49	0.00
1.00	2.000	0.00	9.85	0.50	0.00
1.50	2.000	0.00	9.77	0.50	0.00
2.00	2.000	0.00	9.70	0.50	0.00
2.50	2.000	0.00	9.62	0.50	0.00
3.00	2.000	0.00	9.54	0.50	0.00
3.50	2.000	0.00	9.47	0.50	0.00
4.00	2.000	0.00	9.39	0.50	0.00
4.50	2.000	0.00	9.31	0.50	0.00

:: Liquefaction potential according to Iwasaki ::					
Depth (ft)	FS	F	wz	Thickness (ft)	I _L
5.00	2.000	0.00	9.24	0.50	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
6.50	2.000	0.00	9.01	0.50	0.00
7.00	2.000	0.00	8.93	0.50	0.00
7.50	2.000	0.00	8.86	0.50	0.00
8.00	0.642	0.36	8.78	0.50	0.48
8.50	0.624	0.38	8.70	0.50	0.50
9.00	0.608	0.39	8.63	0.50	0.52
9.50	0.550	0.45	8.55	0.50	0.59
10.00	0.578	0.42	8.48	0.50	0.55
10.50	0.610	0.39	8.40	0.50	0.50
11.00	0.597	0.40	8.32	0.50	0.51
11.50	0.542	0.46	8.25	0.50	0.58
12.00	0.532	0.47	8.17	0.50	0.58
12.50	0.523	0.48	8.10	0.50	0.59
13.00	0.515	0.49	8.02	0.50	0.59
13.50	0.507	0.49	7.94	0.50	0.60
14.00	0.465	0.54	7.87	0.50	0.64
14.50	0.459	0.54	7.79	0.50	0.64
15.00	0.453	0.55	7.71	0.50	0.64
15.50	0.367	0.63	7.64	0.50	0.74
16.00	0.364	0.64	7.56	0.50	0.73
16.50	0.410	0.59	7.49	0.50	0.67
17.00	0.380	0.62	7.41	0.50	0.70
17.50	0.377	0.62	7.33	0.50	0.70
18.00	0.374	0.63	7.26	0.50	0.69
18.50	0.371	0.63	7.18	0.50	0.69
19.00	0.368	0.63	7.10	0.50	0.68
19.50	0.365	0.63	7.03	0.50	0.68
20.00	0.341	0.66	6.95	0.50	0.70
20.50	0.339	0.66	6.88	0.50	0.69
21.00	0.337	0.66	6.80	0.50	0.69
21.50	2.000	0.00	6.72	0.50	0.00
22.00	2.000	0.00	6.65	0.50	0.00
22.50	2.000	0.00	6.57	0.50	0.00

Overall potential I_L : 16.87

- 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)	a	b	γ	ε ₁₅	N _c	ε _{Nc} weight factor	ε _{Nc} (%)	Δh (ft)	ΔS (in)
0.01	28	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.49	0.000
0.50	28	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.000

:: Vertical settlements estimation for dry sands ::													
Depth (ft)	(N ₁) ₆₀	T _{av}	p	G _{max} (tsf)	a	b	γ	ε ₁₅	N _c	ε _{Nc} weight factor	ε _{Nc} (%)	Δh (ft)	ΔS (in)
1.00	28	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.000
1.50	28	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.000
2.00	28	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.000
2.50	28	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.000
3.00	32	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.000
3.50	32	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.000
4.00	31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.000
4.50	30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.000
5.00	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	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	0.00	0.00	0.50	0.000
6.00	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.000
6.50	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.000
7.00	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.000
7.50	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.000

Cumulative settlements: **0.000**

Abbreviations

- T_{av}: Average cyclic shear stress
- p: Average stress
- G_{max}: Maximum shear modulus (tsf)
- a, b: Shear strain formula variables
- γ: Average shear strain
- ε₁₅: 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 displacements estimation for saturated sands ::										
Depth (ft)	(N ₁) _{60cs}	γ _{lim} (%)	F _a	FS _{liq}	γ _{max} (%)	e _v weight factor	e _v (%)	dz (ft)	S _{v-1D} (in)	LDI (ft)
8.00	22	12.67	0.41	0.642	12.05	0.88	1.86	0.50	0.112	0.00
8.50	22	12.67	0.41	0.624	12.67	0.87	1.85	0.50	0.111	0.00
9.00	22	12.67	0.41	0.608	12.67	0.86	1.83	0.50	0.110	0.00
9.50	21	14.20	0.46	0.550	14.20	0.85	1.89	0.50	0.113	0.00
10.00	22	12.67	0.41	0.578	12.67	0.85	1.80	0.50	0.108	0.00
10.50	23	11.27	0.35	0.610	11.27	0.84	1.71	0.50	0.103	0.00
11.00	23	11.27	0.35	0.597	11.27	0.83	1.70	0.50	0.102	0.00
11.50	22	12.67	0.41	0.542	12.67	0.82	1.75	0.50	0.105	0.00
12.00	22	12.67	0.41	0.532	12.67	0.82	1.73	0.50	0.104	0.00
12.50	22	12.67	0.41	0.523	12.67	0.81	1.72	0.50	0.103	0.00
13.00	22	12.67	0.41	0.515	12.67	0.80	1.70	0.50	0.102	0.00
13.50	22	12.67	0.41	0.507	12.67	0.79	1.68	0.50	0.101	0.00
14.00	21	14.20	0.46	0.465	14.20	0.78	1.74	0.50	0.104	0.00
14.50	21	14.20	0.46	0.459	14.20	0.78	1.72	0.50	0.103	0.00
15.00	21	14.20	0.46	0.453	14.20	0.77	1.70	0.50	0.102	0.00
15.50	18	19.85	0.62	0.367	19.85	0.76	1.91	0.50	0.115	0.00
16.00	18	19.85	0.62	0.364	19.85	0.75	1.89	0.50	0.113	0.00
16.50	20	15.90	0.52	0.410	15.90	0.75	1.72	0.50	0.103	0.00
17.00	19	17.78	0.57	0.380	17.78	0.74	1.77	0.50	0.106	0.00

:: Vertical & Lateral displacements estimation for saturated sands ::										
Depth (ft)	(N₁)_{60cs}	Y_{lim} (%)	F_σ	FS_{liq}	Y_{max} (%)	e_v weight factor	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
17.50	19	17.78	0.57	0.377	17.78	0.73	1.76	0.50	0.105	0.00
18.00	19	17.78	0.57	0.374	17.78	0.72	1.74	0.50	0.104	0.00
18.50	19	17.78	0.57	0.371	17.78	0.72	1.72	0.50	0.103	0.00
19.00	19	17.78	0.57	0.368	17.78	0.71	1.70	0.50	0.102	0.00
19.50	19	17.78	0.57	0.365	17.78	0.70	1.68	0.50	0.101	0.00
20.00	18	19.85	0.62	0.341	19.85	0.69	1.74	0.50	0.104	0.00
20.50	18	19.85	0.62	0.339	19.85	0.68	1.72	0.50	0.103	0.00
21.00	18	19.85	0.62	0.337	19.85	0.68	1.70	0.50	0.102	0.00
21.50	37	1.56	-0.58	2.000	0.00	0.67	0.00	0.50	0.000	0.00
22.00	37	1.56	-0.58	2.000	0.00	0.66	0.00	0.50	0.000	0.00
22.50	37	1.56	-0.58	2.000	0.00	0.65	0.00	0.50	0.000	0.00

Cumulative settlements: 2.845 0.00

Abbreviations

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

References

- Ronald D. Andrus, Hossein Hayati, Nisha P. Mohanan, 2009. Correcting Liquefaction Resistance for Aged Sands Using Measured to Estimated Velocity Ratio, *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 135, No. 6, June 1
- Boulanger, R.W. and Idriss, I. M., 2014. CPT AND SPT BASED LIQUEFACTION TRIGGERING PROCEDURES. DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING COLLEGE OF ENGINEERING UNIVERSITY OF CALIFORNIA AT DAVIS
- Dipl.-Ing. Heinz J. Priebe, Vibro Replacement to Prevent Earthquake Induced Liquefaction, Proceedings of the Geotechnique-Colloquium at Darmstadt, Germany, on March 19th, 1998 (also published in *Ground Engineering*, September 1998), Technical paper 12-57E
- Robertson, P.K. and Cabal, K.L., 2007, Guide to Cone Penetration Testing for Geotechnical Engineering. Available at no cost at <http://www.geologismiki.gr/>
- Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J., Liao, S., Marcuson III, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R., and Stokoe, K.H., Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils, ASCE, *Journal of Geotechnical & Geoenvironmental Engineering*, Vol. 127, October, pp 817-833
- Zhang, G., Robertson. P.K., Brachman, R., 2002, Estimating Liquefaction Induced Ground Settlements from the CPT, *Canadian Geotechnical Journal*, 39: pp 1168-1180
- Zhang, G., Robertson. P.K., Brachman, R., 2004, Estimating Liquefaction Induced Lateral Displacements using the SPT and CPT, ASCE, *Journal of Geotechnical & Geoenvironmental Engineering*, Vol. 130, No. 8, 861-871
- Pradel, D., 1998, Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils, ASCE, *Journal of Geotechnical & Geoenvironmental Engineering*, Vol. 124, No. 4, 364-368
- R. Kayen, R. E. S. Moss, E. M. Thompson, R. B. Seed, K. O. Cetin, A. Der Kiureghian, Y. Tanaka, K. Tokimatsu, 2013. Shear-Wave Velocity-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential, *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 139, No. 3, March 1