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GEOTECHNICAL ■ MATERIALS ■ SPECIAL INSPECTIONS
SBE ■ SLBE ■ SCOOP

Hines
Attn: Mr. Tom Lawless
4000 MacArthur Blvd., Suite 110
Newport Beach, CA 92660

July 20, 2017

Subject: Response to Review Comments
Preliminary Geotechnical Investigation
Proposed Pacific Center Development
1071 N. Tustin Ave., 1065 Pacific Center Dr., and 1041 W. Pacific Center Dr.,
Anaheim, California

References:

1. Anaheim 2017. *Geotechnical Engineering Review Sheet, City of Anaheim, Plan Check #OTH 2017-00946*, Albus-Keefe & Associates, AKA Project Number 2608.0, April 26, 2017.
2. NOVA 2017. *Report, Preliminary Geotechnical Investigation, Proposed Pacific Center Development, Anaheim, California*, NOVA Services, Inc., February 17, 2017.

Dear Mr. Lawless,

The intent of this letter is to respond to review comments by the City of Anaheim of the above-referenced geotechnical report. This response has been prepared by NOVA Services, Inc. (NOVA) for Hines. NOVA is retained by Hines as Geotechnical Engineer-of-Record for the project. In no

Review Comments and Responses

The following text reproduces the comments from Anaheim 2017. The comments are reproduced in italics, followed directly by responses by NOVA.

Comment 1, CPT Data. *Section 1.4 (and a few other sections) state that Cone Penetration Test (CPT) data were provided in Appendix C. We could not find CPT data in this or other appendices. Please provide CPT data.*

Response 1. Attachment 1 to this letter provides *Summary of Cone Penetration Test Data* by Kehoe Testing and Engineering, dated January 14, 2017.



Comment 2, Resistivity Data. *Minimum resistivity of 4900 ohm-cm has been reported (Section 5.3.7 and Table 6-2). The discussion that follows Table 6-2 cites Caltrans criteria, concluding that the soil would not be considered corrosive. However, with minimum resistivity of 4900 ohm-cm, the soil should be considered as moderately corrosive. Please clarify and provide mitigating recommendations.*

Response 2. The report should state that the soils will be corrosive to buried metals. The resistivity data provided in Appendix C of the report includes estimates of the life expectancy of a variety of gauges of exposed ferrous metals.

There are several methods of preventing corrosion of buried pipes, though most of these methods are only practical for application prior to installation of the piping. For existing buried pipes the most cost-effective method of minimizing or eliminating corrosion is cathodic protection. Cathodic protection involves the use of sacrificial magnesium or zinc anodes connected to the pipe material, which acts as the cathode. Under corrosive environments, the anode will corrode, sparing the cathode (pipe material). Both the replacement pipe sections and existing pipes can be effectively protected by this method.

This section of NOVA 2017 concludes with the following advice:

Testing to determine several chemical parameters that indicate a potential for soils to be corrosive to construction materials are traditionally completed by the Geotechnical Engineer, comparing testing results with a variety of indices regarding corrosion potential. Like most geotechnical consultants, NOVA does not practice in the field of corrosion protection, since this is not specifically a geotechnical issue. Should more information be required, a specialty corrosion consultant should be retained to address these issues.

Comment 3, Groundwater Level for Analysis. *Groundwater depth of 35 feet is used in liquefaction analysis. According to Seismic Hazard Zone Report 011 (Orange 7.5-Minute Quadrangle, 1997), in the area of the project the historically high groundwater level is estimated as shallow as approximately 10 feet below the ground surface. Please provide the reasoning and data to support the assumed groundwater depth of 35 feet. Otherwise, liquefaction potential should be re-evaluated with consideration of the historic high ground water levels, followed by the associated seismic settlement and lateral spread.*

Response 3. CPT soundings were extended to 71 feet below ground surface (bgs). No groundwater was encountered to this level. NOVA considers it unlikely the groundwater levels will rise from below 70 feet below ground surface (bgs) to within 10 feet bgs during the lives of structures at the site. As is well-established by OCWD 2015, the Orange County Water District is making concerted efforts to recharge groundwater in the area. However, groundwater withdrawals remain high and it is not expected that groundwater levels will recover for decades.

Liquefaction analyses have been reconsidered using an assumed groundwater level of 10 feet bgs. Records of this evaluation are provided as Attachment 2 to this letter. The analyses employed procedures developed by Robertson (2009), available by the software CLiq. Figure 1 and Figure 2 (following page) provides the indications of the re-evaluation of liquefaction.

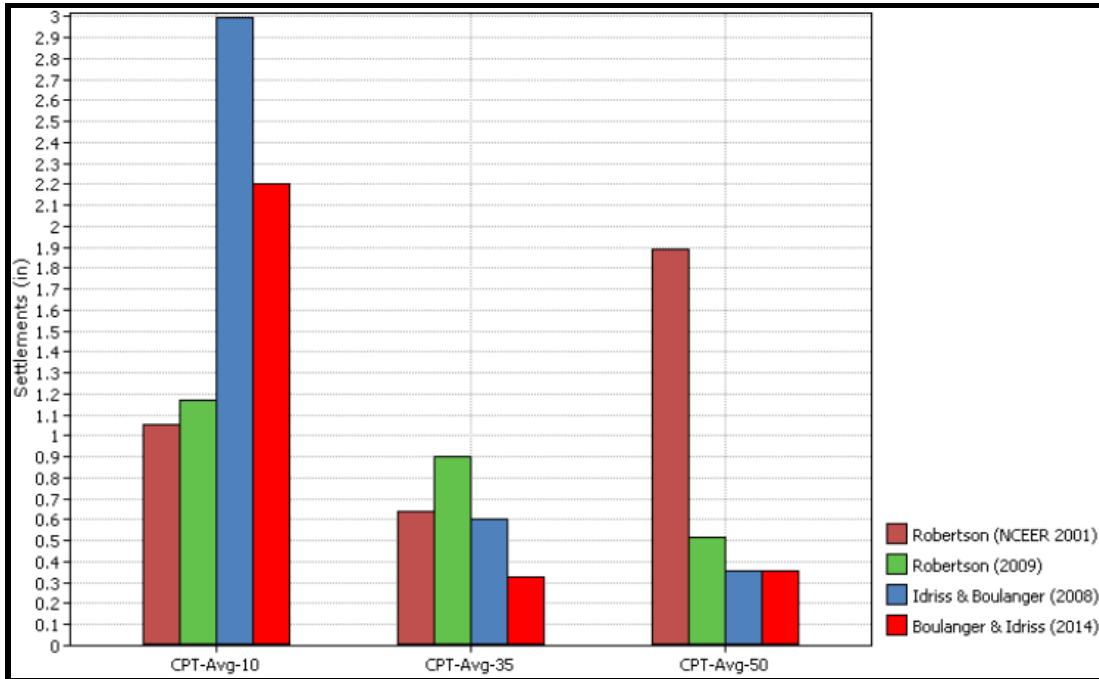


Figure 1. Estimates of Seismic Settlement for Average CPT Data and Groundwater Levels At 10 Feet, 35 Feet and 50 Feet Below Ground Surface

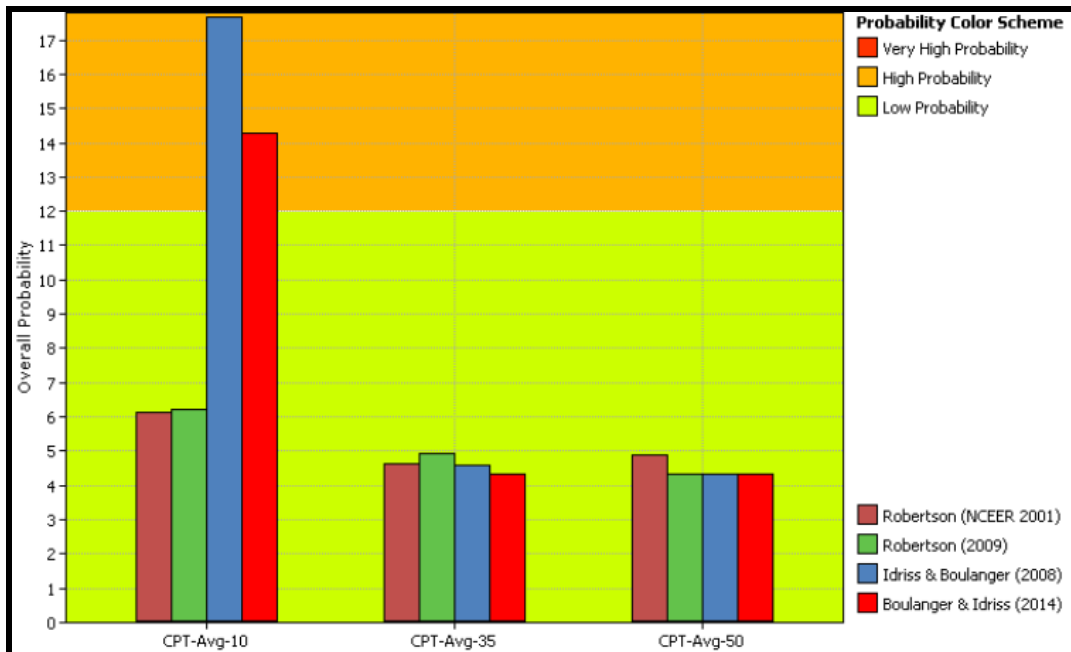


Figure 2. Estimates of Liquefaction Probability for Average CPT Data and Groundwater Levels At 10 Feet, 35 Feet and 50 Feet Below Ground Surface



As may be seen by review of Figure 1, if groundwater levels rise to about 10 feet bgs, the Robertson method indicates that liquefaction and related dry sand movement will be on the order of 1 inch. As may be seen by review of Figure 2, the potential for liquefaction at this site is low if groundwater does not rise to within about 10 feet bgs. However, if the groundwater level rises to about 10 feet bgs, it is likely liquefaction will occur.

NOVA considers it improbable that groundwater levels will rise to within 10 feet of the ground surface in the lives of the planned structures. If this level must be considered by design, NOVA will reissue its report recommending alternative foundation solutions to the shallow foundations described in NOVA 2017.

Comment 4, Factor Safety against Liquefaction. *Figure 5-1 suggests that factor of safety $FS=1.0$ is used for evaluation of liquefaction potential. However, California Special Publication 117A (Guidelines for Evaluating and Mitigating Seismic Hazards in California; California Geological Survey, 2008) requires a factor of safety greater than or equal to 1.3. Please comment or revise liquefaction analysis accordingly.*

Response 4. The CDMG guidelines in Special Publication 117A do not require a specific factor safety. As noted above, NOVA has reconsidered the liquefaction analyses utilizing a groundwater level at 10 feet bgs.

As may be seen by review the data provided in Attachment 2, analyses using the Robertson 2001 method indicate an overall low probability of liquefaction. Moreover, zones of soils at risk from liquefaction are relatively deeper. NOVA's assessment of the risk of damage to structures is that this deeper-seated potential movement beneath densely compacted surface sands will be associated with lower potential for differential foundation movement that could be damaging to structures.

Comment 5, Lateral Spreading. *In Section 5.3.4, potential for lateral spread is reported to be non-existent. This evaluation is based on the gentle ground slope in the vicinity of the site, among other things. However, lateral spread due to slope face (free face) nearby has not been discussed. Both Santa Ana River and Warner Basin may provide a slope face, facilitating lateral spread. Please comment.*

Response 5. As may be seen by review of the liquefaction analyses provided as Attachment 2, no lateral spreading is forecast for the approximately level site.

The slope faces at both the Santa Ana River and Warner Basin are a considerable distance from the property. Warner basin is greater than 1,000 feet distant from the planned structures, an interval of approximately level ground (most of this distance is elevated slightly above the site). The Santa Ana River is about 1,900 feet from the closest structure over approximately level ground.

NOVA concurs that it is reasonable to assume that there may be liquefaction and related lateral spreading at or very near the Santa Ana River and Warner Basin. NOVA judged that the potential for this lateral spread to affect the site is very low due to the distance and related damping of the spreading ground that will occur over distances of 1,000 feet or greater.



Comment 6, Rate of Settlement. *Settlement of 1.0 to 1.5 inches is reported for shallow foundations (e.g. Section 6.5.1 and 6.5.5). It is also stated that 80% of this settlement occurs before completion of construction. What is the basis for the assertion that 80% of settlement would occur during construction?*

Please also address whether the estimated settlement is considered tolerable for the proposed site development. If not, discuss what magnitude of settlement is considered tolerable and what measures may be recommended to mitigate excessive settlement.

Response 6. The basis of the statement is that loads (DL +LL) to shallow foundations are dominated by dead load. Foundation settlements at this sandy site will occur elastically, with foundation movement occurring approximately as the foundations are loaded. As a practical matter, most of the settlements will occur as the structures are constructed, responding to dead loads that dominate live loads. Dead loads for typical apartment structures might accumulate at about 100 psf to 120 psf. Live loads will accumulate at about 20 psf to 30 psf. About 80% of the total structural load will be the structure itself.

The cited distribution of settlement from construction phase to post-construction reflects NOVA's experience monitoring construction of this genre at predominantly sandy sites. The guidance can be removed if the reviewers prefer.

Based on experience with similar structures and construction had numerous sites, NOVA considers that total settlements on the order of 1 inch will be tolerable for structures. Most significantly, as is discussed in the report, NOVA expects that differential movement will be low- well below thresholds of differential movement that can be damaging to structures.

Comment 7, Direct Shear Data. *Direct shear test results are discussed in Section 3.5.2 (item #1 in this section provides friction angle and cohesion). However, results for this test were not found in Appendix B. Please provide. Furthermore, it is not discussed if this test was performed on a remolded specimen or on an undisturbed sample?*

Response 7. The direct shear data is provided as Attachment 3 to this letter.

The direct shear testing was conducted on an undisturbed sample of the Unit 2 sands obtained by the Modified California Sampler ASTM D3550. Near the ground surface these sands are of loose consistency, intended to be improved by the earthwork described in Section 6 of NOVA 2017.



Closure

NOVA hopes this letter adequately responds to comments by the City of Anaheim. Should you have any questions regarding this report or other matters, please do not hesitate to call.

Sincerely,
NOVA Services, Inc.

Jesse D. Bearfield, P.E.
Senior Engineer



John F. O'Brien, P.E., G.E.
Principal Geotechnical Engineer



Attachments:

- Attachment 1: CPT Data
- Attachment 2: Analyses of Liquefaction with Groundwater Level at 10 Feet
- Attachment 3: Direct Shear Data

Attachment 1
CPT Data

SUMMARY
OF
CONE PENETRATION TEST DATA

Project:

**1091 N. Tustin Avenue
Anaheim, CA
January 14, 2017**

Prepared for:

**Mr. Alan Smith
NOVA Services, Inc.
4373 Viewridge Avenue, Ste B
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Office (858) 292-7575 / Fax (858) 292-7570**

Prepared by:



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- Summary of Shear Wave Velocities
- Pore Pressure Dissipation Graphs
- CPeT-IT Calculation Formulas

SUMMARY OF CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at 1091 N. Tustin Avenue in Anaheim, California. The work was performed by Kehoe Testing & Engineering (KTE) on January 14, 2017. The scope of work was performed as directed by NOVA Services, Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at five locations to determine the soil lithology. Groundwater measurements and hole collapse depths provided in **TABLE 2.1** are for information only. The readings indicate the apparent depth to which the hole is open and the apparent water level (if encountered) in the CPT probe hole at the time of measurement upon completion of the CPT. KTE does not warranty the accuracy of the measurements and the reported water levels may not represent the true or stabilized groundwater levels.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	60	Hole open to 18 ft (dry)
CPT-2	15	Refusal, no cave depth taken
CPT-2A	45	Hole open to 45 ft (dry)
CPT-3	71	Refusal, hole open to 47 ft (dry)
CPT-4	60	Hole open to 13 ft (dry)

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm² cone and recorded the following parameters at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed
- Pore Pressure Dissipation (at selected depths)

At locations CPT-1, CPT-2, CPT-3 & CPT-4, shear wave measurements were obtained at various depths. The shear wave is generated using an air-actuated hammer, which is located inside the front jack of the CPT rig. The cone has a triaxial geophone, which recorded the shear wave signal generated by the air hammer.

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil classification on the CPT plots is derived from the attached CPT Classification Chart (Robertson) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (q_c), sleeve friction (f_s), and penetration pore pressure (u). The friction ratio (R_f), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

Tables of basic CPT output from the interpretation program CPeT-IT are provided for CPT data averaged over one foot intervals in the Appendix. We recommend a geotechnical engineer review the assumed input parameters and the calculated output from the CPeT-IT program. A summary of the equations used for the tabulated parameters is provided in the Appendix.

It should be noted that it is not always possible to clearly identify a soil type based on q_c , f_s and u . In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING



Richard W. Koester, Jr.
General Manager

APPENDIX



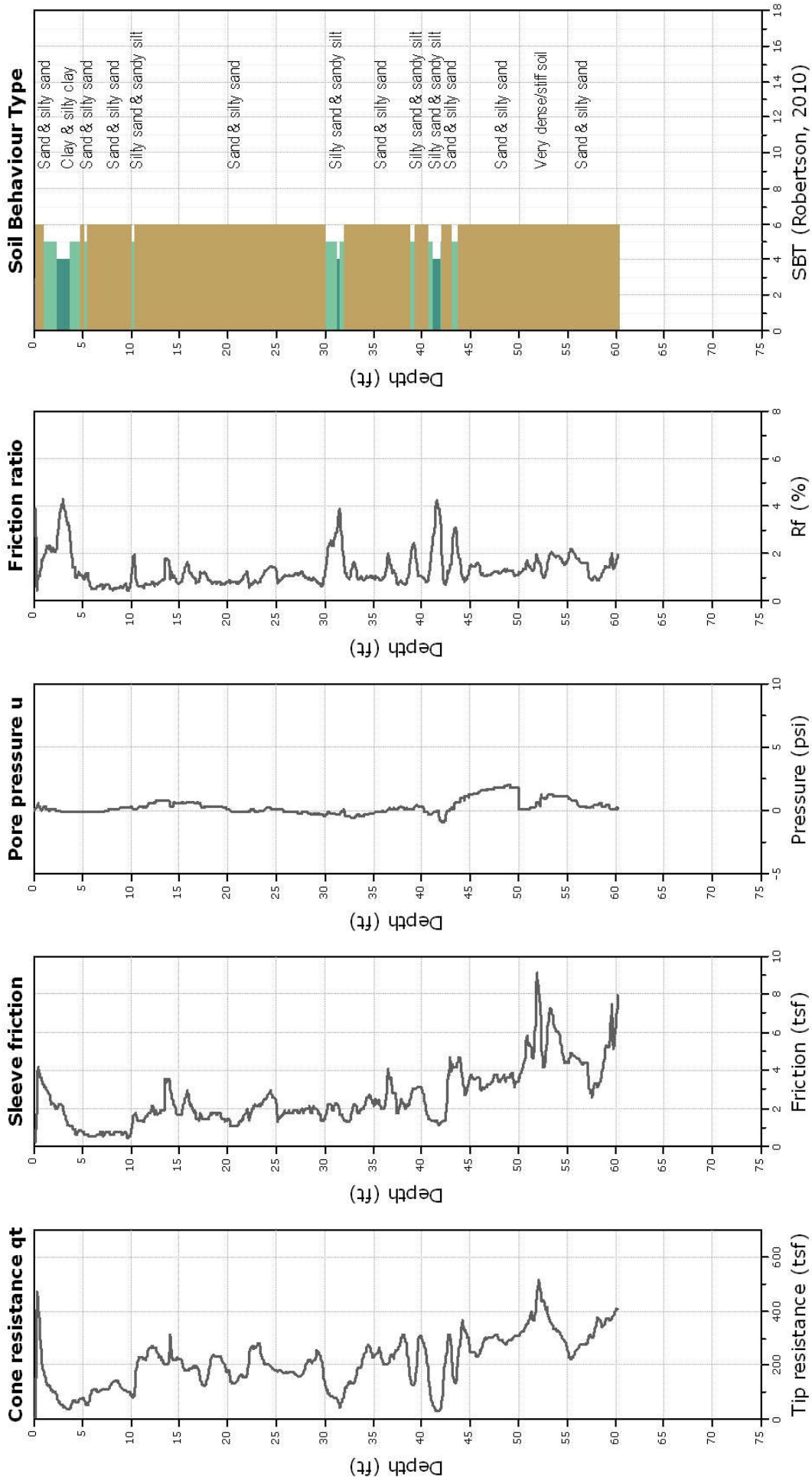
Keheo Testing and Engineering
 714-901-7270
 rich@kehoetesting.com
 www.kehoetesting.com

Project: NOVA Services, Inc.
Location: 1091 N. Tustin Ave Anaheim, CA

CPT-1

Total depth: 60.24 ft, Date: 1/14/2017

Cone Type: Vertek



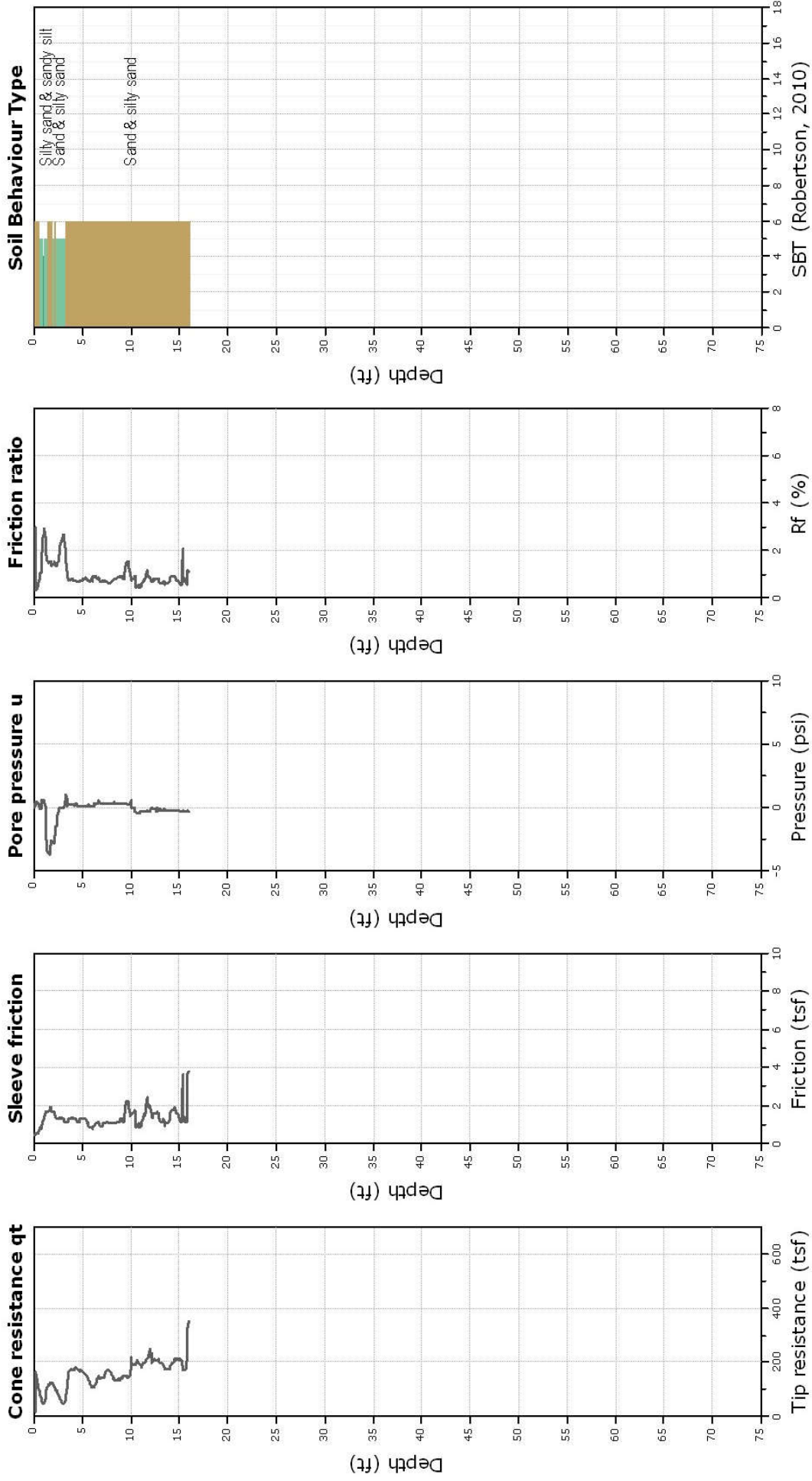


Kehoe Testing and Engineering
714-901-7270
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Project: NOVA Services, Inc.
Location: 1091 N. Tustin Ave Anaheim, CA

CPT-2

Total depth: 15.94 ft, Date: 1/14/2017
Cone Type: Vertek





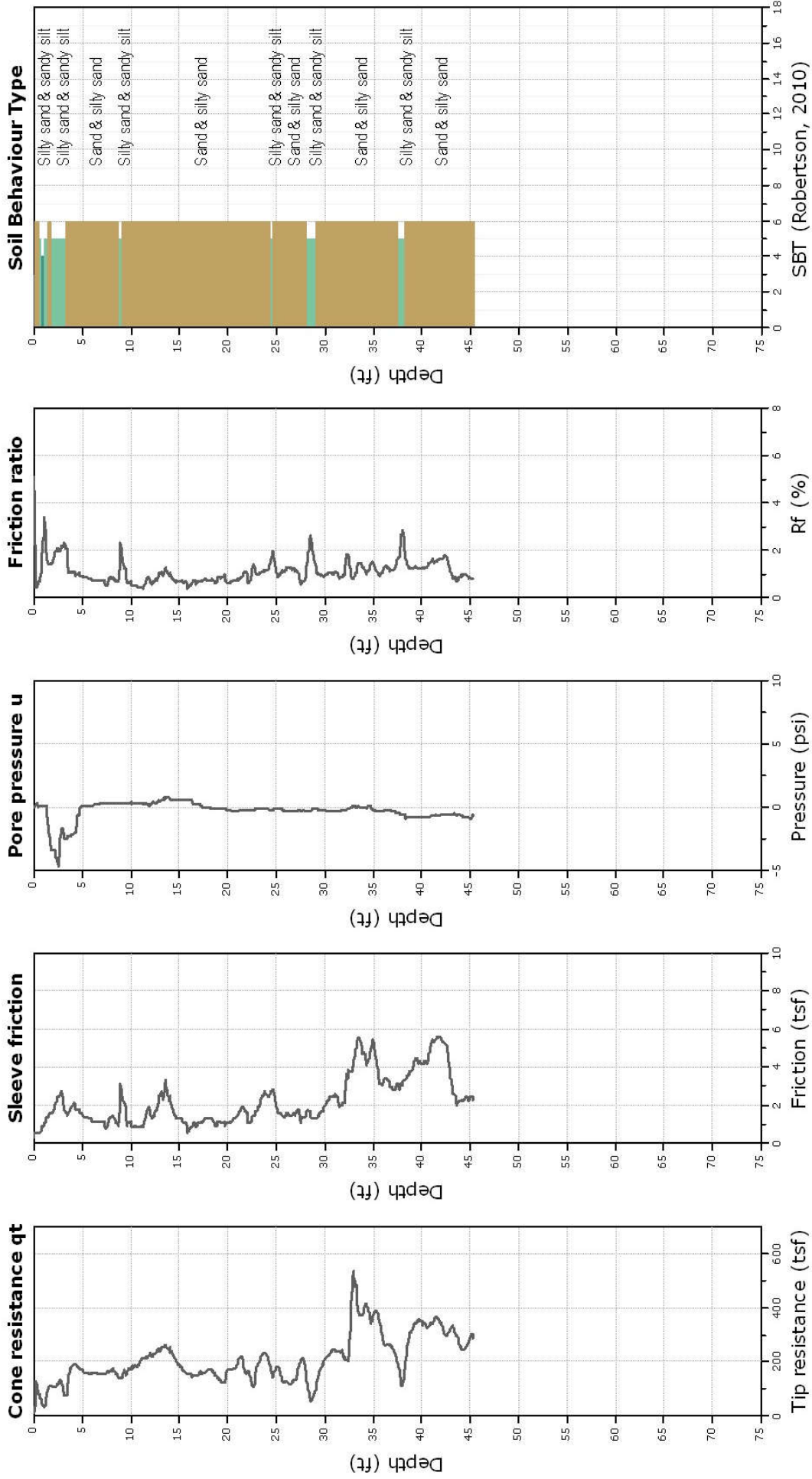
Keheo Testing and Engineering
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www.kehoetesting.com

Project: NOVA Services, Inc.
Location: 1091 N. Tustin Ave Anaheim, CA

CPT-2A

Total depth: 45.28 ft, Date: 1/14/2017

Cone Type: Vertek





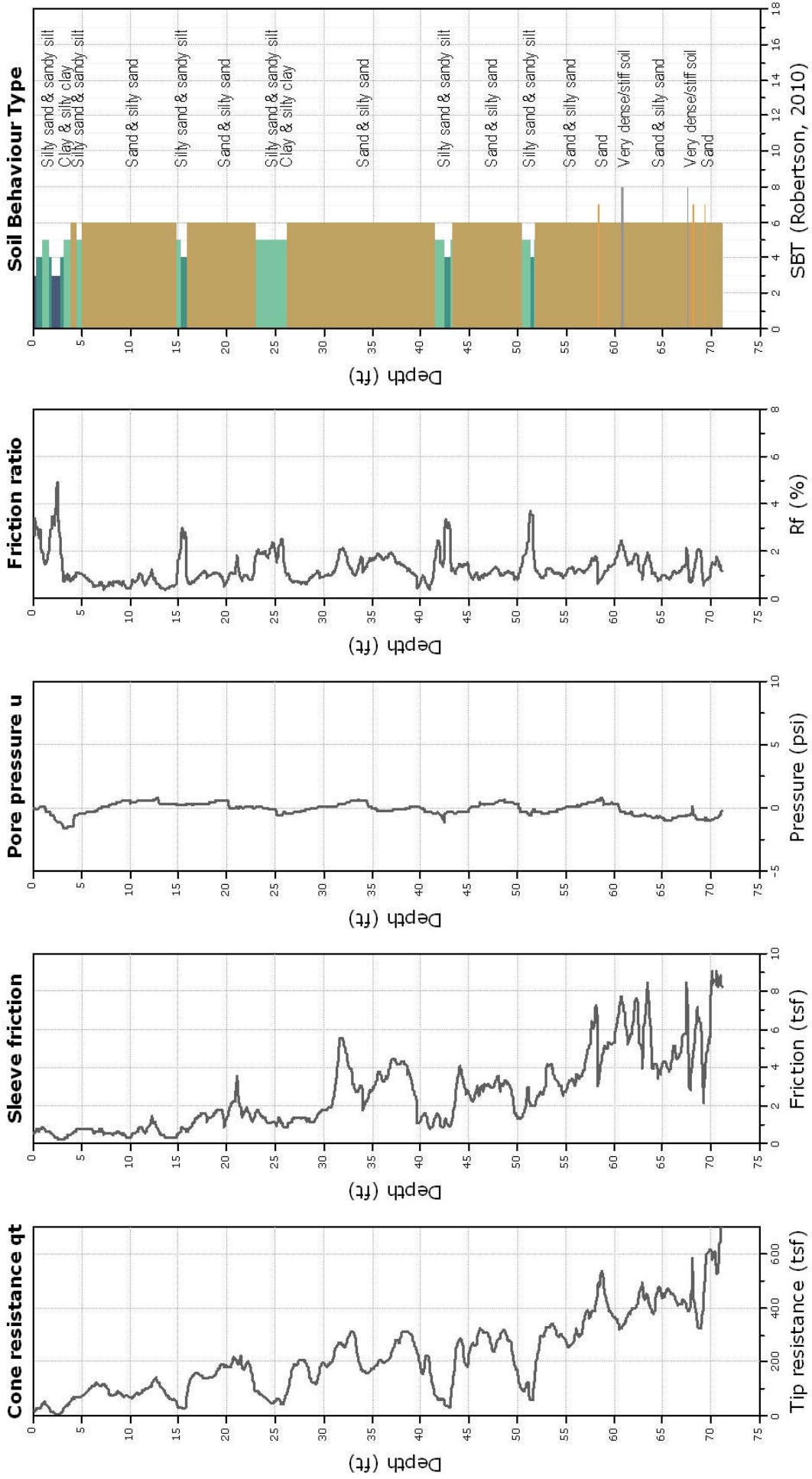
Kehoe Testing and Engineering
 714-901-7270
 rich@kehoetesting.com
 www.kehoetesting.com

Project: NOVA Services, Inc.
Location: 1091 N. Tustin Ave Anaheim, CA

CPT-3

Total depth: 71.14 ft, Date: 1/14/2017

Cone Type: Vertek





Kehoe Testing and Engineering

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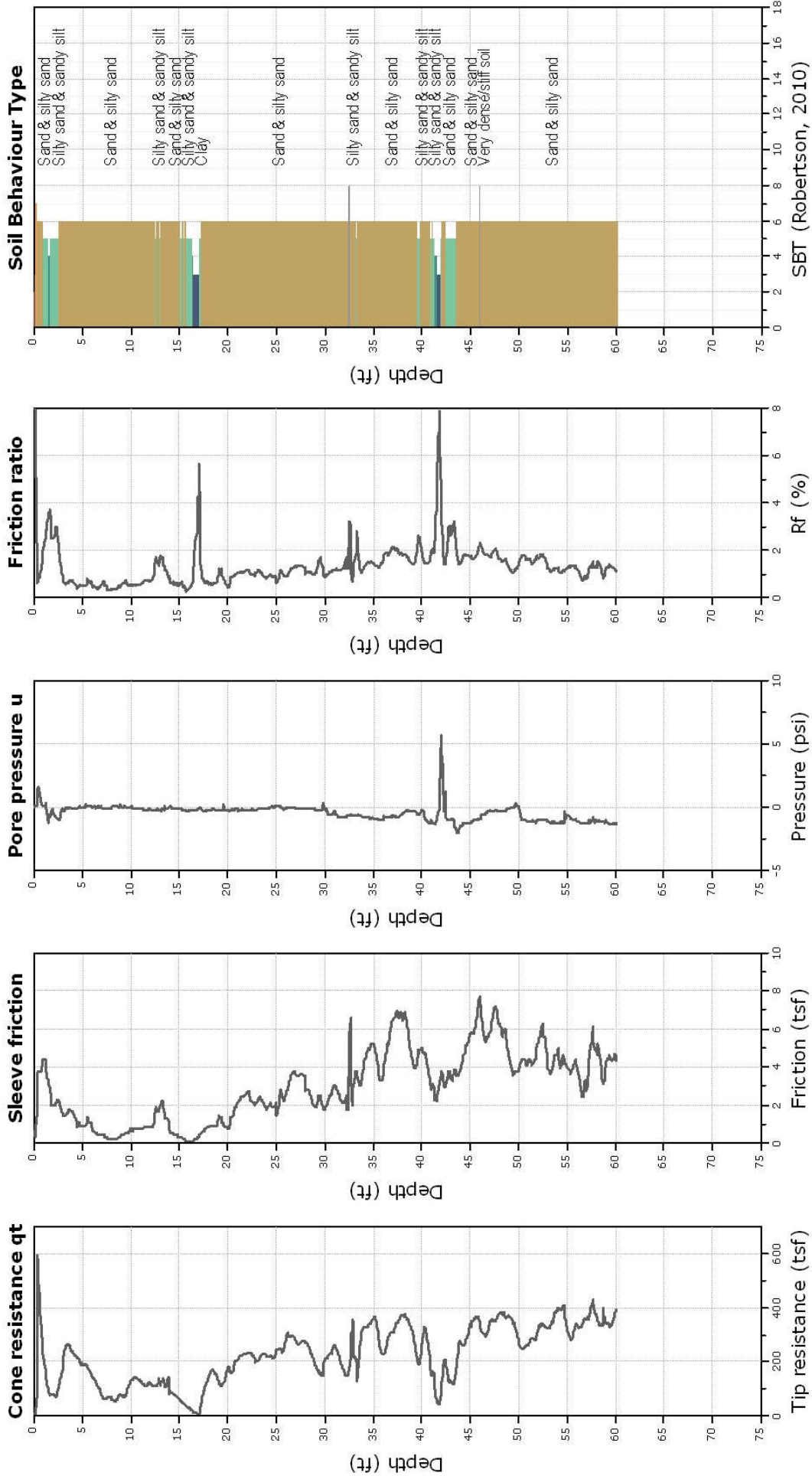
Project: NOVA Services, Inc.

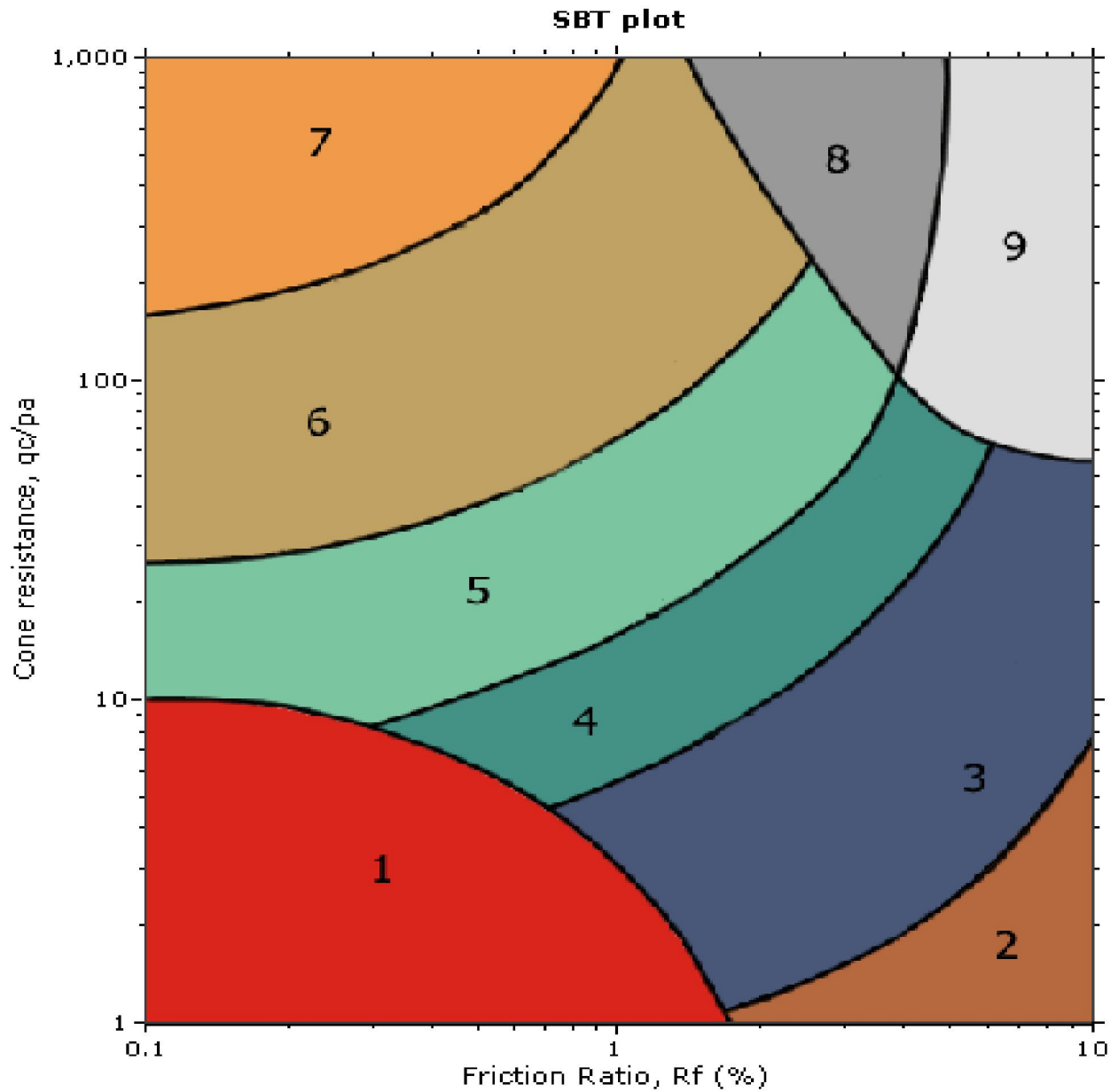
Location: 1091 N. Tustin Ave Anaheim, CA

CPT-4

Total depth: 60.10 ft, Date: 1/14/2017

Cone Type: Vertek





SBT legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

Depth (ft)	CPT-2 In situ data				Basic output data																		
	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic.SBT	\bar{a} (pcf)	\acute{o},v (tsf)	u0 (tsf)	\acute{o}',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn	U2	l(B)	Mod. SBTn
1	46.26	1.25	0.54	-0.3	46.27	2.71	4	2.47	121.27	0.06	0	0.06	761.53	2.71	0	8	0.61	5.66	1.97	247.24	0.64	34.74	7
2	118.52	1.67	-2.77	-0.6	118.49	1.41	6	1.97	125.67	0.12	0	0.12	958.01	1.41	0	6	0.49	2.89	1.67	323.24	-1.62	63.32	7
3	47.1	1.25	0	-0.7	47.1	2.66	5	2.45	121.32	0.18	0	0.18	254.88	2.67	0	5	0.66	3.19	2.11	141.39	0	33.82	7
4	170.32	1.25	0.23	-0.8	170.32	0.74	6	1.67	124.45	0.25	0	0.25	690.47	0.74	0	6	0.42	1.85	1.47	298.11	0.07	106.37	7
5	166.14	1.25	0.15	-1	166.15	0.75	6	1.68	124.39	0.31	0	0.31	537.46	0.76	0	6	0.44	1.72	1.51	269.82	0.04	102.17	7
6	107.04	0.84	0.15	-1.1	107.04	0.78	6	1.84	120.35	0.37	0	0.37	289.26	0.78	0	6	0.5	1.69	1.66	170.87	0.03	88.74	7
7	141.6	0.94	0.31	-1.2	141.61	0.66	6	1.7	121.9	0.43	0	0.43	328.5	0.67	0	6	0.46	1.52	1.56	202.81	0.05	103.8	7
8	155.91	1.04	0.38	-1.2	155.91	0.67	6	1.67	122.9	0.49	0	0.49	316.51	0.67	0	6	0.46	1.43	1.55	209.87	0.06	104.2	7
9	139.62	1.25	0.31	-1.3	139.62	0.9	6	1.79	123.97	0.55	0	0.55	251.45	0.9	0	6	0.52	1.4	1.68	183.84	0.04	82.26	7
10	188.7	1.57	0.54	-1.3	188.71	0.83	6	1.67	126.34	0.62	0	0.62	305.2	0.83	0	6	0.48	1.3	1.59	230.97	0.06	91.85	7
11	190.27	0.94	-0.38	-1.1	190.26	0.49	6	1.52	122.62	0.68	0	0.68	279.77	0.5	0	6	0.44	1.21	1.46	217.66	-0.04	127.97	7
12	223.47	1.88	-0.27	-1.5	223.47	0.84	6	1.62	128.08	0.74	0	0.74	300.29	0.84	0	6	0.48	1.19	1.57	249.9	-0.03	92.52	7
13	193.82	1.15	-0.23	-2	193.81	0.59	6	1.56	124.13	0.8	0	0.8	240.17	0.6	0	6	0.47	1.14	1.52	207.48	-0.02	112.4	7
14	174.81	1.25	-0.23	-2.2	174.81	0.72	6	1.65	124.52	0.87	0	0.87	200.87	0.72	0	6	0.51	1.11	1.62	182.02	-0.02	95.47	7
15	208.75	1.25	-0.27	-2.3	208.75	0.6	6	1.54	124.95	0.93	0	0.93	223.84	0.6	0	6	0.47	1.06	1.52	208.98	-0.02	111.72	7
16	183.9	2.51	0.69	-1.4	183.9	1.36	6	1.83	129.71	0.99	0	0.99	184.14	1.37	0	6	0.59	1.04	1.82	179.44	0.05	59.97	7

1091 N. Tustin Ave
Anaheim, CA

CPT Shear Wave Measurements

	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
CPT-1	10.14	9.14	10.42	20.12	517.80	
	20.21	19.21	19.85	32.44	611.90	765.57
	30.12	29.12	29.55	42.40	696.84	973.50
	40.12	39.12	39.44	53.20	741.32	915.93
	50.30	49.30	49.55	63.80	776.69	954.21
	60.24	59.24	59.45	72.36	821.60	1156.28
CPT-2	10.07	9.07	10.36	16.48	628.45	
	16.01	15.01	15.82	24.28	651.60	700.51

Shear Wave Source Offset = 5 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

CPT-3

10.10	9.10	10.38	18.08	574.29	
20.08	19.08	19.72	31.24	631.38	709.81
25.03	24.03	24.54	38.52	637.19	662.14
30.12	29.12	29.55	45.92	643.43	675.87
35.37	34.37	34.73	52.12	666.38	836.39
40.22	39.22	39.54	57.80	684.04	846.06
45.14	44.14	44.42	63.36	701.11	878.57
50.13	49.13	49.38	68.96	716.12	885.98
55.12	54.12	54.35	74.40	730.52	913.00
60.43	59.43	59.64	80.40	741.79	881.58
65.12	64.12	64.31	86.72	741.64	739.67
70.11	69.11	69.29	92.72	747.31	829.33
71.19	70.19	70.37	93.70	750.99	1099.21

CPT-4

10.10	9.10	10.38	14.00	741.65	
20.14	19.14	19.78	27.92	708.54	675.23
30.15	29.15	29.58	38.84	761.48	896.83
40.19	39.19	39.51	51.36	769.23	793.29
50.07	49.07	49.32	62.08	794.52	915.71
60.14	59.14	59.35	71.92	825.24	1018.99

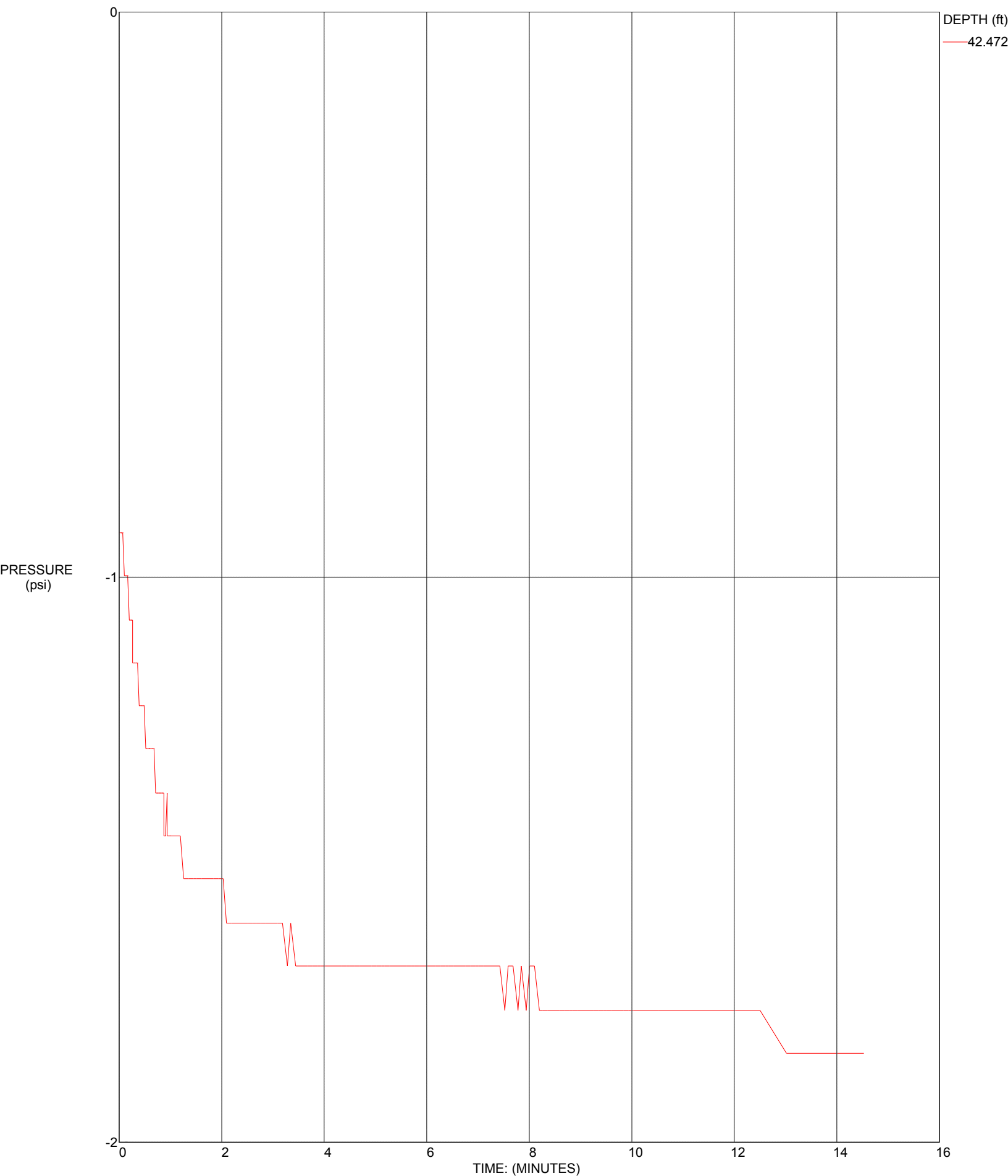
Shear Wave Source Offset = 5 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)



DISSIPATION

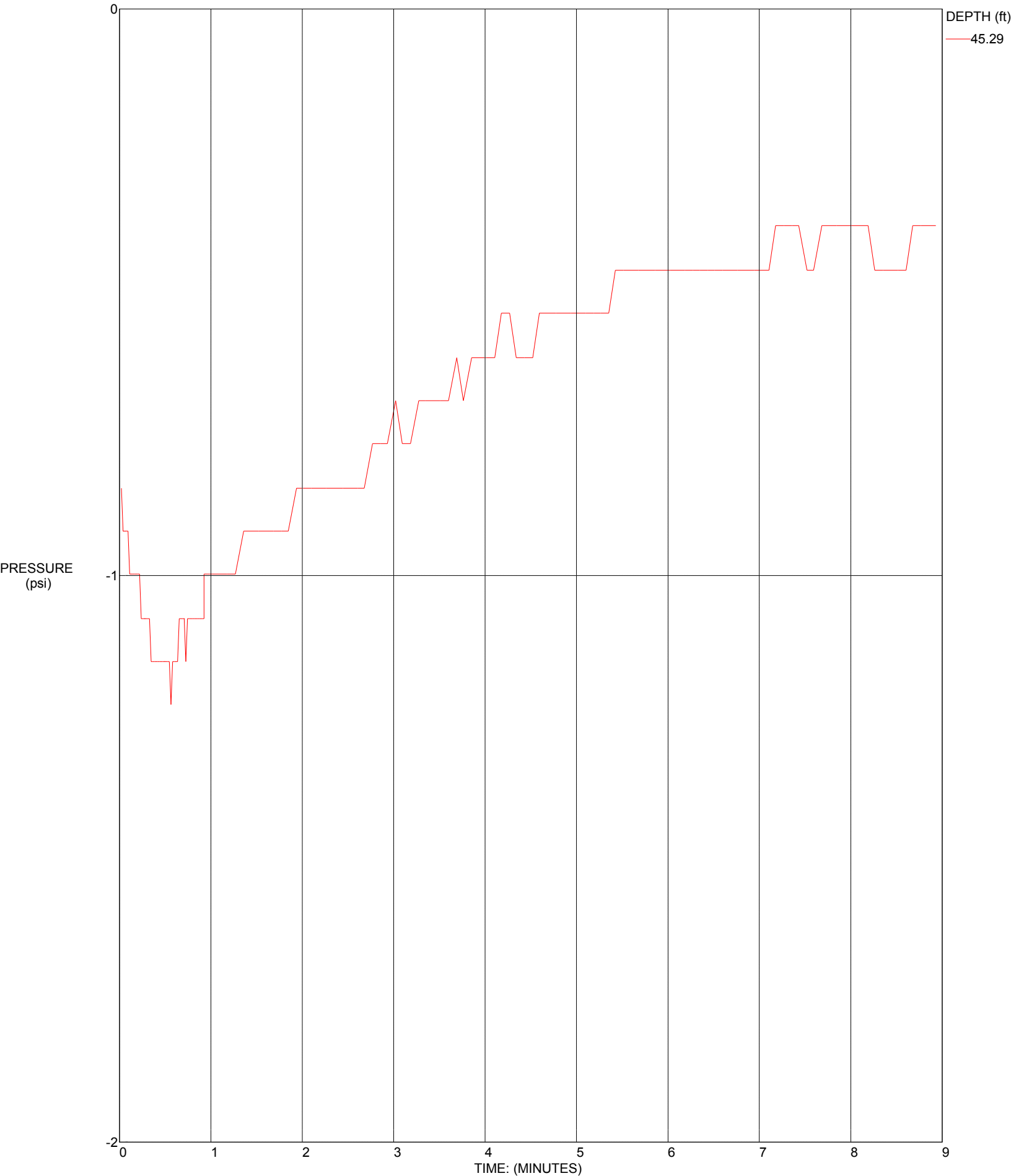
TEST ID: CPT-1
LOCATION: Anaheim
TEST DATE: Sat 14/Jan/2017
CLIENT: NOVA Services, Inc.





DISSIPATION

TEST ID: CPT-2A
LOCATION: Anaheim
TEST DATE: Sat 14/Jan/2017
CLIENT: NOVA Services, Inc.



Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log\left(\frac{q_t}{p_a}\right) + 1.236 \right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$$I_c < 3.27 \text{ and } I_c > 1.00 \text{ then } k = 10^{0.952-3.04 \cdot I_c}$$

$$I_c \leq 4.00 \text{ and } I_c > 3.27 \text{ then } k = 10^{-4.52-1.37 \cdot I_c}$$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{p_a} \right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{I(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, E_s (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c_cutoff}$)

:: Relative Density, D_r (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}} \quad \text{(applicable only to SBT}_n\text{: 5, 6, 7 and 8 or } I_c < I_{c_cutoff}\text{)}$$

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Peak drained friction angle, ϕ (°) ::

$$\phi = 17.60 + 11 \cdot \log(Q_{tn})$$

(applicable only to SBT_n: 5, 6, 7 and 8)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$

$$\alpha = 14 \text{ for } Q_{tn} > 14$$

$$\alpha = Q_{tn} \text{ for } Q_{tn} \leq 14$$

$$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$$

If $I_c \leq 2.20$

$$M_{CPT} = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Small strain shear Modulus, G_0 (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, V_s (m/s) ::

$$V_s = \left(\frac{G_0}{\rho} \right)^{0.50}$$

:: Undrained peak shear strength, S_u (kPa) ::

$$N_{kt} = 10.50 + 7 \cdot \log(F_r) \text{ or user defined}$$

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c_cutoff}$)

:: Remolded undrained shear strength, $S_{u(rem)}$ (kPa) ::

$$S_{u(rem)} = f_s \quad \text{(applicable only to SBT}_n\text{: 1, 2, 3, 4 and 9 or } I_c > I_{c_cutoff}\text{)}$$

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 + 7 \cdot \log(F_r))} \right]^{1.25} \text{ or user defined}$$

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c_cutoff}$)

:: In situ Stress Ratio, K_0 ::

$$K_0 = (1 - \sin \phi') \cdot OCR^{\sin \phi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_s}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c_cutoff}$)

:: Effective Stress Friction Angle, ϕ' (°) ::

$$\phi' = 29.5^\circ \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

References

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)

Attachment 2

Analyses of Liquefaction with Groundwater Level at 10 Feet

LIQUEFACTION ANALYSIS REPORT

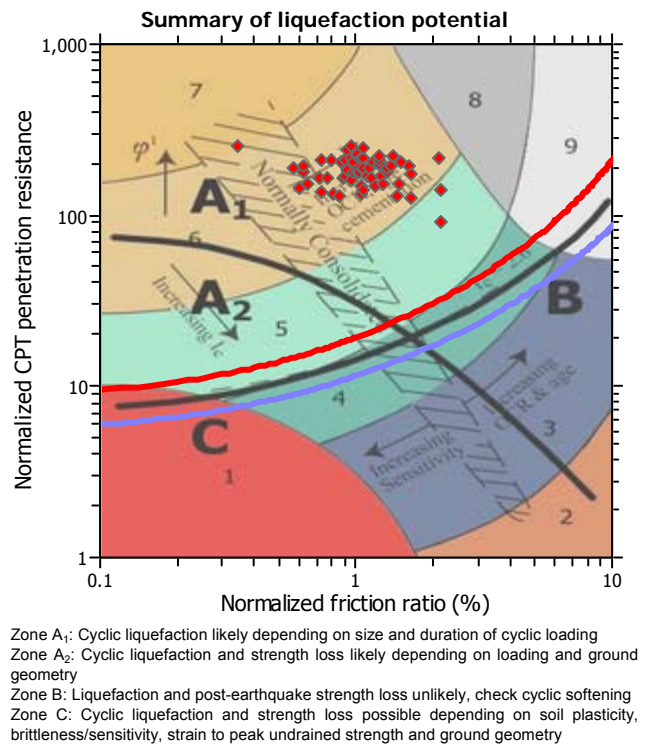
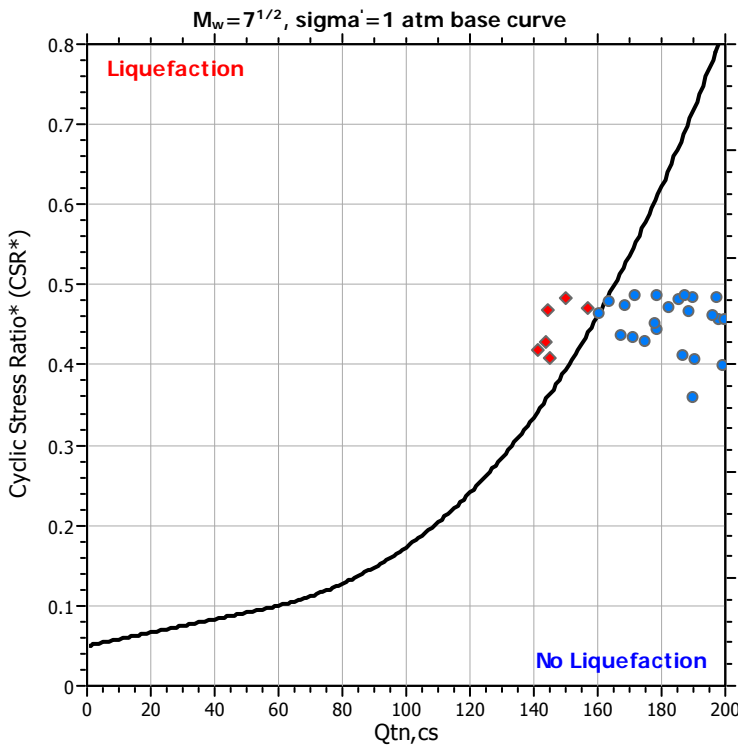
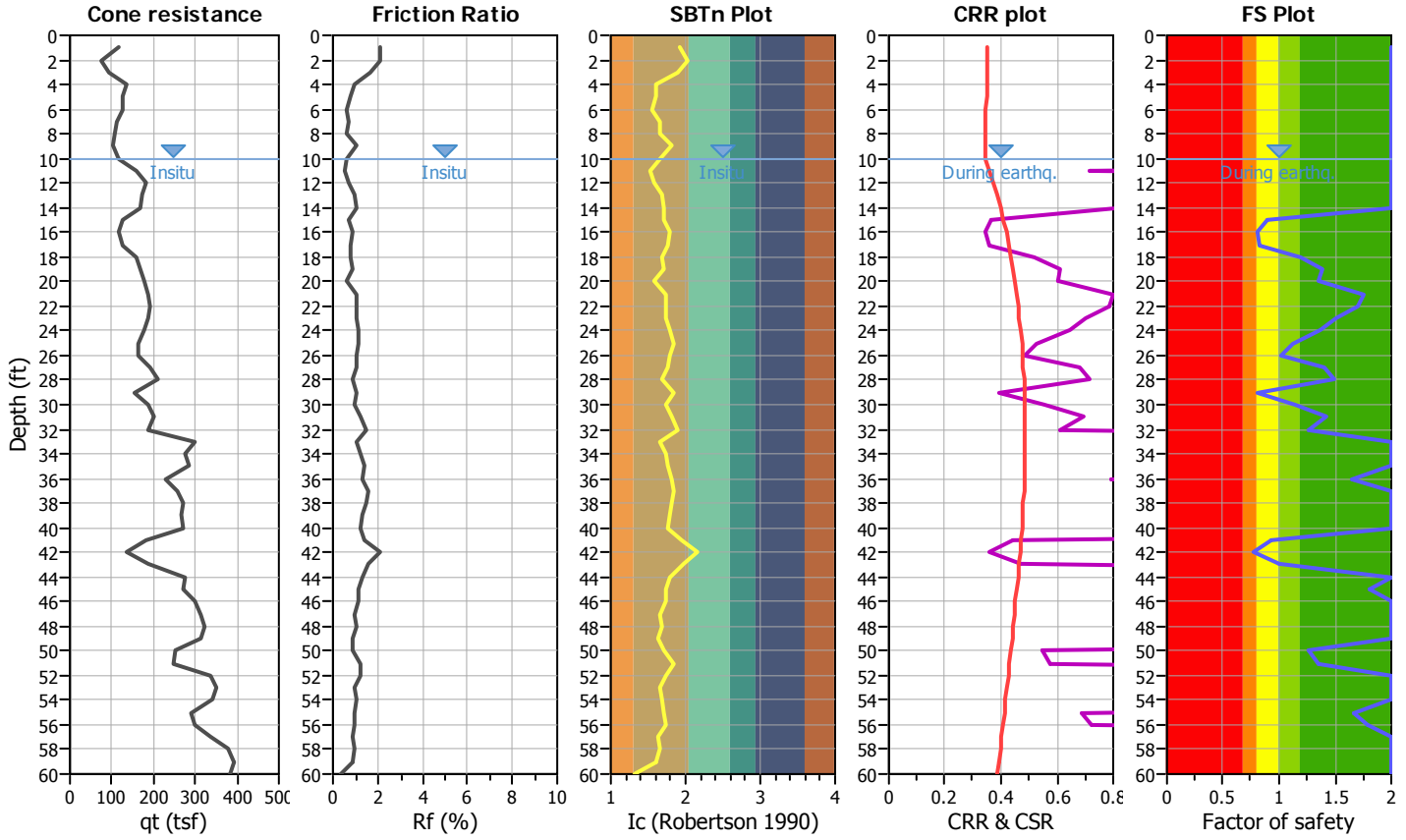
Project title : Pacific Center

Location : Anaheim, CA

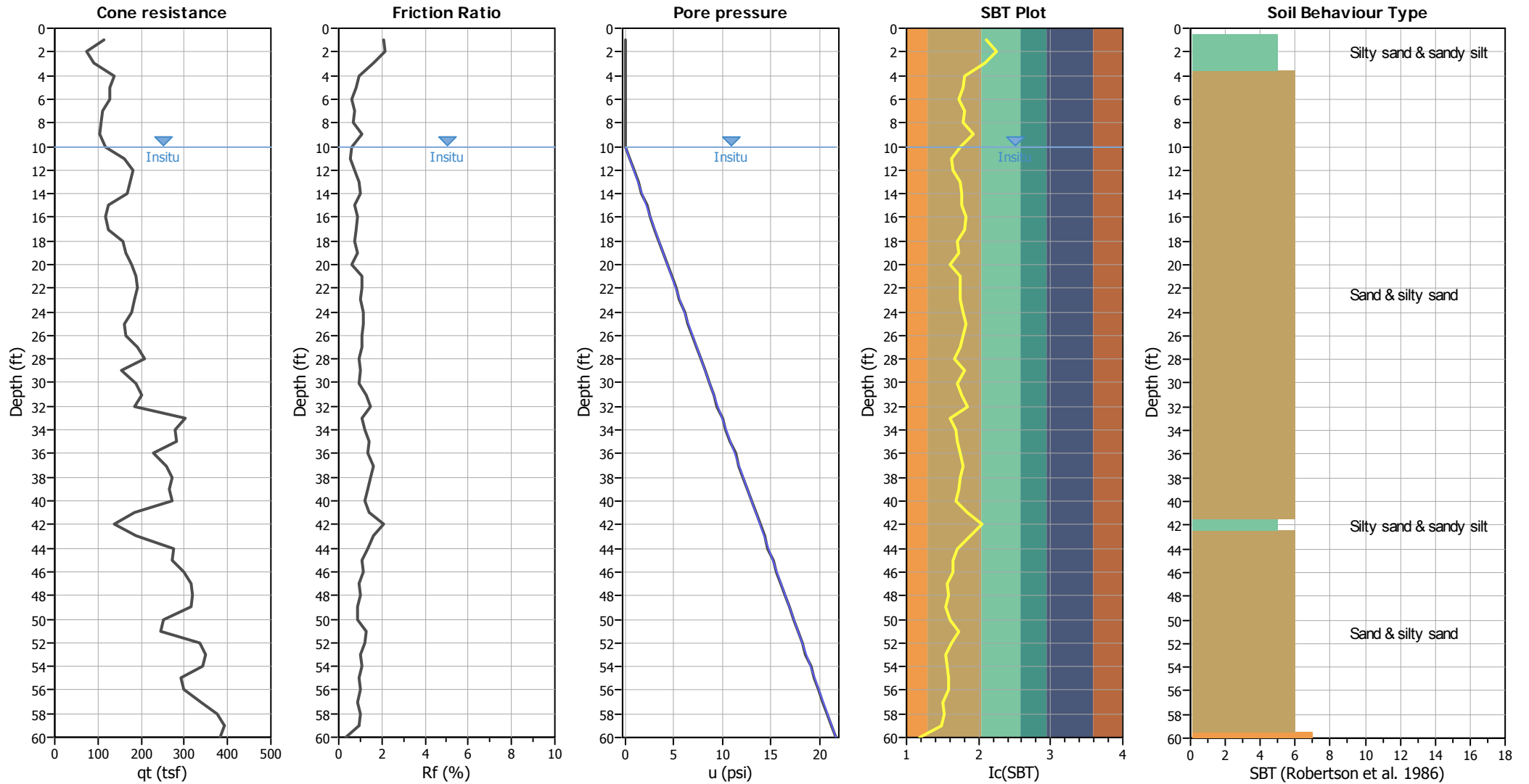
CPT file : CPT-Avg-10

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w :	7.20	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_0 applied:	No		



CPT basic interpretation plots



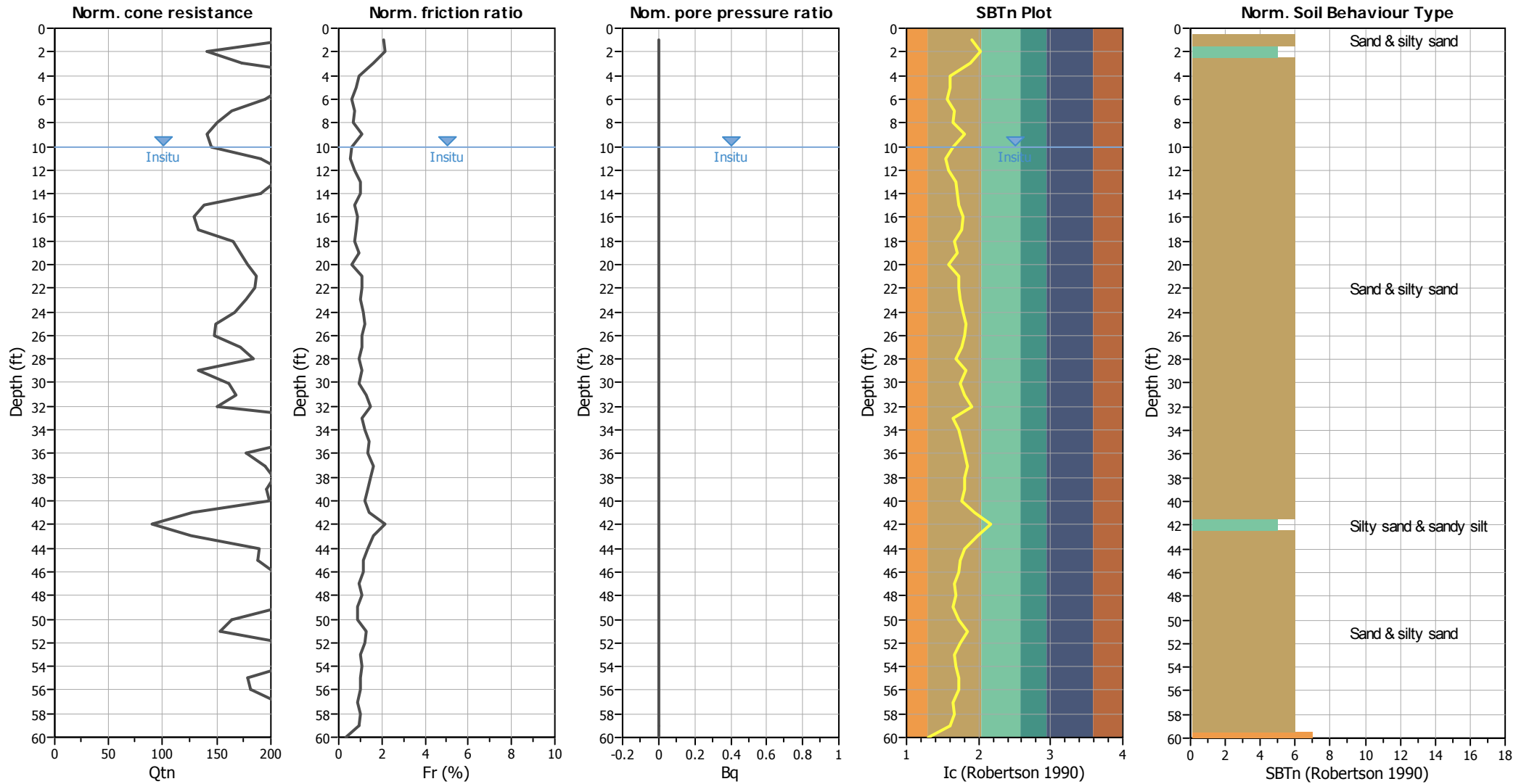
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w :	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots (normalized)



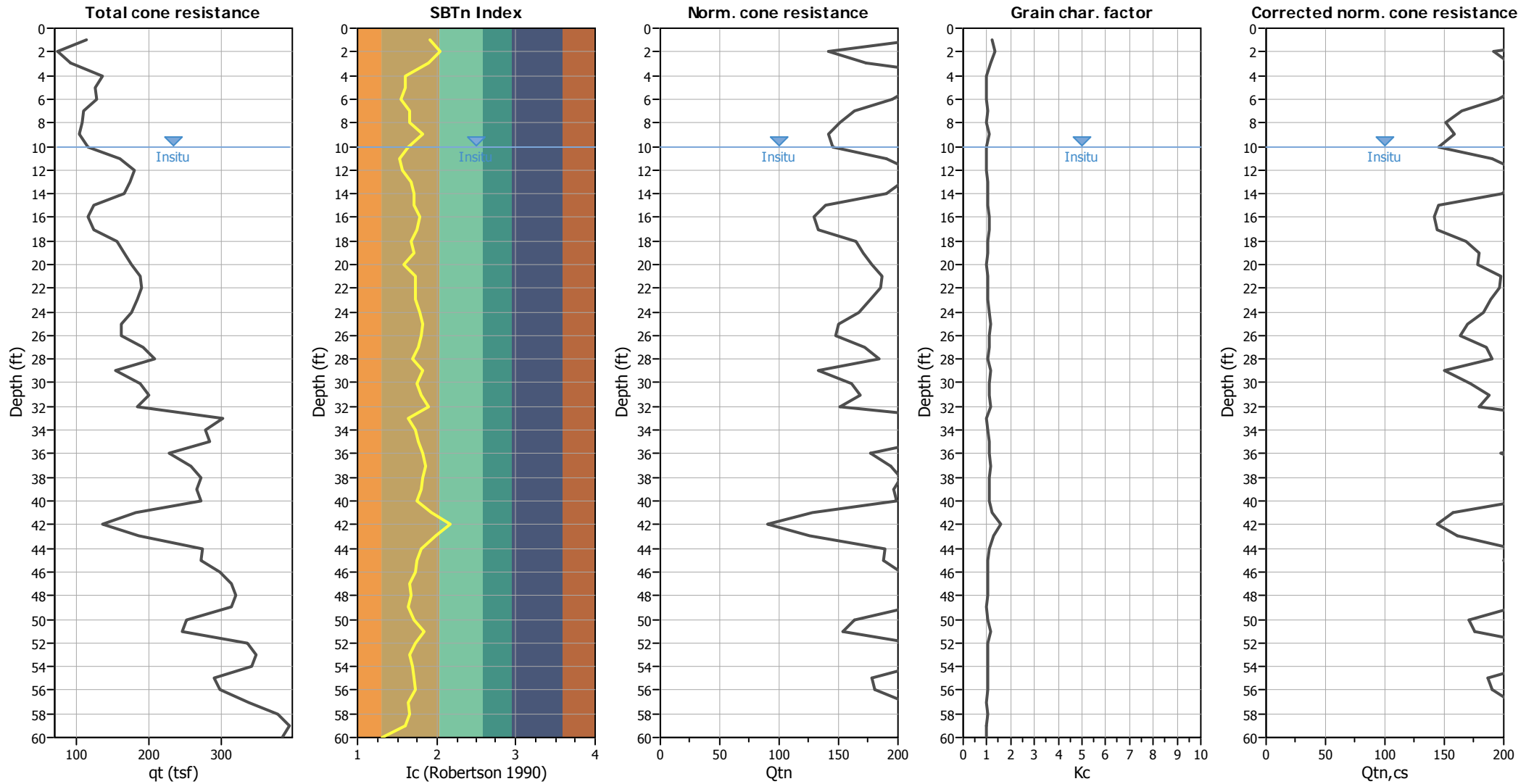
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w :	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

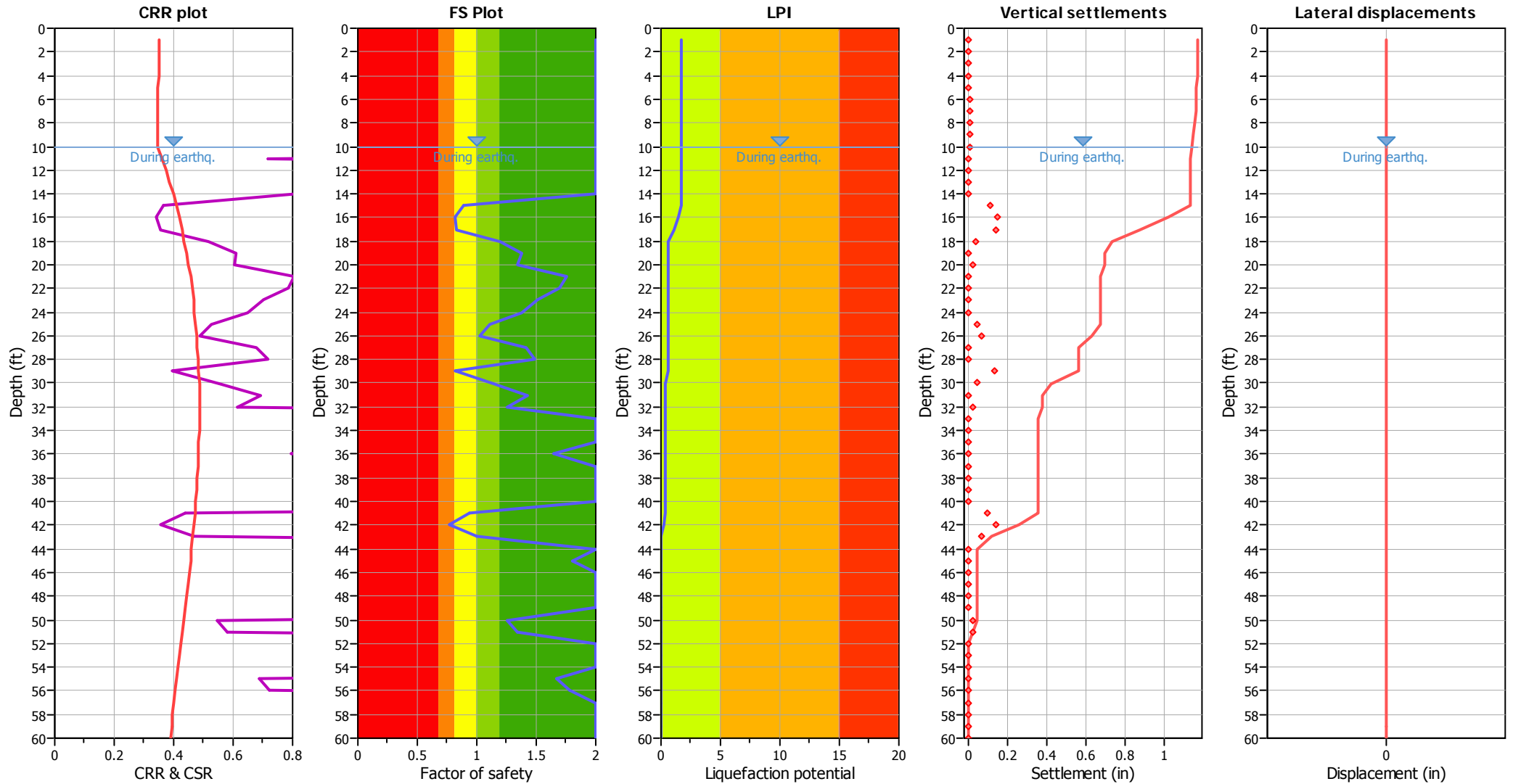
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{cs} applied:	No
Earthquake magnitude M_w :	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (earthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w :	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

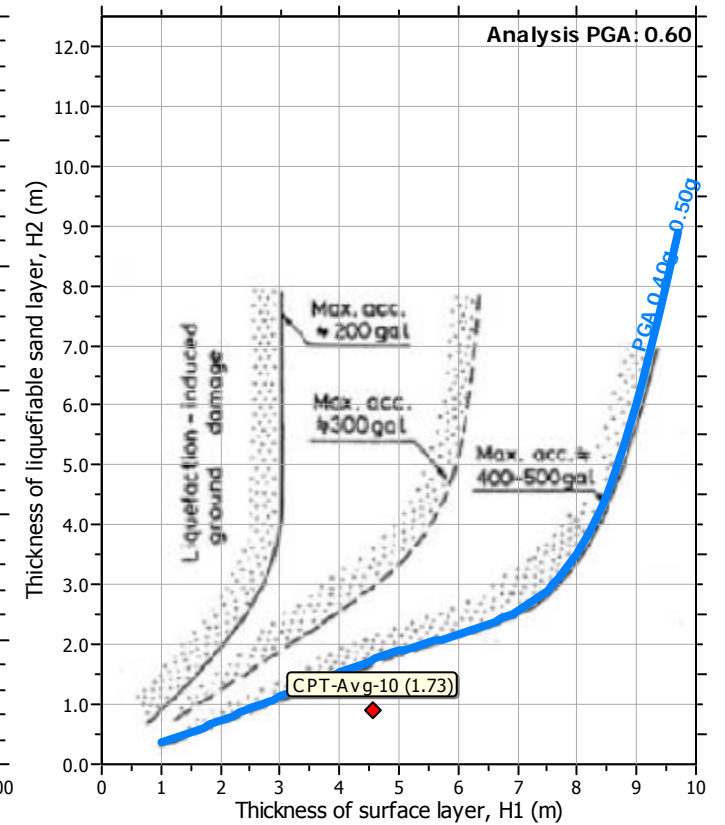
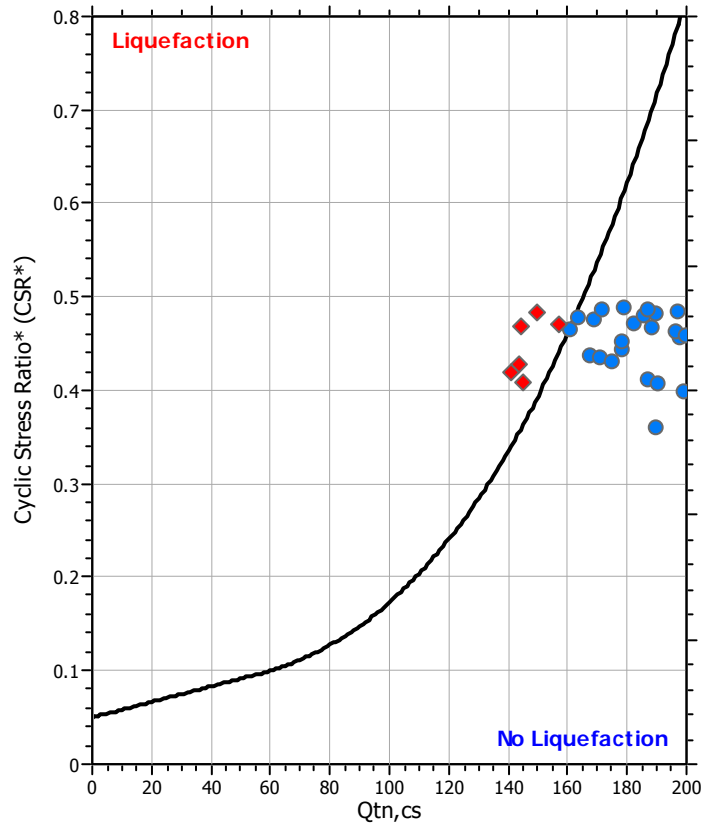
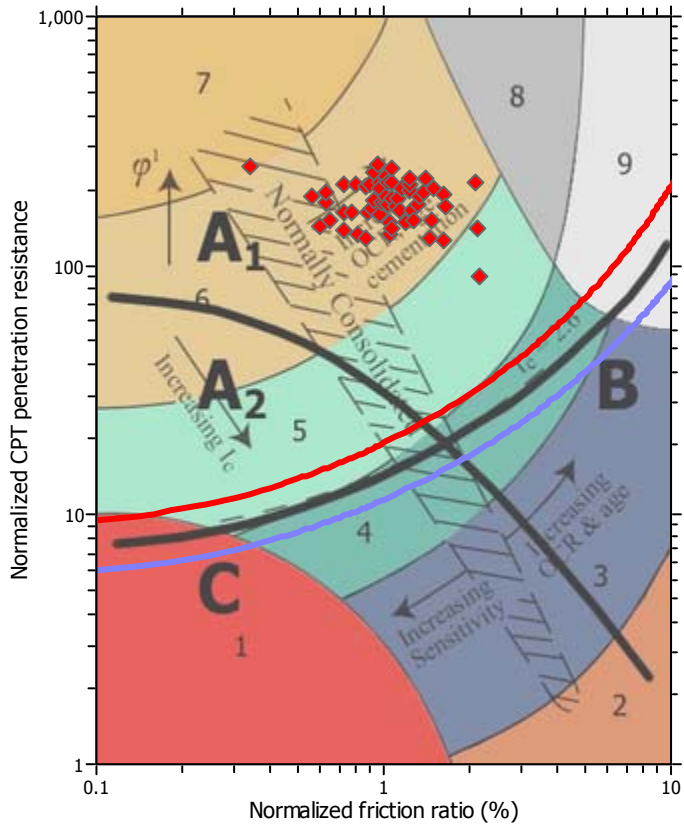
F.S. color scheme

- 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

Liquefaction analysis summary plots

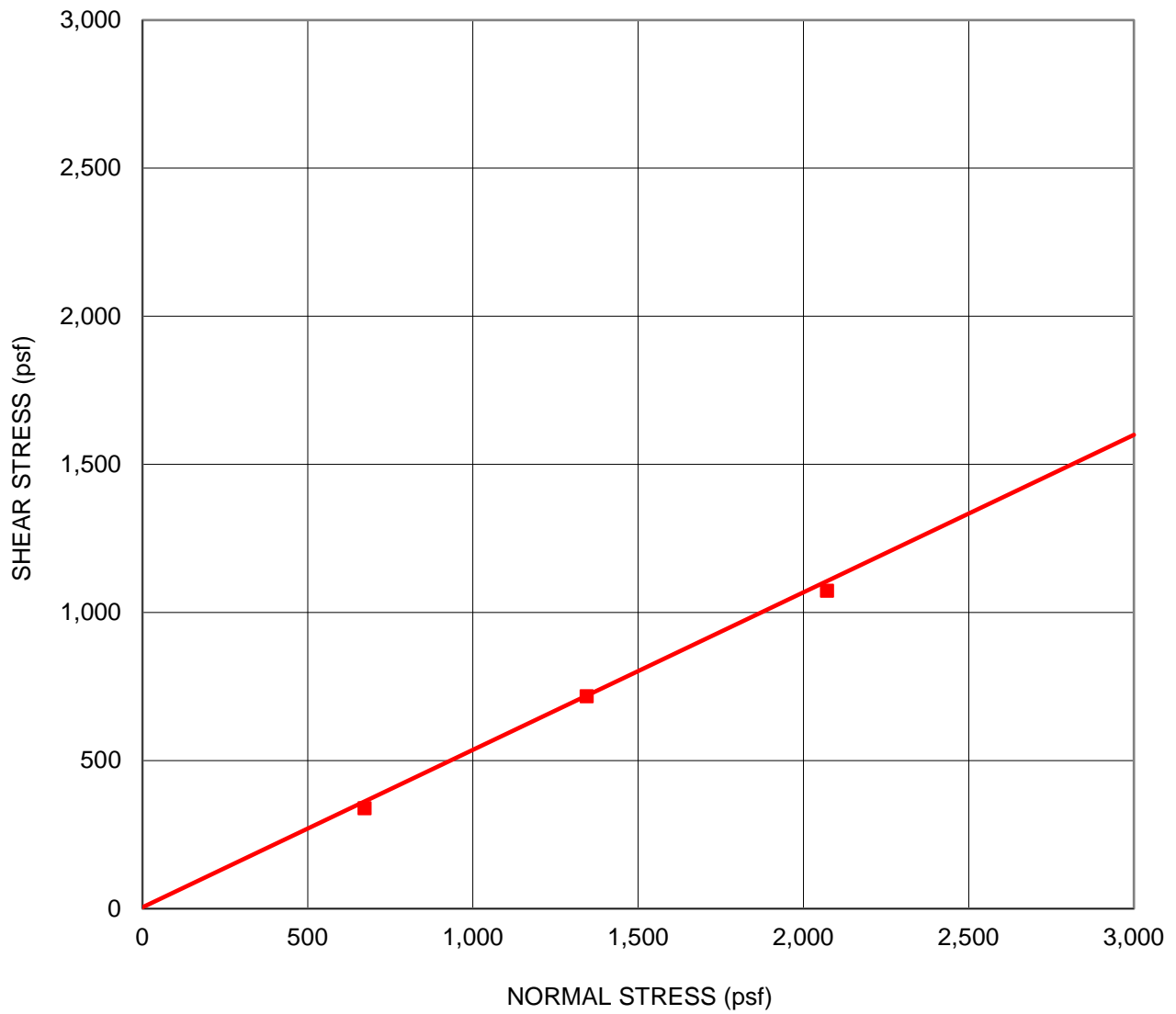


Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (earthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_v applied:	No
Earthquake magnitude M_w :	7.20	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

Attachment 3

Direct Shear Data



Friction Angle (Φ):	28 °
Apparent Cohesion (C):	5 psf

Sample Location:	B-1
Sample Depth (ft.):	3.0'
USCS Soil Type:	SP



DIRECT SHEAR TEST RESULTS

PACIFICENTER
 E. LA PALMA AVE & N. TUSTIN AVE
 ANAHEIM, CA

DATE
 FEB 2017

PROJECT NO.
 2016564