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

September 2, 2025

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

RE: Plan Review & 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 reviewed the architectural plans by MacPherson Construction and Design dated April 1, 2025, civil plans by Ethos Civil dated October 1, 2024, and structural plans by Mulhern Kulp dated October 22, 2024 and updated March 27, 2025. The plans appear to include relevant information from the geotechnical report. We have no comments at this time.

The following is the City-requested information regarding an updated statement of risk and discussion of subsurface soils and liquefaction hazards/mitigation from their letter dated August 27, 2025.

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.

CPT Boring and Liquefaction

The risk and amount of differential settlement between current auger-cast pile foundations and new pin pile supported foundation systems would be less than 1/2 inch over a 20 foot span. This of course relies on proper design and placement of both types of systems (older and upcoming). Pin piles driven to refusal typically experience very little post-construction settlement. Load testing to 200 percent of the design load commonly indicate less than 0.25 inches of movement. The risk of movements between the two types of foundation is low if piles are driven to refusal and existing foundations are confirmed through as built information (auger cast or piers).

Downdrag is not a common discussion or risk with piles smaller than about 6 inches in diameter. Larger piles are more likely to develop skin friction that could result in marginal to significant downdrag. It is our opinion that the risk of downdrag is low and does not pose a life safety risk. Mitigation is not warranted.

We recently advanced a Cone Penetrometer Test boring (CPT) in the driveway at the site. This log is attached with seismic information and soil behavior. To summarize, the boring encountered areas of fine grained soils within the upper 12.5 feet which were locally loose. Minor interbeds of these soils were present near 20 and 23 feet below grade. All other soils were mostly coarse grained with gravels and became consistently dense about 12.5 feet below grade. Very dense soils were encountered consistently below 34 feet with refusal.

We used the CPT information along with groundwater data from our previous boring to perform liquefaction analyses, which are attached. The analyses indicate up to 8.5 inches of total settlement due to liquefaction with differential settlement of about 4.25 inches over a span of 20 feet. We note that these analyses were likely conservative with lower average soil density for the units. The liquefaction zone was primarily 4 to 20 feet below grade.

The seismic testing/analysis indicates that Site Class E is present from about 0 to 20 feet below grade, Site Class D from 20 to 35 feet below grade, and Site Class C at 35 feet and below. Pin piles will likely achieve refusal in the coarse grained deposits between about 20 and 30 feet below grade. Site Class D appears to be reasonable. We recommend piles extending into dense soils at least 5 feet to refusal.

We analyzed the risk of lateral spread by the Bartlett and Youd method (1993). We determined a 1.45 kilometer distance between the site and Seattle Fault Zone from the DNR boring database and relevant thickness of soils with an N value of 15 or less (average), grain size, and slope between the property and Lake Washington.

The analyses yield an estimated 0.15 meters of lateral spread. This is about 6 inches of lateral movement. A mat foundation systems including interconnecting grade beams supported by pin piles could be utilized to resist lateral spread, if required. Battered piles may be an option but could result in some structural distress of the residence during/after certain seismic events.

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

Mitigation Sequencing (Liquefaction)

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;

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

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;

Pile installation is the least impactful method to create a stable building pad and foundation system.

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

Not applicable. Piles reduce the impact by transferring building loads to deeper bearing strata.

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

Not applicable.

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

Not applicable.

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

Monitoring would include observation of pile installation and load testing. Post construction monitoring is not required.

Sincerely,

Cobalt Geosciences, LLC

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STATE OF WASHINGTON
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REGISTERED PROFESSIONAL ENGINEER

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State of Washington
Engineering Geologist
2513
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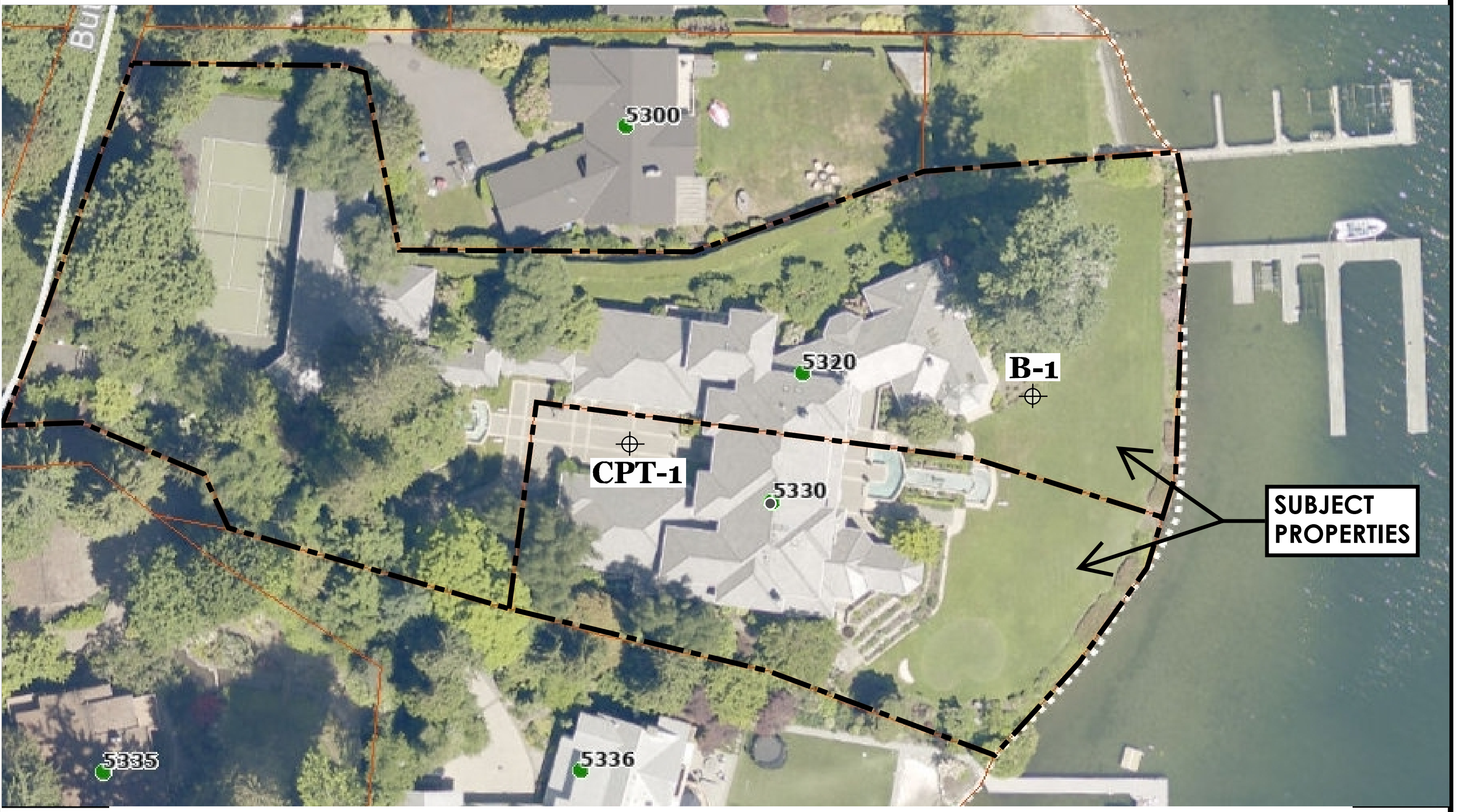
PHILLIP HABERMAN

Phil Haberman

9/2/2025
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
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Kenmore, WA 98028
(206) 331-1097
www.cobaltgeo.com
cobaltgeo@gmail.com

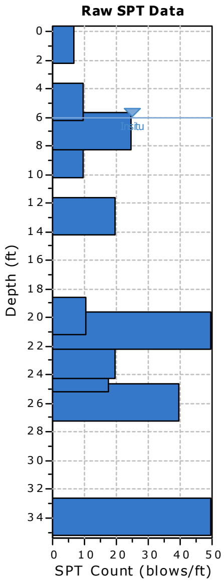
SPT BASED LIQUEFACTION ANALYSIS REPORT

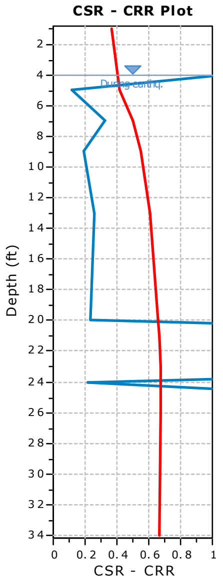
Project title : 5320 Butterworth **SPT Name: SPT #1**

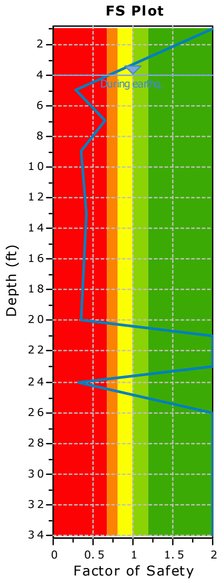
Location : Mercer Island

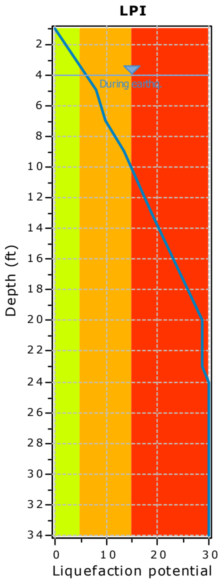
:: Input parameters and analysis properties ::

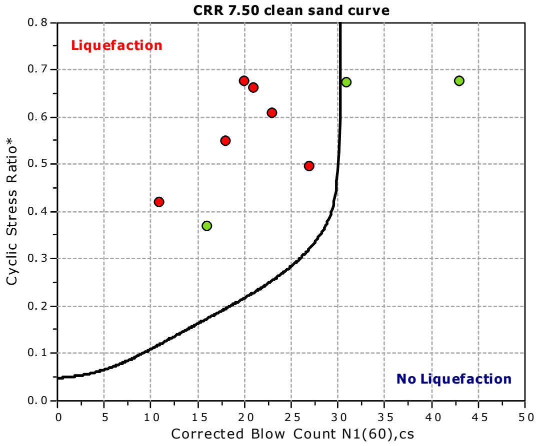
Analysis method:	NCEER 1998	G.W.T. (in-situ):	6.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	4.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.00
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.68 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		











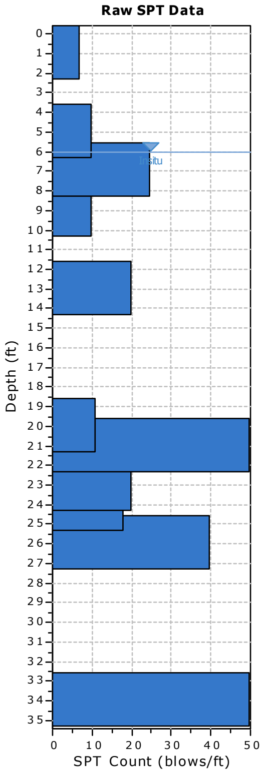
F.S. color scheme

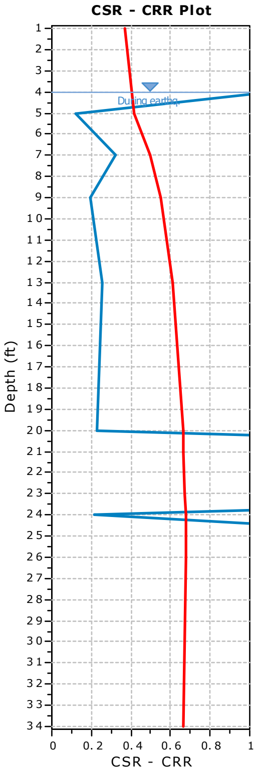
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

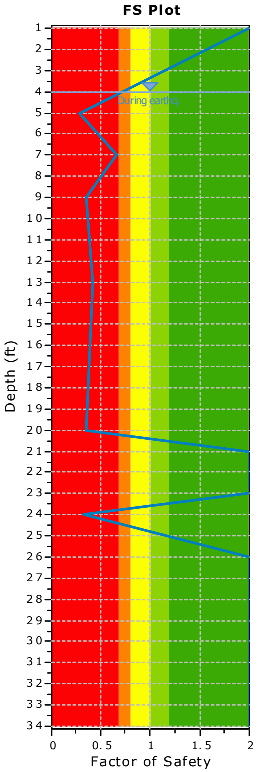
LPI color scheme

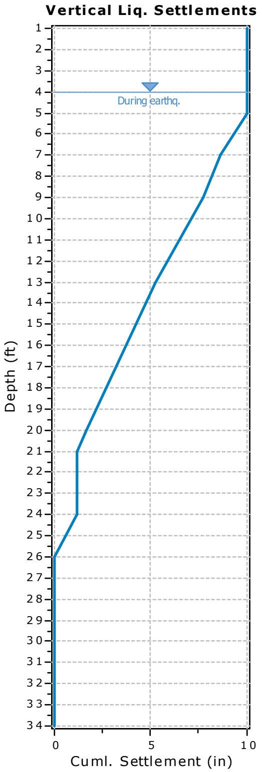
- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

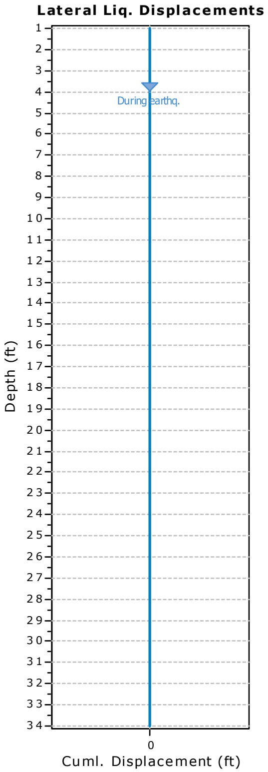
:: Overall Liquefaction Assessment Analysis Plots ::











:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
1.00	7	50.00	100.00	5.00	Yes
5.00	10	5.00	100.00	2.00	Yes
7.00	25	5.00	105.00	2.00	Yes
9.00	10	35.00	105.00	4.00	Yes
13.00	20	5.00	115.00	7.00	Yes
20.00	11	40.00	110.00	1.00	Yes
21.00	50	5.00	115.00	2.00	Yes
23.00	20	40.00	110.00	1.00	Yes
24.00	18	5.00	110.00	2.00	Yes
26.00	40	5.00	115.00	8.00	Yes
34.00	50	5.00	115.00	15.00	Yes

Abbreviations

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

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

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	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	7	100.00	0.05	0.00	0.05	1.70	1.00	1.00	0.75	1.00	9	50.00	5.00	1.20	16	4.000
5.00	10	100.00	0.25	0.00	0.25	1.53	1.00	1.00	0.75	1.00	11	5.00	0.00	1.00	11	0.120
7.00	25	105.00	0.35	0.03	0.32	1.46	1.00	1.00	0.75	1.00	27	5.00	0.00	1.00	27	0.323
9.00	10	105.00	0.46	0.09	0.37	1.42	1.00	1.00	0.75	1.00	11	35.00	5.00	1.20	18	0.196
13.00	20	115.00	0.69	0.22	0.47	1.34	1.00	1.00	0.85	1.00	23	5.00	0.00	1.00	23	0.255
20.00	11	110.00	1.07	0.44	0.64	1.22	1.00	1.00	0.95	1.00	13	40.00	5.00	1.20	21	0.229
21.00	50	115.00	1.13	0.47	0.66	1.20	1.00	1.00	0.95	1.00	57	5.00	0.00	1.00	57	4.000
23.00	20	110.00	1.24	0.53	0.71	1.17	1.00	1.00	0.95	1.00	22	40.00	5.00	1.20	31	4.000
24.00	18	110.00	1.30	0.56	0.74	1.16	1.00	1.00	0.95	1.00	20	5.00	0.00	1.00	20	0.218
26.00	40	115.00	1.41	0.62	0.79	1.13	1.00	1.00	0.95	1.00	43	5.00	0.00	1.00	43	4.000
34.00	50	115.00	1.87	0.87	1.00	1.03	1.00	1.00	1.00	1.00	51	5.00	0.00	1.00	51	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
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
$CRR_{7.5}$: 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

:: 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	100.00	0.05	0.00	0.05	1.00	1.00	0.442	1.19	0.370	1.00	0.370	2.000	🟢
5.00	100.00	0.25	0.03	0.22	0.99	1.00	0.500	1.19	0.419	1.00	0.419	0.287	🔴
7.00	105.00	0.35	0.09	0.26	0.99	1.00	0.592	1.19	0.496	1.00	0.496	0.651	🔴
9.00	105.00	0.46	0.16	0.30	0.98	1.00	0.656	1.19	0.550	1.00	0.550	0.355	🔴
13.00	115.00	0.69	0.28	0.41	0.97	1.00	0.725	1.19	0.608	1.00	0.608	0.419	🔴
20.00	110.00	1.07	0.50	0.58	0.96	1.00	0.790	1.19	0.662	1.00	0.662	0.347	🔴
21.00	115.00	1.13	0.53	0.60	0.95	1.00	0.793	1.19	0.665	1.00	0.665	2.000	🟢
23.00	110.00	1.24	0.59	0.65	0.95	1.00	0.802	1.19	0.672	1.00	0.672	2.000	🟢
24.00	110.00	1.30	0.62	0.67	0.95	1.00	0.805	1.19	0.675	1.00	0.675	0.323	🔴
26.00	115.00	1.41	0.69	0.73	0.94	1.00	0.807	1.19	0.676	1.00	0.676	2.000	🟢
34.00	115.00	1.87	0.94	0.94	0.90	1.00	0.793	1.19	0.665	1.00	0.665	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	4.00	0.00
5.00	0.287	0.71	9.24	4.00	8.03
7.00	0.651	0.35	8.93	2.00	1.90
9.00	0.355	0.64	8.63	2.00	3.39
13.00	0.419	0.58	8.02	4.00	5.68
20.00	0.347	0.65	6.95	7.00	9.69
21.00	2.000	0.00	6.80	1.00	0.00
23.00	2.000	0.00	6.49	2.00	0.00
24.00	0.323	0.68	6.34	1.00	1.31
26.00	2.000	0.00	6.04	2.00	0.00
34.00	2.000	0.00	4.82	8.00	0.00

Overall potential I_L : 30.00

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

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
1.00	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.00	0.000

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)

Cumulative settlemetns: 0.000

Abbreviations

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

:: Vertical settlements estimation for saturated sands ::

Depth (ft)	D_{50} (in)	q_c/N	e_v (%)	Δh (ft)	s (in)
5.00	0.01	2.10	5.80	2.00	1.392
7.00	0.01	2.10	3.72	2.00	0.892
9.00	0.01	2.10	5.18	4.00	2.488
13.00	0.01	2.10	4.24	7.00	3.562
20.00	0.01	2.10	4.57	1.00	0.548
21.00	0.01	2.10	0.00	2.00	0.000
23.00	0.01	2.10	0.00	1.00	0.000
24.00	0.01	2.10	4.76	2.00	1.141
26.00	0.01	2.10	0.00	8.00	0.000
34.00	0.10	4.04	0.00	15.00	0.000

Cumulative settlements: 10.024

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	9	42.00	0.00	5.00	0.000	0.00
5.00	11	46.43	34.10	2.00	0.000	0.00
7.00	27	72.75	11.04	2.00	0.000	0.00
9.00	11	46.43	34.10	4.00	0.000	0.00
13.00	23	67.14	14.50	7.00	0.000	0.00
20.00	13	50.48	34.10	1.00	0.000	0.00
21.00	57	100.00	0.00	2.00	0.000	0.00
23.00	22	65.67	0.00	1.00	0.000	0.00
24.00	20	62.61	22.70	2.00	0.000	0.00
26.00	43	100.00	0.00	8.00	0.000	0.00
34.00	51	100.00	0.00	15.00	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)

Cumulative lateral displacements: **0.00**

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)

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

PROJECT:

ESTIMATE OF LATERAL SPREAD BY METHOD OF BARTLETT & YOUD (1993)

FOR GROUND SLOPE CONDITION:

$$\text{Log (DH)} = -15.787+1.178M - 0.927\log R - 0.013R+0.429\text{Log } S +0.348 \log T15 + 4.527 \log (100-F15) - 0.922D5015$$

M = Earthquake magnitude
R = Horiz. Distance to nearest seismic energy source or fault rupture (km)
S = Slope of ground surface (%)
T15 = Thickness of saturated layers with (n1)60 less than 15 blow per foot (m)
F15 = Average fines content in T15 (%)
D5015 = Average D50 in T15 (mm)

INPUT:
M = 7
R = 1.45
S = 2
T15 = 1
F15 = 50
D5015 = 1

INTERMEDIATE VALUES CALCULATED
A = log R = 0.161368
B = log S = 0.30103
C = log T15 = 0
D = log (100 - F15) = 1.699

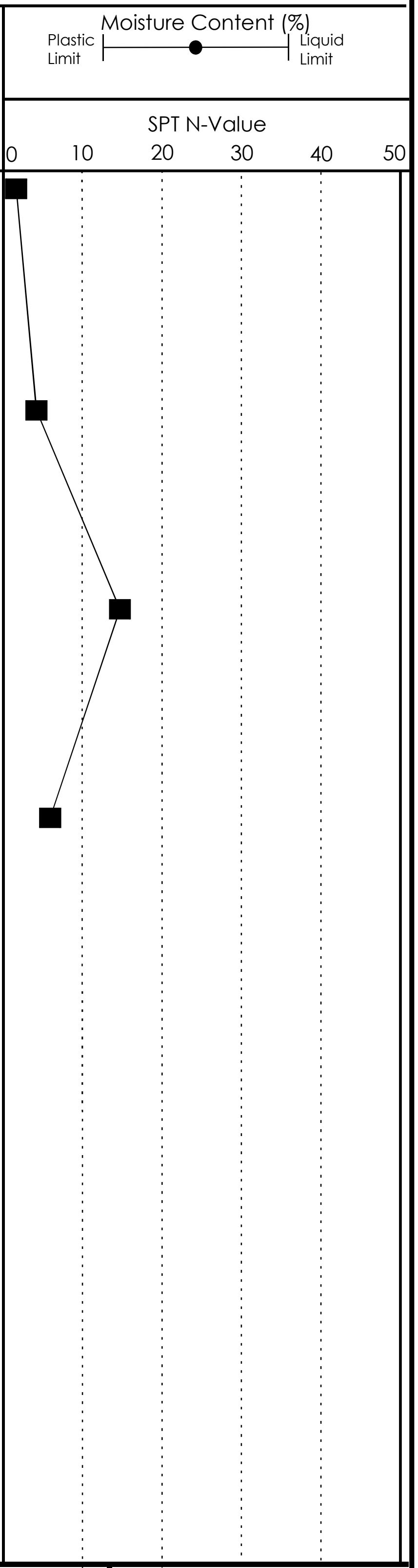
OUTPUT:
Log DH = -0.81106
DH = 0.15 meters

Ref.:
Youd, T.L., Hansen, C.M. and Bartlett, S.F. , "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement", ASCE Journal of Geotechnical and Geoenvironmental Engineering, December 2002, pg 1007 - 1017.

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



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Proposed Residence
5320 Butterworth Road
Mercer Island, Washington

Boring Log

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

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Soil Classification Chart

Figure C1