Cobalt Geosciences, LLC
P.O. Box 1792
North Bend, WA 98045

May 21, 2024
Updated February 2, 2026

MacPherson Construction and Design
Attn: Mr. Dan Buchser
dan@macphersonconstruction.com

RE: Geotechnical Evaluation
Proposed Additions/Remodel
5320 Butterworth Road – CAO25-011
Mercer Island, Washington

In accordance with your authorization, Cobalt Geosciences, LLC has prepared this report to discuss foundation design, grading, and geologic hazards for the proposed project at the above-referenced site location.

Site & Project Description

The site is located at 5330 (5320 new address) Butterworth Road in Mercer Island, Washington. The site consists of one irregularly shaped parcel (No. 8661400040) with a total area of 82,328 square feet.

The property is developed with a residence, sport court, pool areas and water features, driveway, and walkways. A short driveway extends onto the property. Site vegetation includes grasses, bushes, and variable diameter trees.

The site slopes downward from west to east at magnitudes ranging from about 5 to 25 percent and relief of about 26 feet. There is a 2 to 3 feet tall rockery along the shoreline (east property line). The site is bordered to the north and south by residential properties, to the east by Lake Washington, and to the west by Butterworth Road.

Based on our review of provided historic documents, it appears that the structure is partially or wholly supported on auger-cast piles extending into presumed dense soils that underlie the area. In general, we anticipate the depth of loose soils will increase to the east and Lake Washington.

The site contains seismic and potential landslide hazard areas per City mapping.

The project includes subdivision of the property and remodeling of the structure into two residences. This includes removal of portions of the current residence along with some new foundation elements where required. Cuts will be 3 feet or less and foundation loads will generally be light.

Area Geology

The Geologic Map of Mercer Island, indicates that the site is underlain by Pre-Olympia Non-Glacial Fine Grained Deposits and possibly Pre-Olympia Coarse Grained Deposits.

These materials include silts and sands deposited prior to the Vashon-era glaciation. Most of these deposits would have been consolidated by this glaciation; however, subsequent fluvial processes resulted in loose zones of variable thickness and extent.

Soil & Groundwater Conditions

As part of our evaluation, we drilled a hollow stem auger boring where accessible. This work was performed on May 15, 2024. We returned in November 2024 to advance a Cone Penetrometer Test (CPT) boring in the driveway of the existing residence.

Disturbed soil samples were obtained during drilling by using the Standard Penetration Test (SPT) as described in ASTM D-1586. The Standard Penetration Test and sampling method consists of driving a standard 2-inch outside-diameter, split barrel sampler into the subsoil with a 140-pound hammer free falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the Standard Penetration Resistance, or N-value. The blow count is presented graphically on the boring logs in this appendix. The resistance, or "N" value, provides a measure of the relative density of granular soils or of the relative consistency of cohesive soils.

The soils encountered were logged in the field and are described in accordance with the Unified Soil Classification System (USCS).

The boring encountered approximately 6 inches of topsoil underlain by approximately 4.5 feet of very loose to loose, silty-fine to medium grained sand trace gravel (Fill). These materials were underlain by loose to medium dense, fine to medium grained sand (Pre-Olympia Deposits), which continued to the termination depth of the exploration.

We achieved refusal due to heave of sands into the augers. We note that denser soils are likely present around 20 feet below grade in the boring based on the level of heave present.

The CPT boring encountered layered silty-sands, sands, and fine grained soils which became denser with depth. Dense to very dense gravelly sands were present below about 28 feet in this boring.

Groundwater was observed about 4 feet below grade during drilling in B-1. Groundwater is likely at shallow depths, generally consistent with the elevation of Lake Washington. Groundwater continues through the encountered soils to the denser soils that underlie this area.

Water table elevations often fluctuate over time. The groundwater level will depend on a variety of factors that may include seasonal precipitation, irrigation, land use, climatic conditions and soil permeability. Water levels at the time of the field investigation may be different from those encountered during the construction phase of the project. It would be necessary to install a piezometer to determine groundwater depths over a typical year.

City of Mercer Island GIS Mapped Hazards

The City of Mercer Island GIS maps indicate that the site contains potential slide and seismic hazard areas.

The potential landslide hazard designation is likely due to the presence of older non-glacial deposits of variable composition and density. Slope magnitudes are generally low in this area; however, groundwater is at shallow depths, which could result in instability with specific geologic conditions present.

Seismic hazards are moderate to high, increasing from west to east toward Lake Washington. This is due to the presence of loose sediments with a high groundwater level. Deep foundations will be utilized to support new foundation elements to minimize the risk of liquefaction induced settlement.

Discussion of mitigation sequencing is as follows:

Except as otherwise provided in this chapter, an applicant for a development proposal or activity shall implement the following sequential measures, listed below in order of preference, to avoid, minimize, and mitigate impacts to environmentally critical areas and associated buffers. Applicants shall document how each measure has been addressed before considering and incorporating the next measure in the sequence:

A. Avoiding the impact altogether by not taking a certain action or parts of an action. The applicant shall consider reasonable, affirmative steps and make best efforts to avoid critical area impacts. However, avoidance shall not be construed to mean mandatory withdrawal or denial of the development proposal or activity if the proposal or activity is an allowed, permitted, or conditional use in this title. In determining the extent to which the proposal should be redesigned to avoid the impact, the code official may consider the purpose, effectiveness, engineering feasibility, commercial availability of technology, best management practices, safety and cost of the proposal and identified changes to the proposal. Development proposals should seek to avoid, minimize and mitigate overall impacts based on the functions and values of all of the relevant critical areas and based on the recommendations of a critical area study. If impacts cannot be avoided through redesign, use of a setback deviation pursuant to section 19.06.110(C), or because of site conditions or project requirements, the applicant shall then proceed with the sequence of steps in subsections B through E of this section;

<u>We have analyzed liquefaction risks and provided recommendations for deep foundations and grade beam systems, similar to what is currently present. Mitigation of liquefaction risks was not avoidable due to the geologic conditions.</u>

B. Minimizing impacts by limiting the degree or magnitude of the action and its implementation, using a setback deviation pursuant to section 19.06.110(C), using appropriate technology, or by taking affirmative steps to avoid or reduce impacts;

<u>Pile installation is the least impactful method to create a stable building pad and foundation system. Interconnecting grade beams are necessary (or a raft foundation) to retain structural integrity in the case of flow failure and/or lateral spread.</u>

C. Rectifying the impact by repairing, rehabilitating, or restoring the affected environment;

<u>Not applicable. Piles reduce the impact by transferring building loads to deeper bearing strata.</u>

D. Reducing or eliminating the impact over time by preservation and maintenance operations during the life of the action;

<u>Not applicable.</u>

E. Compensating for the impact by replacing, enhancing, or providing substitute resources or environments; and/or

<u>Not applicable.</u>

F. Monitoring the impact and taking appropriate corrective measures to maintain the integrity of compensating measures.

<u>Monitoring would include observation of foundation excavations, pile installation and load testing. Post construction monitoring is not required.</u>

Statement of Risk

Per Section 19.07.160B3 of the Mercer Island City Code, development within geologic hazard areas require that a Geotechnical Engineer licensed within the State of Washington provide a statement of risk with supporting documentation indicating that one of the following conditions can be met:

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; or

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

The project meets the criteria of C from above. The construction will render the affected area as safe as if it were not located in a geologic hazard area. This includes deep foundation elements to support new foundations. The risk of landslide activity is low and will not be increased or decreased.

Erosion Hazard

The Natural Resources Conservation Services (NRCS) maps for King County indicate that the site is underlain by Kitsap silt loam (2 to 8 and 15 to 30 percent slopes). These soils would have a slight to very severe erosion potential in a disturbed state depending on the slope magnitude.

It is our opinion that soil erosion potential at this project site can be reduced through landscaping and surface water runoff control. Typically, erosion of exposed soils will be most noticeable during periods of rainfall and may be controlled by the use of normal temporary erosion control measures, such as silt fences, hay bales, mulching, control ditches and diversion trenches. The typical wet weather season, with regard to site grading, is from October 1st to April 1st. Erosion control measures should be in place before the onset of wet weather.

Seismic Parameters

The overall subsurface profile corresponds to a Site Class *F* as defined by Table 1613.5.2 of the International Building Code (IBC). A Site Class *F* applies to an overall profile consisting of medium dense to very dense soils within the upper 100 feet.

We referenced the U.S. Geological Survey (USGS) Earthquake Hazards Program Website to obtain values for S_S, S_1, F_a, and F_v. The USGS website includes the most updated published data on seismic conditions. The following tables provide seismic parameters from the USGS web site with referenced parameters from ASCE 7-16.

Seismic Design Parameters (ASCE 7-16)

Site Class	Spectral Acceleration at 0.2 sec. (g)	Spectral Acceleration at 1.0 sec. (g)	Site Coefficients		Design Spectral Response Parameters		Design PGA
			F_a	F_v	S_{DS}	S_{D1}	
F	1.437	0.499	Null	Null	Null	Null	0.615

For items listed as "Null" see Section 11.4.8 of the ASCE.

Additional seismic considerations include liquefaction potential and amplification of ground motions by soft/loose soil deposits. The liquefaction potential is highest for loose sand with a high groundwater table.

Soil liquefaction is a state where soil particles lose contact with each other and become suspended in a viscous fluid. This suspension of the soil grains results in a complete loss of strength as the effective stress drops to zero as a result of increased pore pressures. Liquefaction normally occurs under saturated conditions in soils such as sand in which the strength is purely frictional. However, liquefaction has occurred in soils other than clean sand, such as low plasticity silt. Liquefaction usually occurs under vibratory conditions such as those induced by seismic events.

To evaluate the liquefaction potential of the site, we analyzed the following factors:

1) Soil type and plasticity

2) Groundwater depth

3) Relative soil density

4) Initial confining pressure

5) Maximum anticipated intensity and duration of ground shaking

The commercially available liquefaction analysis software, LiqSVS was used to evaluate the liquefaction potential and the possible liquefaction induced settlement for the existing site soil conditions. Maximum Considered Earthquake (MCE) was selected in accordance with the ASCE, *International Building Code* (IBC) and the U.S. Geological Survey (USGS) Earthquake Hazards Program website.

We have updated these analyses utilizing more detailed interpretation of the CPT log with narrower delineation of soil types and relative density. The lateral spread analysis was in accordance with Zhang et al. (2004) method, used in the LiqSVS software with the specific site location.

The liquefaction settlement was found to be about 7 inches with a differential settlement of about 3.5 inches over a 25 foot span. Lateral spread was determined to be on the order of about 6.5 feet.

Based on these results, lateral spread is most likely to occur near the eastern margin of the residential structure, decreasing to the west and generally increasing to the east.

At this time, the planned foundation system includes interconnecting grade beams bearing on pipe piles driven to refusal in the denser soils that underlie this area. We have discussed options for mitigation with the structural engineer and potential pile contractor.

The proposed mitigation/risk reduction system includes vertical pipe piles (as before), grade beams, and additional battered piles (4 inches in diameter) as determined by the structural engineer and their upcoming design. The batter angles and directions will be determined by the structural engineer.

Slope Stability Flow Analyses

We performed slope stability (flow) analyses through a representational cross section through the property and borings. Analyses were performed using data from the explorations, location and anticipated elevations of the proposed structure, and topography from City and County GIS maps. Groundwater depths were interpreted from nearby explorations on downslope properties. We used a nautical chart of Lake Washington from Fishermap (lakebed elevations).

The commercially available slope stability computer program Slope/W was used to evaluate the global stability of the slope within the property. The slope stability was analyzed under static and seismic (pseudo-static method) conditions for the existing topography with residual soil properties during/after liquefaction. Residual values were determined through correlation of N values/CPT blows (equivalent N values) and fines content (described in more detail below).

The computer program calculates factors of safety for potential slope failures and generates the potential failure planes. This software calculates the slope stability under seismic conditions using pseudo-static methods. The stability of the described configuration was analyzed by comparing observed factors of safety to minimum values as set by standard geotechnical practice.

In accordance with typical engineering standards, we used a seismic acceleration equal to one half of the horizontal peak ground acceleration. At this location, the site modified PGA is 0.615 with one half equal to 0.32.

The following estimated soil parameters were used in our analyses:

Soil Description (Existing Conditions)	Unit Weight (pcf)	Undrained Shear Strength (psf)
Loose to Medium Dense Alluvium Silty-Sand and Fine Grained Soils	110	50
Medium Dense to Dense Sands/Silty-Sands	115	100
Dense to Very Dense Gravelly Sands	125	400

Analysis Results

Cross Section	Static Factor of Safety	0.32g Seismic Factor of Safety
Post Liquefaction (residual soil strength)	1.010	0.211

The analyses indicate movements are likely to occur during/after certain seismic events. Displacements are noted in the lateral spread analyses. These confirm lateral movements to occur toward and into Lake Washington during the design event.

Note that residual values were correlated to the N values from the borings (correlated in the case of CPT-1) using Olson and Stark (2002), Idriss and Boulanger (2007), and Kramer-Wang from the publication Evaluation of Liquefaction Hazards in Washington State by Steven Kramer December 2008. We also reviewed Residual Shear Streng of Liquefied Soils by Ross Boulanger 2007 which included actual examples of shear strength based on fines content and blow counts.

Conclusions and Recommendations

General

The site is underlain by areas of fill and areas of loose deposits which overlie denser glacially consolidated materials at variable depths. The near-surface materials have potential for liquefaction during/after certain seismic events.

The new building areas will be supported on pipe piles driven to refusal, battered piles driven to refusal and interconnecting grade beams throughout the new foundation areas.

In the event of several lateral spread and flow, the interconnecting grade beam system would serve to support the structure in one piece, even if significant tilting occurs. The grade beam systems will serve to maintain life/safety for severe seismic events.

We estimate piles to extend 30 to 70 feet below grade (or more) depending on the loading required, elevations, and hammer sizes. Deeper penetrations may be observed toward the east with lower depths likely to be observed further west. Final depths may vary with location.

Plan Review

We have reviewed the architectural plans by MacPherson Construction and Design dated February 3, 2026, civil plans by Ethos dated September 2, 2025, and structural plans by Mulhern & Kulp dated February 3, 2026 which includes the pin piles and interconnected grade beams.

The plans appear to include relevant information from the geotechnical report. We have no comments at this time.

We must be on site to observe pile placement and other aspects of earthwork construction as noted in the previous geotechnical report and letters.

Site Preparation

Trees, shrubs and other vegetation should be removed prior to stripping of surficial organic-rich soil and fill. Based on observations from the site investigation program, it is anticipated that the stripping depth will be 6 to 18 inches. Deeper excavations will be necessary in areas of loose soils, if they remain once building and grading elevations are achieved.

The native soils consist of silty-sand with gravel and poorly graded sands. Some of the native soils may be used as structural fill provided they achieve compaction requirements and are within 3 percent of the optimum moisture. Some of these soils may only be suitable for use as fill during the summer months, as they will be above the optimum moisture levels in their current state. These soils are variably moisture sensitive and may degrade during periods of wet weather and under equipment traffic.

Imported structural fill should consist of a sand and gravel mixture with a maximum grain size of 3 inches and less than 5 percent fines (material passing the U.S. Standard No. 200 Sieve). Structural fill should be placed in maximum lift thicknesses of 12 inches and should be compacted to a minimum of 95 percent of the modified proctor maximum dry density, as determined by the ASTM D 1557 test method.

Temporary Excavations

Based on our understanding of the project, we anticipate that the grading could include local cuts on the order of approximately 3 feet or less for foundation and most of the utility placement. Temporary excavations should be sloped no steeper than 1.5H:1V (Horizontal:Vertical) in loose native soils and fill. If an excavation is subject to heavy vibration or surcharge loads, we recommend that the excavations be sloped no steeper than 2H:1V, where room permits.

Temporary cuts should be in accordance with the Washington Administrative Code (WAC) Part N, Excavation, Trenching, and Shoring. Temporary slopes should be visually inspected daily by a qualified person during construction activities and the inspections should be documented in daily reports. The contractor is responsible for maintaining the stability of the temporary cut slopes and reducing slope erosion during construction.

Temporary cut slopes should be covered with visqueen to help reduce erosion during wet weather, and the slopes should be closely monitored until the permanent retaining systems or slope configurations are complete. Materials should not be stored or equipment operated within 10 feet of the top of any temporary cut slope.

Soil conditions may not be completely known from the geotechnical investigation. In the case of temporary cuts, the existing soil conditions may not be completely revealed until the excavation work exposes the soil. Typically, as excavation work progresses the maximum inclination of temporary slopes will need to be re-evaluated by the geotechnical engineer so that supplemental recommendations can be made. Soil and groundwater conditions can be highly variable. Scheduling for soil work will need to be adjustable, to deal with unanticipated conditions, so that the project can proceed and required deadlines can be met.

If any variations or undesirable conditions are encountered during construction, we should be notified so that supplemental recommendations can be made. If room constraints or groundwater conditions do not permit temporary slopes to be cut to the maximum angles allowed by the WAC, temporary shoring systems may be required. The contractor should be responsible for developing temporary shoring systems, if needed. We recommend that Cobalt Geosciences and the project structural engineer review temporary shoring designs prior to installation, to verify the suitability of the proposed systems.

Foundation Design

New foundation elements may be supported on variable diameter pipe piles extending to refusal in dense soils below the site with battered piles and interconnecting grade beams to limit structural distress during flow failures/lateral spread.

Pin Piles

To effectively eliminate the effects of differential and total settlement due to liquefaction, variable diameter steel pipe piles should be driven beneath foundation elements. The pile spacing will be determined by the project structural engineer during their design work.

We estimate piles to extend 30 to 70 feet below grade (or more) depending on the loading required, elevations, and hammer sizes. Deeper penetrations may be observed toward the east. If pile depths are consistently more than about 50 feet, closed couplers may be considered with additional load testing.

Pipe piles should consist of Schedule 40 galvanized steel with mechanical couplers for splices. Battered piles may be necessary to provide lateral support to the structures.

The number of piles required depends on the magnitude of the design load. Allowable axial compression capacities of 10, 15, and 25 tons may be used for 4-, 6-, and 8 inch diameter pin piles, respectively, with an approximate factor of safety of 2 for piles driven to refusal. Penetration resistance required to achieve the (refusal) capacities will be determined based on the hammer used to install the pile. Tensile capacity of pin piles should be ignored in design calculations.

It is our experience that the driven pipe pile foundations should provide adequate support with total settlements on the order of $^1/_2$-inch or less. We note that 4 inch piles are likely to be used.

For 4-, 6-, and 8-inch pin piles, the following table is a summary of driving refusal criteria for different hammer sizes that are commonly used:

Hammer Model	Hammer Weight (lb) / Blows per minute	4" Pile Refusal Criteria (s/inch penetration)	6" Pile Refusal Criteria (s/inch penetration)	8" Pile Refusal Criteria (s/inch penetration)
Hydraulic TB 325	850 / 900	16		
Hydraulic TB 425	1,100 / 900	10	20	
Hydraulic TB 725X	2,000 / 600	4	10	

Hydraulic TB 830X	3,000 / 500		6	10
Hydraulic BXR-50	5,000 / 500		4-6	8

Please note that these refusal criteria were established empirically based on previous load tests on 4-, 6-, and 8-inch pin piles. Contractors may select a different hammer for driving these piles and propose a different driving criterion. In this case, it is the contractor's responsibility to demonstrate to the geotechnical engineer's satisfaction that the design load can be achieved based on their selected equipment and driving criteria.

Load testing of at least 3 percent of the piles is required (one pile minimum). The load test should be performed in 25 percent increments of the design load up to 200 percent. Deflections should be measured with dial gauges to determine suitability.

A passive pressure of 250 pcf may be used in the design, neglecting the upper 12 inches. Any fill used to create the passive resistance should be compacted as structural fill. Battered piles could be considered to increase passive resistance to reduce the effects of potential lateral spread and driven at a batter of 4V:1H (vertical to horizontal).

We anticipate that battered piles will extend to greater depths than vertical piles since they will be battered outward or inward from specific foundation elements. We note that this will be based on the structural design and analyses. Actual directions will likely vary. All piles should and are anticipated to be driven to refusal.

Selection of the hammer size will depend on the pile diameter among other factors. The driving system and criteria should conservatively meet the required ultimate capacities without damaging the piles or excess vibrations. Load testing will be required on at least 3 percent of the battered piles to 200 percent the design load.

A structural engineer shall perform the structural design of the pile including spacing and reinforcing steel. The structural engineer also should determine the buckling load for the slender piles and make sure that is not exceeded.

We note that structural distress that could result in pipe pile buckling and failure (lateral conditions) will be managed through the use of interconnecting grade beams. The structural engineer is preparing (or has prepared) updated designs with interconnecting grade beams spanning between new and existing foundation elements.

Slab-on-Grade

We recommend that the upper 24 inches of the existing native soils within slab areas be re-compacted to at least 95 percent of the modified proctor (ASTM D1557 Test Method). Note that settlement could occur in these areas unless more significant ground improvement is utilized. We can provide additional input if slab on grade floors are proposed.

Often, a vapor barrier is considered below concrete slab areas. However, the usage of a vapor barrier could result in curling of the concrete slab at joints. Floor covers sensitive to moisture typically requires the usage of a vapor barrier. A materials or structural engineer should be consulted regarding the detailing of the vapor barrier below concrete slabs. Exterior slabs typically do not utilize vapor barriers.

The American Concrete Institutes ACI 360R-06 Design of Slabs on Grade and ACI 302.1R-04 Guide for Concrete Floor and Slab Construction are recommended references for vapor barrier selection and floor slab detailing.

Slabs on grade may be designed using a coefficient of subgrade reaction of 125 pounds per cubic inch (pci) assuming the slab-on-grade base course is underlain by structural fill placed and compacted as outlined above. A 4- to 6-inch-thick capillary break layer should be placed over the prepared subgrade. This material should consist of pea gravel or 5/8 inch clean angular rock.

A perimeter drainage system is recommended unless interior slab areas are elevated a minimum of 12 inches above adjacent exterior grades. If installed, a perimeter drainage system should consist of a 4-inch diameter perforated drain pipe surrounded by a minimum 6 inches of drain rock wrapped in a non-woven geosynthetic filter fabric to reduce migration of soil particles into the drainage system. The perimeter drainage system should discharge by gravity flow to a suitable stormwater system.

Exterior grades surrounding buildings should be sloped at a minimum of one percent to facilitate surface water flow away from the building and preferably with a relatively impermeable surface cover immediately adjacent to the building.

Erosion and Sediment Control

Erosion and sediment control (ESC) is used to reduce the transportation of eroded sediment to wetlands, streams, lakes, drainage systems, and adjacent properties. Erosion and sediment control measures should be implemented, and these measures should be in general accordance with local regulations. At a minimum, the following basic recommendations should be incorporated into the design of the erosion and sediment control features for the site:

- Schedule the soil, foundation, utility, and other work requiring excavation or the disturbance of the site soils, to take place during the dry season (generally May through September). However, provided precautions are taken using Best Management Practices (BMP's), grading activities can be completed during the wet season (generally October through April).
- All site work should be completed and stabilized as quickly as possible.
- Additional perimeter erosion and sediment control features may be required to reduce the possibility of sediment entering the surface water. This may include additional silt fences, silt fences with a higher Apparent Opening Size (AOS), construction of a berm, or other filtration systems.
- Any runoff generated by dewatering discharge should be treated through construction of a sediment trap if there is sufficient space. If space is limited other filtration methods will need to be incorporated.

Utilities

Utility trenches should be excavated according to accepted engineering practices following OSHA (Occupational Safety and Health Administration) standards, by a contractor experienced in such work. The contractor is responsible for the safety of open trenches. Traffic and vibration adjacent to trench walls should be reduced; cyclic wetting and drying of excavation side slopes should be avoided. Depending upon the location and depth of some utility trenches, groundwater flow into open excavations could be experienced, especially during or shortly following periods of precipitation.

In general, silty and sandy soils were encountered at shallow depths in the explorations at this site. These soils have low cohesion and density and will have a tendency to cave or slough in excavations. Shoring or sloping back trench sidewalls is required within these soils in excavations greater than 4 feet deep.

All utility trench backfill should consist of imported structural fill or suitable on site soils. Utility trench backfill placed in or adjacent to buildings and exterior slabs should be compacted to at least 95 percent of the maximum dry density based on ASTM Test Method D1557. The upper 5 feet of utility trench backfill placed in pavement areas should be compacted to at least 95 percent of the maximum dry density based on ASTM Test Method D1557. Below 5 feet, utility trench backfill in pavement areas should be compacted to at least 90 percent of the maximum dry density based on ASTM Test Method D1557. Pipe bedding should be in accordance with the pipe manufacturer's recommendations.

The contractor is responsible for removing all water-sensitive soils from the trenches regardless of the backfill location and compaction requirements. Depending on the depth and location of the proposed utilities, we anticipate the need to re-compact existing fill soils below the utility structures and pipes. The contractor should use appropriate equipment and methods to avoid damage to the utilities and/or structures during fill placement and compaction procedures.

CONSTRUCTION FIELD REVIEWS

Cobalt Geosciences should be retained to provide part time field review during construction in order to verify that the soil conditions encountered are consistent with our design assumptions and that the intent of our recommendations is being met. This will require field and engineering review to:

- Monitor and test structural fill placement and soil compaction
- Verify pile embedments and refusal criteria
- Observe excavation stability

Geotechnical design services should also be anticipated during the subsequent final design phase to support the structural design and address specific issues arising during this phase. Field and engineering review services will also be required during the construction phase in order to provide a Final Letter for the project.

CLOSURE

This report was prepared for the exclusive use of MacPherson Construction and Design and their appointed consultants. Any use of this report or the material contained herein by third parties, or for other than the intended purpose, should first be approved in writing by Cobalt Geosciences, LLC.

The recommendations contained in this report are based on assumed continuity of soils with those of our test holes and assumed structural loads. Cobalt Geosciences should be provided with final architectural and civil drawings when they become available in order that we may review our design recommendations and advise of any revisions, if necessary.

Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of MacPherson Construction and Design who is identified as "the Client" within the Statement of General Conditions, and its agents to review the conditions and to notify Cobalt Geosciences should any of these not be satisfied.

Sincerely,

Cobalt Geosciences, LLC

2/2/2026
Phil Haberman, PE, LG, LEG
Principal

Statement of General Conditions

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Cobalt Geosciences and the Client. Any use which a third party makes of this report is the responsibility of such third party.

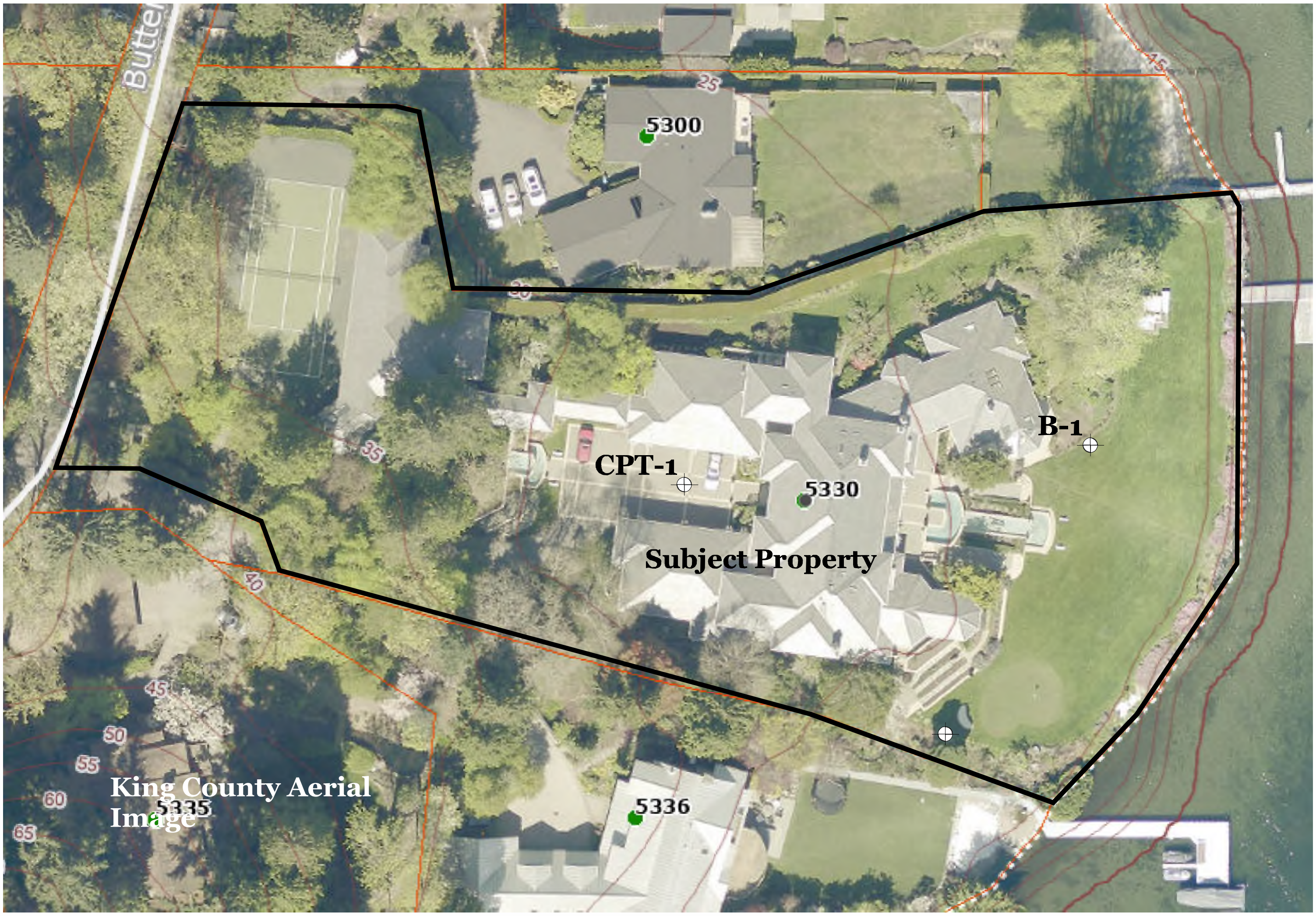
BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Cobalt Geosciences present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Cobalt Geosciences is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state of execution for the specific professional service provided to the Client. No other warranty is made.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Cobalt Geosciences at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Cobalt Geosciences must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Cobalt Geosciences will not be responsible to any party for damages incurred as a result of failing to notify Cobalt Geosciences that differing site or sub-surface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Cobalt Geosciences, sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Cobalt Geosciences cannot be responsible for site work carried out without being present.



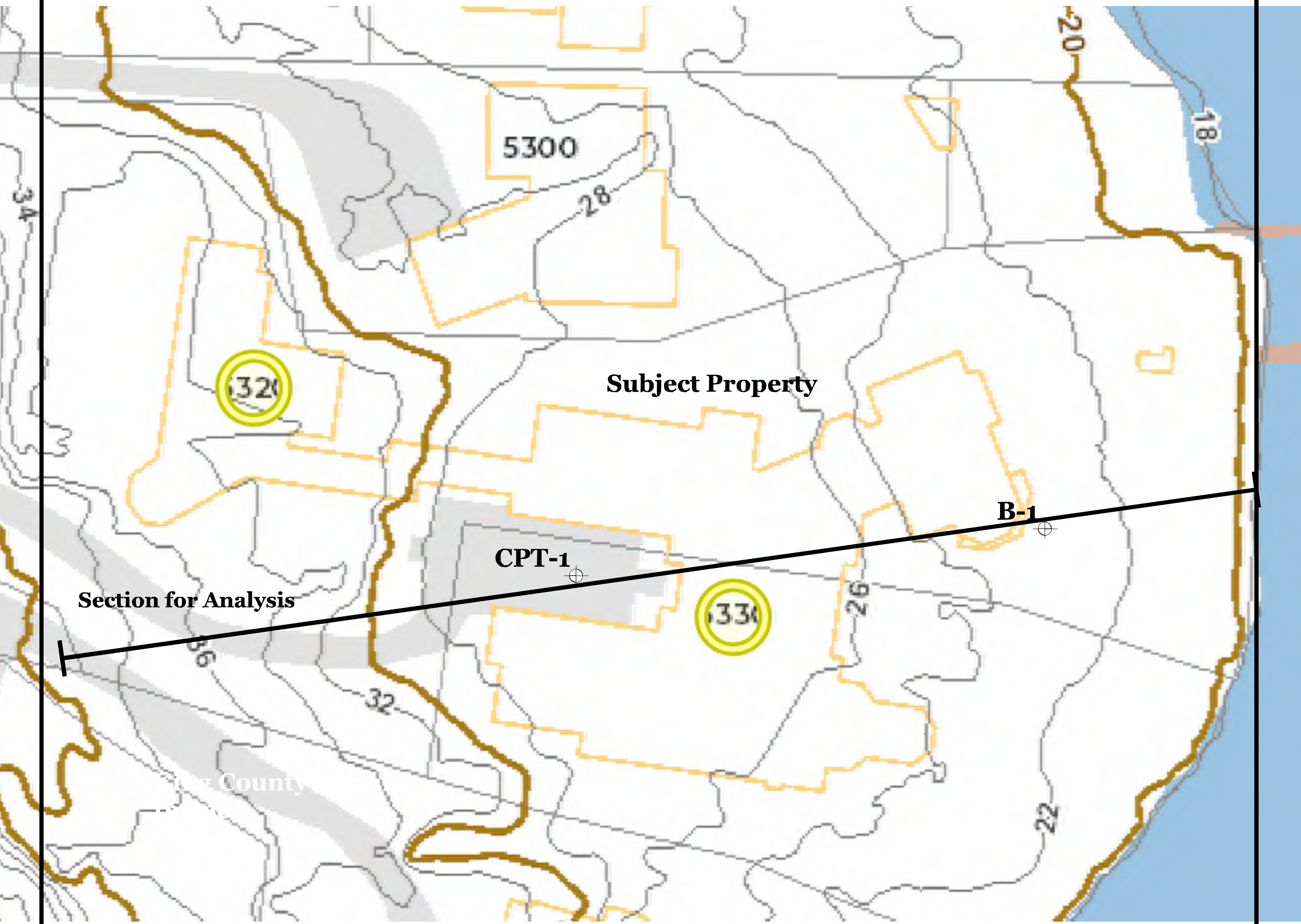
B-1
CPT-1

Approximate
Boring and CPT Boring Locations

N

Not to Scale

COBALT GEOSCIENCES	Proposed Remodel 5320 and 5330 Butterworth Road Mercer Island, Washington	SITE MAP FIGURE 1	Cobalt Geosciences, LLC P.O. Box 82243 Kenmore, WA 98028 (206) 331-1097 www.cobaltgeo.com cobaltgeo@gmail.com



Note: Section continues to east with topography from Lake Washington Nautical Chart and Water Depth Map

Approximate Scale 1”=50’

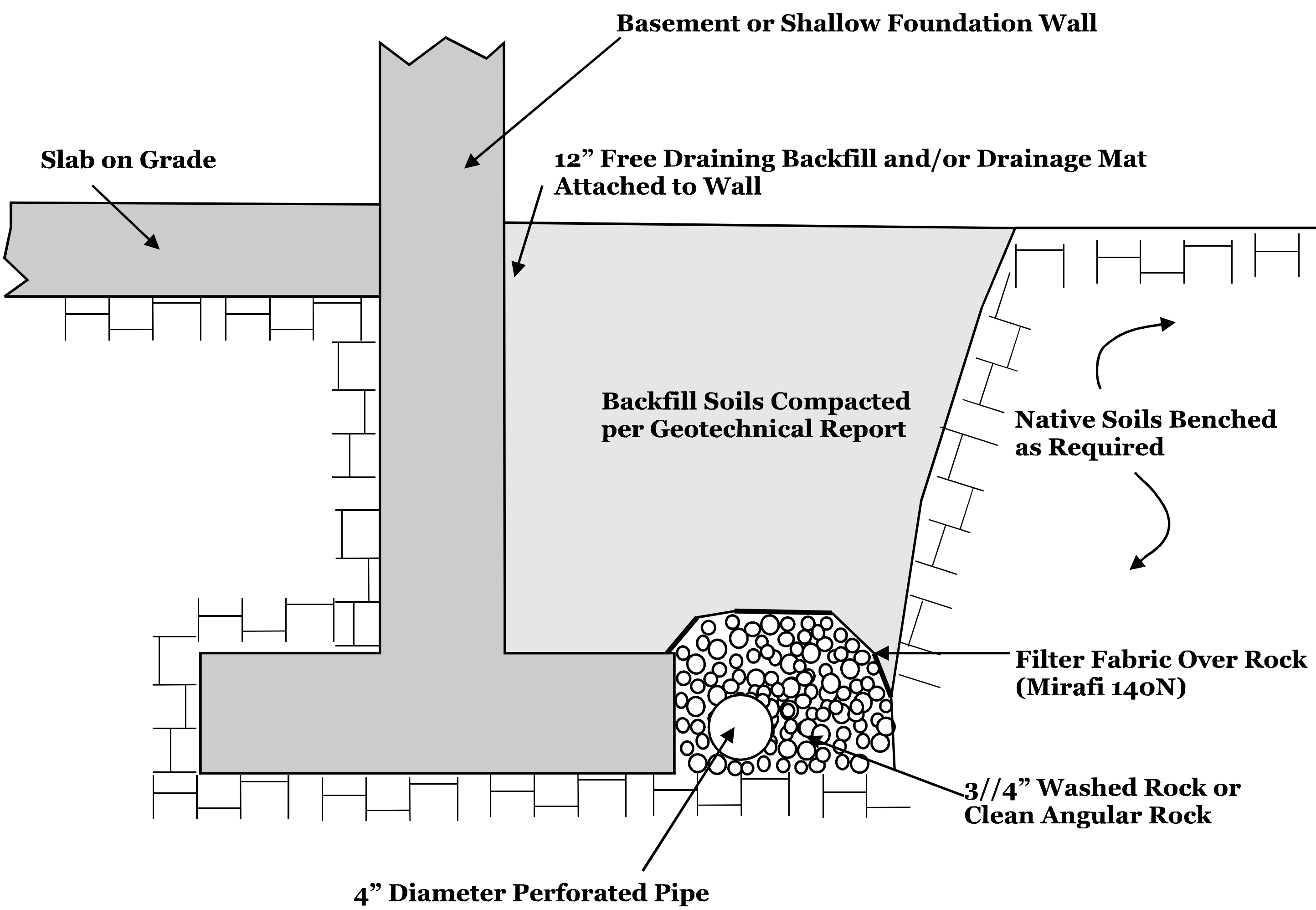
B-1
CPT-1

Approximate
Boring and CPT Boring Locations

N

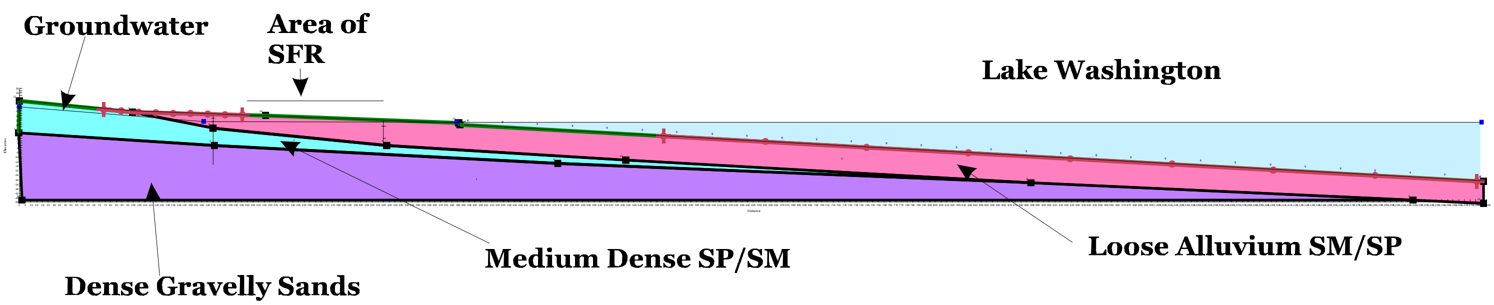
Not to Scale

COBALT GEOSCIENCES	Proposed Residence 5320 Butterworth Road Mercer Island, Washington	**GIS MAP** **FIGURE 2**	Cobalt Geosciences, LLC P.O. Box 82243 Kenmore, WA 98028 (206) 331-1097 www.cobaltgeo.com cobaltgeo@gmail.com

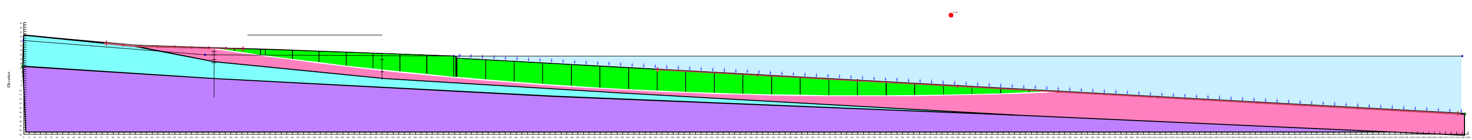


Not to Scale

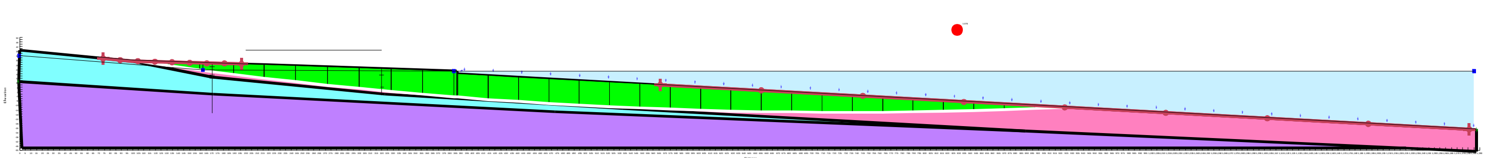
COBALT GEOSCIENCES	**Typical Foundation Drain Detail**	**Attachment**	Cobalt Geosciences, LLC PO Box 1792 North Bend, WA 98045 (206) 331-1097 www.cobaltgeo.com phil@cobaltgeo.com



Residual Conditions
Static FS 1.010



Residual Conditions
0.32g Seismic FS 0.211



COBALT GEOSCIENCES	Proposed Residence 5320 Butterworth Road Mercer Island, Washington	**SLOPE STABILITY**	Cobalt Geosciences, LLC P.O. Box 82243 Kenmore, WA 98028 (206) 331-1097 www.cobaltgeo.com cobaltgeo@gmail.com

Unified Soil Classification System (USCS)

MAJOR DIVISIONS			SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS (more than 50% retained on No. 200 sieve)	Gravels (more than 50% of coarse fraction retained on No. 4 sieve)	Clean Gravels (less than 5% fines)	GW	Well-graded gravels, gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with Fines (more than 12% fines)	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (50% or more of coarse fraction passes the No. 4 sieve)	Clean Sands (less than 5% fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly graded sand, gravelly sands, little or no fines
		Sands with Fines (more than 12% fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS (50% or more passes the No. 200 sieve)	Silts and Clays (liquid limit less than 50)	Inorganic	ML	Inorganic silts of low to medium plasticity, sandy silts, gravelly silts, or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays silty clays, lean clays
		Organic	OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays (liquid limit 50 or more)	Inorganic	MH	Inorganic silts, micaceous or diatomaceous fine sands or silty soils, elastic silt
			CH	Inorganic clays of medium to high plasticity, sandy fat clay, or gravelly fat clay
		Organic	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat, humus, swamp soils with high organic content (ASTM D4427)

Classification of Soil Constituents

MAJOR constituents compose more than 50 percent, by weight, of the soil. Major constituents are capitalized (i.e., SAND).

Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (i.e., silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (i.e., slightly silty SAND).

Trace constituents compose 0 to 5 percent of the soil (i.e., slightly silty SAND, trace gravel).

Grain Size Definitions

Description	Sieve Number and/or Size
Fines	<#200 (0.08 mm)
Sand	
-Fine	#200 to #40 (0.08 to 0.4 mm)
-Medium	#40 to #10 (0.4 to 2 mm)
-Coarse	#10 to #4 (2 to 5 mm)
Gravel	
-Fine	#4 to 3/4 inch (5 to 19 mm)
-Coarse	3/4 to 3 inches (19 to 76 mm)
Cobbles	3 to 12 inches (75 to 305 mm)
Boulders	>12 inches (305 mm)

Relative Density (Coarse Grained Soils)

N, SPT, Blows/FT	Relative Density
0 - 4	Very loose
4 - 10	Loose
10 - 30	Medium dense
30 - 50	Dense
Over 50	Very dense

Consistency (Fine Grained Soils)

N, SPT, Blows/FT	Relative Consistency
Under 2	Very soft
2 - 4	Soft
4 - 8	Medium stiff
8 - 15	Stiff
15 - 30	Very stiff
Over 30	Hard

Moisture Content Definitions

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

COBALT GEOSCIENCES
Cobalt Geosciences, LLC
P.O. Box 82243
Kenmore, WA 98028
(206) 331-1097
www.cobaltgeo.com
cobaltgeo@gmail.com

Soil Classification Chart

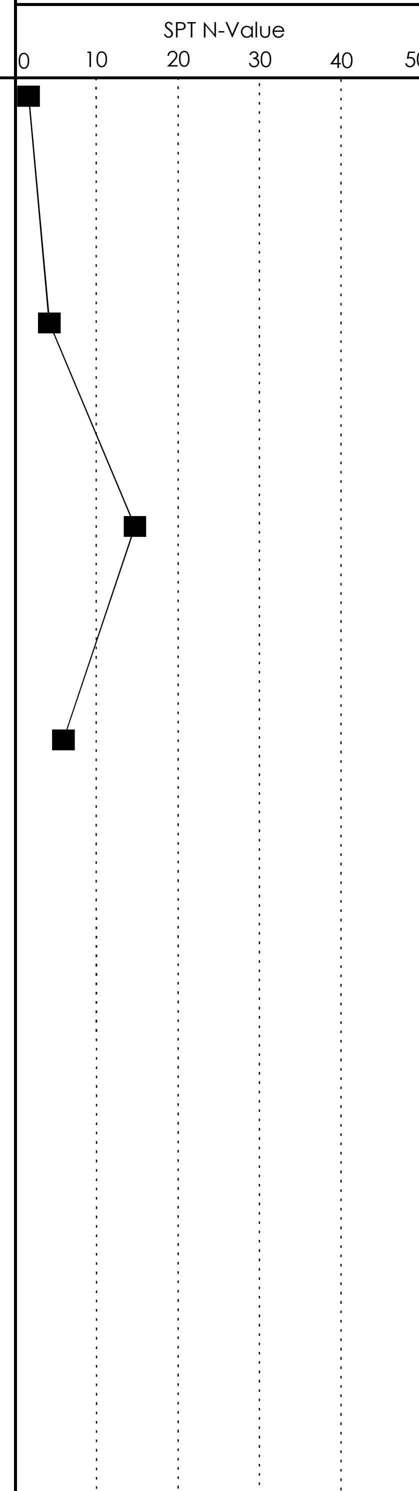
Figure C1

Log of Boring B-1

Date: May 15, 2024	Depth: 16.5'	Initial Groundwater: 4'
Contractor: CN	Elevation:	Sample Type: Split Spoon
Method: Hollow Stem Auger	Logged By: PH Checked By: SC	Final Groundwater: 4'

Depth (Feet)	Interval	% Recovery	Blows/6"	Graphic Log	USCS Symbol	Material Description	Groundwater
0	■		1			Vegetation/Topsoil	
			0 1		SM	Very loose to loose, silty-fine to medium grained sand, dark yellowish brown, moist to wet. (Fill)	
2							
4							
6	■		1 2 1		SP	Loose to medium dense, fine to medium grained sand, mottled olive gray, moist to wet. (Pre-Olympia Deposits)	
8							
10	■		3 4 10				
12							
14							
16	■		2 3 2				
						End of Boring 16.5' Refusal due to heave.	
18							
20							
22							
24							
26							
28							
30							
32							
34							

Moisture Content (%)
Plastic Limit ● Liquid Limit



COBALT GEOSCIENCES
Cobalt Geosciences, LLC
P.O. Box 82243
Kenmore, WA 98028
(206) 331-1097
www.cobaltgeo.com
cobaltgeo@gmail.com

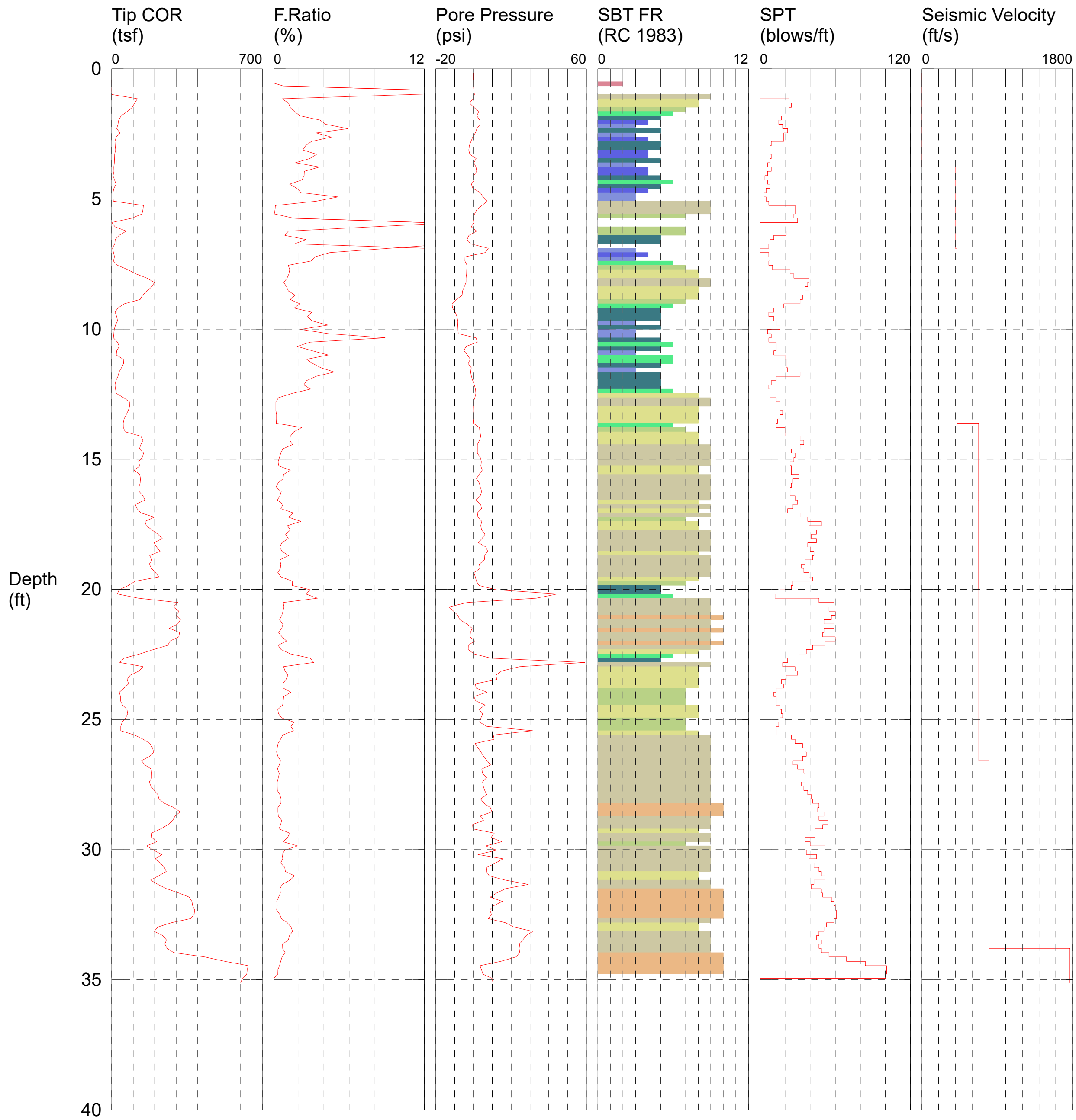
Proposed Residence
5330 Butterworth Road
Mercer Island, Washington

Boring Log

sCPT-01

CPT Contractor: In SItu Engineering
CUSTOMER: Cobalt Geo
LOCATION: Mercer Isd
JOB NUMBER:
COMMENT: Butterworth

OPERATOR: Okbay
CONE ID: DDG1351
TEST DATE: 11/19/2024 10:29:18 AM
PREDRILL: 1 ft
BACKFILL: 20% Bentonite slurry & Chips
SURFACE PATCH: Concrete Patch

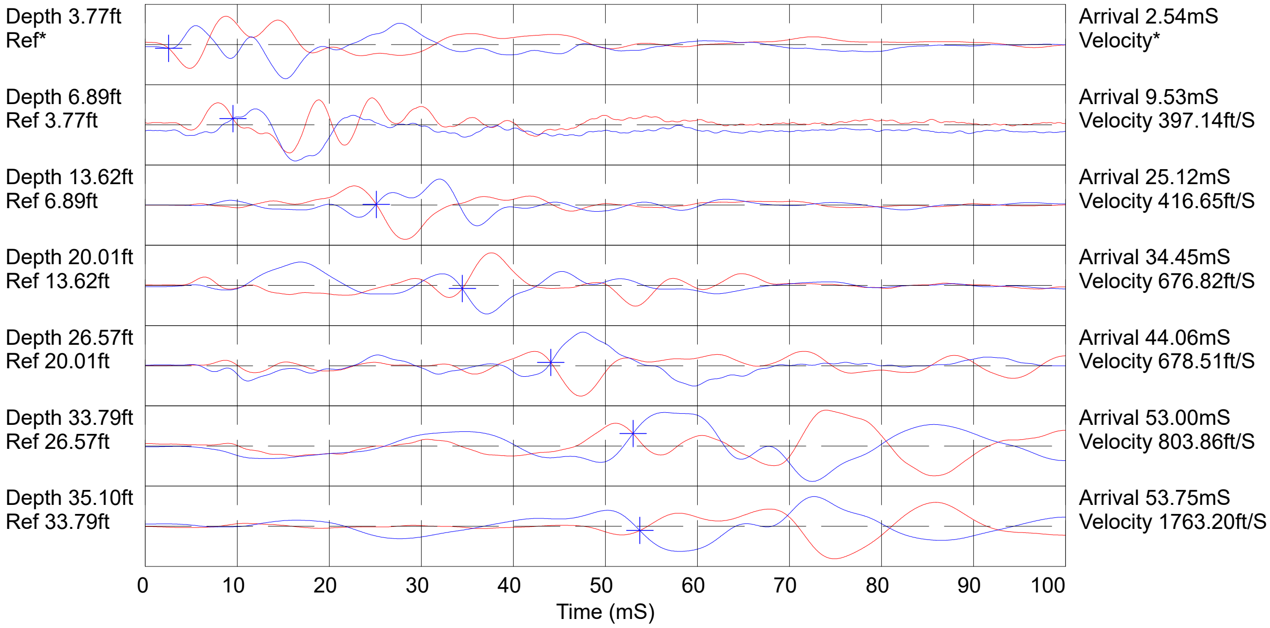


TOTAL DEPTH: 35.105 ft

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

HOLE NUMBER: sCPT-01



Hammer to Rod String Distance (ft): 2.62
* = Not Determined

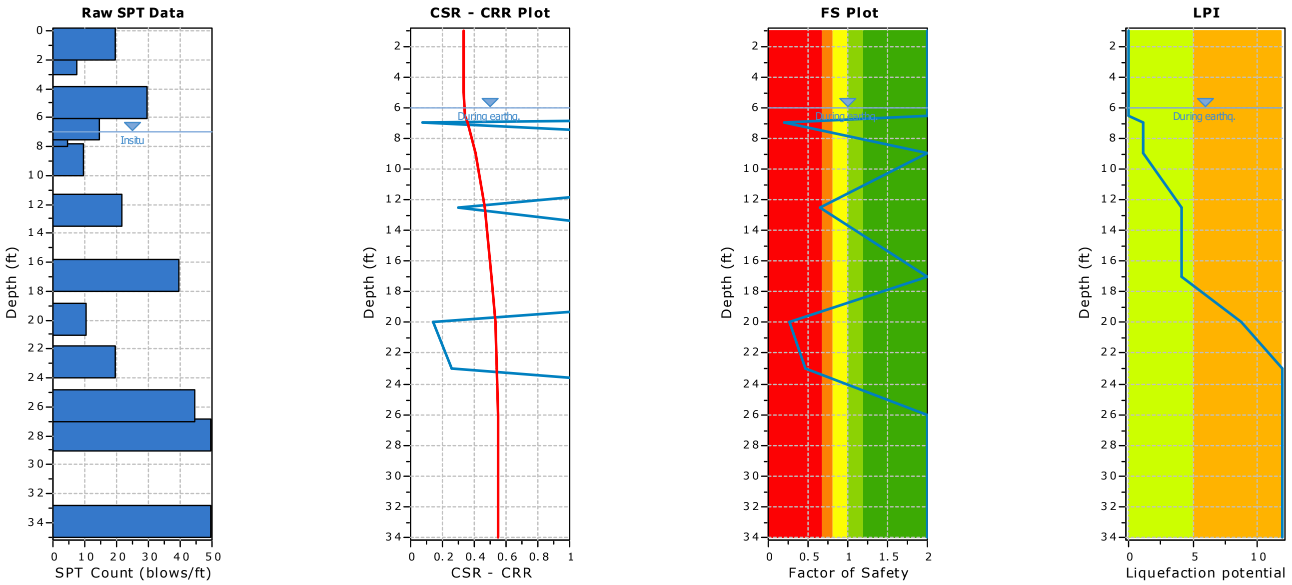
SPT BASED LIQUEFACTION ANALYSIS REPORT

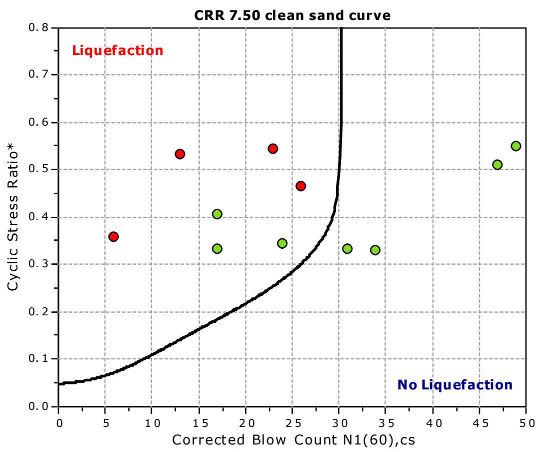
Project title : Butterworth **SPT Name: SPT #1**

Location : Mercer Island

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	7.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	6.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.00
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.61 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		





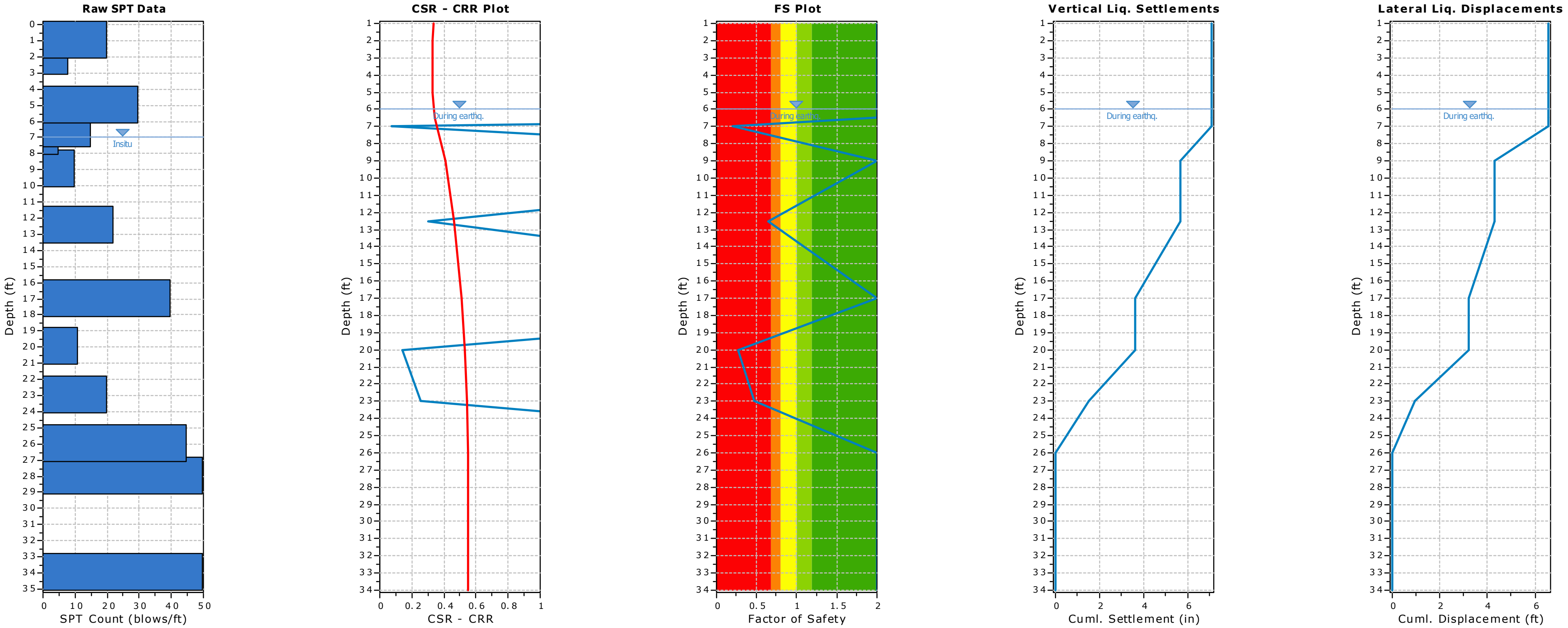
F.S. color scheme

- 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
1.00	20	20.00	105.00	1.00	No
2.00	8	60.00	105.00	3.00	No
5.00	30	5.00	105.00	1.50	Yes
6.50	15	50.00	105.00	0.50	No
7.00	5	10.00	105.00	2.00	Yes
9.00	10	60.00	105.00	3.50	No
12.50	22	10.00	110.00	4.50	Yes
17.00	40	5.00	115.00	3.00	No
20.00	11	10.00	110.00	3.00	Yes
23.00	20	10.00	115.00	3.00	Yes
26.00	45	10.00	120.00	2.00	No
28.00	50	10.00	120.00	6.00	No
34.00	50	10.00	120.00	1.00	No

Abbreviations

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

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

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
1.00	20	105.00	0.05	0.00	0.05	1.70	1.00	1.00	0.75	1.00	25	20.00	3.61	1.08	31	4.000
2.00	8	105.00	0.10	0.00	0.10	1.69	1.00	1.00	0.75	1.00	10	60.00	5.00	1.20	17	4.000
5.00	30	105.00	0.26	0.00	0.26	1.52	1.00	1.00	0.75	1.00	34	5.00	0.00	1.00	34	4.000
6.50	15	105.00	0.34	0.00	0.34	1.44	1.00	1.00	0.75	1.00	16	50.00	5.00	1.20	24	4.000
7.00	5	105.00	0.37	0.00	0.37	1.42	1.00	1.00	0.75	1.00	5	10.00	0.87	1.02	6	0.073
9.00	10	105.00	0.47	0.06	0.41	1.39	1.00	1.00	0.75	1.00	10	60.00	5.00	1.20	17	4.000
12.50	22	110.00	0.67	0.17	0.49	1.32	1.00	1.00	0.85	1.00	25	10.00	0.87	1.02	26	0.303
17.00	40	115.00	0.92	0.31	0.61	1.24	1.00	1.00	0.95	1.00	47	5.00	0.00	1.00	47	4.000
20.00	11	110.00	1.09	0.41	0.68	1.19	1.00	1.00	0.95	1.00	12	10.00	0.87	1.02	13	0.142
23.00	20	115.00	1.26	0.50	0.76	1.15	1.00	1.00	0.95	1.00	22	10.00	0.87	1.02	23	0.255
26.00	45	120.00	1.44	0.59	0.85	1.10	1.00	1.00	0.95	1.00	47	10.00	0.87	1.02	49	4.000
28.00	50	120.00	1.56	0.66	0.91	1.07	1.00	1.00	0.95	1.00	51	10.00	0.87	1.02	53	4.000
34.00	50	120.00	1.92	0.84	1.08	0.99	1.00	1.00	1.00	1.00	50	10.00	0.87	1.02	52	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)
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
α, β:	Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$:	Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$:	Cydic 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	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
1.00	105.00	0.05	0.00	0.05	1.00	1.00	0.396	1.19	0.332	1.00	0.332	2.000	●
2.00	105.00	0.10	0.00	0.10	1.00	1.00	0.395	1.19	0.332	1.00	0.332	2.000	●
5.00	105.00	0.26	0.00	0.26	0.99	1.00	0.393	1.19	0.329	1.00	0.329	2.000	●
6.50	105.00	0.34	0.02	0.33	0.99	1.00	0.410	1.19	0.344	1.00	0.344	2.000	●
7.00	105.00	0.37	0.03	0.34	0.99	1.00	0.427	1.19	0.358	1.00	0.358	0.203	●
9.00	105.00	0.47	0.09	0.38	0.98	1.00	0.485	1.19	0.407	1.00	0.407	2.000	●
12.50	110.00	0.67	0.20	0.46	0.97	1.00	0.556	1.19	0.466	1.00	0.466	0.650	●
17.00	115.00	0.92	0.34	0.58	0.96	1.00	0.608	1.19	0.510	1.00	0.510	2.000	●
20.00	110.00	1.09	0.44	0.65	0.96	1.00	0.634	1.19	0.531	1.00	0.531	0.267	●
23.00	115.00	1.26	0.53	0.73	0.95	1.00	0.649	1.19	0.544	1.00	0.544	0.469	●
26.00	120.00	1.44	0.62	0.82	0.94	1.00	0.656	1.19	0.550	1.00	0.550	2.000	●
28.00	120.00	1.56	0.69	0.87	0.93	1.00	0.658	1.19	0.552	1.00	0.552	2.000	●
34.00	120.00	1.92	0.87	1.05	0.90	1.00	0.653	1.19	0.547	1.00	0.547	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 (adjusted for improvement)
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
1.00	2.000	0.00	9.85	1.00	0.00
2.00	2.000	0.00	9.70	1.00	0.00
5.00	2.000	0.00	9.24	3.00	0.00
6.50	2.000	0.00	9.01	1.50	0.00
7.00	0.203	0.80	8.93	0.50	1.08
9.00	2.000	0.00	8.63	2.00	0.00
12.50	0.650	0.35	8.10	3.50	3.02
17.00	2.000	0.00	7.41	4.50	0.00
20.00	0.267	0.73	6.95	3.00	4.66
23.00	0.469	0.53	6.49	3.00	3.16
26.00	2.000	0.00	6.04	3.00	0.00
28.00	2.000	0.00	5.73	2.00	0.00
34.00	2.000	0.00	4.82	6.00	0.00

Overall potential I_L : 11.92

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} (%)	Δh (ft)	ΔS (in)
1.00	25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
2.00	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.00	0.000
5.00	34	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.50	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 settlements estimation for saturated sands ::

Depth (ft)	D_{50} (in)	q_c/N	e_v (%)	Δh (ft)	s (in)
6.50	0.01	2.10	0.00	0.50	0.000
7.00	0.01	2.10	5.80	2.00	1.392
9.00	0.01	2.10	0.00	3.50	0.000
12.50	0.01	2.10	3.83	4.50	2.071
17.00	0.01	2.10	0.00	3.00	0.000
20.00	0.01	2.10	5.80	3.00	2.088
23.00	0.01	2.10	4.24	3.00	1.527
26.00	0.01	2.10	0.00	2.00	0.000
28.00	0.01	2.10	0.00	6.00	0.000
34.00	0.01	2.10	0.00	1.00	0.000

Cumulative settlements: 7.077

Abbreviations

D_{50}: Median grain size (in)
q_c/N: Ratio of cone resistance to SPT
e_v: Post liquefaction volumetric strain (%)
Δh: Thickness of soil layer to be considered (ft)
s: Estimated settlement (in)

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
1.00	25	70.00	0.00	1.00	0.000	0.00
2.00	10	44.27	0.00	3.00	0.000	0.00
5.00	34	81.63	0.00	1.50	0.000	0.00
6.50	16	56.00	0.00	0.50	0.000	0.00
7.00	5	31.30	51.20	2.00	1.024	2.25
9.00	10	44.27	0.00	3.50	0.000	0.00
12.50	25	70.00	11.12	4.50	0.500	1.10
17.00	47	100.00	0.00	3.00	0.000	0.00

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
20.00	12	48.50	34.10	3.00	1.023	2.25
23.00	22	65.67	14.50	3.00	0.435	0.96
26.00	47	100.00	0.00	2.00	0.000	0.00
28.00	51	100.00	0.00	6.00	0.000	0.00
34.00	50	100.00	0.00	1.00	0.000	0.00

Cumulative lateral displacements: 6.56

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

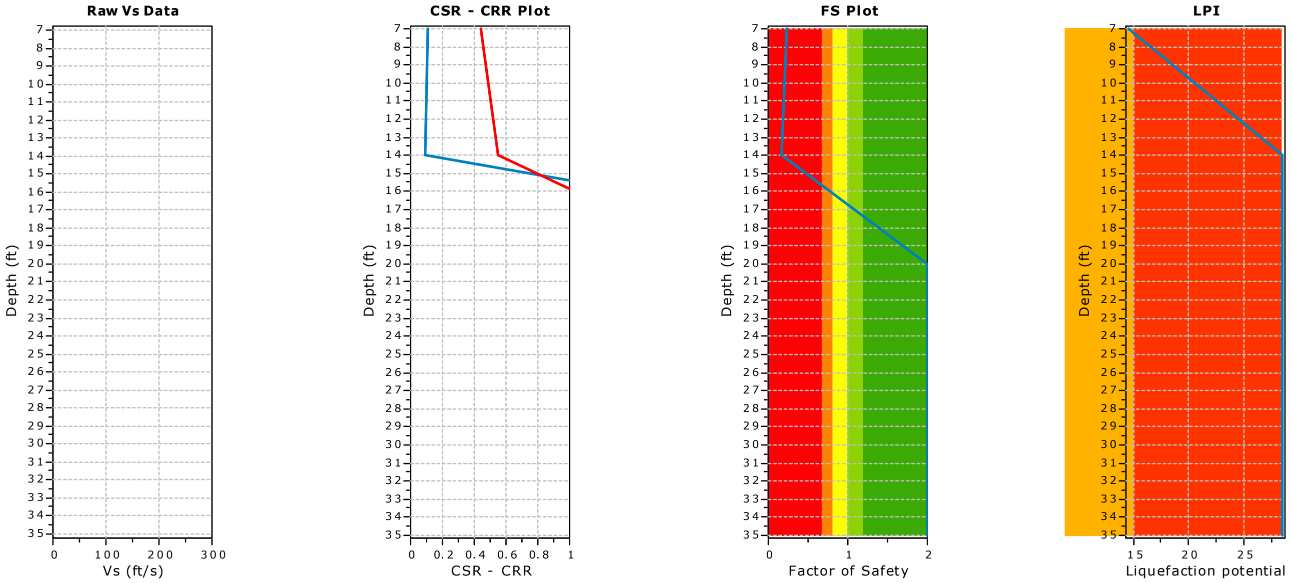
V_s BASED LIQUEFACTION ANALYSIS REPORT

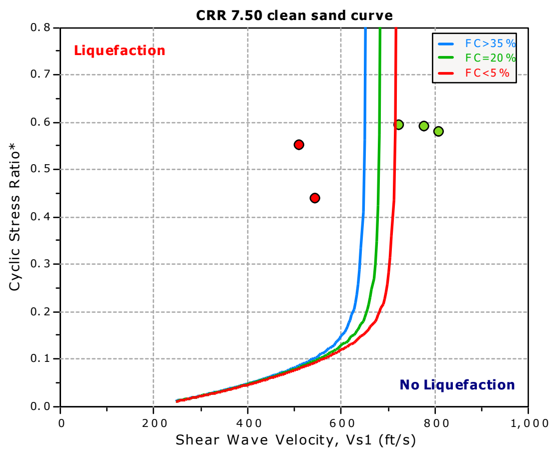
Project title : Butterworth **V_s Name: Vs #2**

Location : Mercer Island

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	5.00 ft
G.W.T. (earthq.):	4.00 ft
Earthquake magnitude M_w:	7.00
Peak ground acceleration:	0.61 g
Eq. external load:	0.00 tsf





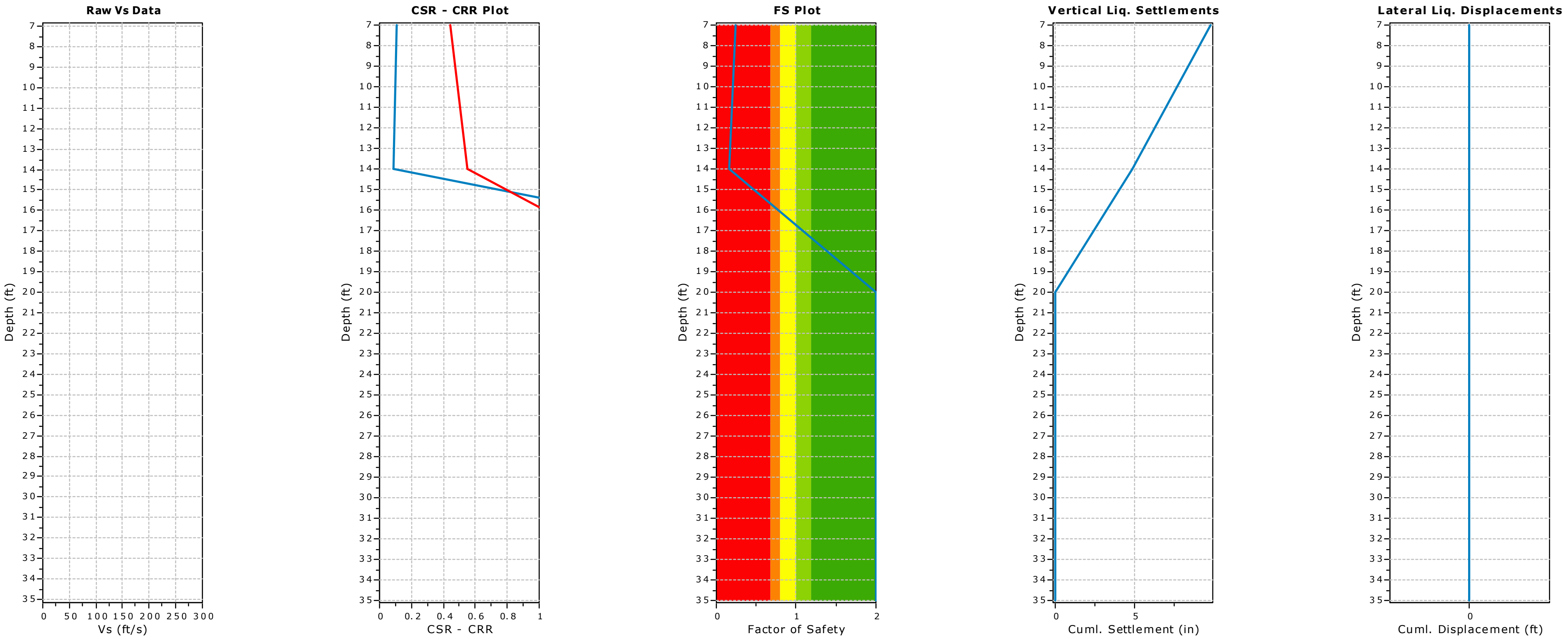
F.S. color scheme

- 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)	V_s Field Value (ft/s)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
7.00	400.00	5.00	105.00	7.00	Yes
14.00	417.00	10.00	110.00	7.00	Yes
20.00	680.00	10.00	110.00	6.00	Yes
27.00	680.00	10.00	120.00	7.00	Yes
34.00	803.00	10.00	120.00	7.00	Yes
35.00	1763.00	10.00	120.00	1.00	No

Abbreviations

Depth: Depth at which test was performed (ft)
Vs Field Value: Measured shear waves velocity (ft/s)
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)	V_s Field Value (ft/s)	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	Norm. Factor	V_{s1} (ft/s)	Vs_{s1}^* (ft/s)	$CRR_{7.5}$
7.00	400.00	105.00	0.37	0.06	0.31	1.36	545.86	215.00	0.105
14.00	417.00	110.00	0.75	0.28	0.47	1.22	510.33	212.50	0.089
20.00	680.00	110.00	1.08	0.47	0.61	1.15	778.95	212.50	4.000
27.00	680.00	120.00	1.50	0.69	0.82	1.07	725.61	212.50	4.000
34.00	803.00	120.00	1.92	0.90	1.02	1.01	810.85	212.50	4.000
35.00	1763.00	120.00	1.98	0.94	1.05	1.00	1767.87	212.50	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)
Norm. Factor: overburden-stress correction factor
V_{S1}: Overburden-stress corrected shear wave velocity
V_{s1}^*: Limiting upper value of V_{s1}
$CRR_{7.5}$: 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	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
7.00	105.00	0.37	0.09	0.27	0.99	1.00	0.524	1.19	0.440	1.00	0.440	0.240
14.00	110.00	0.75	0.31	0.44	0.97	1.00	0.657	1.19	0.551	1.00	0.551	0.162
20.00	110.00	1.08	0.50	0.58	0.96	1.00	0.704	1.19	0.590	1.00	0.590	2.000
27.00	120.00	1.50	0.72	0.78	0.93	1.00	0.709	1.19	0.594	1.00	0.594	2.000
34.00	120.00	1.92	0.94	0.99	0.90	1.00	0.693	1.19	0.581	1.00	0.581	2.000
35.00	120.00	1.98	0.97	1.02	0.89	1.00	0.690	1.19	0.578	1.00	0.578	2.000

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

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS

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 : Cydic 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
7.00	0.240	0.76	8.93	7.00	14.49
14.00	0.162	0.84	7.87	7.00	14.07
20.00	2.000	0.00	6.95	6.00	0.00
27.00	2.000	0.00	5.89	7.00	0.00
34.00	2.000	0.00	4.82	7.00	0.00
35.00	2.000	0.00	4.67	1.00	0.00

Overall potential I_L: 28.55

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 saturated sands ::

Depth (ft)	$V_{s1,cs}$ (ft/s)	$q_{t1N,cs}$	e_v (%)	Δh (ft)	s (in)
7.00	545.86	11788.58	5.80	7.00	4.872
14.00	513.49	9047.75	5.80	7.00	4.872
20.00	796.64	60561.89	0.00	6.00	0.000
27.00	738.24	43557.68	0.00	7.00	0.000
34.00	832.23	73175.72	0.00	7.00	0.000
35.00	2303.54	6004100.49	0.00	1.00	0.000

Cumulative settlements: 9.744

Abbreviations

$V_{s1,cs}$: Normalized shear wave velocity clean sand equivalent
$q_{t1N,cs}$: Estimated normalized corrected clean sand cone resitance
e_v: Post liquefaction volumetric strain (%)
Δh: Thickness of soil layer to be considered (ft)
s: Estimated settlement (in)

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$V_{s1,cs}$ (ft/s)	$(N_1)_{60,cs}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
7.00	545.86	50	100.00	6.20	7.00	0.000	0.00
14.00	513.49	50	100.00	6.20	7.00	0.000	0.00
20.00	796.64	50	100.00	0.00	6.00	0.000	0.00
27.00	738.24	50	100.00	0.00	7.00	0.000	0.00
34.00	832.23	50	100.00	0.00	7.00	0.000	0.00
35.00	2303.54	50	100.00	0.00	1.00	0.000	0.00

Cumulative lateral displacements: 0.00

Abbreviations

$V_{s1,cs}$: Normalized shear wave velocity clean sand equivalent
$(N_1)_{60,cs}$: Estimated normalized corrected clean sand SPT
D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated 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