COBALT GEOSCIENCES

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

November 25, 2024

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

RE: Geotechnical Addendum
Proposed Additions/Remodel
5330 Butterworth Road
Mercer Island, Washington

In accordance with your authorization, Cobalt Geosciences, LLC has prepared this letter to present responses to City comments. Our responses are as follows:

The risk and amount of differential settlement between current auger-cast pile foundations and new pin pile supported foundation systems would be less than ½ 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 and 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.

Sincerely,

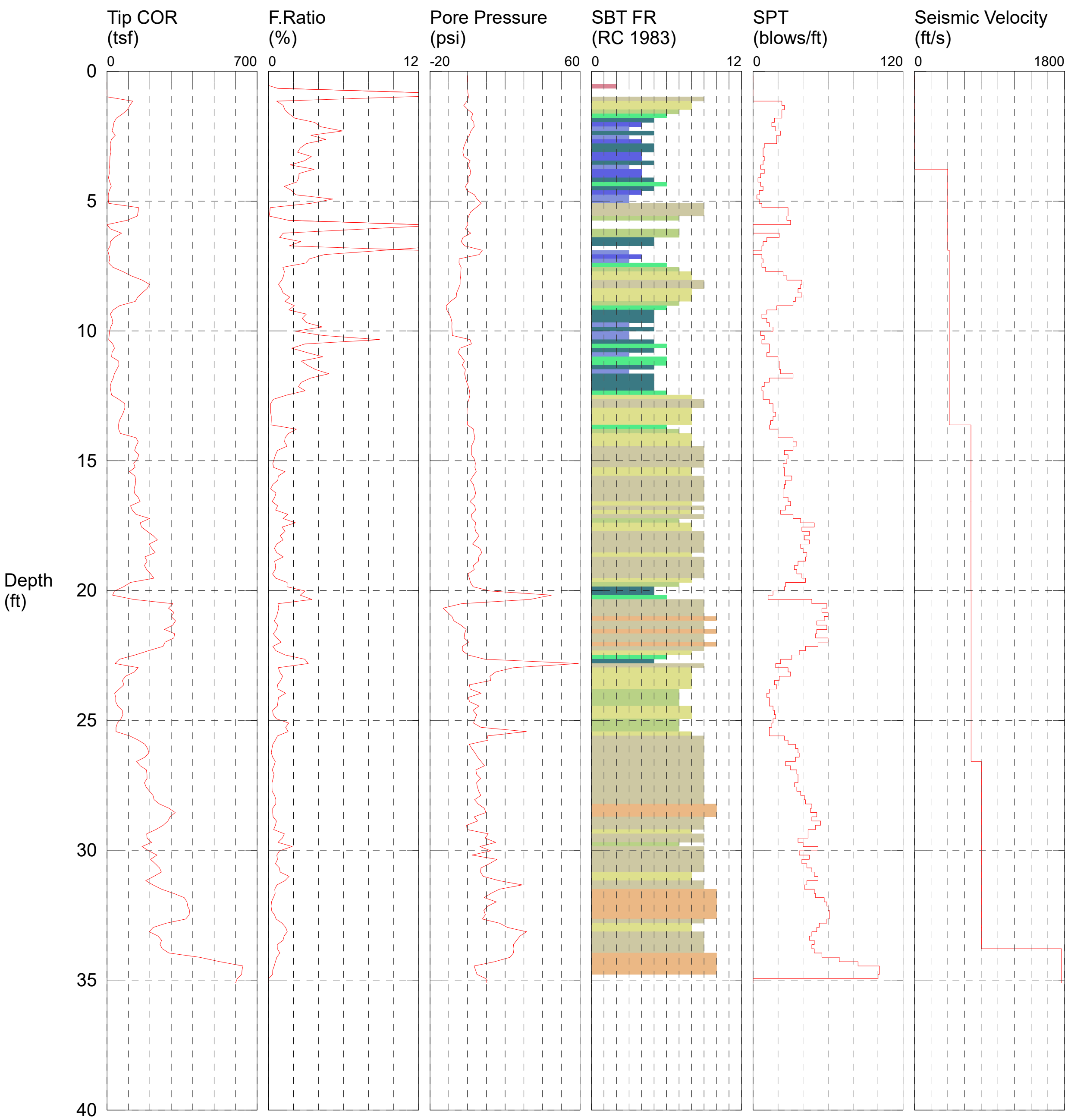
Cobalt Geosciences, LLC

11/25/2024
Phil Haberman, PE, LG, LEG
Principal

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

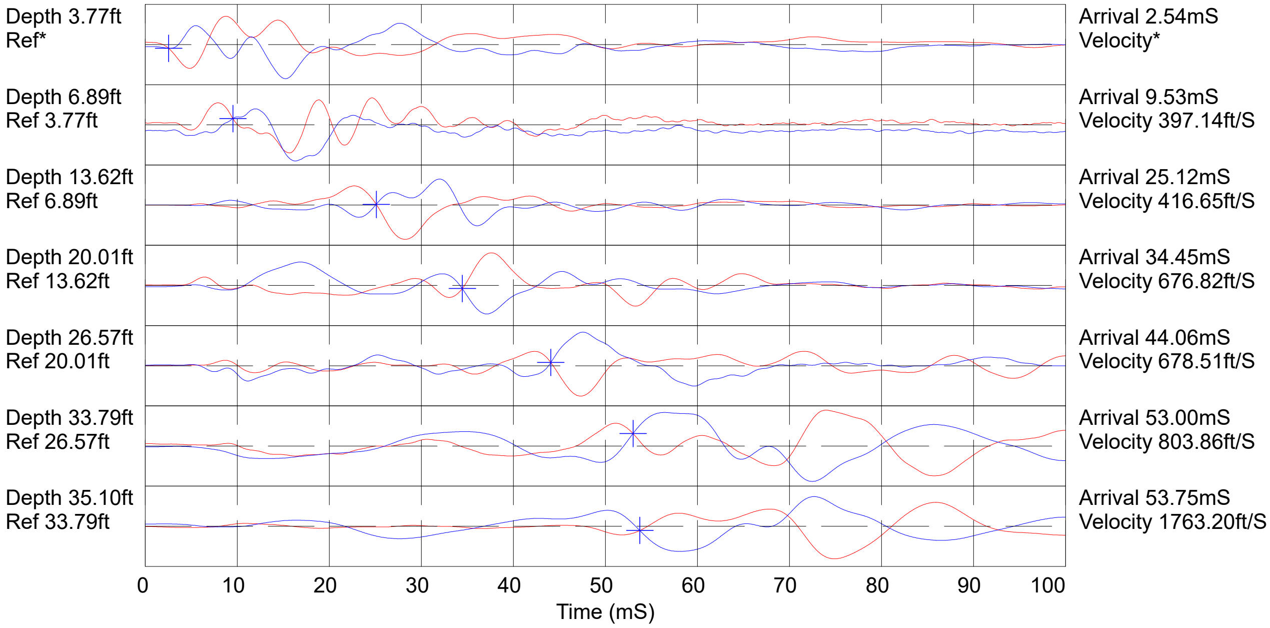


TOTAL DEPTH: 35.105 ft

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

*SBT/SPT CORRELATION: UBC-1983

HOLE NUMBER: sCPT-01



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

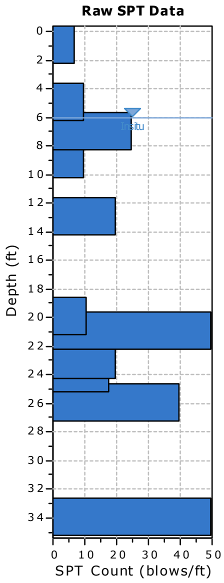
SPT BASED LIQUEFACTION ANALYSIS REPORT

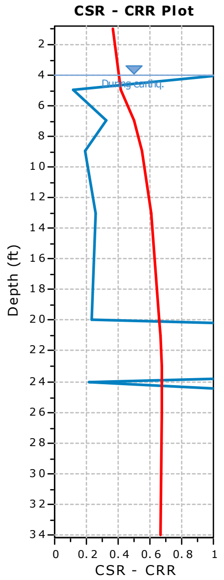
Project title : 5330 Butterworth **SPT Name: SPT #1**

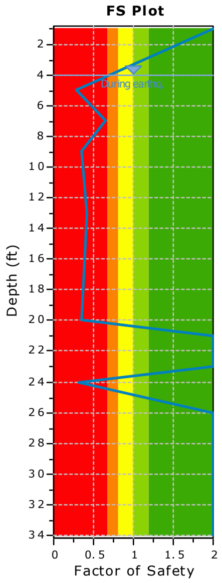
Location : Mercer Island

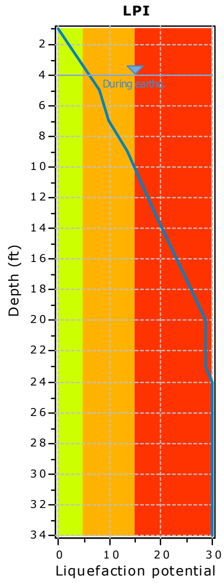
:: Input parameters and analysis properties ::

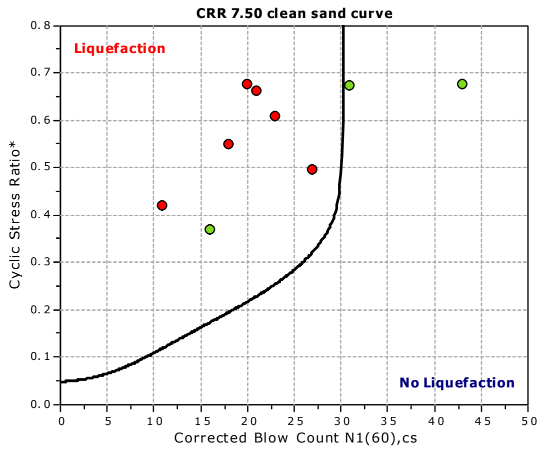
Analysis method:	NCEER 1998	G.W.T. (in-situ):	6.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	4.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.00
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.68 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer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











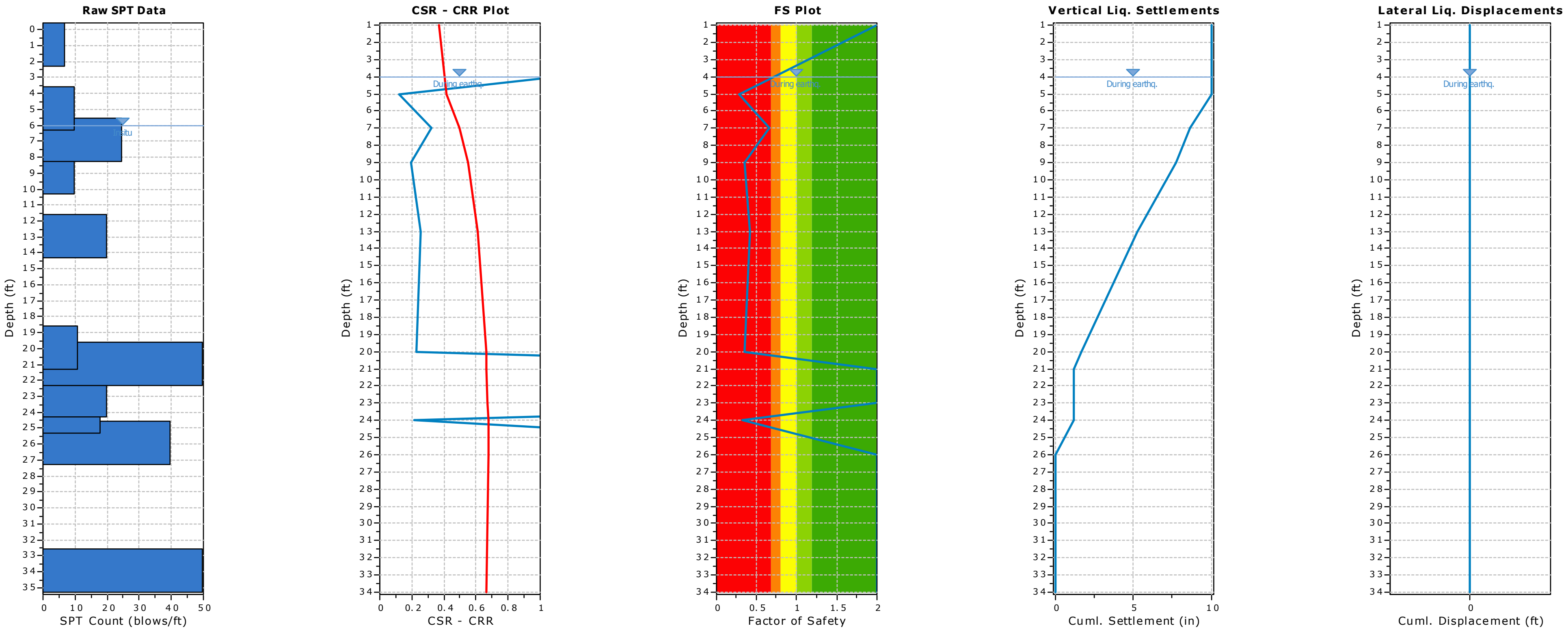
F.S. color scheme

- 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
$CRR_{7.5}$: 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

:: 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)

<u>FOR GROUND SLOPE CONDITION:</u>

Log (DH) = -15.787+1.178M - 0.927log R - 0.013R+0.429Log 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:		INTERMEDIATE VALUES CALCULATED	
M =	7	A = log R =	0.161368
R =	1.45	B = log S =	0.30103
S =	2	C = log T15 =	0
T15 =	1	D = log (100 - F15) =	1.699
F15 =	50		
D5015 =	1		

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.