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

December 11, 2025

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

RE: Additional Comment Responses
Proposed Additions/Remodel
5320 Butterworth Road – CAO25--011
Mercer Island, Washington

In accordance with your authorization, Cobalt Geosciences, LLC has prepared a plan review letter for the project. We have included comments and responses from the November 13, 2025 letter from the City of Mercer Island.

There are comments regarding structural design and how it relates to potential lateral spread, ground loss, and structural deformation.

In preparation of this letter, we have reviewed the CPT boring log and seismic velocity information and re-analyzed liquefaction and lateral spread analyses.

The updated analyses for liquefaction utilized 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. This is an estimate of flow failure toward the east.

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 or more inches in diameter) as determined by the structural engineer and their upcoming design.

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 25 to 50 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 40 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 the 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.

For 3-, 4-, and 6-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. A typical batter is 1H:6H; however, we understand that Pile King can batter piles at 4V:1H.

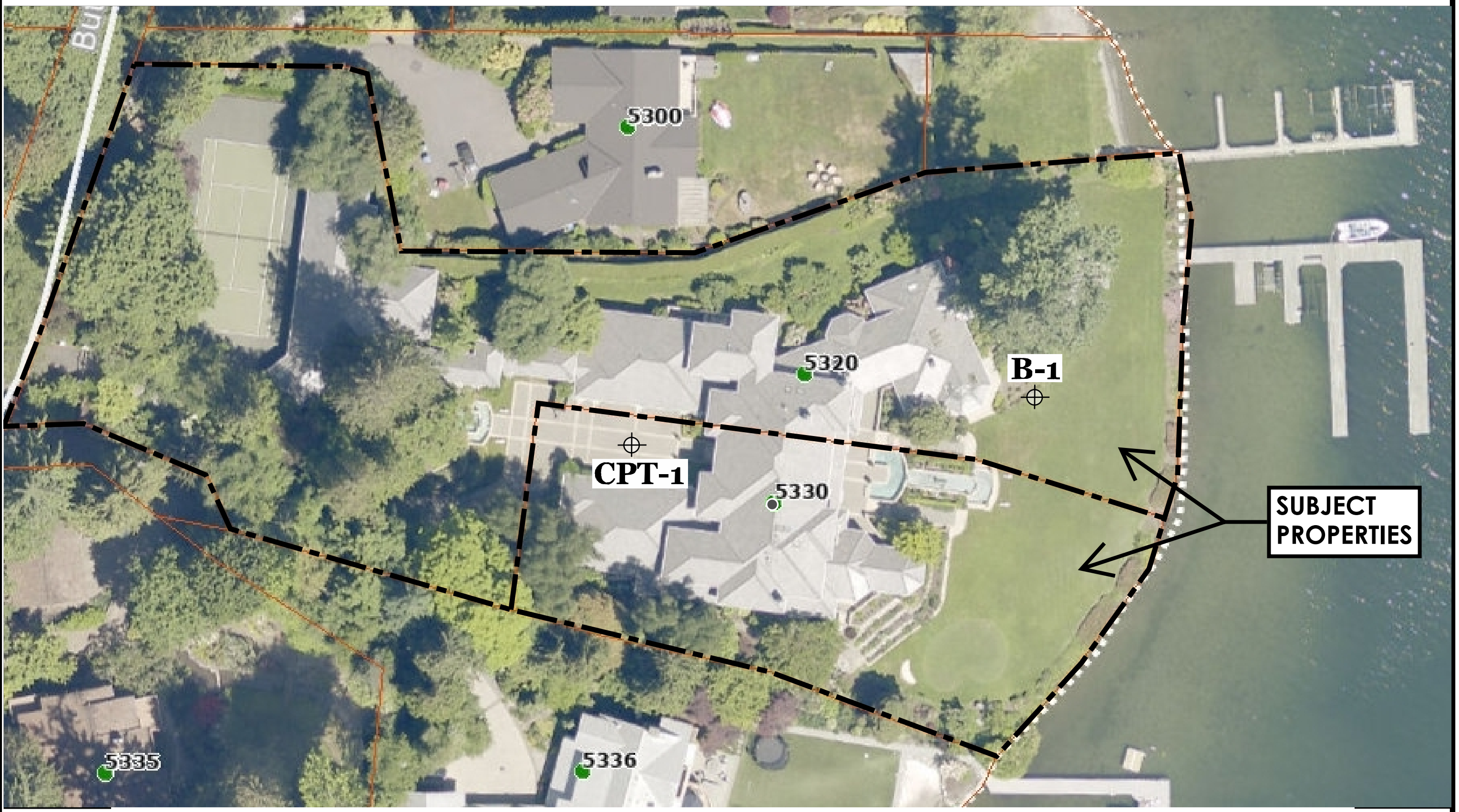
We anticipate that battered piles will extend to greater depths than vertical piles since they will be battered outward toward Lake Washington (base of pile to the east of the surface location). Selection of the hammer size will depend on the pile diameter among other factors. The driving system and 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.

Sincerely,

Cobalt Geosciences, LLC

12/11/2025
Phil Haberman, PE, LG, LEG
Princip



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

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).

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

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)

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

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Soil Classification Chart

Figure C1

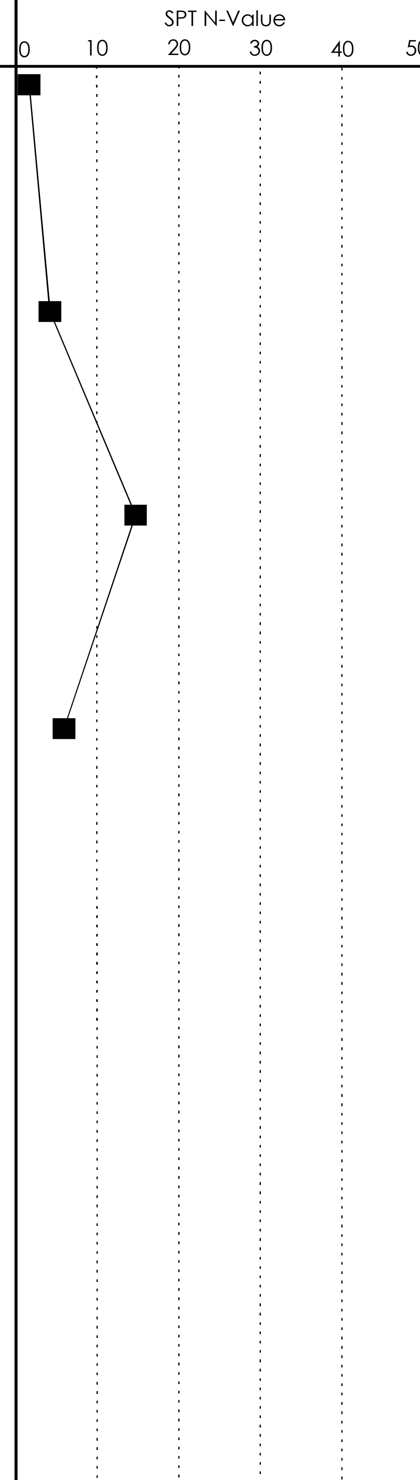
Log of Boring B-1

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
0–0.5			1			Vegetation/Topsoil	
0.5–5			0 1		SM	Very loose to loose, silty-fine to medium grained sand, dark yellowish brown, moist to wet. (Fill)	
5–16.5			1 2 1		SP	Loose to medium dense, fine to medium grained sand, mottled olive gray, moist to wet. (Pre-Olympia Deposits)	
10			3 4 10				
15			2 3 2				
16.5						End of Boring 16.5' Refusal due to heave.	

Depth scale (feet): 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32, 34

Moisture Content (%): Plastic Limit — Liquid Limit



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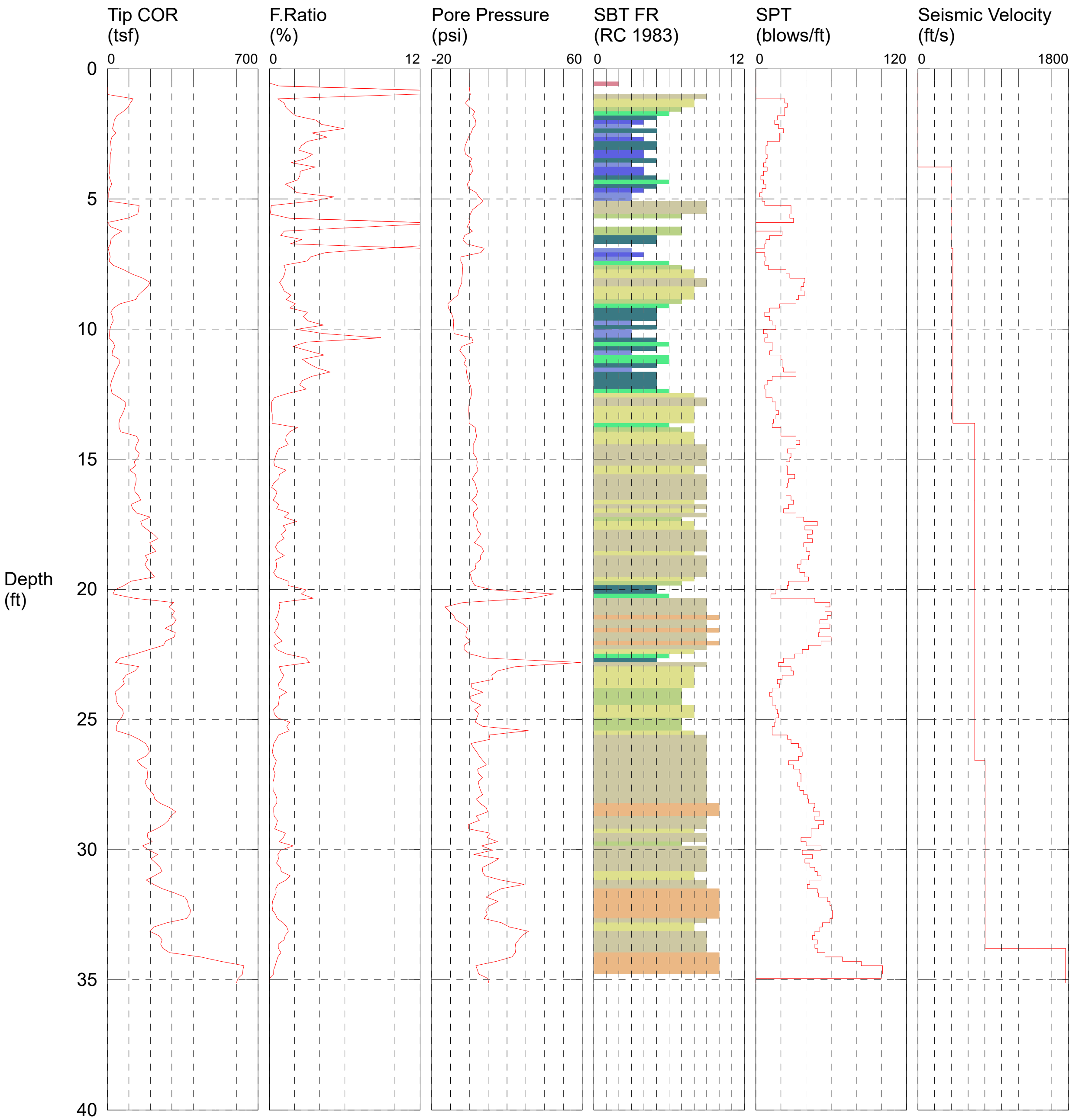
Proposed Residence
5330 Butterworth Road
Mercer Island, Washington

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

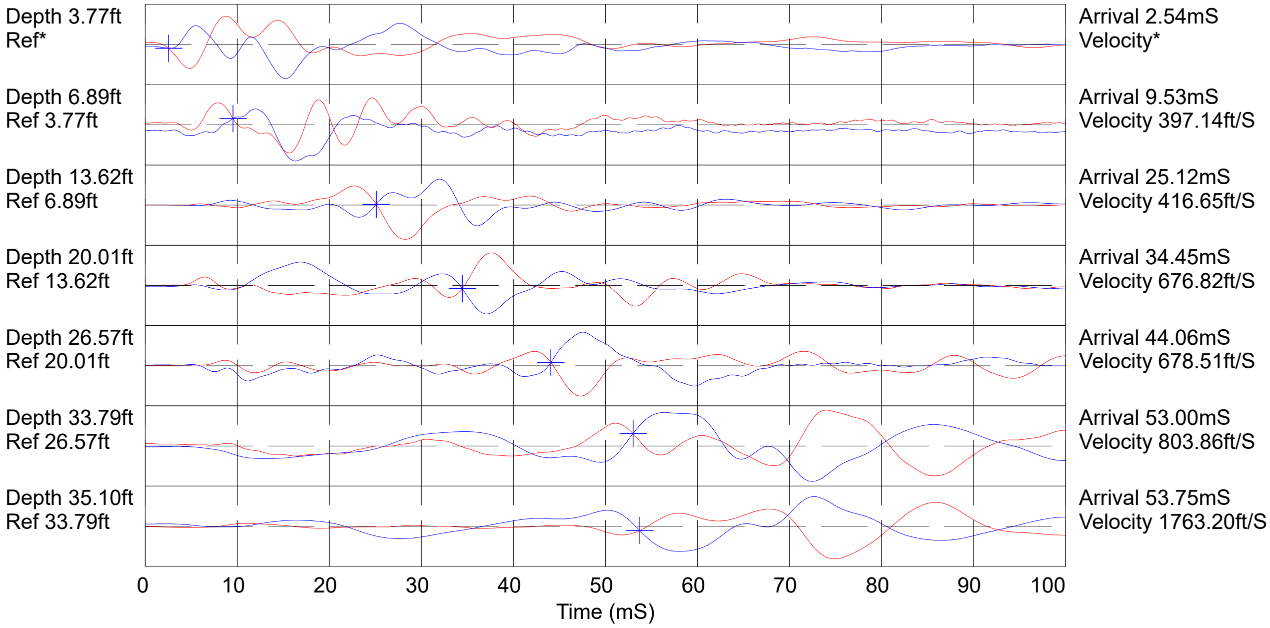


TOTAL DEPTH: 35.105 ft

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

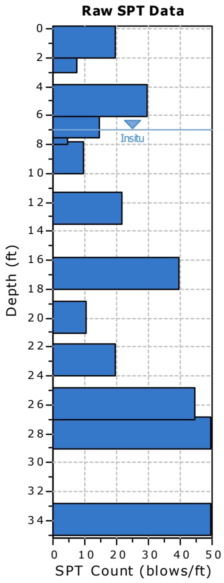
SPT BASED LIQUEFACTION ANALYSIS REPORT

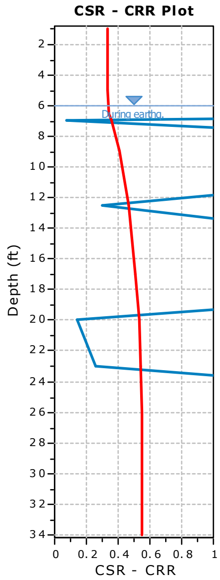
Project title : Butterworth **SPT Name: SPT #1**

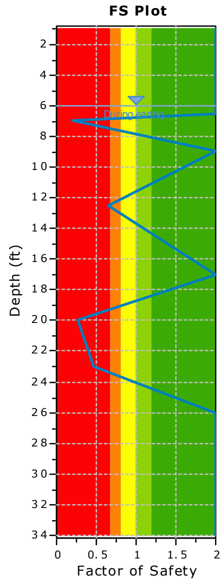
Location : Mercer Island

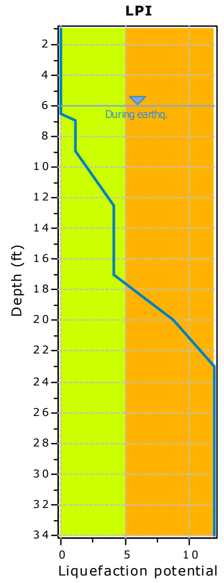
:: Input parameters and analysis properties ::

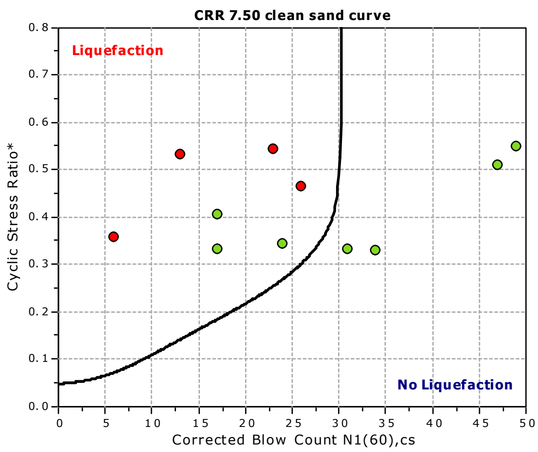
Analysis method:	NCEER 1998	G.W.T. (in-situ):	7.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	6.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.00
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.61 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer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











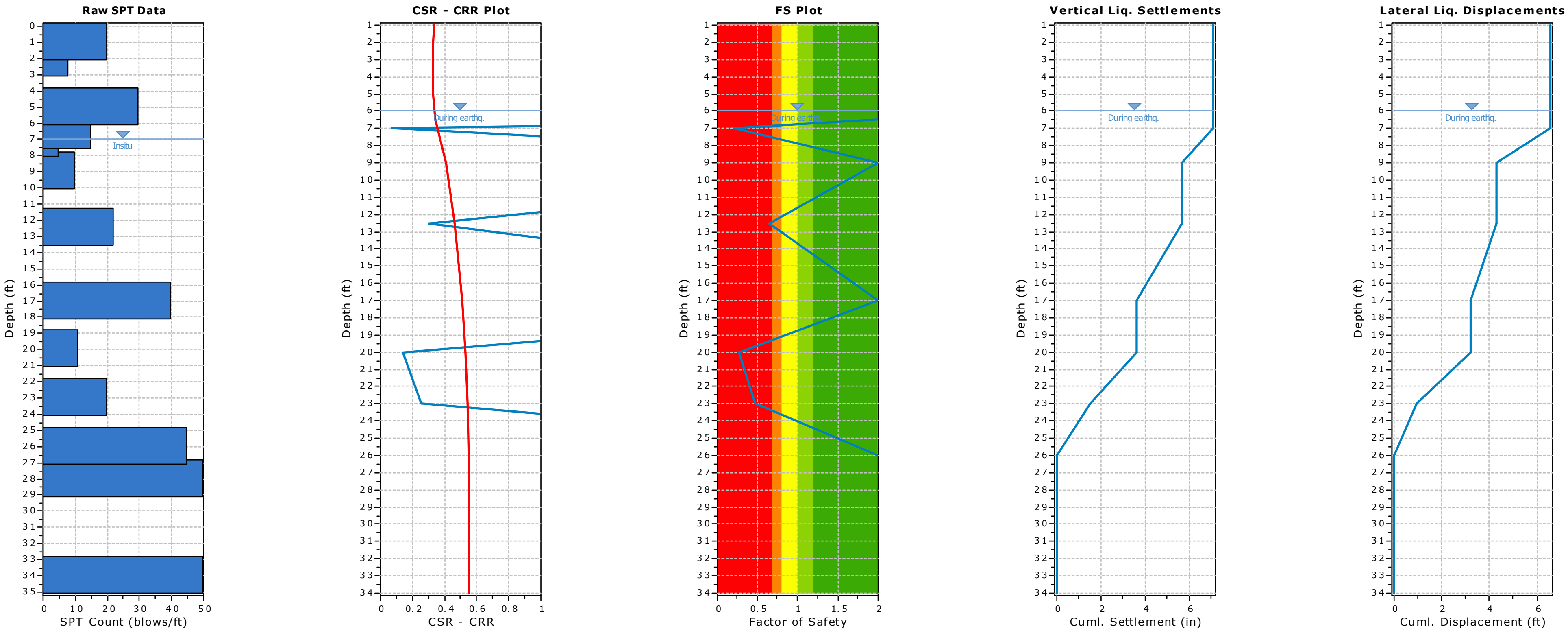
F.S. color scheme

- 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)

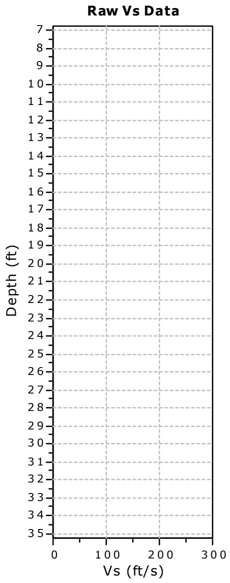
V_s BASED LIQUEFACTION ANALYSIS REPORT

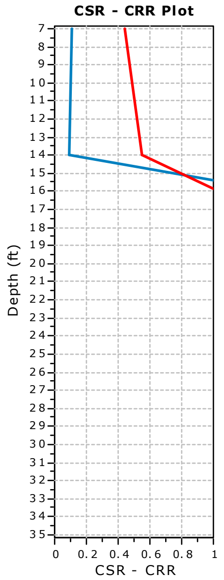
Project title : Butterworth **V_s Name: Vs #2**

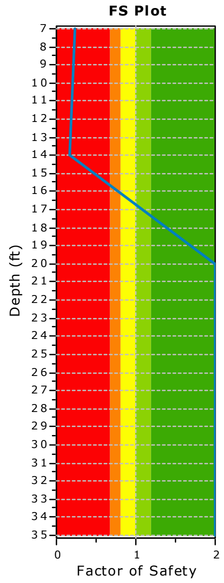
Location : Mercer Island

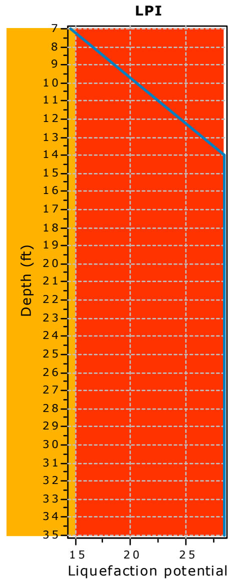
:: Input parameters and analysis properties ::

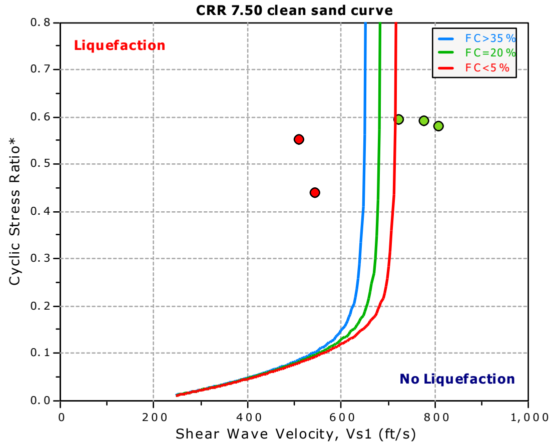
Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	5.00 ft
G.W.T. (earthq.):	4.00 ft
Earthquake magnitude M_w:	7.00
Peak ground acceleration:	0.61 g
Eq. external load:	0.00 tsf











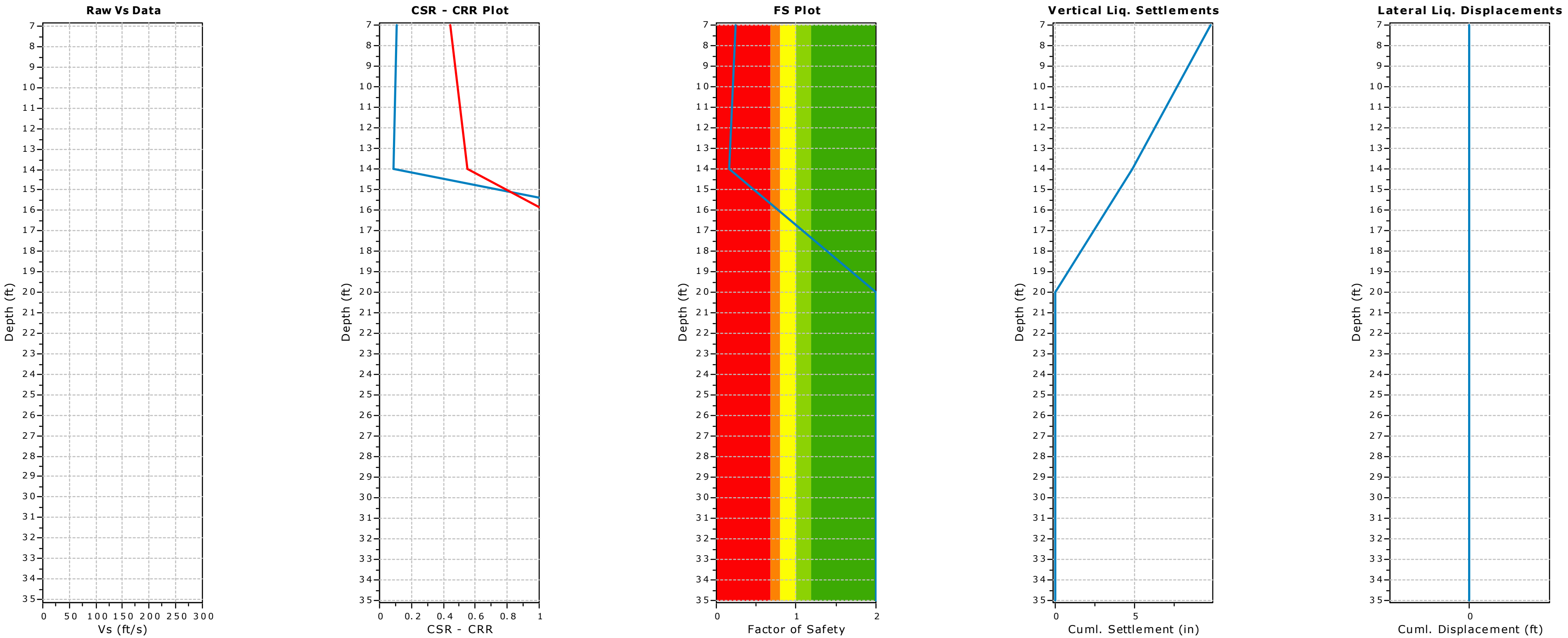
F.S. color scheme

- 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 : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
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)

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