11.3 Geotechnical Investigation

February 26, 2026

9th Street Partners, LLC
880 Apollo Street, Suite 356
El Segundo, California 90245

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Attention: Mr. Rosendo Solis
Vice President of Development

Project No.: **24G159-3**

Subject: **Response to Third-Party Geotechnical Review**
Proposed Industrial Building
3800 Via Oro Avenue
Long Beach, California

Reference: Geotechnical Investigation, Proposed Industrial Building, 3800 Via Oro Avenue, Long Beach, California, prepared by Southern California Geotechnical, Inc. (SCG) for 9th Street Partners, LLC, SCG Project No. 24G159-1, dated October 21, 2024.

Mr. Solis:

In accordance with your request, we have prepared this letter to respond to comments issued by Michael Baker International (MBI), Third-Party Reviewer for the city of Long Beach, following their review of the above referenced report. The review comments prepared by MBI are included below, followed by our responses. This letter will also serve as an update to the project soils report. Updated seismic design parameters in accordance with the 2025 California Building Code, are included herein.

MBI 106: *The geotechnical report references the 2022 California Building Code (CBC). As of January 1, 2026 the 2025 CBC has become effective. The geotechnical consultant should provide updated recommendations in accordance with the 2025 CBC including updated seismic design criteria, liquefaction analysis, and any appropriate updates to the foundation design recommendations.*

SCG 106: This report includes updated seismic design parameters which have been prepared in accordance with the 2025 CBC. These values are included in a subsequent section of this report. We have also updated the liquefaction analysis, included herein, using the 2025 CBC derived seismic design criteria.

MBI 107: *The geotechnical report is more than 1 year old. The consultant should observe the current site conditions and confirm that their recommendations remain applicable.*

SCG 107: This letter will serve as an update to the referenced geotechnical investigation. The recommendations presented in the referenced report remain applicable to the currently proposed development. Some additional recommendations, related to the 2025 CBC, have been provided in a subsequent section of this report.

MBI 108: *Two conceptual site plans were submitted to the City. In Section 3.2 of the report, the consultant describes the redevelopment of the site that will include one large concrete tilt-up building, which corresponds to the Option 1 plan that was submitted to the City. The geotechnical consultant should review the Option 2 conceptual site plan that shows two separate buildings and a substation, and provide any updated conclusions and recommendations, as appropriate.*

SCG 108: Based on information provided by the client, Option 2 has been eliminated. The proposed development will include one tilt-up building. Attached to this report is the Updated Boring and CPT Location Plan, included as Plate 1 with this letter, indicating the proposed development.

MBI 109: *In this section, it mentions identifying liquifiable soils at all three CPT locations. This section of the report should be corrected to reflect all four CPT locations.*

SCG 109: The updated liquefaction analysis, which reflects a total for four (4) CPT soundings, is included in a subsequent section of this report. This letter will serve as an amendment to the referenced soils report.

MBI 110: *The consultant states that the differential dynamic settlement of about 1.5 inches "could be assumed to occur across a distance of 100 feet". The consultant should explain how they decided on a horizontal span of 100 feet for the dynamic differential settlement. That span length seems excessive considering the liquefaction analysis results presented in Appendix G. The vertical settlements plots indicate that liquefaction-induced dynamic settlement was limited to up to about a depth of 48 feet below the ground surface at all CPT locations.*

SCG 110: The differential dynamic settlement of 1½± inches is considered to be applicable to the proposed development. The greatest change in total dynamic settlement is 1.2± inches and occurs between CPT-1 and CPT-4, which are approximately 550± feet apart. Based on the change in total dynamic settlement and linear distance between CPT-1 and CPT-4, the associated differential dynamic settlement is substantially less than 0.001 inches per inch. Therefore the recommended 1½± inch differential dynamic settlement over a distance of 100 feet, with a maximum angular distortion of less than 0.002 inches per inch, is considered to be conservative, and applicable to the proposed development.

MBI 111: *The consultant states that where remedial grading for retaining walls and site walls is constrained by property boundaries that the foundations for those walls should be designed using a reduced allowable bearing pressure. However, the consultant did not provide a recommendation for the reduced allowable bearing pressure. The consultant should provide the reduced bearing capacity value.*

SCG 111: The applicable reduced bearing pressure for new foundations along property line is based on the limited lateral extent of remedial grading which will be feasible during grading. Based on our experience with similar projects in the vicinity of the site, the reduced soil bearing pressure at property line condition is expected to be

1,500 lbs/ft^2. This value should be verified during a detailed grading plan review to be performed by our office once the plans become available.

Updated Seismic Design Parameters

The 2025 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

The 2025 CBC Seismic Design Parameters have been generated using the ASCE Hazard Tool, a web-based software application available at the website www.ascehazardtool.org. This software application calculates seismic design parameters based on the project site class in accordance with several building code reference documents, including ASCE 7-22, upon which the 2025 CBC is based. The application utilizes a database of risk-targeted maximum considered earthquake (MCE_R) site accelerations at 0.01-degree intervals for each of the code documents. The table below was created using data obtained from the application. The output generated from this program is included with this report.

Chapter 20 of ASCE 7-22 indicates that the site soil shall be classified based on the average shear wave velocity parameter which is derived from the measured shear wave velocity profile from the ground surface to a depth of 100 feet. Furthermore, Section 20.3 of ASCE 7-22 indicates that where shear wave velocity data does not extend to depths of 50 feet, default site classes shall be used. Based on the lack of shear wave velocity measurements for this project, the Default Site Class is considered appropriate for this project at this time. It may be beneficial to perform a shear wave velocity to a depth of at least of 100 feet below the existing site grades to determine the actual Site Class for this project.

2025 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.91
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.76
Site Class	---	Default
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	2.05
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.56
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.37
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	1.04

Updated Ground Motion Parameters

For the purposes of the liquefaction analysis performed for this study, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2025 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-22. The parameter PGA_M is the maximum considered earthquake geometric mean

(MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-22. The web-based software application ASCE Hazard Tool (described in the previous section) was used to determine PGA_M, which is 0.82g. A portion of the program output is included with this report. An associated earthquake magnitude was obtained from the USGS Earthquake Hazard Toolbox, Disaggregation application available on the USGS website. The disaggregated mean magnitude is 6.99, based on the peak ground acceleration and soil classification D.

Updated Liquefaction

The liquefaction potential for the on-site soils was evaluated using data obtained at the four (4) CPT locations. This data was analyzed using the computer program CLiq V3.5.2.10, which was developed by Geologismiki, copyright 2006. The analysis method is based on Boulanger and Idriss 2014. The liquefaction potential of the site was analyzed utilizing a PGA_M of 0.82g for a magnitude 6.99 seismic event.

The results of the liquefaction analysis have identified potentially liquefiable soils at all four (4) of the CPT soundings performed at the site. Soils which are located above the historic groundwater table or possess factors of safety of at least 1.3 are considered non-liquefiable. Several clayey strata located below the ground water table are also considered to be non-liquefiable due to their cohesive characteristics and the results of the Atterberg limits testing with respect to the criteria of Bray and Sancio (2006). Settlement analyses were conducted for each of the potentially liquefiable strata. The results of the dynamic settlement analyses are included the CLiq program output in Appendix G and are presented below:

- CPT-1: 1.74± inches
- CPT-2: 2.83± inches
- CPT-3: 2.86± inches
- CPT-4: 2.05± inches

Based on these total settlements, differential settlements of up to 1½± inches should be expected to occur during a liquefaction inducing seismic event. The estimated differential settlement could be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of less than 0.002 inches per inch.

Geotechnical Report Update

Based on our review of the latest architectural site plan, the proposed development is consistent with that which was proposed at the time of the referenced geotechnical investigation. Therefore, this letter may serve as an update to the original geotechnical report. Provided that the updated recommendations contained within this letter are implemented, the previous geotechnical report is considered valid for the currently proposed development. With the exception of the 2025 seismic design parameters noted above, no other changes to the recommendations provided in the referenced reports are considered necessary. All references to the 2022 CBC are hereby updated to the 2025 CBC.

Closure

We sincerely appreciate the opportunity to be of continued service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

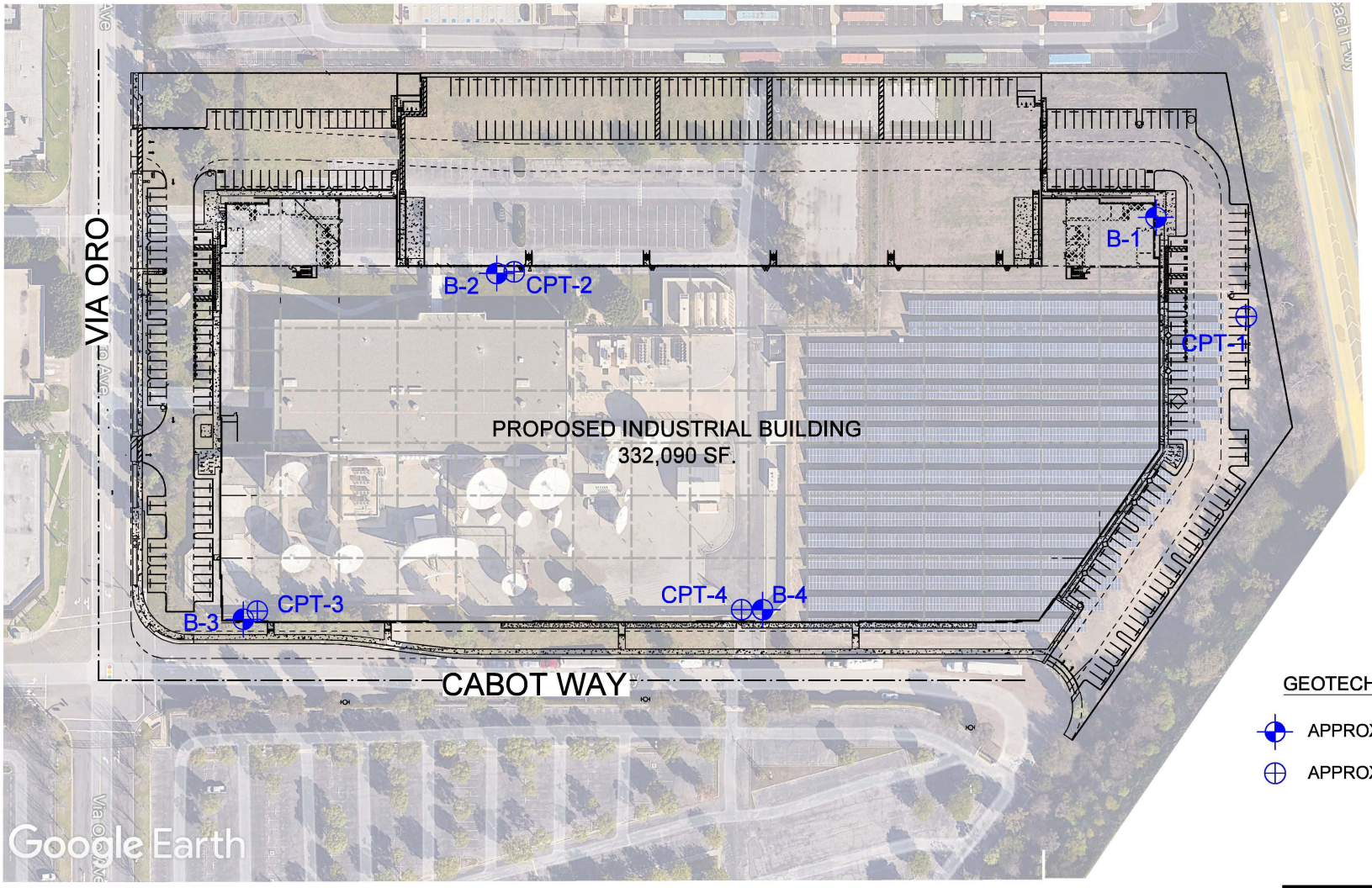
SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Pablo Montes Jr.
Project Engineer

Daniel W. Nielsen, GE 3166
Principal Engineer



Enclosures: Plate 1: Updated Boring and CPT Location Plan
Seismic Design Parameters
CPT Liquefaction Analysis



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

APPROXIMATE CPT LOCATION

NORTH

NOTE: BASE SITE PLAN PREPARED BY HERDMAN ARCHITECTURE + DESIGN
AIR PHOTO OBTAINED FROM GOOGLE EARTH.

UPDATED BORING AND CPT LOCATION PLAN

PROPOSED INDUSTRIAL BUILDING

LONG BEACH, CALIFORNIA

SCALE: 1" = 100'	
DRAWN: PM CHKD: DWN	
SCG PROJECT 24G159-3	
PLATE 1	

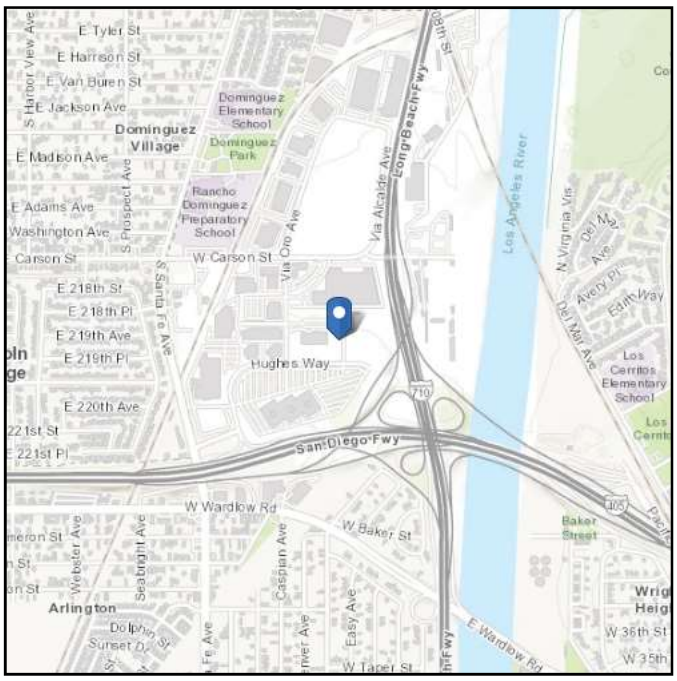
SOUTHERN CALIFORNIA GEOTECHNICAL

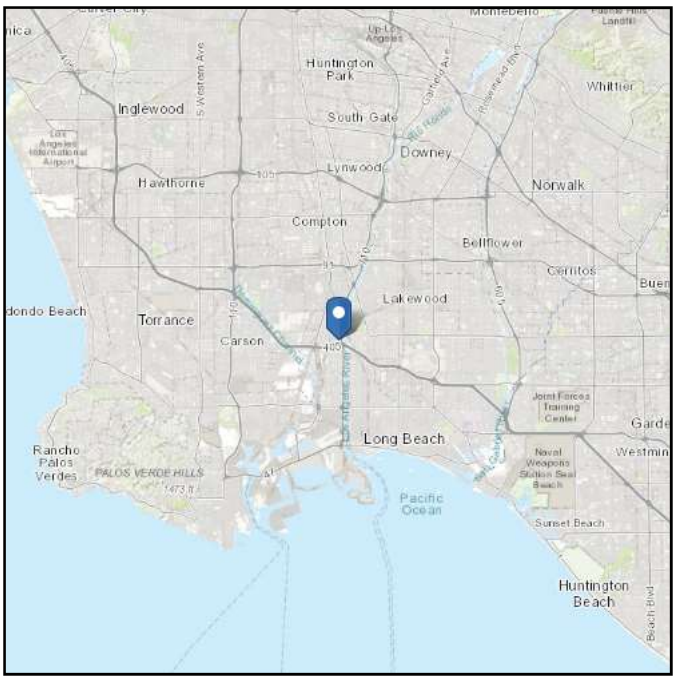
ASCE Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-22
Risk Category: II
Soil Class: Default

Latitude: 33.829311
Longitude: -118.210698
Elevation: 32.91231545936204 ft (NAVD 88)





Seismic

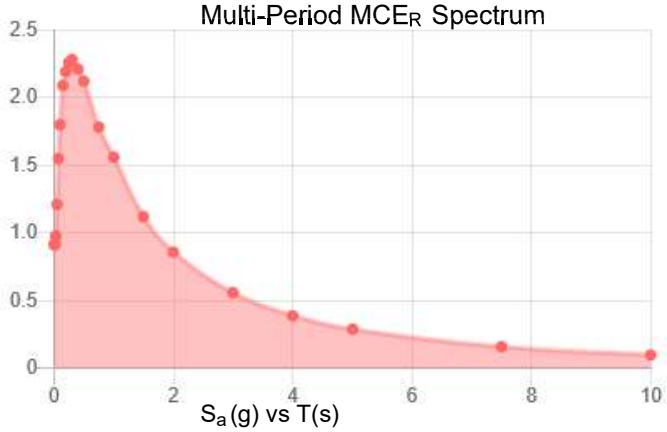
Site Soil Class: Default

Results:

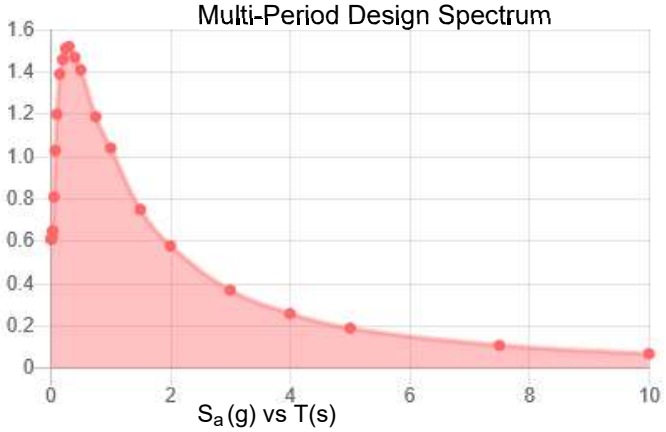
PGA_M :	0.82	T_L :	8
S_{MS} :	2.05	S_S :	1.91
S_{M1} :	1.56	S_1 :	0.76
S_{DS} :	1.37	V_{S30} :	260
S_{D1} :	1.04		

Seismic Design Category: E

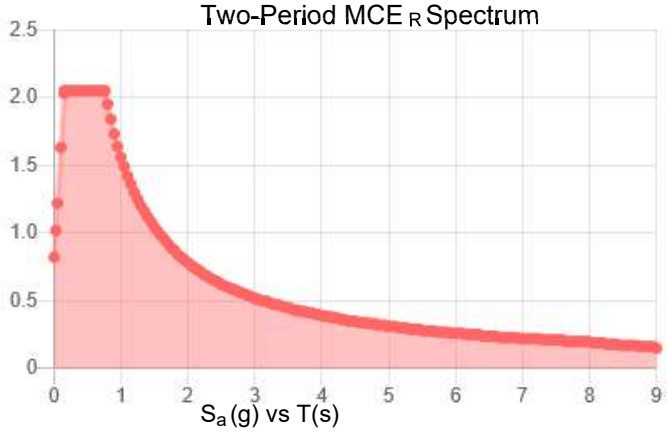
Multi-Period MCE_R Spectrum



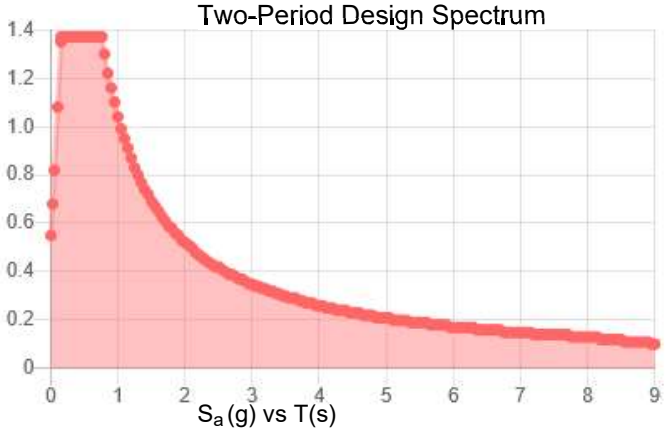
Multi-Period Design Spectrum



Two-Period MCE_R Spectrum



Two-Period Design Spectrum



MCE_R Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Data Accessed: Tue Feb 24 2026

Date Source:
USGS Seismic Design Maps based on ASCE/SEI 7-22 and ASCE/SEI 7-22 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-22 Ch. 21 are available from USGS.

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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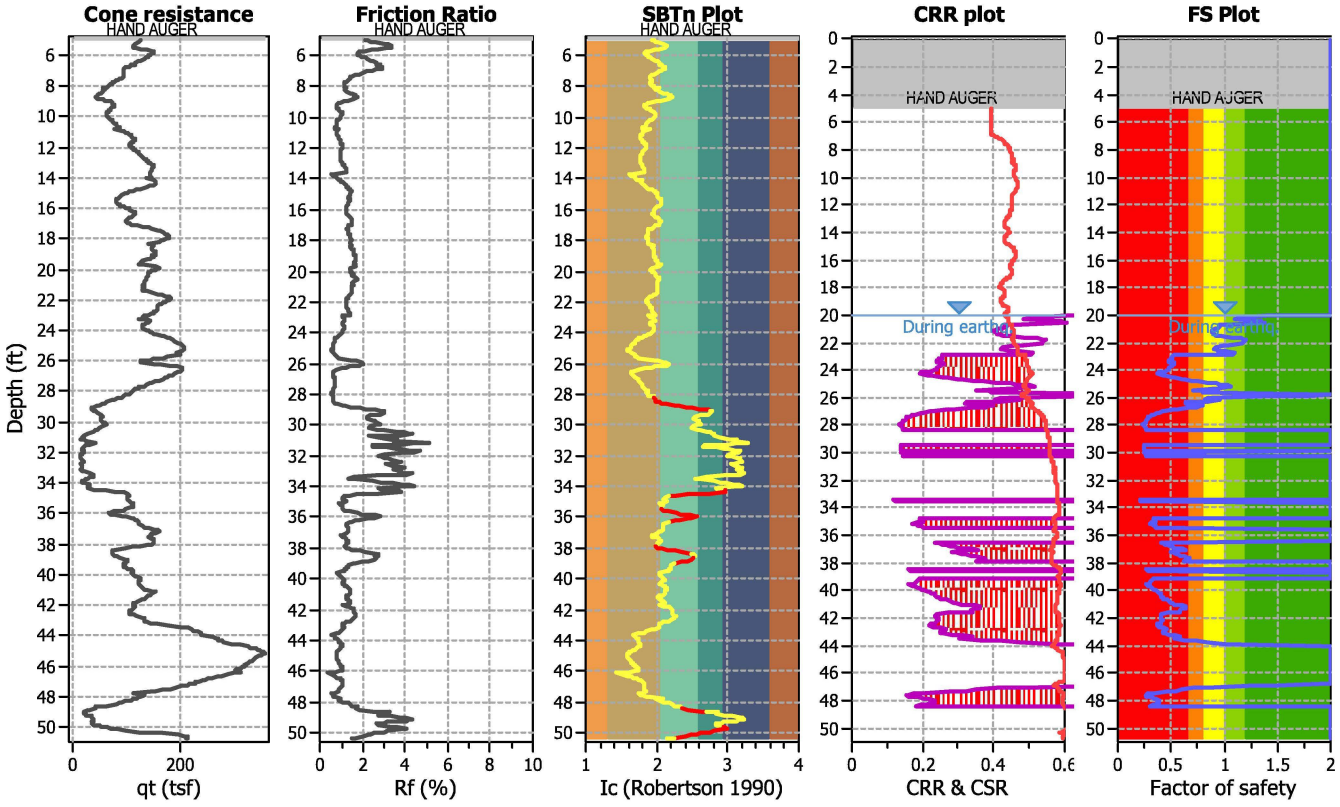
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

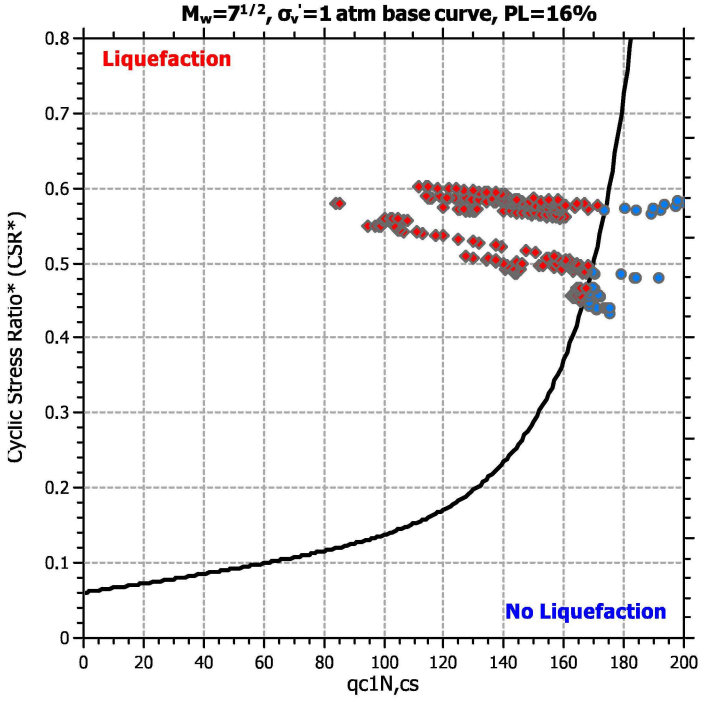
LIQUEFACTION ANALYSIS REPORT

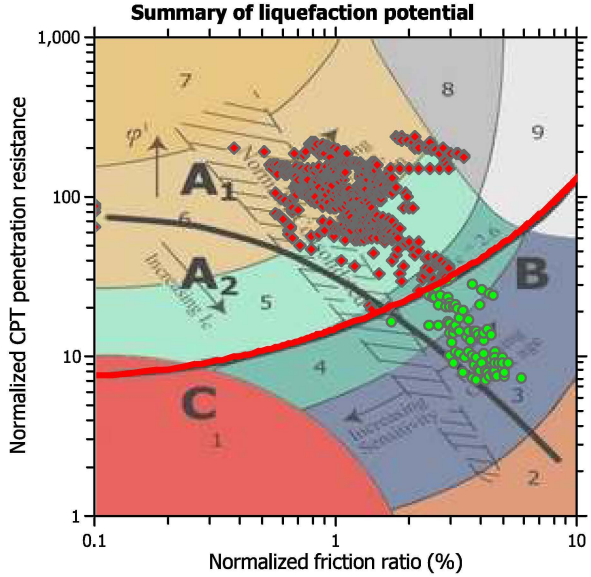
Project title : Southern California Geotechnical / Proposed Industrial Building **Location : 3800 Via Oro Ave, Long Beach, CA**
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	27.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.99	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

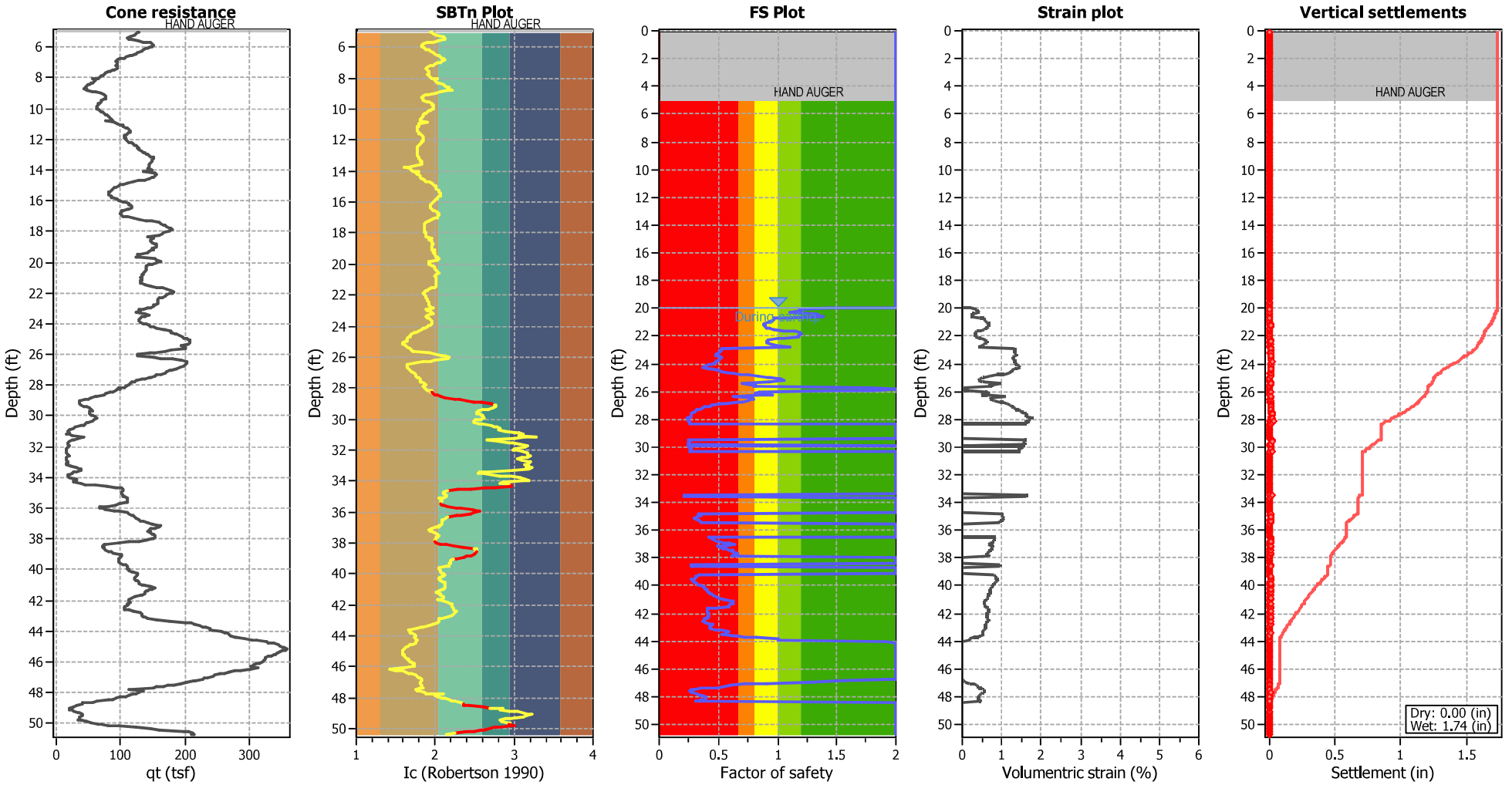






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
20.02	175.84	1.43	0.20	0.66	0.00	20.11	170.89	1.19	0.35	0.66	0.00
20.15	170.58	1.17	0.36	0.66	0.00	20.22	171.00	1.19	0.35	0.66	0.00
20.30	168.88	1.10	0.42	0.66	0.00	20.41	173.47	1.30	0.28	0.65	0.00
20.44	173.97	1.32	0.26	0.65	0.00	20.48	174.58	1.35	0.24	0.65	0.00
20.57	174.62	1.35	0.24	0.65	0.00	20.61	175.26	1.38	0.23	0.65	0.00
20.71	168.51	1.08	0.44	0.65	0.01	20.76	165.22	0.96	0.57	0.65	0.00
20.82	165.27	0.96	0.57	0.65	0.00	20.89	165.08	0.96	0.58	0.65	0.00
20.94	164.60	0.94	0.60	0.65	0.00	21.00	163.78	0.91	0.63	0.64	0.00
21.09	162.92	0.89	0.67	0.64	0.01	21.13	162.60	0.88	0.69	0.64	0.00
21.22	162.98	0.89	0.67	0.64	0.01	21.27	163.19	0.89	0.66	0.64	0.00
21.36	163.35	0.90	0.65	0.64	0.01	21.41	163.79	0.91	0.64	0.64	0.00
21.47	164.55	0.93	0.60	0.64	0.00	21.54	165.81	0.96	0.55	0.63	0.00
21.61	168.60	1.06	0.45	0.63	0.00	21.68	171.08	1.15	0.36	0.63	0.00
21.73	171.88	1.18	0.34	0.63	0.00	21.82	172.40	1.21	0.32	0.63	0.00
21.91	171.82	1.18	0.34	0.63	0.00	21.93	171.86	1.18	0.34	0.63	0.00
22.00	171.83	1.18	0.34	0.63	0.00	22.06	171.87	1.18	0.34	0.63	0.00
22.14	168.78	1.05	0.45	0.62	0.00	22.19	167.44	1.00	0.49	0.62	0.00
22.28	164.78	0.92	0.60	0.62	0.01	22.33	164.78	0.92	0.60	0.62	0.00
22.42	164.38	0.90	0.62	0.62	0.01	22.47	164.45	0.91	0.62	0.62	0.00
22.52	164.73	0.91	0.60	0.62	0.00	22.60	165.74	0.94	0.56	0.62	0.01
22.65	167.40	0.99	0.50	0.62	0.00	22.78	170.49	1.10	0.39	0.61	0.01
22.80	170.31	1.10	0.40	0.61	0.00	22.84	169.96	1.08	0.41	0.61	0.00
22.93	144.45	0.53	1.32	0.61	0.01	22.98	143.75	0.52	1.33	0.61	0.01
23.04	144.15	0.52	1.32	0.61	0.01	23.11	143.74	0.52	1.32	0.61	0.01
23.20	142.64	0.50	1.33	0.61	0.02	23.24	143.14	0.51	1.33	0.61	0.01
23.30	140.74	0.48	1.35	0.61	0.01	23.41	140.76	0.48	1.35	0.60	0.02
23.45	142.76	0.50	1.32	0.60	0.01	23.50	144.66	0.52	1.30	0.60	0.01
23.57	145.31	0.53	1.29	0.60	0.01	23.64	144.43	0.52	1.30	0.60	0.01
23.71	143.22	0.51	1.31	0.60	0.01	23.82	141.29	0.48	1.32	0.60	0.02
23.91	141.21	0.48	1.32	0.59	0.01	23.97	139.87	0.47	1.33	0.59	0.01
24.05	137.04	0.44	1.36	0.59	0.01	24.10	133.96	0.42	1.39	0.59	0.01
24.18	129.63	0.39	1.44	0.59	0.01	24.23	127.54	0.37	1.46	0.59	0.01
24.28	127.35	0.37	1.47	0.59	0.01	24.37	134.62	0.42	1.37	0.59	0.01
24.42	137.39	0.44	1.34	0.59	0.01	24.51	144.16	0.51	1.27	0.58	0.01
24.55	145.80	0.53	1.25	0.58	0.01	24.65	151.42	0.60	1.20	0.58	0.01
24.67	153.03	0.63	1.18	0.58	0.00	24.74	156.43	0.68	1.06	0.58	0.01
24.82	159.48	0.75	0.87	0.58	0.01	24.92	163.10	0.83	0.68	0.58	0.01
24.97	166.23	0.92	0.55	0.58	0.00	25.02	167.36	0.95	0.51	0.58	0.00
25.09	169.61	1.03	0.43	0.57	0.00	25.16	170.51	1.06	0.40	0.57	0.00
25.20	169.96	1.04	0.42	0.57	0.00	25.30	167.09	0.94	0.52	0.57	0.01
25.34	166.46	0.92	0.54	0.57	0.00	25.43	157.61	0.70	0.98	0.57	0.01
25.48	161.06	0.77	0.78	0.57	0.00	25.56	162.58	0.81	0.71	0.57	0.01
25.62	163.02	0.82	0.69	0.57	0.00	25.66	162.14	0.80	0.73	0.57	0.00
25.75	183.93	1.79	0.05	0.56	0.00	25.79	191.71	2.00	0.00	0.56	0.00
25.89	184.20	1.81	0.05	0.56	0.00	25.94	179.21	1.46	0.16	0.56	0.00
26.03	165.74	0.89	0.57	0.56	0.01	26.08	164.36	0.85	0.63	0.56	0.00
26.12	162.75	0.81	0.70	0.56	0.00	26.22	168.10	0.96	0.48	0.56	0.01
26.26	164.35	0.84	0.63	0.55	0.00	26.31	155.10	0.64	1.11	0.55	0.01

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
26.40	154.38	0.63	1.11	0.55	0.01	26.49	162.26	0.79	0.72	0.55	0.01
26.54	162.64	0.80	0.70	0.55	0.00	26.58	161.60	0.77	0.75	0.55	0.00
26.67	160.37	0.74	0.81	0.55	0.01	26.72	158.17	0.70	0.94	0.55	0.01
26.77	156.55	0.67	1.05	0.55	0.01	26.86	150.38	0.57	1.13	0.54	0.01
26.90	147.47	0.53	1.15	0.54	0.01	27.00	139.49	0.44	1.22	0.54	0.01
27.04	137.09	0.42	1.24	0.54	0.01	27.14	131.89	0.39	1.29	0.54	0.01
27.18	130.06	0.37	1.31	0.54	0.01	27.27	124.52	0.34	1.37	0.54	0.02
27.32	119.79	0.32	1.43	0.54	0.01	27.36	117.34	0.31	1.46	0.54	0.01
27.46	112.57	0.29	1.52	0.53	0.02	27.51	110.80	0.28	1.54	0.53	0.01
27.60	106.80	0.27	1.60	0.53	0.02	27.64	105.24	0.27	1.62	0.53	0.01
27.73	105.06	0.27	1.62	0.53	0.02	27.78	104.78	0.26	1.62	0.53	0.01
27.83	105.22	0.27	1.61	0.53	0.01	27.92	94.47	0.24	1.79	0.53	0.02
27.97	97.06	0.24	1.74	0.53	0.01	28.11	98.71	0.25	1.70	0.52	0.03
28.14	99.28	0.25	1.69	0.52	0.01	28.15	104.28	0.26	1.61	0.52	0.00
28.25	103.22	0.26	1.62	0.52	0.02	28.30	103.50	0.26	1.61	0.52	0.01
28.38	102.68	2.00	0.00	0.52	0.00	28.43	102.30	2.00	0.00	0.52	0.00
28.49	102.37	2.00	0.00	0.52	0.00	28.57	102.35	2.00	0.00	0.52	0.00
28.62	103.12	2.00	0.00	0.51	0.00	28.71	101.99	2.00	0.00	0.51	0.00
28.75	101.25	2.00	0.00	0.51	0.00	28.81	99.13	2.00	0.00	0.51	0.00
28.89	96.92	2.00	0.00	0.51	0.00	28.94	92.77	2.00	0.00	0.51	0.00
29.01	26.90	2.00	0.00	0.51	0.00	29.08	26.69	2.00	0.00	0.51	0.00
29.14	26.12	2.00	0.00	0.51	0.00	29.21	26.45	2.00	0.00	0.50	0.00
29.30	27.83	2.00	0.00	0.50	0.00	29.36	30.69	2.00	0.00	0.50	0.00
29.41	34.07	2.00	0.00	0.50	0.00	29.50	99.54	0.25	1.61	0.50	0.02
29.54	101.26	0.25	1.58	0.50	0.01	29.63	102.34	0.25	1.56	0.50	0.02
29.68	100.05	0.25	1.60	0.50	0.01	29.77	100.19	0.25	1.59	0.50	0.02
29.79	100.26	0.25	1.59	0.50	0.00	29.91	38.22	2.00	0.00	0.49	0.00
29.96	102.07	0.25	1.55	0.49	0.01	30.00	102.99	0.25	1.53	0.49	0.01
30.08	106.71	0.26	1.47	0.49	0.01	30.14	107.90	0.27	1.45	0.49	0.01
30.19	108.04	0.27	1.45	0.49	0.01	30.28	104.97	0.26	1.49	0.49	0.02
30.32	40.73	2.00	0.00	0.49	0.00	30.42	36.67	2.00	0.00	0.48	0.00
30.47	34.68	2.00	0.00	0.48	0.00	30.55	33.10	2.00	0.00	0.48	0.00
30.58	31.82	2.00	0.00	0.48	0.00	30.65	28.07	2.00	0.00	0.48	0.00
30.73	20.70	2.00	0.00	0.48	0.00	30.79	18.87	2.00	0.00	0.48	0.00
30.88	14.75	2.00	0.00	0.48	0.00	30.92	13.80	2.00	0.00	0.48	0.00
30.97	12.53	2.00	0.00	0.48	0.00	31.06	13.72	2.00	0.00	0.47	0.00
31.14	10.33	2.00	0.00	0.47	0.00	31.20	13.63	2.00	0.00	0.47	0.00
31.24	17.19	2.00	0.00	0.47	0.00	31.33	30.29	2.00	0.00	0.47	0.00
31.39	30.90	2.00	0.00	0.47	0.00	31.47	23.73	2.00	0.00	0.47	0.00
31.53	19.57	2.00	0.00	0.47	0.00	31.57	17.68	2.00	0.00	0.47	0.00
31.66	14.71	2.00	0.00	0.46	0.00	31.71	12.78	2.00	0.00	0.46	0.00
31.76	12.77	2.00	0.00	0.46	0.00	31.84	12.76	2.00	0.00	0.46	0.00
31.93	14.41	2.00	0.00	0.46	0.00	31.98	14.20	2.00	0.00	0.46	0.00
32.03	12.81	2.00	0.00	0.46	0.00	32.12	11.22	2.00	0.00	0.46	0.00
32.16	10.37	2.00	0.00	0.45	0.00	32.26	10.36	2.00	0.00	0.45	0.00
32.35	10.35	2.00	0.00	0.45	0.00	32.44	11.26	2.00	0.00	0.45	0.00
32.49	12.50	2.00	0.00	0.45	0.00	32.58	13.61	2.00	0.00	0.45	0.00
32.63	12.75	2.00	0.00	0.45	0.00	32.72	11.23	2.00	0.00	0.45	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
32.76	10.77	2.00	0.00	0.44	0.00	32.81	10.83	2.00	0.00	0.44	0.00
32.90	11.54	2.00	0.00	0.44	0.00	32.95	11.80	2.00	0.00	0.44	0.00
33.05	10.94	2.00	0.00	0.44	0.00	33.09	11.66	2.00	0.00	0.44	0.00
33.18	10.80	2.00	0.00	0.44	0.00	33.21	11.51	2.00	0.00	0.44	0.00
33.27	14.92	2.00	0.00	0.44	0.00	33.37	22.11	2.00	0.00	0.43	0.00
33.46	83.78	0.21	1.66	0.43	0.02	33.47	85.04	0.21	1.63	0.43	0.00
33.55	85.35	0.21	1.62	0.43	0.02	33.66	21.43	2.00	0.00	0.43	0.00
33.69	19.29	2.00	0.00	0.43	0.00	33.73	17.44	2.00	0.00	0.43	0.00
33.81	14.59	2.00	0.00	0.43	0.00	33.87	13.01	2.00	0.00	0.43	0.00
33.96	12.02	2.00	0.00	0.42	0.00	34.01	13.71	2.00	0.00	0.42	0.00
34.10	20.89	2.00	0.00	0.42	0.00	34.15	22.35	2.00	0.00	0.42	0.00
34.20	21.81	2.00	0.00	0.42	0.00	34.26	19.02	2.00	0.00	0.42	0.00
34.33	19.34	2.00	0.00	0.42	0.00	34.42	27.55	2.00	0.00	0.42	0.00
34.47	32.16	2.00	0.00	0.42	0.00	34.52	103.16	2.00	0.00	0.41	0.00
34.58	114.96	2.00	0.00	0.41	0.00	34.65	123.15	2.00	0.00	0.41	0.00
34.71	126.55	2.00	0.00	0.41	0.00	34.79	127.58	0.33	1.02	0.41	0.01
34.85	128.25	0.34	1.01	0.41	0.01	34.93	129.86	0.35	0.99	0.41	0.01
34.98	130.65	0.35	0.99	0.41	0.01	35.07	125.67	0.32	1.02	0.41	0.01
35.13	119.95	0.30	1.07	0.40	0.01	35.21	124.83	0.32	1.03	0.40	0.01
35.25	126.79	0.33	1.01	0.40	0.01	35.34	129.88	0.35	0.98	0.40	0.01
35.39	130.62	0.35	0.97	0.40	0.01	35.45	130.75	0.35	0.97	0.40	0.01
35.54	132.51	2.00	0.00	0.40	0.00	35.59	133.89	2.00	0.00	0.40	0.00
35.63	135.42	2.00	0.00	0.40	0.00	35.73	133.17	2.00	0.00	0.39	0.00
35.77	131.20	2.00	0.00	0.39	0.00	35.86	120.51	2.00	0.00	0.39	0.00
35.91	113.16	2.00	0.00	0.39	0.00	35.96	111.59	2.00	0.00	0.39	0.00
36.05	114.74	2.00	0.00	0.39	0.00	36.12	117.68	2.00	0.00	0.39	0.00
36.19	127.54	2.00	0.00	0.39	0.00	36.23	133.04	2.00	0.00	0.39	0.00
36.33	139.41	2.00	0.00	0.38	0.00	36.35	139.79	2.00	0.00	0.38	0.00
36.46	139.24	2.00	0.00	0.38	0.00	36.50	140.11	0.41	0.85	0.38	0.00
36.56	142.25	0.43	0.84	0.38	0.01	36.62	143.97	0.45	0.82	0.38	0.01
36.69	145.85	0.47	0.81	0.38	0.01	36.79	148.23	0.49	0.79	0.38	0.01
36.83	149.46	0.50	0.78	0.38	0.00	36.88	151.92	0.54	0.77	0.37	0.00
36.97	155.88	0.59	0.74	0.37	0.01	37.02	158.52	0.64	0.68	0.37	0.00
37.11	154.44	0.57	0.74	0.37	0.01	37.16	148.82	0.50	0.78	0.37	0.00
37.24	150.33	0.51	0.76	0.37	0.01	37.29	152.45	0.54	0.75	0.37	0.00
37.34	154.15	0.57	0.74	0.37	0.00	37.43	155.86	0.59	0.73	0.37	0.01
37.49	157.82	0.62	0.68	0.36	0.00	37.53	157.27	0.61	0.69	0.36	0.00
37.62	159.35	0.65	0.64	0.36	0.01	37.67	160.16	0.67	0.62	0.36	0.00
37.73	160.34	0.67	0.62	0.36	0.00	37.81	158.73	0.64	0.65	0.36	0.01
37.87	157.64	0.62	0.67	0.36	0.00	37.95	156.92	2.00	0.00	0.36	0.00
38.00	156.66	2.00	0.00	0.36	0.00	38.08	156.77	2.00	0.00	0.35	0.00
38.13	153.36	2.00	0.00	0.35	0.00	38.20	145.68	2.00	0.00	0.35	0.00
38.27	134.54	2.00	0.00	0.35	0.00	38.37	119.28	2.00	0.00	0.35	0.00
38.41	116.34	2.00	0.00	0.35	0.00	38.46	114.91	0.27	0.97	0.35	0.01
38.55	116.08	0.28	0.95	0.35	0.01	38.59	115.50	0.28	0.96	0.35	0.00
38.68	116.17	2.00	0.00	0.34	0.00	38.73	117.57	2.00	0.00	0.34	0.00
38.82	120.86	2.00	0.00	0.34	0.00	38.87	125.04	2.00	0.00	0.34	0.00
38.91	126.76	2.00	0.00	0.34	0.00	39.01	131.90	2.00	0.00	0.34	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
39.05	132.26	2.00	0.00	0.34	0.00	39.15	129.87	2.00	0.00	0.34	0.00
39.19	128.47	0.33	0.83	0.34	0.00	39.28	125.82	0.32	0.84	0.33	0.01
39.32	124.91	0.31	0.85	0.33	0.00	39.38	124.15	0.31	0.85	0.33	0.01
39.45	123.77	0.31	0.85	0.33	0.01	39.51	121.64	0.30	0.86	0.33	0.01
39.58	114.29	0.27	0.92	0.33	0.01	39.66	118.35	0.29	0.88	0.33	0.01
39.70	121.05	0.30	0.86	0.33	0.01	39.83	126.51	0.32	0.82	0.32	0.01
39.84	126.99	0.32	0.81	0.32	0.00	39.92	129.22	0.33	0.79	0.32	0.01
39.98	131.48	0.35	0.78	0.32	0.00	40.05	134.41	0.36	0.75	0.32	0.01
40.11	136.51	0.38	0.74	0.32	0.01	40.16	137.50	0.39	0.73	0.32	0.00
40.25	139.25	0.40	0.72	0.32	0.01	40.30	139.43	0.40	0.71	0.32	0.00
40.39	140.12	0.40	0.71	0.32	0.01	40.43	140.96	0.41	0.70	0.31	0.00
40.53	142.11	0.42	0.69	0.31	0.01	40.57	143.08	0.43	0.68	0.31	0.00
40.62	144.17	0.44	0.68	0.31	0.00	40.69	146.10	0.46	0.66	0.31	0.01
40.76	148.00	0.48	0.65	0.31	0.01	40.85	149.82	0.50	0.64	0.31	0.01
40.90	150.67	0.51	0.63	0.31	0.00	40.99	152.70	0.53	0.62	0.31	0.01
41.03	154.15	0.56	0.61	0.30	0.00	41.09	157.41	0.61	0.57	0.30	0.00
41.16	158.84	0.63	0.54	0.30	0.00	41.27	158.88	0.63	0.54	0.30	0.01
41.31	157.92	0.61	0.56	0.30	0.00	41.35	157.05	0.60	0.57	0.30	0.00
41.41	156.11	0.58	0.59	0.30	0.00	41.49	156.26	0.59	0.58	0.30	0.01
41.54	156.72	0.59	0.57	0.30	0.00	41.63	148.25	0.48	0.62	0.29	0.01
41.67	140.65	0.41	0.66	0.29	0.00	41.80	140.09	0.40	0.65	0.29	0.01
41.86	140.26	0.40	0.65	0.29	0.01	41.91	140.29	0.40	0.65	0.29	0.00
41.96	141.01	0.41	0.64	0.29	0.00	42.00	141.70	0.42	0.64	0.29	0.00
42.09	141.52	0.41	0.64	0.29	0.01	42.14	142.26	0.42	0.63	0.29	0.00
42.23	140.52	0.41	0.63	0.28	0.01	42.28	141.26	0.41	0.63	0.28	0.00
42.36	139.79	0.40	0.63	0.28	0.01	42.41	137.48	0.38	0.64	0.28	0.00
42.46	136.84	0.38	0.64	0.28	0.00	42.55	136.21	0.37	0.64	0.28	0.01
42.60	136.35	0.37	0.64	0.28	0.00	42.65	139.08	0.39	0.63	0.28	0.00
42.74	143.33	0.43	0.60	0.28	0.01	42.83	146.03	0.45	0.59	0.27	0.01
42.87	144.57	0.44	0.59	0.27	0.00	42.92	143.39	0.43	0.60	0.27	0.00
42.99	143.46	0.43	0.59	0.27	0.00	43.05	143.57	0.43	0.59	0.27	0.00
43.12	144.02	0.43	0.58	0.27	0.00	43.19	148.42	0.48	0.56	0.27	0.00
43.24	152.77	0.53	0.54	0.27	0.00	43.33	156.45	0.58	0.52	0.27	0.01
43.42	154.84	0.56	0.53	0.26	0.01	43.47	154.85	0.56	0.53	0.26	0.00
43.51	152.32	0.53	0.54	0.26	0.00	43.61	158.89	0.62	0.47	0.26	0.01
43.66	160.29	0.65	0.44	0.26	0.00	43.70	163.65	0.72	0.39	0.26	0.00
43.77	167.97	0.82	0.29	0.26	0.00	43.84	173.73	1.01	0.19	0.26	0.00
43.92	173.72	1.01	0.19	0.26	0.00	43.97	173.75	1.01	0.19	0.25	0.00
44.06	189.13	1.91	0.01	0.25	0.00	44.10	192.46	2.00	0.00	0.25	0.00
44.16	197.64	2.00	0.00	0.25	0.00	44.23	200.15	2.00	0.00	0.25	0.00
44.30	201.48	2.00	0.00	0.25	0.00	44.39	207.22	2.00	0.00	0.25	0.00
44.43	210.26	2.00	0.00	0.25	0.00	44.52	222.86	2.00	0.00	0.25	0.00
44.58	233.58	2.00	0.00	0.24	0.00	44.63	238.59	2.00	0.00	0.24	0.00
44.71	253.53	2.00	0.00	0.24	0.00	44.76	256.17	2.00	0.00	0.24	0.00
44.82	263.91	2.00	0.00	0.24	0.00	44.90	267.66	2.00	0.00	0.24	0.00
44.95	269.76	2.00	0.00	0.24	0.00	45.04	272.99	2.00	0.00	0.24	0.00
45.08	275.39	2.00	0.00	0.24	0.00	45.17	278.53	2.00	0.00	0.23	0.00
45.22	276.32	2.00	0.00	0.23	0.00	45.28	272.42	2.00	0.00	0.23	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
45.36	269.38	2.00	0.00	0.23	0.00	45.45	265.60	2.00	0.00	0.23	0.00
45.50	264.03	2.00	0.00	0.23	0.00	45.55	261.68	2.00	0.00	0.23	0.00
45.64	259.06	2.00	0.00	0.23	0.00	45.68	255.04	2.00	0.00	0.23	0.00
45.75	251.57	2.00	0.00	0.22	0.00	45.82	251.11	2.00	0.00	0.22	0.00
45.88	247.23	2.00	0.00	0.22	0.00	45.94	248.96	2.00	0.00	0.22	0.00
46.00	246.61	2.00	0.00	0.22	0.00	46.06	242.29	2.00	0.00	0.22	0.00
46.14	237.53	2.00	0.00	0.22	0.00	46.20	233.08	2.00	0.00	0.22	0.00
46.28	229.25	2.00	0.00	0.22	0.00	46.33	228.14	2.00	0.00	0.21	0.00
46.41	239.07	2.00	0.00	0.21	0.00	46.47	226.89	2.00	0.00	0.21	0.00
46.56	213.81	2.00	0.00	0.21	0.00	46.61	205.40	2.00	0.00	0.21	0.00
46.69	198.27	2.00	0.00	0.21	0.00	46.74	194.04	2.00	0.00	0.21	0.00
46.80	189.98	1.96	0.00	0.21	0.00	46.88	184.03	1.51	0.05	0.21	0.00
46.93	180.34	1.29	0.09	0.20	0.00	47.02	171.20	0.91	0.19	0.20	0.00
47.07	166.66	0.78	0.26	0.20	0.00	47.12	164.23	0.72	0.30	0.20	0.00
47.21	157.73	0.60	0.37	0.20	0.00	47.26	154.77	0.55	0.40	0.20	0.00
47.34	149.54	0.49	0.41	0.20	0.00	47.39	133.32	0.35	0.47	0.20	0.00
47.48	121.48	0.29	0.51	0.20	0.01	47.53	113.89	0.26	0.55	0.19	0.00
47.58	111.62	0.26	0.56	0.19	0.00	47.66	114.59	0.27	0.54	0.19	0.01
47.71	117.82	0.28	0.52	0.19	0.00	47.78	126.46	0.31	0.48	0.19	0.00
47.86	131.69	0.34	0.45	0.19	0.00	47.90	134.65	0.36	0.44	0.19	0.00
47.99	139.34	0.39	0.42	0.19	0.00	48.04	140.13	0.40	0.42	0.19	0.00
48.10	139.20	0.39	0.42	0.18	0.00	48.18	137.11	0.37	0.42	0.18	0.00
48.23	133.87	0.35	0.43	0.18	0.00	48.32	129.87	0.33	0.44	0.18	0.00
48.37	124.18	0.30	0.46	0.18	0.00	48.45	119.24	2.00	0.00	0.18	0.00
48.51	115.40	2.00	0.00	0.18	0.00	48.60	40.87	2.00	0.00	0.18	0.00
48.65	36.23	2.00	0.00	0.18	0.00	48.69	25.92	2.00	0.00	0.17	0.00
48.76	25.39	2.00	0.00	0.17	0.00	48.84	19.11	2.00	0.00	0.17	0.00
48.90	16.08	2.00	0.00	0.17	0.00	49.01	12.74	2.00	0.00	0.17	0.00
49.03	12.28	2.00	0.00	0.17	0.00	49.10	12.68	2.00	0.00	0.17	0.00
49.17	14.46	2.00	0.00	0.17	0.00	49.22	15.69	2.00	0.00	0.17	0.00
49.31	20.67	2.00	0.00	0.16	0.00	49.35	22.15	2.00	0.00	0.16	0.00
49.45	24.79	2.00	0.00	0.16	0.00	49.49	24.72	2.00	0.00	0.16	0.00
49.59	22.66	2.00	0.00	0.16	0.00	49.61	21.71	2.00	0.00	0.16	0.00
49.67	22.75	2.00	0.00	0.16	0.00	49.77	21.09	2.00	0.00	0.16	0.00
49.81	22.83	2.00	0.00	0.16	0.00	49.91	30.13	2.00	0.00	0.15	0.00
49.94	33.24	2.00	0.00	0.15	0.00	50.04	41.30	2.00	0.00	0.15	0.00
50.08	108.42	2.00	0.00	0.15	0.00	50.13	116.98	2.00	0.00	0.15	0.00
50.21	129.06	2.00	0.00	0.15	0.00	50.30	148.71	2.00	0.00	0.15	0.00
50.37	162.26	2.00	0.00	0.15	0.00	50.41	100.88	2.00	0.00	0.15	0.00
50.50	130.91	2.00	0.00	0.14	0.00	50.55	137.62	2.00	0.00	0.14	0.00
50.60	143.26	2.00	0.00	0.14	0.00	50.69	145.18	2.00	0.00	0.14	0.00
50.73	140.64	2.00	0.00	0.14	0.00						

Total estimated settlement: 1.74

Abbreviations

$Q_{tn,cs}$:	Equivalent clean sand normalized cone resistance
FS:	Factor of safety against liquefaction
e_v (%):	Post-liquefaction volumentric strain
DF:	e_v depth weighting factor
Settlement:	Calculated settlement

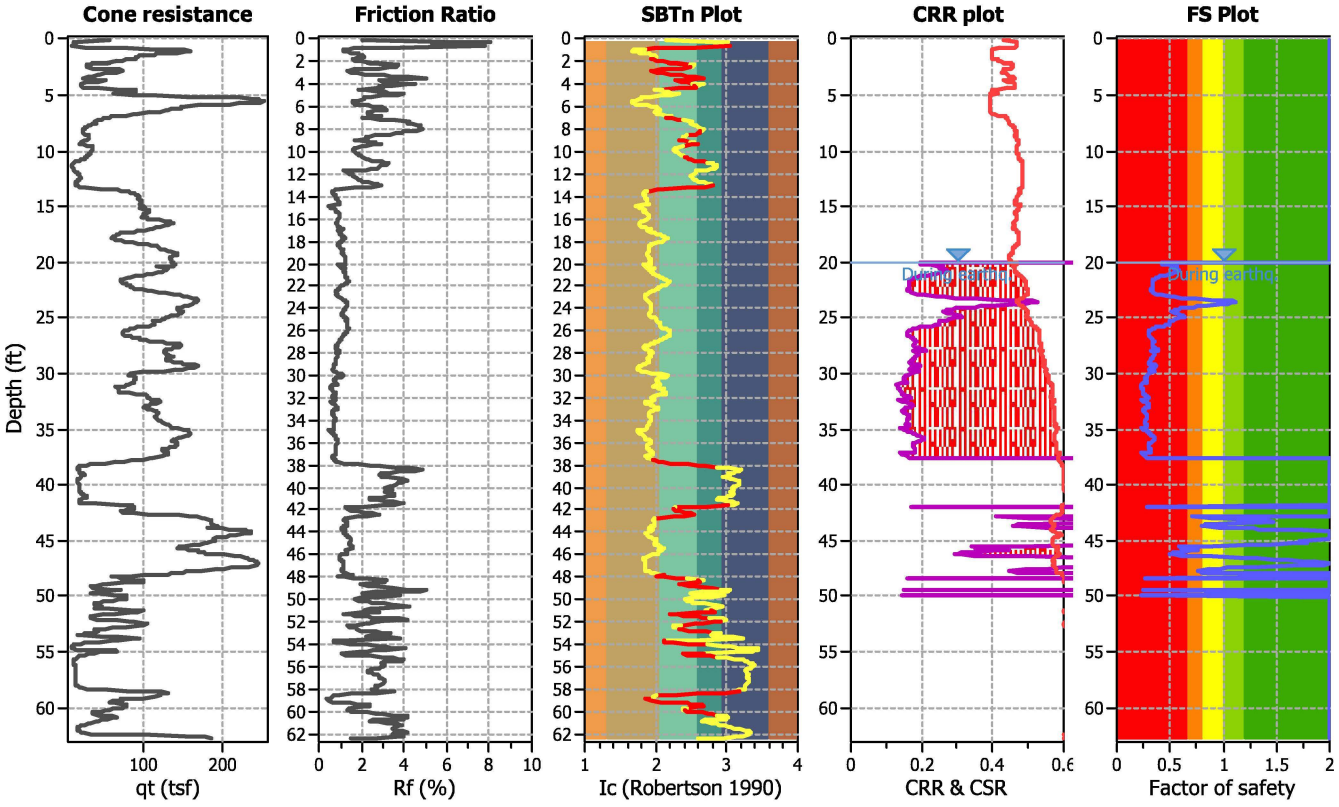
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

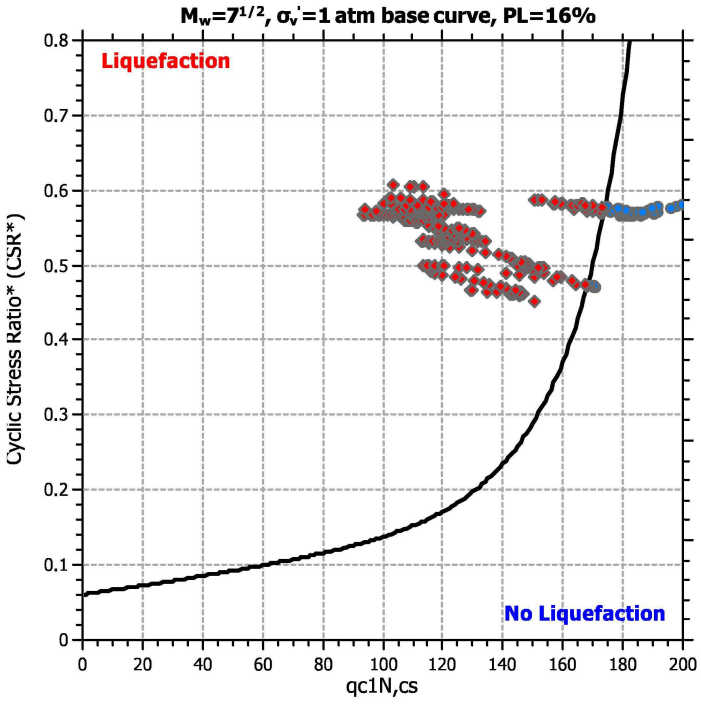
LIQUEFACTION ANALYSIS REPORT

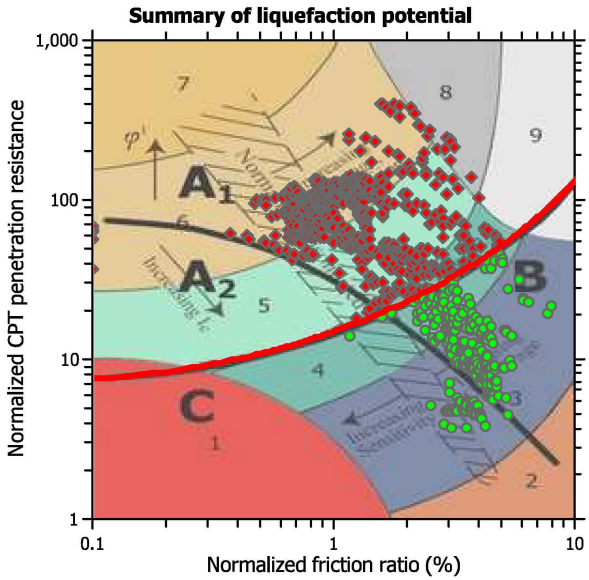
Project title : Southern California Geotechnical / Proposed Industrial Building **Location : 3800 Via Oro Ave, Long Beach, CA**
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	26.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.99	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

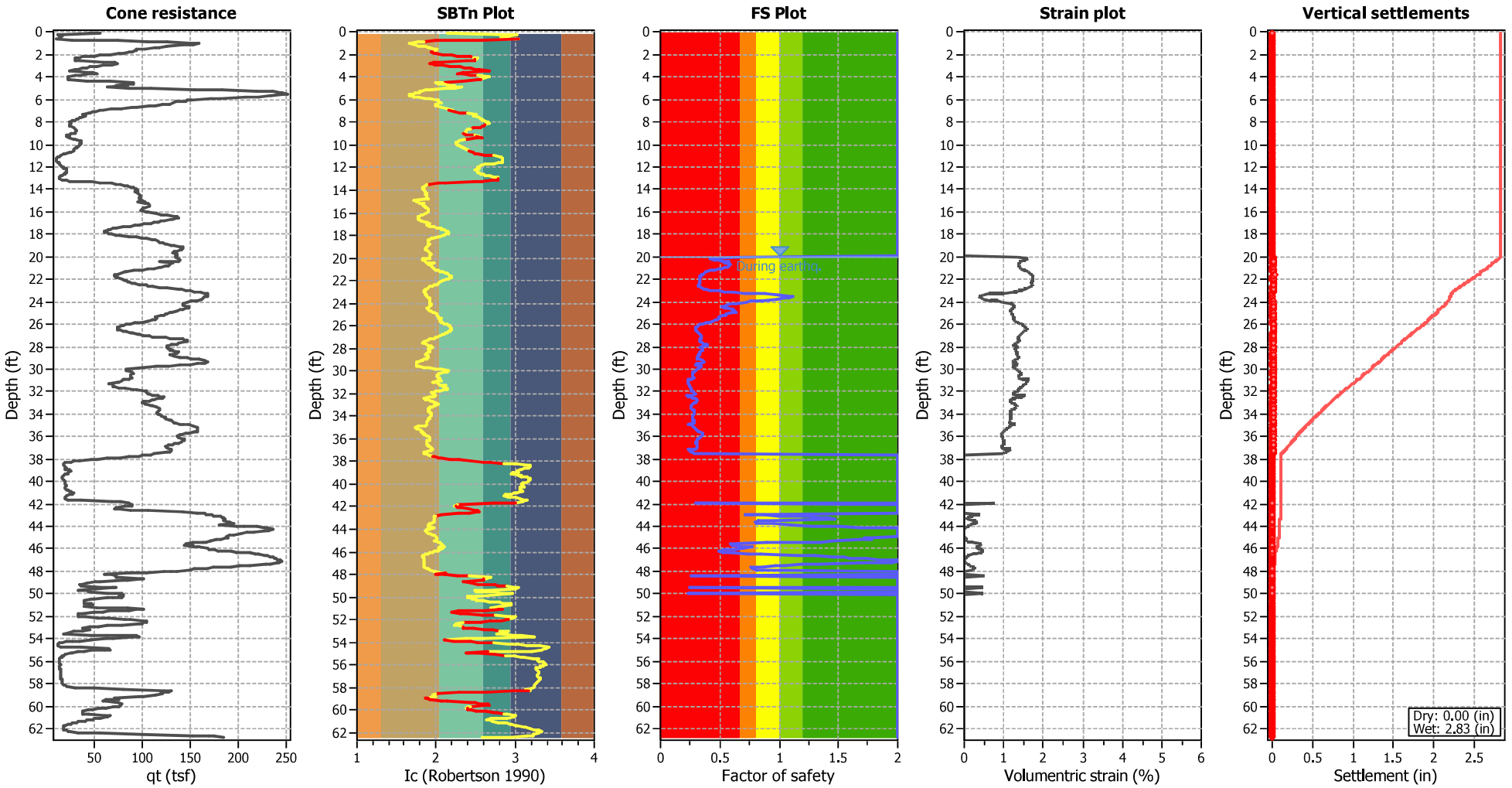






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
20.03	150.68	0.65	1.36	0.66	0.01	20.10	129.34	0.42	1.61	0.66	0.01
20.15	129.63	0.42	1.61	0.66	0.01	20.24	134.76	0.46	1.54	0.66	0.02
20.28	137.79	0.49	1.50	0.66	0.01	20.38	142.09	0.53	1.44	0.65	0.02
20.42	145.69	0.57	1.40	0.65	0.01	20.52	142.84	0.54	1.43	0.65	0.02
20.57	143.28	0.54	1.42	0.65	0.01	20.61	144.60	0.56	1.41	0.65	0.01
20.70	146.79	0.58	1.38	0.65	0.01	20.75	146.81	0.58	1.38	0.65	0.01
20.84	146.08	0.57	1.38	0.65	0.02	20.89	145.18	0.56	1.39	0.65	0.01
20.93	143.95	0.54	1.40	0.65	0.01	21.03	141.32	0.51	1.43	0.64	0.02
21.08	139.28	0.49	1.45	0.64	0.01	21.17	135.48	0.46	1.49	0.64	0.02
21.21	133.47	0.44	1.51	0.64	0.01	21.26	130.30	0.42	1.55	0.64	0.01
21.35	125.92	0.39	1.61	0.64	0.02	21.39	124.40	0.38	1.63	0.64	0.01
21.54	119.74	0.35	1.69	0.63	0.03	21.63	117.47	0.34	1.72	0.63	0.02
21.66	117.17	0.34	1.72	0.63	0.01	21.77	116.07	0.33	1.74	0.63	0.02
21.79	116.13	0.33	1.73	0.63	0.00	21.86	116.17	0.33	1.73	0.63	0.01
21.95	116.52	0.33	1.72	0.63	0.02	22.00	116.38	0.33	1.72	0.63	0.01
22.09	116.22	0.33	1.72	0.63	0.02	22.12	116.20	0.33	1.72	0.63	0.01
22.18	116.20	0.33	1.71	0.62	0.01	22.26	116.65	0.33	1.70	0.62	0.02
22.32	116.99	0.33	1.70	0.62	0.01	22.38	116.42	0.33	1.70	0.62	0.01
22.46	115.24	0.32	1.72	0.62	0.02	22.55	113.30	0.32	1.74	0.62	0.02
22.58	114.57	0.32	1.72	0.62	0.01	22.64	117.18	0.33	1.68	0.62	0.01
22.73	119.74	0.34	1.64	0.61	0.02	22.78	120.19	0.34	1.63	0.61	0.01
22.88	125.25	0.37	1.55	0.61	0.02	22.92	127.84	0.39	1.52	0.61	0.01
22.98	131.77	0.41	1.46	0.61	0.01	23.06	141.35	0.49	1.35	0.61	0.01
23.10	145.25	0.54	1.31	0.61	0.01	23.17	150.68	0.61	1.25	0.61	0.01
23.24	157.05	0.72	1.01	0.61	0.01	23.33	164.21	0.89	0.63	0.60	0.01
23.38	167.79	1.00	0.49	0.60	0.00	23.47	170.35	1.09	0.40	0.60	0.00
23.51	171.15	1.12	0.37	0.60	0.00	23.57	171.13	1.12	0.37	0.60	0.00
23.65	170.38	1.08	0.40	0.60	0.00	23.70	169.17	1.04	0.44	0.60	0.00
23.76	167.66	0.98	0.49	0.60	0.00	23.87	163.17	0.85	0.67	0.60	0.01
23.89	162.76	0.84	0.69	0.60	0.00	23.98	159.53	0.76	0.86	0.59	0.01
24.02	157.69	0.72	0.97	0.59	0.01	24.12	153.40	0.64	1.20	0.59	0.01
24.16	151.56	0.61	1.21	0.59	0.01	24.21	149.78	0.58	1.23	0.59	0.01
24.30	147.48	0.55	1.24	0.59	0.01	24.39	146.59	0.54	1.25	0.59	0.01
24.44	145.41	0.52	1.26	0.59	0.01	24.49	145.12	0.52	1.26	0.58	0.01
24.57	147.11	0.54	1.24	0.58	0.01	24.62	148.89	0.57	1.22	0.58	0.01
24.72	151.83	0.61	1.19	0.58	0.01	24.76	152.55	0.62	1.18	0.58	0.01
24.80	152.97	0.63	1.18	0.58	0.01	24.90	153.77	0.64	1.17	0.58	0.01
24.95	151.48	0.60	1.18	0.58	0.01	25.04	148.40	0.56	1.21	0.58	0.01
25.07	148.35	0.56	1.21	0.58	0.00	25.17	147.93	0.55	1.21	0.57	0.01
25.22	147.03	0.54	1.22	0.57	0.01	25.27	146.39	0.53	1.22	0.57	0.01
25.35	143.49	0.50	1.24	0.57	0.01	25.40	143.20	0.49	1.25	0.57	0.01
25.49	142.89	0.49	1.25	0.57	0.01	25.53	142.89	0.49	1.24	0.57	0.01
25.60	142.55	0.48	1.24	0.57	0.01	25.67	140.79	0.47	1.26	0.56	0.01
25.72	138.36	0.44	1.28	0.56	0.01	25.81	133.93	0.41	1.33	0.56	0.01
25.85	129.94	0.38	1.37	0.56	0.01	25.95	125.14	0.35	1.42	0.56	0.02
25.99	122.09	0.34	1.46	0.56	0.01	26.07	119.55	0.32	1.49	0.56	0.01
26.13	117.47	0.31	1.51	0.56	0.01	26.19	115.60	0.31	1.54	0.56	0.01
26.27	113.68	0.30	1.56	0.55	0.01	26.32	113.25	0.30	1.56	0.55	0.01

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
26.41	112.75	0.29	1.57	0.55	0.02	26.45	112.67	0.29	1.57	0.55	0.01
26.54	113.67	0.30	1.55	0.55	0.02	26.59	115.14	0.30	1.52	0.55	0.01
26.65	116.81	0.31	1.50	0.55	0.01	26.73	118.71	0.32	1.47	0.55	0.01
26.77	119.44	0.32	1.46	0.55	0.01	26.87	121.17	0.33	1.43	0.54	0.02
26.90	121.76	0.33	1.42	0.54	0.01	27.01	123.57	0.34	1.40	0.54	0.02
27.05	124.25	0.34	1.38	0.54	0.01	27.10	125.12	0.35	1.37	0.54	0.01
27.19	124.85	0.34	1.37	0.54	0.01	27.24	124.84	0.34	1.37	0.54	0.01
27.33	123.62	0.34	1.38	0.54	0.02	27.38	123.66	0.34	1.38	0.54	0.01
27.47	122.33	0.33	1.39	0.53	0.02	27.50	122.32	0.33	1.39	0.53	0.00
27.56	122.92	0.33	1.38	0.53	0.01	27.70	127.88	0.36	1.32	0.53	0.02
27.75	129.80	0.37	1.29	0.53	0.01	27.79	131.97	0.38	1.27	0.53	0.01
27.83	132.96	0.39	1.25	0.53	0.01	27.89	134.12	0.40	1.24	0.53	0.01
27.97	127.53	0.36	1.31	0.53	0.01	28.02	121.71	0.33	1.37	0.53	0.01
28.12	125.81	0.35	1.32	0.52	0.02	28.20	128.04	0.36	1.29	0.52	0.01
28.23	127.62	0.35	1.30	0.52	0.00	28.30	126.93	0.35	1.30	0.52	0.01
28.35	127.99	0.36	1.29	0.52	0.01	28.42	123.83	0.33	1.33	0.52	0.01
28.52	123.31	0.33	1.33	0.52	0.02	28.56	123.97	0.33	1.32	0.52	0.01
28.66	123.48	0.33	1.32	0.51	0.02	28.70	122.45	0.33	1.33	0.51	0.01
28.74	122.23	0.32	1.33	0.51	0.01	28.84	120.43	0.32	1.35	0.51	0.02
28.89	120.01	0.31	1.35	0.51	0.01	28.97	118.30	0.31	1.37	0.51	0.01
29.02	117.64	0.30	1.38	0.51	0.01	29.07	118.63	0.31	1.36	0.51	0.01
29.14	122.73	0.33	1.31	0.51	0.01	29.21	127.56	0.35	1.25	0.50	0.01
29.30	130.41	0.37	1.22	0.50	0.01	29.35	130.03	0.36	1.22	0.50	0.01
29.44	127.04	0.35	1.25	0.50	0.01	29.48	124.99	0.34	1.27	0.50	0.01
29.53	120.19	0.31	1.32	0.50	0.01	29.63	115.45	0.29	1.38	0.50	0.02
29.67	116.86	0.30	1.36	0.50	0.01	29.76	124.60	0.33	1.26	0.50	0.01
29.81	127.52	0.35	1.23	0.49	0.01	29.90	125.54	0.34	1.25	0.49	0.01
29.99	118.38	0.30	1.32	0.49	0.02	30.08	115.40	0.29	1.36	0.49	0.01
30.12	114.96	0.29	1.36	0.49	0.01	30.22	111.61	0.28	1.40	0.49	0.02
30.25	110.89	0.28	1.41	0.49	0.01	30.32	109.69	0.27	1.42	0.49	0.01
30.41	107.95	0.27	1.44	0.48	0.02	30.45	107.40	0.26	1.44	0.48	0.01
30.55	108.47	0.27	1.42	0.48	0.02	30.59	109.11	0.27	1.41	0.48	0.01
30.66	110.70	0.27	1.39	0.48	0.01	30.72	112.08	0.28	1.37	0.48	0.01
30.82	113.38	0.28	1.35	0.48	0.02	30.86	103.62	0.25	1.48	0.48	0.01
30.96	93.16	0.23	1.64	0.48	0.02	30.98	94.07	0.23	1.62	0.47	0.00
31.05	95.98	0.23	1.59	0.47	0.01	31.11	96.67	0.24	1.57	0.47	0.01
31.19	96.88	0.24	1.56	0.47	0.01	31.24	96.53	0.23	1.57	0.47	0.01
31.32	98.25	0.24	1.53	0.47	0.02	31.37	98.90	0.24	1.52	0.47	0.01
31.46	100.58	0.24	1.49	0.47	0.02	31.51	101.37	0.25	1.48	0.47	0.01
31.60	102.95	0.25	1.45	0.46	0.02	31.65	104.13	0.25	1.43	0.46	0.01
31.74	105.21	0.25	1.41	0.46	0.01	31.79	105.07	0.25	1.41	0.46	0.01
31.83	105.48	0.26	1.40	0.46	0.01	31.92	106.66	0.26	1.38	0.46	0.02
31.97	109.39	0.27	1.34	0.46	0.01	32.07	114.25	0.28	1.28	0.46	0.01
32.10	115.58	0.29	1.26	0.46	0.01	32.16	117.31	0.29	1.24	0.46	0.01
32.23	118.85	0.30	1.22	0.45	0.01	32.29	109.14	0.27	1.33	0.45	0.01
32.38	93.98	0.23	1.54	0.45	0.02	32.43	97.81	0.24	1.48	0.45	0.01
32.52	105.66	0.25	1.36	0.45	0.02	32.58	112.49	0.28	1.27	0.45	0.01
32.62	117.04	0.29	1.22	0.45	0.00	32.68	119.55	0.30	1.19	0.45	0.01

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
32.78	120.29	0.30	1.18	0.44	0.01	32.82	120.53	0.30	1.17	0.44	0.01
32.91	120.22	0.30	1.17	0.44	0.01	32.96	118.75	0.30	1.19	0.44	0.01
33.05	114.77	0.28	1.22	0.44	0.01	33.10	112.08	0.27	1.25	0.44	0.01
33.14	109.64	0.26	1.28	0.44	0.01	33.23	106.16	0.25	1.32	0.44	0.01
33.28	106.13	0.25	1.32	0.44	0.01	33.37	105.20	0.25	1.32	0.43	0.01
33.41	105.60	0.25	1.32	0.43	0.01	33.51	107.42	0.26	1.29	0.43	0.01
33.55	109.07	0.26	1.27	0.43	0.01	33.60	111.97	0.27	1.23	0.43	0.01
33.70	115.22	0.28	1.19	0.43	0.01	33.74	115.78	0.28	1.18	0.43	0.01
33.88	116.49	0.29	1.17	0.43	0.02	33.92	116.52	0.29	1.16	0.43	0.01
33.97	116.25	0.29	1.17	0.42	0.01	34.02	115.96	0.28	1.17	0.42	0.01
34.06	115.07	0.28	1.17	0.42	0.01	34.16	113.16	0.27	1.19	0.42	0.01
34.19	112.79	0.27	1.19	0.42	0.01	34.29	112.95	0.27	1.19	0.42	0.01
34.34	112.93	0.27	1.18	0.42	0.01	34.39	113.15	0.27	1.18	0.42	0.01
34.48	112.37	0.27	1.18	0.42	0.01	34.52	114.17	0.28	1.16	0.41	0.01
34.62	115.88	0.28	1.14	0.41	0.01	34.66	115.98	0.28	1.14	0.41	0.01
34.75	116.82	0.29	1.12	0.41	0.01	34.83	112.10	0.27	1.17	0.41	0.01
34.89	100.62	0.24	1.30	0.41	0.01	34.93	102.89	0.24	1.27	0.41	0.01
34.98	107.93	0.26	1.21	0.41	0.01	35.07	111.70	0.27	1.16	0.41	0.01
35.12	114.51	0.28	1.13	0.40	0.01	35.17	115.40	0.28	1.12	0.40	0.01
35.27	118.15	0.29	1.09	0.40	0.01	35.31	119.57	0.30	1.07	0.40	0.01
35.40	122.21	0.31	1.04	0.40	0.01	35.45	123.44	0.31	1.03	0.40	0.01
35.54	125.60	0.32	1.01	0.40	0.01	35.59	126.71	0.33	0.99	0.40	0.01
35.63	127.78	0.33	0.98	0.40	0.01	35.73	131.37	0.35	0.95	0.39	0.01
35.77	132.14	0.36	0.94	0.39	0.00	35.86	132.50	0.36	0.93	0.39	0.01
35.90	132.03	0.36	0.94	0.39	0.00	36.00	130.11	0.34	0.95	0.39	0.01
36.03	129.76	0.34	0.95	0.39	0.00	36.09	128.52	0.34	0.96	0.39	0.01
36.18	123.98	0.31	0.99	0.39	0.01	36.23	122.48	0.31	1.00	0.39	0.01
36.32	122.62	0.31	1.00	0.38	0.01	36.39	122.58	0.31	0.99	0.38	0.01
36.46	121.47	0.30	1.00	0.38	0.01	36.51	120.77	0.30	1.01	0.38	0.01
36.56	121.26	0.30	1.00	0.38	0.01	36.64	122.50	0.31	0.98	0.38	0.01
36.68	123.01	0.31	0.98	0.38	0.00	36.78	123.22	0.31	0.97	0.38	0.01
36.83	123.76	0.31	0.97	0.38	0.01	36.92	124.07	0.31	0.96	0.37	0.01
36.97	123.81	0.31	0.96	0.37	0.01	37.01	123.71	0.31	0.96	0.37	0.01
37.09	120.45	0.30	0.98	0.37	0.01	37.15	102.08	0.24	1.16	0.37	0.01
37.24	102.78	0.24	1.15	0.37	0.01	37.29	105.68	0.25	1.12	0.37	0.01
37.34	108.97	0.26	1.08	0.37	0.01	37.43	113.31	0.27	1.03	0.37	0.01
37.52	116.79	0.28	1.00	0.36	0.01	37.54	116.16	0.28	1.00	0.36	0.00
37.61	115.21	2.00	0.00	0.36	0.00	37.70	116.51	2.00	0.00	0.36	0.00
37.75	117.93	2.00	0.00	0.36	0.00	37.83	117.05	2.00	0.00	0.36	0.00
37.88	113.49	2.00	0.00	0.36	0.00	37.95	106.75	2.00	0.00	0.36	0.00
38.02	37.14	2.00	0.00	0.36	0.00	38.07	33.39	2.00	0.00	0.35	0.00
38.16	24.37	2.00	0.00	0.35	0.00	38.21	19.81	2.00	0.00	0.35	0.00
38.26	18.03	2.00	0.00	0.35	0.00	38.34	14.20	2.00	0.00	0.35	0.00
38.39	13.04	2.00	0.00	0.35	0.00	38.48	12.77	2.00	0.00	0.35	0.00
38.53	12.89	2.00	0.00	0.35	0.00	38.62	13.27	2.00	0.00	0.35	0.00
38.67	13.45	2.00	0.00	0.34	0.00	38.72	13.58	2.00	0.00	0.34	0.00
38.81	14.15	2.00	0.00	0.34	0.00	38.85	13.88	2.00	0.00	0.34	0.00
38.95	13.61	2.00	0.00	0.34	0.00	38.99	16.37	2.00	0.00	0.34	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
39.09	18.23	2.00	0.00	0.34	0.00	39.11	18.23	2.00	0.00	0.34	0.00
39.18	15.19	2.00	0.00	0.34	0.00	39.27	16.14	2.00	0.00	0.33	0.00
39.31	15.05	2.00	0.00	0.33	0.00	39.38	13.31	2.00	0.00	0.33	0.00
39.45	12.55	2.00	0.00	0.33	0.00	39.55	11.77	2.00	0.00	0.33	0.00
39.59	11.65	2.00	0.00	0.33	0.00	39.64	11.58	2.00	0.00	0.33	0.00
39.70	11.58	2.00	0.00	0.33	0.00	39.77	11.69	2.00	0.00	0.33	0.00
39.87	12.51	2.00	0.00	0.32	0.00	39.91	12.94	2.00	0.00	0.32	0.00
40.01	13.38	2.00	0.00	0.32	0.00	40.05	13.30	2.00	0.00	0.32	0.00
40.10	13.05	2.00	0.00	0.32	0.00	40.16	13.04	2.00	0.00	0.32	0.00
40.24	13.03	2.00	0.00	0.32	0.00	40.33	13.98	2.00	0.00	0.32	0.00
40.38	13.97	2.00	0.00	0.32	0.00	40.47	13.90	2.00	0.00	0.31	0.00
40.50	13.70	2.00	0.00	0.31	0.00	40.56	13.86	2.00	0.00	0.31	0.00
40.63	13.82	2.00	0.00	0.31	0.00	40.70	14.44	2.00	0.00	0.31	0.00
40.79	15.89	2.00	0.00	0.31	0.00	40.84	16.46	2.00	0.00	0.31	0.00
40.89	17.02	2.00	0.00	0.31	0.00	40.97	18.60	2.00	0.00	0.31	0.00
41.02	19.80	2.00	0.00	0.30	0.00	41.08	19.87	2.00	0.00	0.30	0.00
41.15	18.20	2.00	0.00	0.30	0.00	41.21	16.09	2.00	0.00	0.30	0.00
41.31	14.50	2.00	0.00	0.30	0.00	41.39	13.61	2.00	0.00	0.30	0.00
41.44	13.16	2.00	0.00	0.30	0.00	41.48	13.10	2.00	0.00	0.30	0.00
41.57	13.78	2.00	0.00	0.30	0.00	41.62	15.79	2.00	0.00	0.29	0.00
41.67	24.72	2.00	0.00	0.29	0.00	41.77	105.77	2.00	0.00	0.29	0.00
41.81	111.40	2.00	0.00	0.29	0.00	41.90	118.97	2.00	0.00	0.29	0.00
41.93	120.09	0.29	0.77	0.29	0.00	42.04	121.70	2.00	0.00	0.29	0.00
42.09	123.95	2.00	0.00	0.29	0.00	42.13	124.98	2.00	0.00	0.29	0.00
42.22	121.86	2.00	0.00	0.28	0.00	42.29	117.75	2.00	0.00	0.28	0.00
42.36	115.31	2.00	0.00	0.28	0.00	42.40	114.90	2.00	0.00	0.28	0.00
42.46	117.17	2.00	0.00	0.28	0.00	42.54	126.53	2.00	0.00	0.28	0.00
42.60	139.66	2.00	0.00	0.28	0.00	42.73	159.24	2.00	0.00	0.28	0.00
42.81	160.35	2.00	0.00	0.27	0.00	42.87	161.23	2.00	0.00	0.27	0.00
42.91	163.50	0.72	0.41	0.27	0.00	43.01	169.77	0.88	0.27	0.27	0.00
43.05	173.22	0.99	0.21	0.27	0.00	43.15	179.36	1.25	0.12	0.27	0.00
43.18	180.91	1.34	0.10	0.27	0.00	43.24	182.16	1.41	0.09	0.27	0.00
43.33	183.11	1.47	0.07	0.27	0.00	43.39	182.75	1.44	0.08	0.26	0.00
43.47	172.16	0.95	0.22	0.26	0.00	43.51	168.03	0.82	0.30	0.26	0.00
43.62	167.45	0.81	0.31	0.26	0.00	43.65	167.12	0.80	0.31	0.26	0.00
43.71	167.65	0.81	0.30	0.26	0.00	43.79	172.35	0.96	0.22	0.26	0.00
43.83	175.97	1.09	0.16	0.26	0.00	43.93	179.83	1.27	0.11	0.26	0.00
43.97	180.67	1.32	0.10	0.25	0.00	44.05	183.85	1.51	0.06	0.25	0.00
44.10	186.18	1.68	0.04	0.25	0.00	44.16	190.81	2.00	0.00	0.25	0.00
44.26	198.28	2.00	0.00	0.25	0.00	44.30	200.99	2.00	0.00	0.25	0.00
44.39	206.19	2.00	0.00	0.25	0.00	44.44	208.33	2.00	0.00	0.25	0.00
44.53	209.43	2.00	0.00	0.25	0.00	44.58	207.28	2.00	0.00	0.24	0.00
44.63	204.69	2.00	0.00	0.24	0.00	44.71	199.87	2.00	0.00	0.24	0.00
44.76	196.47	2.00	0.00	0.24	0.00	44.85	192.05	2.00	0.00	0.24	0.00
44.89	190.64	2.00	0.00	0.24	0.00	44.99	187.26	1.75	0.03	0.24	0.00
45.04	186.81	1.72	0.03	0.24	0.00	45.08	187.31	1.75	0.03	0.24	0.00
45.15	187.67	1.78	0.02	0.23	0.00	45.22	186.30	1.68	0.03	0.23	0.00
45.31	183.03	1.45	0.07	0.23	0.00	45.36	181.37	1.35	0.08	0.23	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
45.41	179.84	1.27	0.10	0.23	0.00	45.48	170.78	0.90	0.22	0.23	0.00
45.54	156.94	0.59	0.44	0.23	0.00	45.64	159.84	0.64	0.39	0.23	0.00
45.69	157.67	0.60	0.42	0.23	0.00	45.78	162.32	0.68	0.36	0.22	0.00
45.82	164.27	0.73	0.33	0.22	0.00	45.87	166.14	0.77	0.29	0.22	0.00
45.96	163.21	0.70	0.34	0.22	0.00	46.01	159.10	0.62	0.39	0.22	0.00
46.10	153.09	0.53	0.44	0.22	0.00	46.14	151.85	0.51	0.45	0.22	0.00
46.23	150.45	0.50	0.45	0.22	0.01	46.27	152.82	0.53	0.44	0.22	0.00
46.33	157.45	0.59	0.40	0.21	0.00	46.41	164.65	0.73	0.31	0.21	0.00
46.46	169.01	0.84	0.23	0.21	0.00	46.56	177.65	1.16	0.12	0.21	0.00
46.60	180.46	1.30	0.09	0.21	0.00	46.65	183.30	1.46	0.06	0.21	0.00
46.72	184.69	1.55	0.05	0.21	0.00	46.79	185.37	1.60	0.04	0.21	0.00
46.88	187.31	1.74	0.02	0.21	0.00	46.93	188.57	1.84	0.01	0.20	0.00
47.02	190.94	2.00	0.00	0.20	0.00	47.06	191.41	2.00	0.00	0.20	0.00
47.11	191.95	2.00	0.00	0.20	0.00	47.20	189.86	1.95	0.00	0.20	0.00
47.25	187.10	1.72	0.02	0.20	0.00	47.34	181.31	1.34	0.07	0.20	0.00
47.39	178.53	1.19	0.10	0.20	0.00	47.48	175.03	1.04	0.14	0.20	0.00
47.53	172.97	0.97	0.16	0.19	0.00	47.59	169.75	0.86	0.20	0.19	0.00
47.68	166.12	0.77	0.25	0.19	0.00	47.71	166.66	0.78	0.24	0.19	0.00
47.80	166.40	0.77	0.25	0.19	0.00	47.85	167.67	0.80	0.22	0.19	0.00
47.99	135.22	2.00	0.00	0.19	0.00	48.00	136.08	2.00	0.00	0.19	0.00
48.04	137.82	2.00	0.00	0.19	0.00	48.13	133.53	2.00	0.00	0.18	0.00
48.17	122.87	2.00	0.00	0.18	0.00	48.23	40.46	2.00	0.00	0.18	0.00
48.32	44.91	2.00	0.00	0.18	0.00	48.37	113.23	0.26	0.51	0.18	0.00
48.46	48.90	2.00	0.00	0.18	0.00	48.51	115.30	2.00	0.00	0.18	0.00
48.63	131.99	2.00	0.00	0.18	0.00	48.65	133.73	2.00	0.00	0.18	0.00
48.69	132.31	2.00	0.00	0.17	0.00	48.78	122.19	2.00	0.00	0.17	0.00
48.83	117.19	2.00	0.00	0.17	0.00	48.92	36.88	2.00	0.00	0.17	0.00
48.97	33.60	2.00	0.00	0.17	0.00	49.06	26.94	2.00	0.00	0.17	0.00
49.12	23.64	2.00	0.00	0.17	0.00	49.15	21.84	2.00	0.00	0.17	0.00
49.25	23.33	2.00	0.00	0.17	0.00	49.29	23.05	2.00	0.00	0.16	0.00
49.38	37.60	2.00	0.00	0.16	0.00	49.43	110.51	0.25	0.47	0.16	0.00
49.48	109.31	0.25	0.47	0.16	0.00	49.57	32.98	2.00	0.00	0.16	0.00
49.61	29.86	2.00	0.00	0.16	0.00	49.71	21.62	2.00	0.00	0.16	0.00
49.75	28.96	2.00	0.00	0.16	0.00	49.84	28.11	2.00	0.00	0.16	0.00
49.89	103.28	0.23	0.48	0.15	0.00	49.98	113.61	0.26	0.43	0.15	0.00
50.03	114.77	2.00	0.00	0.15	0.00	50.08	113.77	2.00	0.00	0.15	0.00
50.17	116.01	2.00	0.00	0.15	0.00	50.21	114.17	2.00	0.00	0.15	0.00
50.30	117.25	2.00	0.00	0.15	0.00	50.35	46.03	2.00	0.00	0.15	0.00
50.40	41.45	2.00	0.00	0.15	0.00	50.49	32.22	2.00	0.00	0.14	0.00
50.53	29.74	2.00	0.00	0.14	0.00	50.59	24.78	2.00	0.00	0.14	0.00
50.67	25.11	2.00	0.00	0.14	0.00	50.80	25.04	2.00	0.00	0.14	0.00
50.85	25.35	2.00	0.00	0.14	0.00	50.92	29.64	2.00	0.00	0.14	0.00
51.00	30.50	2.00	0.00	0.14	0.00	51.06	27.35	2.00	0.00	0.13	0.00
51.14	25.02	2.00	0.00	0.13	0.00	51.19	34.22	2.00	0.00	0.13	0.00
51.27	114.90	2.00	0.00	0.13	0.00	51.32	124.64	2.00	0.00	0.13	0.00
51.41	123.67	2.00	0.00	0.13	0.00	51.45	119.74	2.00	0.00	0.13	0.00
51.55	104.74	2.00	0.00	0.13	0.00	51.59	37.77	2.00	0.00	0.13	0.00
51.64	31.97	2.00	0.00	0.12	0.00	51.74	23.72	2.00	0.00	0.12	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
51.78	21.35	2.00	0.00	0.12	0.00	51.85	21.19	2.00	0.00	0.12	0.00
51.92	21.15	2.00	0.00	0.12	0.00	51.97	21.01	2.00	0.00	0.12	0.00
52.05	26.13	2.00	0.00	0.12	0.00	52.10	34.18	2.00	0.00	0.12	0.00
52.19	104.38	2.00	0.00	0.12	0.00	52.26	115.51	2.00	0.00	0.11	0.00
52.30	119.03	2.00	0.00	0.11	0.00	52.40	126.94	2.00	0.00	0.11	0.00
52.44	130.53	2.00	0.00	0.11	0.00	52.54	132.62	2.00	0.00	0.11	0.00
52.58	129.68	2.00	0.00	0.11	0.00	52.64	129.51	2.00	0.00	0.11	0.00
52.69	127.91	2.00	0.00	0.11	0.00	52.78	113.86	2.00	0.00	0.11	0.00
52.83	40.48	2.00	0.00	0.10	0.00	52.92	34.19	2.00	0.00	0.10	0.00
52.97	28.96	2.00	0.00	0.10	0.00	53.05	23.97	2.00	0.00	0.10	0.00
53.10	24.74	2.00	0.00	0.10	0.00	53.15	26.61	2.00	0.00	0.10	0.00
53.22	27.97	2.00	0.00	0.10	0.00	53.29	23.94	2.00	0.00	0.10	0.00
53.38	17.38	2.00	0.00	0.10	0.00	53.42	14.93	2.00	0.00	0.09	0.00
53.52	11.54	2.00	0.00	0.09	0.00	53.56	11.99	2.00	0.00	0.09	0.00
53.65	28.76	2.00	0.00	0.09	0.00	53.70	98.53	2.00	0.00	0.09	0.00
53.75	109.76	2.00	0.00	0.09	0.00	53.83	111.97	2.00	0.00	0.09	0.00
53.88	112.52	2.00	0.00	0.09	0.00	53.98	104.51	2.00	0.00	0.09	0.00
54.03	99.68	2.00	0.00	0.08	0.00	54.07	28.71	2.00	0.00	0.08	0.00
54.16	20.92	2.00	0.00	0.08	0.00	54.21	16.35	2.00	0.00	0.08	0.00
54.27	12.90	2.00	0.00	0.08	0.00	54.35	9.73	2.00	0.00	0.08	0.00
54.40	8.01	2.00	0.00	0.08	0.00	54.49	7.18	2.00	0.00	0.08	0.00
54.53	7.01	2.00	0.00	0.08	0.00	54.62	6.95	2.00	0.00	0.07	0.00
54.67	7.23	2.00	0.00	0.07	0.00	54.76	8.93	2.00	0.00	0.07	0.00
54.80	21.91	2.00	0.00	0.07	0.00	54.89	98.39	2.00	0.00	0.07	0.00
54.95	101.37	2.00	0.00	0.07	0.00	55.00	98.57	2.00	0.00	0.07	0.00
55.06	29.05	2.00	0.00	0.07	0.00	55.14	24.18	2.00	0.00	0.07	0.00
55.19	20.04	2.00	0.00	0.06	0.00	55.28	16.24	2.00	0.00	0.06	0.00
55.33	15.26	2.00	0.00	0.06	0.00	55.42	12.24	2.00	0.00	0.06	0.00
55.46	11.28	2.00	0.00	0.06	0.00	55.52	10.16	2.00	0.00	0.06	0.00
55.60	9.55	2.00	0.00	0.06	0.00	55.66	8.77	2.00	0.00	0.06	0.00
55.75	8.21	2.00	0.00	0.06	0.00	55.83	8.04	2.00	0.00	0.05	0.00
55.85	7.99	2.00	0.00	0.05	0.00	55.93	7.82	2.00	0.00	0.05	0.00
55.99	8.09	2.00	0.00	0.05	0.00	56.04	8.15	2.00	0.00	0.05	0.00
56.13	8.36	2.00	0.00	0.05	0.00	56.17	8.41	2.00	0.00	0.05	0.00
56.26	8.68	2.00	0.00	0.05	0.00	56.31	8.62	2.00	0.00	0.05	0.00
56.41	8.45	2.00	0.00	0.04	0.00	56.45	8.40	2.00	0.00	0.04	0.00
56.50	8.40	2.00	0.00	0.04	0.00	56.59	8.39	2.00	0.00	0.04	0.00
56.63	8.44	2.00	0.00	0.04	0.00	56.73	8.66	2.00	0.00	0.04	0.00
56.77	8.71	2.00	0.00	0.04	0.00	56.87	8.98	2.00	0.00	0.04	0.00
56.91	9.09	2.00	0.00	0.04	0.00	56.96	9.14	2.00	0.00	0.03	0.00
57.05	9.13	2.00	0.00	0.03	0.00	57.10	8.91	2.00	0.00	0.03	0.00
57.19	8.80	2.00	0.00	0.03	0.00	57.23	8.85	2.00	0.00	0.03	0.00
57.32	8.95	2.00	0.00	0.03	0.00	57.38	9.00	2.00	0.00	0.03	0.00
57.42	9.00	2.00	0.00	0.03	0.00	57.51	9.22	2.00	0.00	0.03	0.00
57.56	9.27	2.00	0.00	0.02	0.00	57.65	9.48	2.00	0.00	0.02	0.00
57.73	9.53	2.00	0.00	0.02	0.00	57.74	9.48	2.00	0.00	0.02	0.00
57.83	9.47	2.00	0.00	0.02	0.00	57.88	9.47	2.00	0.00	0.02	0.00
57.97	9.47	2.00	0.00	0.02	0.00	58.02	9.85	2.00	0.00	0.02	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
58.10	10.72	2.00	0.00	0.02	0.00	58.15	11.82	2.00	0.00	0.01	0.00
58.25	12.26	2.00	0.00	0.01	0.00	58.29	12.81	2.00	0.00	0.01	0.00
58.34	17.44	2.00	0.00	0.01	0.00	58.43	30.16	2.00	0.00	0.01	0.00
58.48	107.46	2.00	0.00	0.01	0.00	58.57	117.72	2.00	0.00	0.01	0.00
58.66	115.53	2.00	0.00	0.01	0.00	58.70	116.06	2.00	0.00	0.01	0.00
58.76	114.44	2.00	0.00	0.00	0.00	58.84	107.61	2.00	0.00	0.00	0.00
58.89	93.07	2.00	0.00	0.00	0.00	58.94	82.86	2.00	0.00	0.00	0.00
59.08	94.74	2.00	0.00	0.00	0.00	59.17	108.55	2.00	0.00	0.00	0.00
59.21	113.19	2.00	0.00	0.00	0.00	59.27	105.10	2.00	0.00	0.00	0.00
59.37	109.26	2.00	0.00	0.00	0.00	59.40	106.29	2.00	0.00	0.00	0.00
59.45	37.83	2.00	0.00	0.00	0.00	59.53	35.86	2.00	0.00	0.00	0.00
59.59	38.38	2.00	0.00	0.00	0.00	59.69	107.85	2.00	0.00	0.00	0.00
59.76	109.72	2.00	0.00	0.00	0.00	59.82	109.54	2.00	0.00	0.00	0.00
59.87	108.96	2.00	0.00	0.00	0.00	59.91	108.27	2.00	0.00	0.00	0.00
60.01	107.79	2.00	0.00	0.00	0.00	60.05	105.37	2.00	0.00	0.00	0.00
60.14	38.45	2.00	0.00	0.00	0.00	60.19	34.59	2.00	0.00	0.00	0.00
60.24	32.73	2.00	0.00	0.00	0.00	60.33	28.75	2.00	0.00	0.00	0.00
60.37	26.79	2.00	0.00	0.00	0.00	60.46	22.40	2.00	0.00	0.00	0.00
60.52	22.27	2.00	0.00	0.00	0.00	60.60	22.26	2.00	0.00	0.00	0.00
60.65	22.14	2.00	0.00	0.00	0.00	60.70	25.91	2.00	0.00	0.00	0.00
60.79	36.87	2.00	0.00	0.00	0.00	60.85	40.57	2.00	0.00	0.00	0.00
60.93	39.16	2.00	0.00	0.00	0.00	60.96	37.64	2.00	0.00	0.00	0.00
61.06	34.20	2.00	0.00	0.00	0.00	61.11	30.46	2.00	0.00	0.00	0.00
61.16	28.91	2.00	0.00	0.00	0.00	61.25	25.65	2.00	0.00	0.00	0.00
61.30	24.31	2.00	0.00	0.00	0.00	61.39	20.08	2.00	0.00	0.00	0.00
61.43	18.77	2.00	0.00	0.00	0.00	61.53	15.67	2.00	0.00	0.00	0.00
61.57	14.61	2.00	0.00	0.00	0.00	61.62	13.07	2.00	0.00	0.00	0.00
61.71	11.59	2.00	0.00	0.00	0.00	61.76	10.73	2.00	0.00	0.00	0.00
61.84	10.45	2.00	0.00	0.00	0.00	61.90	10.55	2.00	0.00	0.00	0.00
61.95	10.82	2.00	0.00	0.00	0.00	62.03	10.65	2.00	0.00	0.00	0.00
62.08	10.54	2.00	0.00	0.00	0.00	62.17	12.32	2.00	0.00	0.00	0.00
62.22	12.96	2.00	0.00	0.00	0.00	62.30	14.49	2.00	0.00	0.00	0.00
62.36	19.40	2.00	0.00	0.00	0.00	62.41	94.36	2.00	0.00	0.00	0.00
62.49	53.94	2.00	0.00	0.00	0.00	62.59	89.38	2.00	0.00	0.00	0.00
62.63	101.58	2.00	0.00	0.00	0.00	62.68	105.91	2.00	0.00	0.00	0.00
62.77	113.74	2.00	0.00	0.00	0.00						

Total estimated settlement: 2.83

Abbreviations

$Q_{tn,cs}$:	Equivalent clean sand normalized cone resistance
FS:	Factor of safety against liquefaction
e_v (%):	Post-liquefaction volumentric strain
DF:	e_v depth weighting factor
Settlement:	Calculated settlement

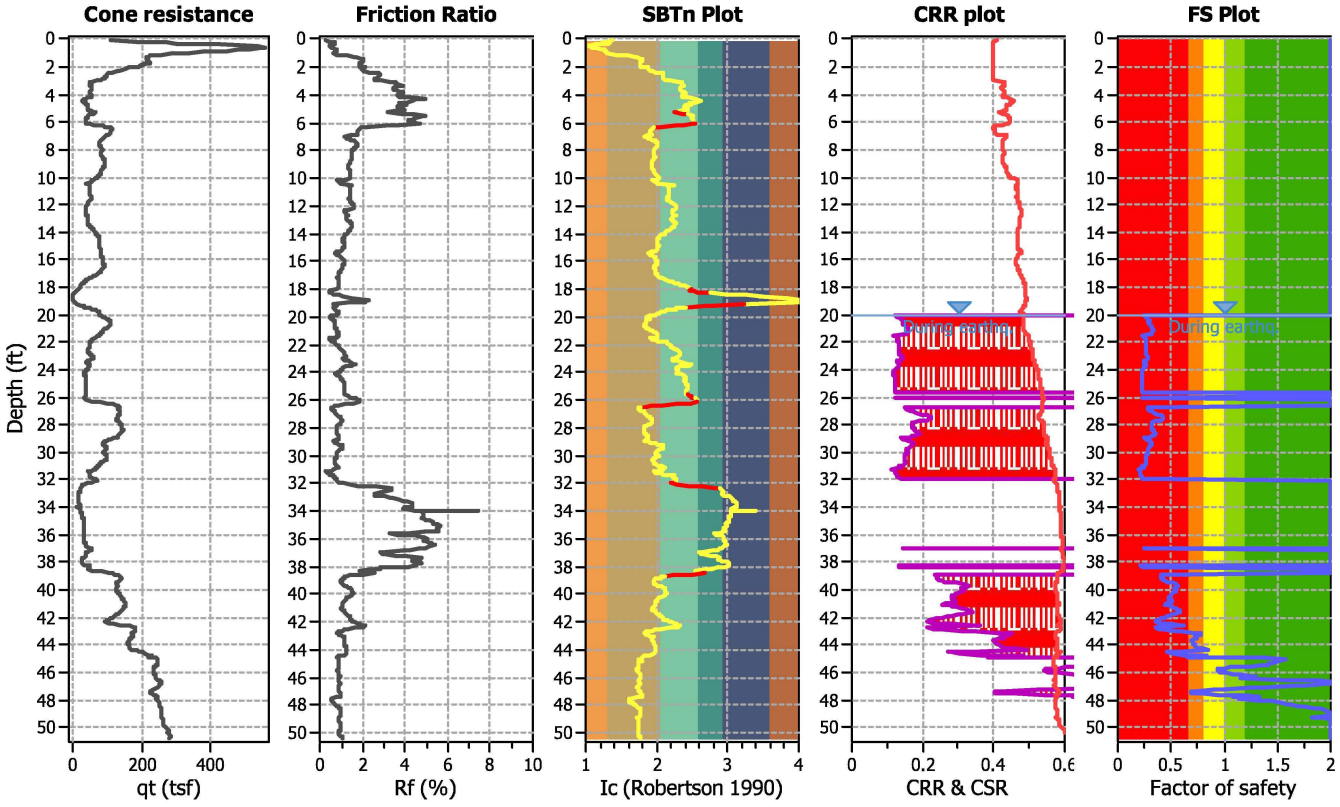
Kehoe Testing and Engineering
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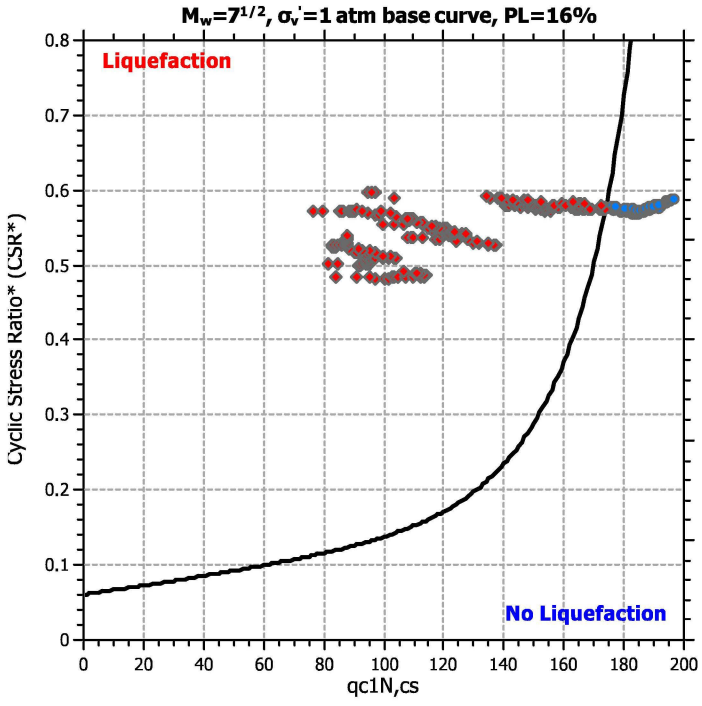
LIQUEFACTION ANALYSIS REPORT

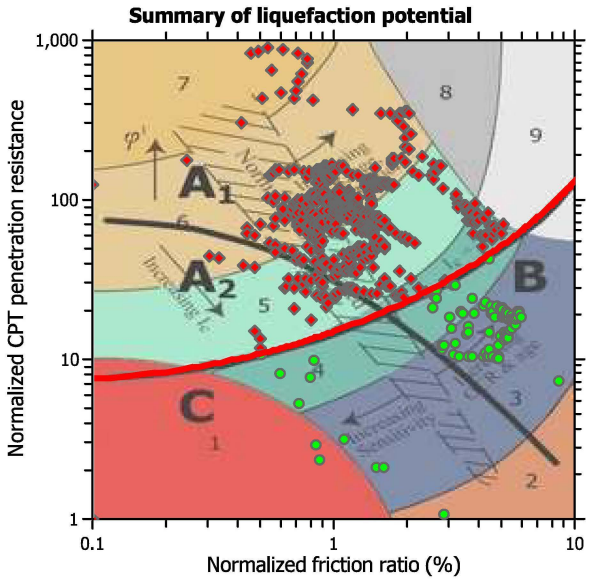
Project title : Southern California Geotechnical / Proposed Industrial Building **Location : 3800 Via Oro Ave, Long Beach, CA**
CPT file : CPT-3

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	23.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.99	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

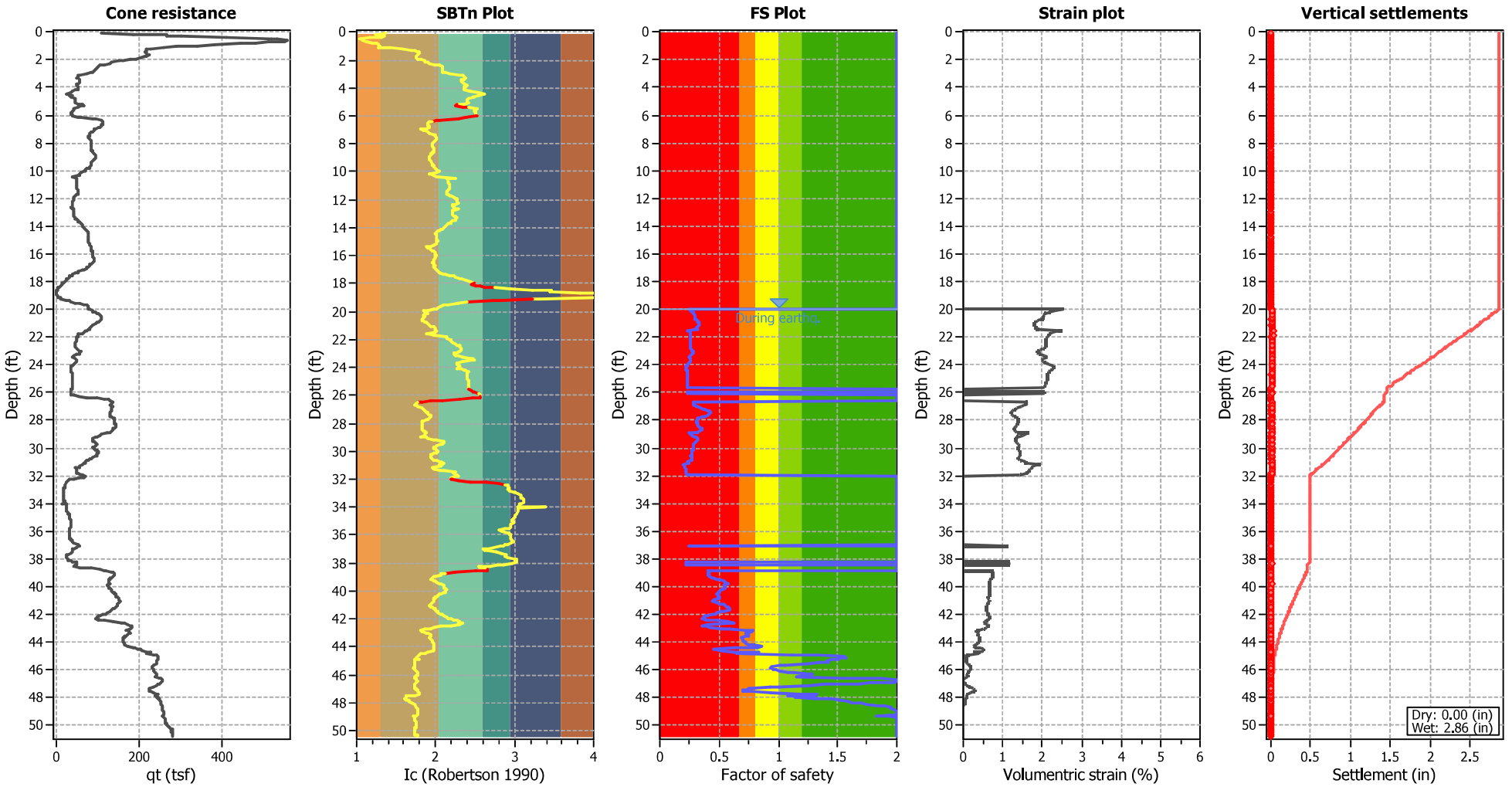






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
20.03	83.94	0.25	2.53	0.66	0.01	20.11	91.07	0.26	2.33	0.66	0.02
20.17	96.88	0.28	2.18	0.66	0.01	20.21	95.08	0.27	2.22	0.66	0.01
20.30	100.47	0.29	2.10	0.66	0.02	20.35	101.26	0.29	2.08	0.66	0.01
20.45	102.68	0.29	2.04	0.65	0.02	20.49	103.01	0.29	2.03	0.65	0.01
20.58	102.97	0.29	2.03	0.65	0.02	20.67	103.71	0.29	2.01	0.65	0.02
20.71	104.73	0.30	1.99	0.65	0.01	20.77	107.40	0.31	1.93	0.65	0.01
20.82	110.00	0.31	1.88	0.65	0.01	20.91	111.93	0.32	1.85	0.65	0.02
20.95	113.22	0.33	1.82	0.64	0.01	21.04	113.56	0.33	1.81	0.64	0.02
21.13	114.08	0.33	1.80	0.64	0.02	21.14	114.22	0.33	1.80	0.64	0.00
21.22	113.38	0.32	1.81	0.64	0.02	21.28	112.13	0.32	1.82	0.64	0.01
21.35	110.40	0.31	1.85	0.64	0.02	21.41	110.92	0.31	1.84	0.64	0.02
21.46	106.78	0.30	1.91	0.64	0.01	21.55	81.34	0.23	2.50	0.63	0.03
21.59	84.84	0.24	2.40	0.63	0.01	21.69	91.23	0.25	2.23	0.63	0.02
21.74	92.32	0.26	2.20	0.63	0.01	21.79	92.80	0.26	2.19	0.63	0.01
21.90	93.81	0.26	2.16	0.63	0.03	21.93	94.06	0.26	2.15	0.63	0.01
22.02	94.80	0.26	2.13	0.63	0.02	22.07	95.11	0.26	2.12	0.63	0.01
22.13	95.38	0.26	2.11	0.62	0.02	22.21	95.48	0.26	2.10	0.62	0.02
22.25	95.26	0.26	2.10	0.62	0.01	22.34	95.03	0.26	2.10	0.62	0.02
22.39	94.46	0.26	2.11	0.62	0.01	22.48	94.15	0.26	2.11	0.62	0.02
22.53	94.34	0.26	2.11	0.62	0.01	22.62	94.70	0.26	2.09	0.62	0.02
22.66	94.84	0.26	2.09	0.62	0.01	22.71	95.09	0.26	2.08	0.62	0.01
22.80	96.51	0.26	2.04	0.61	0.02	22.84	97.28	0.26	2.02	0.61	0.01
22.94	99.80	0.27	1.97	0.61	0.02	22.99	101.76	0.27	1.93	0.61	0.01
23.07	103.50	0.28	1.89	0.61	0.02	23.13	103.83	0.28	1.88	0.61	0.01
23.18	102.40	0.28	1.90	0.61	0.01	23.26	99.44	0.27	1.96	0.61	0.02
23.35	93.92	0.25	2.07	0.60	0.02	23.40	91.44	0.25	2.12	0.60	0.01
23.45	90.55	0.24	2.14	0.60	0.01	23.49	91.04	0.25	2.13	0.60	0.01
23.59	94.92	0.25	2.03	0.60	0.02	23.63	96.92	0.26	1.99	0.60	0.01
23.69	96.98	0.26	1.98	0.60	0.01	23.77	95.60	0.25	2.01	0.60	0.02
23.86	92.66	0.25	2.07	0.60	0.02	23.91	91.51	0.24	2.09	0.59	0.01
23.95	88.49	0.24	2.16	0.59	0.01	24.05	84.02	0.23	2.26	0.59	0.02
24.10	83.34	0.23	2.28	0.59	0.02	24.18	82.76	0.23	2.29	0.59	0.02
24.22	82.71	0.23	2.29	0.59	0.01	24.28	83.07	0.23	2.27	0.59	0.02
24.36	83.43	0.23	2.26	0.59	0.02	24.41	84.84	0.23	2.22	0.59	0.02
24.48	85.94	0.23	2.19	0.59	0.02	24.56	86.97	0.23	2.16	0.58	0.02
24.64	87.77	0.23	2.13	0.58	0.02	24.68	87.94	0.23	2.13	0.58	0.01
24.74	88.06	0.23	2.12	0.58	0.02	24.82	87.97	0.23	2.12	0.58	0.02
24.88	87.68	0.23	2.12	0.58	0.01	24.97	87.28	0.23	2.12	0.58	0.02
25.01	87.17	0.23	2.12	0.58	0.01	25.11	87.05	0.23	2.12	0.57	0.02
25.15	87.05	0.23	2.12	0.57	0.01	25.20	87.23	0.23	2.11	0.57	0.01
25.28	87.41	0.23	2.10	0.57	0.02	25.34	87.66	0.23	2.09	0.57	0.01
25.40	87.68	0.23	2.09	0.57	0.02	25.50	87.78	0.23	2.08	0.57	0.02
25.57	87.66	0.23	2.08	0.57	0.02	25.60	87.67	0.23	2.08	0.57	0.01
25.70	87.46	2.00	0.00	0.56	0.00	25.75	87.31	2.00	0.00	0.56	0.00
25.80	87.49	2.00	0.00	0.56	0.00	25.89	87.80	2.00	0.00	0.56	0.00
25.93	87.77	2.00	0.00	0.56	0.00	25.99	87.50	0.23	2.06	0.56	0.01
26.08	87.48	0.23	2.05	0.56	0.02	26.13	88.21	2.00	0.00	0.56	0.00
26.21	91.61	2.00	0.00	0.56	0.00	26.26	99.60	2.00	0.00	0.55	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
26.35	108.67	2.00	0.00	0.55	0.00	26.40	108.70	2.00	0.00	0.55	0.00
26.48	98.86	2.00	0.00	0.55	0.00	26.54	102.41	2.00	0.00	0.55	0.00
26.58	105.77	2.00	0.00	0.55	0.00	26.67	109.06	0.28	1.61	0.55	0.02
26.72	109.49	0.28	1.60	0.55	0.01	26.77	107.94	0.28	1.62	0.55	0.01
26.87	109.91	0.28	1.59	0.54	0.02	26.91	112.56	0.29	1.55	0.54	0.01
27.01	117.20	0.31	1.48	0.54	0.02	27.04	118.69	0.32	1.46	0.54	0.01
27.16	124.44	0.34	1.38	0.54	0.02	27.24	130.08	0.37	1.31	0.54	0.01
27.26	131.90	0.39	1.29	0.54	0.00	27.33	135.10	0.41	1.25	0.54	0.01
27.38	136.55	0.42	1.24	0.54	0.01	27.46	137.55	0.43	1.22	0.53	0.01
27.51	137.17	0.42	1.22	0.53	0.01	27.56	136.39	0.42	1.23	0.53	0.01
27.63	134.72	0.40	1.24	0.53	0.01	27.70	131.14	0.38	1.28	0.53	0.01
27.76	127.60	0.36	1.32	0.53	0.01	27.84	121.92	0.33	1.38	0.53	0.01
27.89	119.54	0.32	1.41	0.53	0.01	27.97	119.10	0.31	1.41	0.53	0.01
28.02	118.92	0.31	1.41	0.53	0.01	28.11	118.96	0.31	1.40	0.52	0.01
28.16	119.36	0.31	1.40	0.52	0.01	28.25	120.37	0.32	1.38	0.52	0.02
28.29	121.05	0.32	1.37	0.52	0.01	28.35	121.69	0.32	1.36	0.52	0.01
28.43	122.73	0.33	1.34	0.52	0.01	28.48	123.58	0.33	1.33	0.52	0.01
28.56	125.79	0.34	1.30	0.52	0.01	28.62	127.30	0.35	1.28	0.51	0.01
28.68	127.48	0.35	1.28	0.51	0.01	28.77	123.47	0.33	1.32	0.51	0.02
28.85	120.02	0.31	1.36	0.51	0.01	28.88	115.82	0.30	1.41	0.51	0.00
28.94	99.73	0.25	1.64	0.51	0.01	29.04	103.74	0.26	1.57	0.51	0.02
29.08	106.68	0.27	1.52	0.51	0.01	29.17	115.03	0.29	1.40	0.51	0.02
29.21	117.49	0.30	1.37	0.50	0.01	29.30	120.36	0.31	1.33	0.50	0.01
29.33	120.70	0.32	1.33	0.50	0.00	29.44	120.48	0.31	1.32	0.50	0.02
29.49	119.58	0.31	1.33	0.50	0.01	29.54	118.77	0.31	1.34	0.50	0.01
29.63	117.11	0.30	1.36	0.50	0.01	29.67	116.14	0.30	1.37	0.50	0.01
29.76	112.88	0.28	1.40	0.50	0.02	29.81	111.24	0.28	1.42	0.49	0.01
29.86	111.00	0.28	1.42	0.49	0.01	29.95	112.53	0.28	1.40	0.49	0.02
29.99	112.58	0.28	1.40	0.49	0.01	30.08	112.72	0.28	1.39	0.49	0.01
30.13	111.25	0.28	1.41	0.49	0.01	30.20	108.41	0.27	1.44	0.49	0.01
30.27	107.89	0.27	1.45	0.49	0.01	30.32	108.34	0.27	1.44	0.49	0.01
30.41	108.21	0.27	1.44	0.48	0.02	30.46	107.73	0.26	1.44	0.48	0.01
30.55	107.29	0.26	1.44	0.48	0.02	30.60	108.28	0.27	1.42	0.48	0.01
30.69	109.16	0.27	1.41	0.48	0.02	30.74	109.59	0.27	1.40	0.48	0.01
30.78	109.22	0.27	1.40	0.48	0.01	30.84	107.84	0.26	1.42	0.48	0.01
30.92	103.84	0.25	1.47	0.48	0.01	31.01	97.38	0.24	1.57	0.47	0.02
31.06	98.72	0.24	1.54	0.47	0.01	31.10	94.37	0.23	1.61	0.47	0.01
31.20	76.67	0.20	1.97	0.47	0.02	31.24	79.75	0.20	1.89	0.47	0.01
31.33	85.08	0.21	1.77	0.47	0.02	31.38	86.66	0.21	1.74	0.47	0.01
31.43	85.69	0.21	1.75	0.47	0.01	31.52	88.33	0.22	1.70	0.47	0.02
31.56	88.91	0.22	1.68	0.47	0.01	31.65	89.84	0.22	1.66	0.46	0.02
31.70	90.33	0.22	1.65	0.46	0.01	31.79	91.05	0.22	1.63	0.46	0.02
31.84	92.61	0.22	1.60	0.46	0.01	31.93	98.98	0.24	1.49	0.46	0.02
31.97	102.46	0.25	1.44	0.46	0.01	32.02	107.28	2.00	0.00	0.46	0.00
32.11	110.24	2.00	0.00	0.46	0.00	32.16	103.45	2.00	0.00	0.45	0.00
32.24	33.02	2.00	0.00	0.45	0.00	32.30	25.50	2.00	0.00	0.45	0.00
32.39	21.06	2.00	0.00	0.45	0.00	32.44	19.42	2.00	0.00	0.45	0.00
32.48	18.98	2.00	0.00	0.45	0.00	32.62	16.57	2.00	0.00	0.45	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
32.71	16.13	2.00	0.00	0.45	0.00	32.76	15.98	2.00	0.00	0.44	0.00
32.85	15.34	2.00	0.00	0.44	0.00	32.90	14.22	2.00	0.00	0.44	0.00
32.94	14.01	2.00	0.00	0.44	0.00	33.04	13.86	2.00	0.00	0.44	0.00
33.08	13.85	2.00	0.00	0.44	0.00	33.17	13.77	2.00	0.00	0.44	0.00
33.21	13.84	2.00	0.00	0.44	0.00	33.31	13.89	2.00	0.00	0.44	0.00
33.36	13.82	2.00	0.00	0.43	0.00	33.43	13.81	2.00	0.00	0.43	0.00
33.50	13.80	2.00	0.00	0.43	0.00	33.54	13.80	2.00	0.00	0.43	0.00
33.63	13.78	2.00	0.00	0.43	0.00	33.68	13.78	2.00	0.00	0.43	0.00
33.77	14.04	2.00	0.00	0.43	0.00	33.82	14.80	2.00	0.00	0.43	0.00
33.86	15.34	2.00	0.00	0.43	0.00	33.96	16.44	2.00	0.00	0.42	0.00
34.01	10.24	2.00	0.00	0.42	0.00	34.05	17.40	2.00	0.00	0.42	0.00
34.15	18.22	2.00	0.00	0.42	0.00	34.19	18.21	2.00	0.00	0.42	0.00
34.29	18.20	2.00	0.00	0.42	0.00	34.33	18.19	2.00	0.00	0.42	0.00
34.45	18.80	2.00	0.00	0.42	0.00	34.47	19.00	2.00	0.00	0.42	0.00
34.52	19.34	2.00	0.00	0.41	0.00	34.61	20.38	2.00	0.00	0.41	0.00
34.66	20.65	2.00	0.00	0.41	0.00	34.74	21.54	2.00	0.00	0.41	0.00
34.80	22.65	2.00	0.00	0.41	0.00	34.84	22.99	2.00	0.00	0.41	0.00
34.93	23.95	2.00	0.00	0.41	0.00	34.98	24.09	2.00	0.00	0.41	0.00
35.07	24.28	2.00	0.00	0.41	0.00	35.11	24.55	2.00	0.00	0.40	0.00
35.21	26.01	2.00	0.00	0.40	0.00	35.24	26.28	2.00	0.00	0.40	0.00
35.30	26.41	2.00	0.00	0.40	0.00	35.39	25.83	2.00	0.00	0.40	0.00
35.44	25.81	2.00	0.00	0.40	0.00	35.53	25.78	2.00	0.00	0.40	0.00
35.58	25.90	2.00	0.00	0.40	0.00	35.63	25.79	2.00	0.00	0.40	0.00
35.72	25.43	2.00	0.00	0.39	0.00	35.81	24.58	2.00	0.00	0.39	0.00
35.85	24.23	2.00	0.00	0.39	0.00	35.90	24.61	2.00	0.00	0.39	0.00
35.99	24.62	2.00	0.00	0.39	0.00	36.03	24.47	2.00	0.00	0.39	0.00
36.13	24.31	2.00	0.00	0.39	0.00	36.18	24.23	2.00	0.00	0.39	0.00
36.23	24.01	2.00	0.00	0.39	0.00	36.32	23.78	2.00	0.00	0.38	0.00
36.36	23.77	2.00	0.00	0.38	0.00	36.46	23.75	2.00	0.00	0.38	0.00
36.50	23.74	2.00	0.00	0.38	0.00	36.55	25.12	2.00	0.00	0.38	0.00
36.64	26.77	2.00	0.00	0.38	0.00	36.69	27.87	2.00	0.00	0.38	0.00
36.78	28.54	2.00	0.00	0.38	0.00	36.82	30.21	2.00	0.00	0.38	0.00
36.97	38.21	2.00	0.00	0.37	0.00	37.01	39.97	2.00	0.00	0.37	0.00
37.02	103.60	0.24	1.15	0.37	0.00	37.10	39.08	2.00	0.00	0.37	0.00
37.15	37.43	2.00	0.00	0.37	0.00	37.24	31.76	2.00	0.00	0.37	0.00
37.29	29.17	2.00	0.00	0.37	0.00	37.38	27.41	2.00	0.00	0.37	0.00
37.42	28.03	2.00	0.00	0.37	0.00	37.47	28.78	2.00	0.00	0.36	0.00
37.56	26.88	2.00	0.00	0.36	0.00	37.61	24.72	2.00	0.00	0.36	0.00
37.70	20.38	2.00	0.00	0.36	0.00	37.73	19.22	2.00	0.00	0.36	0.00
37.80	19.10	2.00	0.00	0.36	0.00	37.88	19.09	2.00	0.00	0.36	0.00
37.93	18.98	2.00	0.00	0.36	0.00	38.00	20.12	2.00	0.00	0.36	0.00
38.11	21.30	2.00	0.00	0.35	0.00	38.16	25.19	2.00	0.00	0.35	0.00
38.21	28.18	2.00	0.00	0.35	0.00	38.27	94.46	0.22	1.20	0.35	0.01
38.34	97.45	0.23	1.15	0.35	0.01	38.39	96.21	0.22	1.17	0.35	0.01
38.48	32.49	2.00	0.00	0.35	0.00	38.53	31.85	2.00	0.00	0.35	0.00
38.60	101.91	2.00	0.00	0.35	0.00	38.67	119.58	2.00	0.00	0.34	0.00
38.73	131.90	2.00	0.00	0.34	0.00	38.81	139.13	2.00	0.00	0.34	0.00
38.86	140.32	0.41	0.76	0.34	0.00	38.93	141.74	0.42	0.75	0.34	0.01

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
38.99	141.36	0.42	0.75	0.34	0.01	39.05	140.78	0.41	0.75	0.34	0.00
39.14	140.61	0.41	0.75	0.34	0.01	39.19	140.94	0.41	0.75	0.34	0.00
39.26	141.81	0.42	0.74	0.33	0.01	39.33	143.71	0.44	0.73	0.33	0.01
39.40	145.68	0.45	0.71	0.33	0.01	39.46	148.01	0.48	0.70	0.33	0.01
39.51	150.20	0.50	0.68	0.33	0.00	39.60	153.30	0.54	0.67	0.33	0.01
39.65	153.92	0.55	0.66	0.33	0.00	39.78	155.38	0.57	0.65	0.33	0.01
39.83	154.24	0.56	0.65	0.32	0.00	39.92	152.94	0.54	0.66	0.32	0.01
39.97	153.88	0.55	0.65	0.32	0.00	40.06	152.88	0.54	0.65	0.32	0.01
40.11	153.65	0.55	0.65	0.32	0.00	40.20	152.56	0.53	0.65	0.32	0.01
40.25	151.01	0.51	0.65	0.32	0.00	40.29	150.37	0.50	0.66	0.32	0.00
40.39	150.26	0.50	0.65	0.32	0.01	40.47	149.26	0.49	0.66	0.31	0.01
40.50	148.87	0.49	0.66	0.31	0.00	40.57	148.55	0.48	0.66	0.31	0.01
40.63	148.73	0.48	0.65	0.31	0.00	40.71	149.42	0.49	0.65	0.31	0.01
40.75	149.86	0.50	0.64	0.31	0.00	40.85	150.51	0.51	0.64	0.31	0.01
40.89	150.17	0.50	0.64	0.31	0.00	40.98	145.78	0.45	0.65	0.31	0.01
41.03	144.22	0.44	0.66	0.30	0.00	41.12	147.83	0.47	0.64	0.30	0.01
41.17	149.19	0.49	0.63	0.30	0.00	41.21	150.71	0.51	0.62	0.30	0.00
41.31	152.85	0.53	0.61	0.30	0.01	41.36	153.73	0.55	0.60	0.30	0.00
41.48	155.35	0.57	0.59	0.30	0.01	41.49	155.57	0.57	0.59	0.30	0.00
41.54	156.30	0.58	0.58	0.30	0.00	41.63	156.56	0.59	0.57	0.29	0.01
41.68	156.25	0.58	0.58	0.29	0.00	41.77	153.12	0.54	0.59	0.29	0.01
41.82	151.72	0.52	0.60	0.29	0.00	41.91	148.51	0.48	0.61	0.29	0.01
41.95	147.38	0.47	0.61	0.29	0.00	42.00	144.32	0.44	0.62	0.29	0.00
42.07	141.25	0.41	0.64	0.29	0.01	42.13	137.82	0.38	0.65	0.29	0.00
42.23	135.36	0.37	0.66	0.28	0.01	42.27	138.30	0.39	0.64	0.28	0.00
42.33	134.21	0.36	0.66	0.28	0.00	42.41	138.73	0.39	0.64	0.28	0.01
42.47	147.93	0.47	0.59	0.28	0.00	42.55	155.93	0.58	0.55	0.28	0.01
42.60	158.72	0.62	0.50	0.28	0.00	42.66	157.91	0.61	0.51	0.28	0.00
42.74	140.96	0.41	0.61	0.28	0.01	42.78	134.29	0.36	0.65	0.27	0.00
42.88	138.99	0.39	0.62	0.27	0.01	42.92	142.89	0.42	0.60	0.27	0.00
43.01	152.54	0.53	0.55	0.27	0.01	43.05	156.62	0.59	0.52	0.27	0.00
43.11	163.51	0.71	0.41	0.27	0.00	43.20	166.47	0.78	0.34	0.27	0.00
43.25	166.21	0.78	0.34	0.27	0.00	43.34	164.07	0.73	0.39	0.27	0.00
43.38	164.09	0.73	0.39	0.26	0.00	43.48	164.80	0.74	0.37	0.26	0.00
43.53	164.90	0.74	0.37	0.26	0.00	43.62	163.87	0.72	0.39	0.26	0.00
43.66	162.33	0.69	0.41	0.26	0.00	43.71	162.47	0.69	0.41	0.26	0.00
43.81	163.00	0.70	0.40	0.26	0.00	43.85	163.11	0.70	0.40	0.26	0.00
43.94	163.32	0.71	0.39	0.26	0.00	43.98	163.49	0.71	0.39	0.25	0.00
44.03	163.79	0.72	0.38	0.25	0.00	44.13	164.23	0.73	0.37	0.25	0.00
44.17	165.04	0.75	0.35	0.25	0.00	44.23	166.89	0.79	0.31	0.25	0.00
44.31	169.25	0.86	0.26	0.25	0.00	44.40	168.06	0.82	0.28	0.25	0.00
44.45	159.16	0.63	0.44	0.25	0.00	44.49	146.86	0.46	0.52	0.25	0.00
44.56	147.74	0.47	0.52	0.24	0.00	44.63	152.22	0.52	0.50	0.24	0.00
44.72	159.07	0.62	0.43	0.24	0.00	44.77	168.67	0.84	0.26	0.24	0.00
44.87	160.81	0.66	0.40	0.24	0.00	44.91	176.61	1.12	0.14	0.24	0.00
44.96	181.14	1.34	0.09	0.24	0.00	45.05	184.64	1.56	0.05	0.24	0.00
45.09	184.82	1.57	0.05	0.24	0.00	45.19	183.95	1.51	0.06	0.23	0.00
45.23	183.89	1.51	0.06	0.23	0.00	45.28	183.07	1.45	0.07	0.23	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
45.37	182.36	1.41	0.07	0.23	0.00	45.42	182.56	1.42	0.07	0.23	0.00
45.51	181.45	1.35	0.08	0.23	0.00	45.56	179.54	1.25	0.10	0.23	0.00
45.65	175.27	1.06	0.15	0.23	0.00	45.70	174.22	1.02	0.17	0.23	0.00
45.75	172.26	0.95	0.19	0.22	0.00	45.83	172.02	0.94	0.20	0.22	0.00
45.88	172.09	0.94	0.19	0.22	0.00	45.97	172.75	0.96	0.18	0.22	0.00
46.00	173.36	0.98	0.17	0.22	0.00	46.06	174.02	1.01	0.16	0.22	0.00
46.15	175.97	1.08	0.14	0.22	0.00	46.20	177.07	1.13	0.13	0.22	0.00
46.30	179.53	1.25	0.10	0.22	0.00	46.33	180.32	1.29	0.09	0.21	0.00
46.43	178.91	1.22	0.10	0.21	0.00	46.48	177.57	1.15	0.12	0.21	0.00
46.57	184.54	1.54	0.05	0.21	0.00	46.62	188.85	1.87	0.01	0.21	0.00
46.66	192.00	2.00	0.00	0.21	0.00	46.75	192.64	2.00	0.00	0.21	0.00
46.80	192.08	2.00	0.00	0.21	0.00	46.89	190.10	1.97	0.00	0.21	0.00
46.94	187.67	1.77	0.02	0.20	0.00	46.98	186.53	1.68	0.03	0.20	0.00
47.07	182.23	1.39	0.07	0.20	0.00	47.12	180.20	1.28	0.09	0.20	0.00
47.21	173.95	1.00	0.15	0.20	0.00	47.26	171.80	0.93	0.18	0.20	0.00
47.39	165.41	0.75	0.27	0.20	0.00	47.42	164.94	0.74	0.28	0.20	0.00
47.45	163.05	0.70	0.30	0.20	0.00	47.53	163.00	0.69	0.30	0.19	0.00
47.58	166.46	0.77	0.25	0.19	0.00	47.66	172.12	0.94	0.17	0.19	0.00
47.72	178.18	1.18	0.10	0.19	0.00	47.77	181.04	1.32	0.07	0.19	0.00
47.84	176.04	1.08	0.12	0.19	0.00	47.93	180.93	1.31	0.07	0.19	0.00
47.98	177.37	1.14	0.11	0.19	0.00	48.05	180.87	1.31	0.07	0.19	0.00
48.12	183.12	1.44	0.05	0.18	0.00	48.17	184.39	1.52	0.04	0.18	0.00
48.29	185.44	1.60	0.04	0.18	0.00	48.30	185.43	1.60	0.04	0.18	0.00
48.40	185.75	1.62	0.03	0.18	0.00	48.44	186.87	1.70	0.02	0.18	0.00
48.53	187.36	1.74	0.02	0.18	0.00	48.58	188.61	1.83	0.01	0.18	0.00
48.63	189.40	1.90	0.01	0.18	0.00	48.70	189.95	1.95	0.00	0.17	0.00
48.76	190.01	1.95	0.00	0.17	0.00	48.84	190.15	1.96	0.00	0.17	0.00
48.90	190.36	1.98	0.00	0.17	0.00	48.97	191.29	2.00	0.00	0.17	0.00
49.09	190.72	2.00	0.00	0.17	0.00	49.11	190.86	2.00	0.00	0.17	0.00
49.18	191.19	2.00	0.00	0.17	0.00	49.22	191.22	2.00	0.00	0.17	0.00
49.32	188.57	1.83	0.01	0.16	0.00	49.36	189.78	1.93	0.00	0.16	0.00
49.42	191.99	2.00	0.00	0.16	0.00	49.50	194.09	2.00	0.00	0.16	0.00
49.55	194.36	2.00	0.00	0.16	0.00	49.62	195.04	2.00	0.00	0.16	0.00
49.69	195.37	2.00	0.00	0.16	0.00	49.78	195.94	2.00	0.00	0.16	0.00
49.82	195.54	2.00	0.00	0.16	0.00	49.88	196.16	2.00	0.00	0.15	0.00
49.96	196.81	2.00	0.00	0.15	0.00	50.01	198.96	2.00	0.00	0.15	0.00
50.10	201.50	2.00	0.00	0.15	0.00	50.14	202.90	2.00	0.00	0.15	0.00
50.24	206.89	2.00	0.00	0.15	0.00	50.27	208.42	2.00	0.00	0.15	0.00
50.33	205.36	2.00	0.00	0.15	0.00	50.41	208.58	2.00	0.00	0.15	0.00
50.47	206.82	2.00	0.00	0.14	0.00	50.54	208.17	2.00	0.00	0.14	0.00
50.59	209.61	2.00	0.00	0.14	0.00	50.67	210.61	2.00	0.00	0.14	0.00
50.73	210.59	2.00	0.00	0.14	0.00	50.80	209.86	2.00	0.00	0.14	0.00

Total estimated settlement: 2.86

Abbreviations

$Q_{tn,cs}$:	Equivalent clean sand normalized cone resistance
FS:	Factor of safety against liquefaction
e_v (%):	Post-liquefaction volumentric strain
DF:	e_v depth weighting factor
Settlement:	Calculated settlement

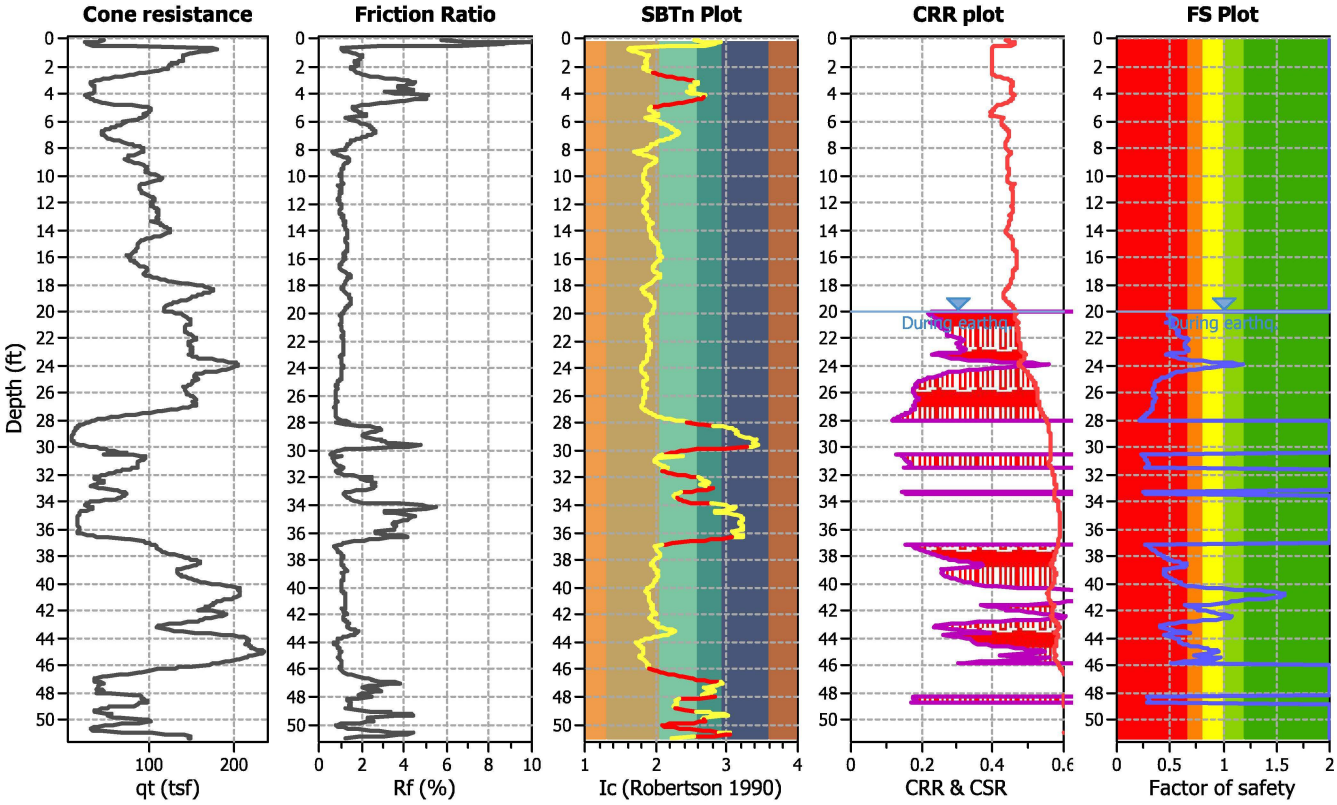
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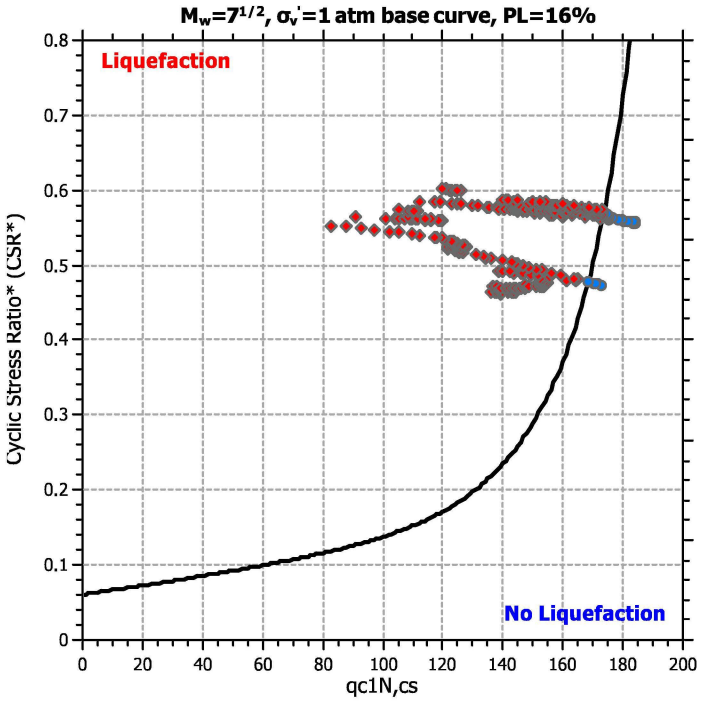
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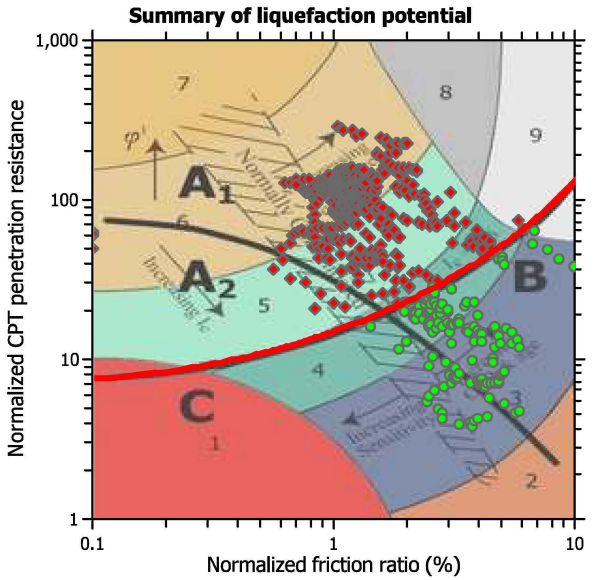
Project title : Southern California Geotechnical / Proposed Industrial Building **Location : 3800 Via Oro Ave, Long Beach, CA**
CPT file : CPT-4

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	24.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.99	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

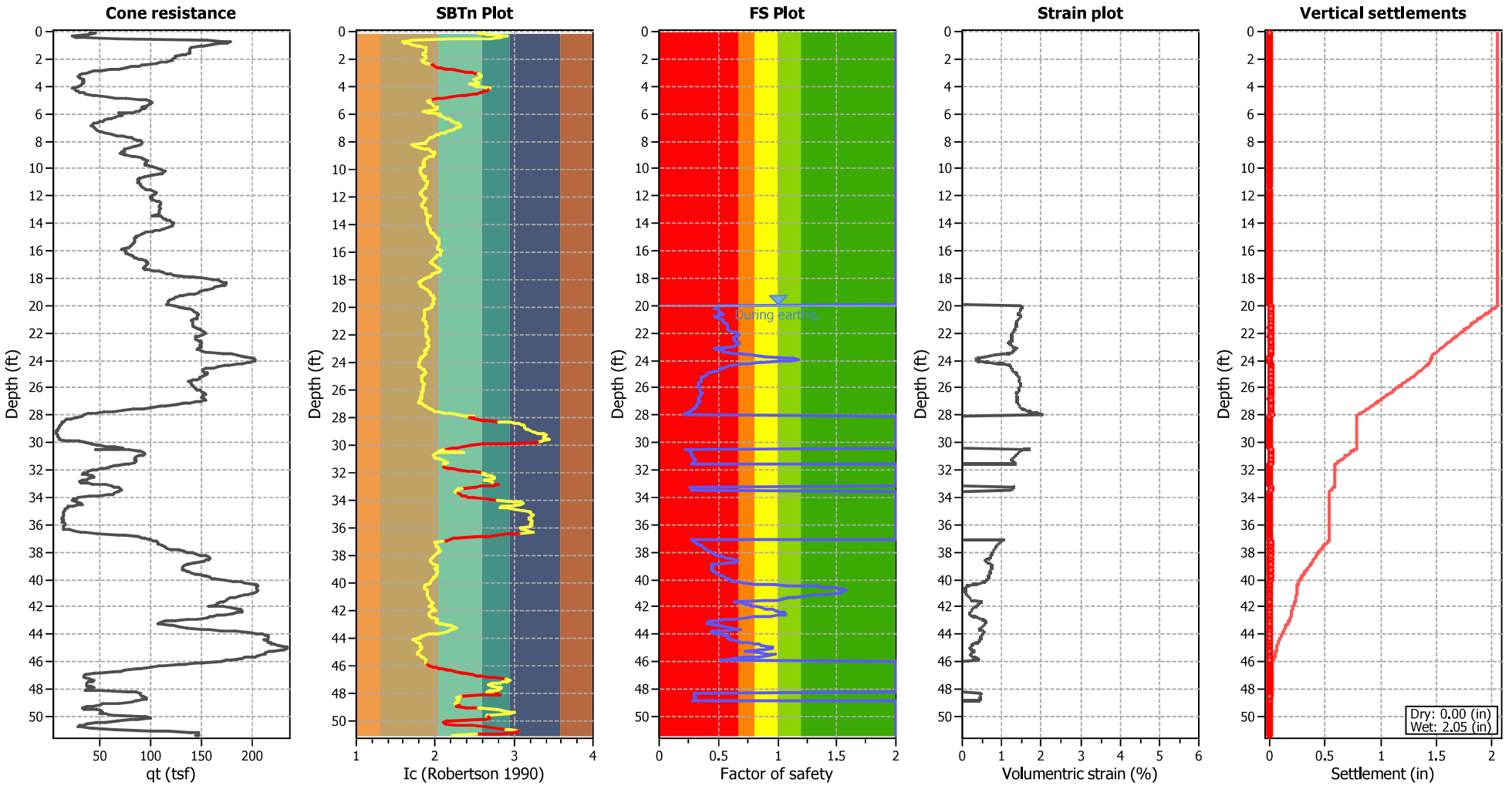






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
20.05	138.98	0.50	1.49	0.66	0.02	20.09	137.56	0.48	1.51	0.66	0.01
20.19	136.10	0.47	1.52	0.66	0.02	20.24	137.89	0.49	1.50	0.66	0.01
20.28	139.02	0.50	1.48	0.66	0.01	20.37	139.74	0.50	1.47	0.65	0.02
20.41	140.09	0.51	1.47	0.65	0.01	20.51	141.78	0.52	1.44	0.65	0.02
20.55	142.75	0.53	1.43	0.65	0.01	20.61	143.90	0.55	1.42	0.65	0.01
20.69	143.43	0.54	1.42	0.65	0.01	20.79	136.57	0.47	1.49	0.65	0.02
20.83	138.11	0.48	1.47	0.65	0.01	20.88	139.11	0.49	1.46	0.65	0.01
20.97	140.77	0.51	1.44	0.64	0.02	21.01	142.30	0.52	1.42	0.64	0.01
21.10	143.59	0.54	1.40	0.64	0.01	21.14	143.65	0.54	1.40	0.64	0.01
21.20	144.06	0.54	1.39	0.64	0.01	21.29	145.28	0.55	1.38	0.64	0.02
21.34	146.29	0.57	1.36	0.64	0.01	21.43	147.47	0.58	1.35	0.64	0.01
21.47	147.43	0.58	1.35	0.64	0.01	21.57	147.12	0.57	1.35	0.63	0.02
21.60	147.17	0.57	1.34	0.63	0.01	21.70	147.14	0.57	1.34	0.63	0.02
21.75	147.45	0.58	1.34	0.63	0.01	21.79	148.60	0.59	1.32	0.63	0.01
21.88	151.09	0.63	1.30	0.63	0.01	21.93	152.44	0.65	1.28	0.63	0.01
21.99	152.99	0.66	1.27	0.63	0.01	22.07	153.63	0.67	1.26	0.63	0.01
22.12	153.83	0.67	1.24	0.63	0.01	22.21	153.39	0.66	1.26	0.62	0.01
22.26	152.96	0.65	1.26	0.62	0.01	22.32	151.58	0.63	1.27	0.62	0.01
22.38	152.08	0.64	1.27	0.62	0.01	22.45	152.44	0.64	1.26	0.62	0.01
22.51	152.44	0.64	1.26	0.62	0.01	22.57	152.34	0.64	1.26	0.62	0.01
22.66	153.28	0.65	1.25	0.62	0.01	22.72	154.73	0.68	1.18	0.61	0.01
22.81	154.74	0.68	1.18	0.61	0.01	22.86	154.13	0.67	1.23	0.61	0.01
22.91	153.03	0.65	1.24	0.61	0.01	22.99	152.52	0.64	1.24	0.61	0.01
23.06	143.59	0.52	1.33	0.61	0.01	23.13	138.84	0.47	1.38	0.61	0.01
23.18	139.86	0.48	1.36	0.61	0.01	23.23	142.61	0.50	1.33	0.61	0.01
23.31	145.84	0.54	1.30	0.60	0.01	23.36	146.43	0.55	1.29	0.60	0.01
23.44	149.05	0.58	1.26	0.60	0.01	23.51	151.90	0.62	1.23	0.60	0.01
23.56	155.60	0.68	1.12	0.60	0.01	23.64	161.05	0.80	0.78	0.60	0.01
23.69	164.25	0.88	0.63	0.60	0.00	23.79	170.47	1.08	0.40	0.60	0.00
23.83	171.64	1.13	0.36	0.60	0.00	23.93	173.00	1.18	0.32	0.59	0.00
24.02	171.09	1.10	0.38	0.59	0.00	24.07	168.75	1.01	0.46	0.59	0.00
24.14	164.84	0.89	0.61	0.59	0.00	24.16	163.45	0.85	0.67	0.59	0.00
24.25	159.15	0.75	0.88	0.59	0.01	24.30	156.10	0.68	1.08	0.59	0.01
24.36	152.75	0.63	1.19	0.59	0.01	24.41	150.86	0.60	1.21	0.59	0.01
24.48	148.39	0.56	1.23	0.59	0.01	24.54	146.56	0.54	1.24	0.58	0.01
24.62	144.74	0.51	1.26	0.58	0.01	24.71	144.40	0.51	1.26	0.58	0.01
24.76	142.95	0.49	1.27	0.58	0.01	24.84	139.74	0.46	1.30	0.58	0.01
24.89	136.24	0.43	1.34	0.58	0.01	24.94	134.22	0.41	1.36	0.58	0.01
25.03	133.40	0.41	1.36	0.58	0.01	25.09	131.27	0.39	1.38	0.57	0.01
25.17	126.93	0.37	1.43	0.57	0.01	25.21	125.63	0.36	1.45	0.57	0.01
25.26	124.94	0.35	1.45	0.57	0.01	25.35	124.40	0.35	1.46	0.57	0.01
25.40	124.71	0.35	1.45	0.57	0.01	25.47	126.26	0.36	1.43	0.57	0.01
25.54	126.18	0.36	1.43	0.57	0.01	25.63	121.78	0.34	1.48	0.57	0.02
25.68	121.42	0.33	1.48	0.56	0.01	25.73	121.51	0.33	1.48	0.56	0.01
25.82	122.03	0.34	1.47	0.56	0.02	25.86	121.93	0.34	1.47	0.56	0.01
25.96	121.80	0.33	1.46	0.56	0.02	26.00	122.68	0.34	1.45	0.56	0.01
26.09	124.22	0.35	1.43	0.56	0.02	26.14	124.07	0.34	1.43	0.56	0.01
26.18	123.83	0.34	1.43	0.56	0.01	26.25	124.41	0.35	1.42	0.56	0.01

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
26.32	126.04	0.35	1.39	0.55	0.01	26.39	128.19	0.37	1.37	0.55	0.01
26.46	127.45	0.36	1.37	0.55	0.01	26.52	126.55	0.36	1.38	0.55	0.01
26.60	124.85	0.35	1.40	0.55	0.01	26.65	124.18	0.34	1.40	0.55	0.01
26.74	123.03	0.34	1.41	0.55	0.02	26.77	123.02	0.34	1.41	0.55	0.01
26.87	123.06	0.34	1.41	0.54	0.02	26.91	122.64	0.33	1.41	0.54	0.01
26.97	122.49	0.33	1.41	0.54	0.01	27.04	123.59	0.34	1.39	0.54	0.01
27.11	123.20	0.34	1.40	0.54	0.01	27.17	122.88	0.33	1.40	0.54	0.01
27.25	120.32	0.32	1.42	0.54	0.01	27.34	119.65	0.32	1.43	0.54	0.02
27.38	118.10	0.31	1.45	0.54	0.01	27.43	116.95	0.31	1.46	0.54	0.01
27.52	112.33	0.29	1.52	0.53	0.02	27.57	109.66	0.28	1.56	0.53	0.01
27.64	105.61	0.27	1.61	0.53	0.01	27.70	101.89	0.26	1.67	0.53	0.01
27.77	97.42	0.25	1.75	0.53	0.01	27.84	92.86	0.23	1.83	0.53	0.02
27.90	87.90	0.22	1.93	0.53	0.01	27.98	82.56	0.21	2.04	0.53	0.02
28.02	80.00	2.00	0.00	0.53	0.00	28.14	21.25	2.00	0.00	0.52	0.00
28.17	19.97	2.00	0.00	0.52	0.00	28.25	16.70	2.00	0.00	0.52	0.00
28.30	13.82	2.00	0.00	0.52	0.00	28.36	12.75	2.00	0.00	0.52	0.00
28.41	11.83	2.00	0.00	0.52	0.00	28.50	10.92	2.00	0.00	0.52	0.00
28.54	10.49	2.00	0.00	0.52	0.00	28.63	9.79	2.00	0.00	0.51	0.00
28.68	9.23	2.00	0.00	0.51	0.00	28.75	8.87	2.00	0.00	0.51	0.00
28.82	8.25	2.00	0.00	0.51	0.00	28.87	7.76	2.00	0.00	0.51	0.00
29.00	7.00	2.00	0.00	0.51	0.00	29.01	6.79	2.00	0.00	0.51	0.00
29.09	6.37	2.00	0.00	0.51	0.00	29.14	5.96	2.00	0.00	0.51	0.00
29.23	5.82	2.00	0.00	0.50	0.00	29.28	6.09	2.00	0.00	0.50	0.00
29.36	6.22	2.00	0.00	0.50	0.00	29.42	6.42	2.00	0.00	0.50	0.00
29.49	6.56	2.00	0.00	0.50	0.00	29.56	6.76	2.00	0.00	0.50	0.00
29.60	6.96	2.00	0.00	0.50	0.00	29.69	7.64	2.00	0.00	0.50	0.00
29.74	8.19	2.00	0.00	0.50	0.00	29.83	12.26	2.00	0.00	0.49	0.00
29.88	15.23	2.00	0.00	0.49	0.00	29.92	21.37	2.00	0.00	0.49	0.00
30.00	83.40	2.00	0.00	0.49	0.00	30.06	86.12	2.00	0.00	0.49	0.00
30.15	88.82	2.00	0.00	0.49	0.00	30.20	88.91	2.00	0.00	0.49	0.00
30.27	89.48	2.00	0.00	0.49	0.00	30.34	94.52	2.00	0.00	0.49	0.00
30.39	97.10	2.00	0.00	0.48	0.00	30.47	90.72	0.22	1.71	0.48	0.02
30.52	101.02	0.25	1.54	0.48	0.01	30.61	103.93	0.25	1.49	0.48	0.02
30.65	105.47	0.26	1.46	0.48	0.01	30.71	107.52	0.26	1.43	0.48	0.01
30.80	108.88	0.27	1.41	0.48	0.01	30.88	109.86	0.27	1.39	0.48	0.01
30.93	110.80	0.27	1.37	0.48	0.01	30.97	111.57	0.28	1.36	0.48	0.01
31.06	114.39	0.29	1.32	0.47	0.01	31.11	116.10	0.29	1.30	0.47	0.01
31.21	119.49	0.30	1.26	0.47	0.01	31.25	118.85	0.30	1.26	0.47	0.01
31.35	118.39	0.30	1.26	0.47	0.01	31.39	116.17	0.29	1.29	0.47	0.01
31.43	114.09	0.28	1.31	0.47	0.01	31.53	108.44	0.26	1.38	0.47	0.02
31.57	106.29	2.00	0.00	0.46	0.00	31.67	102.41	2.00	0.00	0.46	0.00
31.71	102.37	2.00	0.00	0.46	0.00	31.76	101.58	2.00	0.00	0.46	0.00
31.85	101.12	2.00	0.00	0.46	0.00	31.89	99.01	2.00	0.00	0.46	0.00
31.99	35.50	2.00	0.00	0.46	0.00	32.04	32.04	2.00	0.00	0.46	0.00
32.10	28.86	2.00	0.00	0.46	0.00	32.17	27.50	2.00	0.00	0.45	0.00
32.22	25.74	2.00	0.00	0.45	0.00	32.31	24.53	2.00	0.00	0.45	0.00
32.36	25.30	2.00	0.00	0.45	0.00	32.46	32.56	2.00	0.00	0.45	0.00
32.50	33.55	2.00	0.00	0.45	0.00	32.55	32.97	2.00	0.00	0.45	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
32.64	28.48	2.00	0.00	0.45	0.00	32.69	27.19	2.00	0.00	0.45	0.00
32.78	24.36	2.00	0.00	0.44	0.00	32.83	22.12	2.00	0.00	0.44	0.00
32.91	23.21	2.00	0.00	0.44	0.00	32.97	87.52	2.00	0.00	0.44	0.00
33.02	90.89	2.00	0.00	0.44	0.00	33.10	98.89	2.00	0.00	0.44	0.00
33.15	102.29	2.00	0.00	0.44	0.00	33.23	105.23	0.25	1.33	0.44	0.01
33.29	108.33	0.26	1.29	0.44	0.01	33.34	109.51	0.26	1.27	0.43	0.01
33.43	111.51	0.27	1.24	0.43	0.01	33.47	110.66	0.27	1.25	0.43	0.01
33.56	110.43	2.00	0.00	0.43	0.00	33.61	109.56	2.00	0.00	0.43	0.00
33.69	108.39	2.00	0.00	0.43	0.00	33.75	107.31	2.00	0.00	0.43	0.00
33.80	105.18	2.00	0.00	0.43	0.00	33.89	35.52	2.00	0.00	0.43	0.00
33.93	29.94	2.00	0.00	0.42	0.00	34.03	21.15	2.00	0.00	0.42	0.00
34.07	19.70	2.00	0.00	0.42	0.00	34.13	17.77	2.00	0.00	0.42	0.00
34.21	16.26	2.00	0.00	0.42	0.00	34.27	17.00	2.00	0.00	0.42	0.00
34.35	20.33	2.00	0.00	0.42	0.00	34.39	22.51	2.00	0.00	0.42	0.00
34.48	23.94	2.00	0.00	0.42	0.00	34.53	20.77	2.00	0.00	0.41	0.00
34.62	17.42	2.00	0.00	0.41	0.00	34.67	14.04	2.00	0.00	0.41	0.00
34.72	13.36	2.00	0.00	0.41	0.00	34.82	11.82	2.00	0.00	0.41	0.00
34.87	11.61	2.00	0.00	0.41	0.00	34.92	11.28	2.00	0.00	0.41	0.00
35.02	10.61	2.00	0.00	0.41	0.00	35.05	10.40	2.00	0.00	0.41	0.00
35.12	10.27	2.00	0.00	0.40	0.00	35.19	10.33	2.00	0.00	0.40	0.00
35.25	10.65	2.00	0.00	0.40	0.00	35.33	10.58	2.00	0.00	0.40	0.00
35.37	10.31	2.00	0.00	0.40	0.00	35.47	9.78	2.00	0.00	0.40	0.00
35.51	9.71	2.00	0.00	0.40	0.00	35.60	9.83	2.00	0.00	0.40	0.00
35.65	9.96	2.00	0.00	0.40	0.00	35.72	9.96	2.00	0.00	0.39	0.00
35.79	9.88	2.00	0.00	0.39	0.00	35.83	10.01	2.00	0.00	0.39	0.00
35.92	11.12	2.00	0.00	0.39	0.00	35.97	11.64	2.00	0.00	0.39	0.00
36.06	11.31	2.00	0.00	0.39	0.00	36.11	10.84	2.00	0.00	0.39	0.00
36.16	10.44	2.00	0.00	0.39	0.00	36.25	10.44	2.00	0.00	0.39	0.00
36.29	10.43	2.00	0.00	0.38	0.00	36.38	13.91	2.00	0.00	0.38	0.00
36.44	17.37	2.00	0.00	0.38	0.00	36.52	22.58	2.00	0.00	0.38	0.00
36.57	25.83	2.00	0.00	0.38	0.00	36.62	93.55	2.00	0.00	0.38	0.00
36.71	107.67	2.00	0.00	0.38	0.00	36.76	110.68	2.00	0.00	0.38	0.00
36.89	117.69	2.00	0.00	0.37	0.00	36.93	119.02	2.00	0.00	0.37	0.00
36.95	119.84	2.00	0.00	0.37	0.00	37.04	109.39	2.00	0.00	0.37	0.00
37.08	111.99	0.27	1.06	0.37	0.01	37.17	117.34	0.28	1.01	0.37	0.01
37.21	119.03	0.29	0.99	0.37	0.01	37.31	122.69	0.31	0.95	0.37	0.01
37.37	124.51	0.31	0.94	0.37	0.01	37.41	125.78	0.32	0.92	0.37	0.00
37.50	129.67	0.34	0.89	0.36	0.01	37.55	131.49	0.35	0.87	0.36	0.00
37.64	134.52	0.37	0.85	0.36	0.01	37.69	135.67	0.38	0.84	0.36	0.00
37.78	137.91	0.39	0.82	0.36	0.01	37.81	138.73	0.40	0.81	0.36	0.00
37.87	140.35	0.41	0.80	0.36	0.01	37.97	142.47	0.43	0.78	0.36	0.01
38.01	143.49	0.44	0.78	0.36	0.00	38.06	144.78	0.45	0.77	0.35	0.00
38.15	146.93	0.47	0.75	0.35	0.01	38.20	147.98	0.48	0.74	0.35	0.00
38.29	150.76	0.52	0.72	0.35	0.01	38.33	152.26	0.54	0.71	0.35	0.00
38.42	155.28	0.58	0.70	0.35	0.01	38.46	156.70	0.60	0.67	0.35	0.00
38.56	159.69	0.65	0.60	0.35	0.01	38.61	159.97	0.66	0.60	0.35	0.00
38.65	160.03	0.66	0.59	0.34	0.00	38.75	159.14	0.64	0.61	0.34	0.01
38.79	155.98	0.59	0.68	0.34	0.00	38.88	147.08	0.47	0.72	0.34	0.01

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
38.92	144.05	0.44	0.74	0.34	0.00	38.98	144.52	0.45	0.73	0.34	0.00
39.07	145.17	0.45	0.73	0.34	0.01	39.12	145.05	0.45	0.73	0.34	0.00
39.18	144.65	0.45	0.73	0.34	0.01	39.27	144.53	0.45	0.72	0.33	0.01
39.32	144.96	0.45	0.72	0.33	0.00	39.41	145.54	0.45	0.71	0.33	0.01
39.45	146.35	0.46	0.71	0.33	0.00	39.51	147.47	0.47	0.70	0.33	0.00
39.59	149.15	0.49	0.69	0.33	0.01	39.64	150.22	0.51	0.68	0.33	0.00
39.73	152.18	0.53	0.67	0.33	0.01	39.78	153.01	0.54	0.66	0.33	0.00
39.87	154.17	0.56	0.65	0.32	0.01	39.92	155.39	0.58	0.64	0.32	0.00
39.97	156.57	0.59	0.63	0.32	0.00	40.06	157.06	0.60	0.61	0.32	0.01
40.11	157.47	0.61	0.60	0.32	0.00	40.20	160.47	0.66	0.54	0.32	0.01
40.23	162.87	0.71	0.49	0.32	0.00	40.32	167.59	0.83	0.36	0.32	0.00
40.38	172.22	0.97	0.26	0.32	0.00	40.43	175.62	1.10	0.20	0.31	0.00
40.52	180.60	1.34	0.12	0.31	0.00	40.56	181.53	1.39	0.10	0.31	0.00
40.62	182.79	1.47	0.09	0.31	0.00	40.70	183.99	1.55	0.07	0.31	0.00
40.75	184.51	1.58	0.06	0.31	0.00	40.82	184.43	1.57	0.06	0.31	0.00
40.88	184.04	1.55	0.07	0.31	0.00	40.98	183.88	1.54	0.07	0.31	0.00
41.03	183.53	1.51	0.08	0.30	0.00	41.12	181.70	1.40	0.10	0.30	0.00
41.16	180.08	1.30	0.12	0.30	0.00	41.21	178.82	1.24	0.14	0.30	0.00
41.30	176.70	1.14	0.17	0.30	0.00	41.35	175.50	1.09	0.19	0.30	0.00
41.41	174.37	1.04	0.21	0.30	0.00	41.48	173.45	1.01	0.23	0.30	0.00
41.54	170.11	0.89	0.29	0.30	0.00	41.62	159.66	0.64	0.51	0.29	0.00
41.67	161.21	0.67	0.49	0.29	0.00	41.76	163.12	0.71	0.45	0.29	0.00
41.80	164.41	0.74	0.41	0.29	0.00	41.89	165.97	0.78	0.37	0.29	0.00
41.93	165.57	0.77	0.38	0.29	0.00	42.01	167.93	0.83	0.33	0.29	0.00
42.10	169.64	0.88	0.29	0.29	0.00	42.14	170.44	0.90	0.27	0.29	0.00
42.21	172.08	0.95	0.24	0.28	0.00	42.26	173.02	0.99	0.22	0.28	0.00
42.33	173.72	1.01	0.21	0.28	0.00	42.42	174.87	1.06	0.19	0.28	0.00
42.46	175.28	1.07	0.18	0.28	0.00	42.55	175.05	1.06	0.19	0.28	0.00
42.60	173.63	1.01	0.21	0.28	0.00	42.66	171.82	0.94	0.24	0.28	0.00
42.74	167.35	0.81	0.33	0.28	0.00	42.79	164.42	0.74	0.39	0.27	0.00
42.88	158.73	0.62	0.49	0.27	0.01	42.93	154.60	0.56	0.55	0.27	0.00
43.02	149.49	0.49	0.56	0.27	0.01	43.06	143.51	0.43	0.59	0.27	0.00
43.11	141.17	0.41	0.60	0.27	0.00	43.20	140.01	0.40	0.60	0.27	0.01
43.25	141.48	0.41	0.59	0.27	0.00	43.34	148.58	0.48	0.56	0.27	0.01
43.38	150.97	0.51	0.55	0.26	0.00	43.44	154.59	0.56	0.53	0.26	0.00
43.52	158.31	0.61	0.48	0.26	0.00	43.62	161.97	0.68	0.42	0.26	0.00
43.66	159.75	0.64	0.45	0.26	0.00	43.71	154.05	0.55	0.52	0.26	0.00
43.80	144.91	0.44	0.56	0.26	0.01	43.85	145.05	0.44	0.55	0.26	0.00
43.94	149.99	0.50	0.53	0.26	0.01	43.98	152.06	0.52	0.52	0.25	0.00
44.07	154.66	0.56	0.51	0.25	0.01	44.12	156.30	0.58	0.49	0.25	0.00
44.17	156.66	0.59	0.49	0.25	0.00	44.26	156.46	0.58	0.49	0.25	0.01
44.31	156.97	0.59	0.48	0.25	0.00	44.37	157.31	0.60	0.47	0.25	0.00
44.42	158.24	0.61	0.45	0.25	0.00	44.51	160.94	0.66	0.41	0.25	0.00
44.55	162.52	0.69	0.39	0.24	0.00	44.64	165.85	0.77	0.32	0.24	0.00
44.69	167.47	0.81	0.29	0.24	0.00	44.76	168.77	0.84	0.26	0.24	0.00
44.83	169.37	0.86	0.25	0.24	0.00	44.92	171.81	0.94	0.21	0.24	0.00
44.97	172.38	0.95	0.20	0.24	0.00	45.02	172.48	0.96	0.20	0.24	0.00
45.11	170.35	0.89	0.23	0.24	0.00	45.15	169.10	0.85	0.25	0.23	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
45.27	164.13	0.72	0.34	0.23	0.01	45.29	164.46	0.73	0.34	0.23	0.00
45.38	168.90	0.84	0.25	0.23	0.00	45.43	171.09	0.91	0.21	0.23	0.00
45.47	173.05	0.98	0.18	0.23	0.00	45.57	171.34	0.92	0.21	0.23	0.00
45.61	168.84	0.84	0.25	0.23	0.00	45.70	163.81	0.72	0.34	0.23	0.00
45.75	159.68	0.63	0.39	0.22	0.00	45.84	154.48	0.55	0.45	0.22	0.01
45.89	152.11	0.52	0.45	0.22	0.00	45.94	149.76	2.00	0.00	0.22	0.00
46.07	148.94	2.00	0.00	0.22	0.00	46.09	148.62	2.00	0.00	0.22	0.00
46.16	146.12	2.00	0.00	0.22	0.00	46.21	144.19	2.00	0.00	0.22	0.00
46.30	137.51	2.00	0.00	0.22	0.00	46.33	135.17	2.00	0.00	0.21	0.00
46.40	130.62	2.00	0.00	0.21	0.00	46.48	125.22	2.00	0.00	0.21	0.00
46.53	123.04	2.00	0.00	0.21	0.00	46.66	114.03	2.00	0.00	0.21	0.00
46.68	111.03	2.00	0.00	0.21	0.00	46.72	108.08	2.00	0.00	0.21	0.00
46.81	33.27	2.00	0.00	0.21	0.00	46.86	30.41	2.00	0.00	0.21	0.00
46.95	23.84	2.00	0.00	0.20	0.00	46.99	22.08	2.00	0.00	0.20	0.00
47.09	22.12	2.00	0.00	0.20	0.00	47.14	22.09	2.00	0.00	0.20	0.00
47.19	22.13	2.00	0.00	0.20	0.00	47.27	26.31	2.00	0.00	0.20	0.00
47.33	29.03	2.00	0.00	0.20	0.00	47.41	28.59	2.00	0.00	0.20	0.00
47.52	25.48	2.00	0.00	0.19	0.00	47.53	24.48	2.00	0.00	0.19	0.00
47.60	24.40	2.00	0.00	0.19	0.00	47.65	24.38	2.00	0.00	0.19	0.00
47.74	24.29	2.00	0.00	0.19	0.00	47.79	27.41	2.00	0.00	0.19	0.00
47.86	29.28	2.00	0.00	0.19	0.00	47.92	28.28	2.00	0.00	0.19	0.00
47.97	25.44	2.00	0.00	0.19	0.00	48.06	23.24	2.00	0.00	0.19	0.00
48.11	32.43	2.00	0.00	0.18	0.00	48.21	111.82	2.00	0.00	0.18	0.00
48.25	117.91	2.00	0.00	0.18	0.00	48.31	120.95	0.29	0.48	0.18	0.00
48.39	122.30	0.29	0.47	0.18	0.00	48.44	122.36	0.29	0.47	0.18	0.00
48.53	122.89	0.30	0.46	0.18	0.01	48.58	124.26	0.30	0.45	0.18	0.00
48.66	125.81	0.31	0.44	0.18	0.00	48.71	126.20	0.31	0.44	0.17	0.00
48.75	124.97	0.31	0.44	0.17	0.00	48.82	119.93	0.28	0.46	0.17	0.00
48.89	116.07	2.00	0.00	0.17	0.00	48.99	117.17	2.00	0.00	0.17	0.00
49.03	109.49	2.00	0.00	0.17	0.00	49.09	34.33	2.00	0.00	0.17	0.00
49.17	32.63	2.00	0.00	0.17	0.00	49.23	28.28	2.00	0.00	0.17	0.00
49.30	24.19	2.00	0.00	0.16	0.00	49.37	21.25	2.00	0.00	0.16	0.00
49.43	21.98	2.00	0.00	0.16	0.00	49.52	30.62	2.00	0.00	0.16	0.00
49.55	33.09	2.00	0.00	0.16	0.00	49.65	34.56	2.00	0.00	0.16	0.00
49.69	33.47	2.00	0.00	0.16	0.00	49.78	32.82	2.00	0.00	0.16	0.00
49.83	36.05	2.00	0.00	0.16	0.00	49.87	102.02	2.00	0.00	0.15	0.00
49.97	116.21	2.00	0.00	0.15	0.00	50.01	117.52	2.00	0.00	0.15	0.00
50.10	116.72	2.00	0.00	0.15	0.00	50.15	117.34	2.00	0.00	0.15	0.00
50.24	115.69	2.00	0.00	0.15	0.00	50.27	113.07	2.00	0.00	0.15	0.00
50.34	105.62	2.00	0.00	0.15	0.00	50.42	36.17	2.00	0.00	0.15	0.00
50.48	30.00	2.00	0.00	0.14	0.00	50.56	23.94	2.00	0.00	0.14	0.00
50.61	22.04	2.00	0.00	0.14	0.00	50.68	19.33	2.00	0.00	0.14	0.00
50.75	18.36	2.00	0.00	0.14	0.00	50.79	19.87	2.00	0.00	0.14	0.00
50.89	26.17	2.00	0.00	0.14	0.00	50.93	103.96	2.00	0.00	0.14	0.00
51.02	125.72	2.00	0.00	0.14	0.00	51.06	71.63	2.00	0.00	0.13	0.00
51.15	87.82	2.00	0.00	0.13	0.00	51.20	93.50	2.00	0.00	0.13	0.00
51.25	94.65	2.00	0.00	0.13	0.00	51.34	93.74	2.00	0.00	0.13	0.00
51.39	92.03	2.00	0.00	0.13	0.00						

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)

Total estimated settlement: 2.05

Abbreviations

$Q_{tn,cs}$:	Equivalent clean sand normalized cone resistance
FS:	Factor of safety against liquefaction
e_v (%):	Post-liquefaction volumentric strain
DF:	e_v depth weighting factor
Settlement:	Calculated settlement

GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL BUILDING

3800 Via Oro Avenue
Long Beach, California
for
9th Street Partners, LLC

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

October 21, 2024

9th Street Partners, LLC
880 Apollo Street, Suite 356
El Segundo, California 90245

Attention: Mr. Tom Ashcraft
Managing Partner

Project No.: **24G159-1**

Subject: **Geotechnical Investigation**
Proposed Industrial Building
3800 Via Oro Avenue
Long Beach, California

Mr. Ashcraft:

In accordance with your request, we have conducted a geotechnical investigation at the subject site. We are pleased to present this report summarizing the conclusions and recommendations developed from our investigation.

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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Plate 2: Boring Location Plan
B Boring Logs
C Laboratory Test Results
D Grading Guide Specifications
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F CPT Report
G CPT Liquefaction Evaluation

1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report.

Geotechnical Design Considerations

- The subject site is located within an area that has been mapped as a liquefaction hazard zone.
- Our site-specific liquefaction evaluation included four (4) CPT soundings advanced to depths of 50± feet. In addition to the four CPTs, two (2) of the borings were advanced to depths of 50± feet to provide samples for laboratory testing and for correlation with the CPT data. Liquefiable soils were encountered at various depths between 20 and 49± feet at all of the CPT locations.
- The total potential liquefaction induced settlement ranges between 1.70 and 2.85± inches at the CPT locations. Differential settlement is expected to be a maximum of 1½± inches.
- Based on the estimated magnitude of the differential settlements, the proposed structure may be supported on shallow foundations. Additional design considerations related to the potentially liquefiable soils are presented within the text of this report.
- The proposed building area is generally underlain by existing undocumented fill soils, extending to depths of 4½ to 8½± feet at the boring locations, as well as potentially compressible native alluvium. These soils are not considered suitable for support of the new structure.
- Some of the on-site soils are low to non-expansive.
- Tested samples of the on-site soils possess negligible to moderate soluble sulfate concentrations. Based on the presence of soils with moderate sulfate concentrations, ACI 318 Sulfate Classification "S1" has been assigned to the site. Concrete mix design recommendations are presented in Section 6.2 of this report.

Site Preparation

- Initial site preparation should include demolition of the improvements associated with the existing facility. This includes all foundations, floor slabs, and any underground improvements that will not be reused with the new development. All topsoil and organics (including tree root masses) should be removed from the landscaped areas.
- Remedial grading should be performed within the proposed building area in order to remove the existing undocumented fill soils, which extend to depths of 4½± to 8½± feet at the boring locations. The overexcavation is also recommended to extend to a depth of at least 6 feet below existing grade, and to a depth of 4 feet below proposed building pad subgrade elevation. Within the foundation influence zones, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade.
- After the recommended overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated. The resulting subgrade should then be scarified to a depth of 12 inches and thoroughly moisture conditioned to 2 to 4 percent above optimum moisture content. The resulting subgrade should then be recompacted to at least 90 percent of the ASTM D-1557

maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

- The new parking area subgrade soils are recommended to be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,500 lbs/ft^2 maximum allowable soil bearing pressure.
- Reinforcement consisting of at least six (6) No. 5 rebars (3 top and 3 bottom) in strip footings, due to the potential liquefaction-induced differential settlements. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade, 6 inches thick.
- Modulus of Subgrade Reaction: k = 125 psi/in.
- Minimum slab reinforcement: No. 3 bars at 18-inches on-center, in both directions, due to the potential liquefaction-induced settlement. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

Pavements

ASPHALT PAVEMENTS (R = 30)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	6	8	10	11	13
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 30)				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 24P289, dated August 23, 2024. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slabs, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. Based on the location of the subject site, this investigation also included a site-specific liquefaction evaluation. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the northeast corner of Via Oro Avenue and Hughes Way in Long Beach, California. The site is also referenced by the street address 3800 Via Oro Avenue. The site is bounded to the north by an existing commercial/industrial building, to the west by Via Oro Avenue, to the south by Hughes Way, and to the east by the Long Beach Freeway (Interstate 710). The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in Appendix A of this report.

The site consists of an irregular-shaped parcel, 13.41± acres in size. The site is developed with one communication building, 35,000± ft² in size, located in the west-central area of the site. In addition, numerous satellite dishes are present on the south and eastern sides of the building. A guard shack is present along the northwestern perimeter of the site. A metal shed, about 1,200± ft², is located in the southcentral portion of the site. Several air conditioners and generators are present near the northeastern portion of the communication building. A brick building, about 1,872± ft² is located south of the southwestern portion of the communication building. Three (3) large above-ground diesel storage tanks (AST), approximately 9 feet in diameter and 34 feet in length, are located in the central area of the site. Several metal storage bins are present in the areas developed with satellite dishes. The southeastern portion of the site is developed with solar panels. The northeastern portion of the site consists of vacant undeveloped land. The ground surface cover in the satellite dish area consists of open-graded gravel with concrete lined pathways and pads for the antennas. Landscaped planter areas with grass, shrubs, and medium to large sized trees are present along the western perimeter and northern perimeter of the building. Ground surface cover in the northeastern portion of the site consists of exposed soil, and moderate native weeds, shrubs, and grass. The ground surface cover in the southeastern portion of the site consists of exposed soil. The northwestern portion of the site is developed as a parking lot, paved with asphaltic concrete. The existing asphalt pavements are in fair to poor condition, with localized areas of severe cracking.

Topographic information was obtained from the ALTA Survey prepared by S.E.C. Civil Engineers, Inc. Based on this survey, the western portion of the site slopes to the south at a gradient of 2± percent. The eastern portion of the site slopes to the southeast at a gradient of 1± percent. The site grades range from a maximum site elevation of 34.1± feet mean sea level (MSL) in the northwest corner of the site to a minimum site elevation of 28.1± feet msl in the southeast corner of the site. There is approximately 6± feet of elevation differential across the site.

3.2 Proposed Development

Based on a conceptual site plan prepared by Herdman Architecture + Design, the site will be developed with one new industrial building, 325,096± ft² in size, located in the south-central area of the site. The building will include two second floor mezzanines, each 3,500± ft² in size, located

in the northeast and northwest corners of the building. The building will be constructed with dock-high doors along a portion of the north building wall. The building will be surrounded by asphaltic concrete pavements in the automobile parking and drive areas, Portland cement concrete pavements in the truck court, and areas of concrete flatwork and landscape planters.

Detailed structural information has not been provided. It is assumed that the new building will be a single-story structure of tilt-up concrete construction, typically supported on conventional shallow foundations with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 4 to 7 kips per linear foot, respectively.

No significant amounts of below-grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. Based on the assumed topography, cuts and fills of 1 to 2± feet are expected to be necessary to achieve the proposed site grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration for this project consisted of five (5) borings (identified as B-1 through B-5) advanced to depths of 20 to 50± feet below existing site grades. Four (4) of the borings were advanced to depths of 25 to 50± feet within the proposed building area. Two of these borings were drilled to a depth of 50± feet, as a part of the liquefaction evaluation. The remaining boring was drilled within the parking area, extending to a depth of 20± feet below existing grades. Upon the completion of drilling, Boring No. B-1 was converted into a temporary piezometer to establish the current groundwater elevation. All of the borings were logged during drilling by a member of our staff. In addition to the borings, four (4) Cone Penetration Test (CPT) soundings were advanced to depths of 51 to 63± feet as a part of the liquefaction evaluation. The CPTs are identified as CPT-1 through CPT-4.

Hollow Stem Auger Borings

The borings were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig. Representative bulk and relatively undisturbed soil samples were taken during drilling. Relatively undisturbed soil samples were taken with a split barrel "California Sampler" containing a series of one-inch-long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. Samples were also taken using a 1.4±-inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

Cone Penetration Test (CPT) Soundings

The CPT soundings were performed by Kehoe Testing and Engineering (KTE) under the supervision of a member of our staff. The cone system used for this project was manufactured by Vertek. The CPT soundings were performed in general accordance with ASTM standards (D-5778). The cone penetrometers were pushed using 30-ton CPT rig. The cones used during the program recorded the cone resistance, sleeve friction, and dynamic core pressure at 2.5-centimeter depth intervals. The CPT soundings were advanced to depths of 50± feet. A more complete description of the CPT program as well as the results of the data interpretation are provided in the report prepared by KTE, enclosed in Appendix F of this report. The CPT soundings do not result in any recovered soil samples. However, correlations have been developed that utilize the cone resistance and the sleeve friction to estimate the soil type that is present at each 2.5-centimeter interval in the subsurface profile. These soil classifications are presented graphically on the CPT output forms enclosed in Appendix F.

The data generated by the cone penetrometer equipment has been reduced by KTE using CPeT-IT, V2.3.1.9, published by Geologismiki Geotechnical Software. The CPeT-IT program output as well as more details regarding the interpretation procedure are presented in a report prepared by KTE, which is provided in Appendix F of this report.

General

The approximate locations of the borings and CPT soundings are indicated on the Boring and CPT Location Plan, included as Plate 2 in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Pavements

Asphaltic concrete pavements were encountered at the ground surface at Boring No. B-2 and B-4. The pavement sections generally consisted of 3± inches underlain by about 4 to 5± inches of aggregate base.

Artificial Fill

Open-graded gravel was encountered at the ground surface at Boring No. B-3, measuring 6± inches in thickness.

Artificial fill soils were encountered at the ground surface at Boring Nos. B-1 and B-5, beneath the gravel at Boring No. B-3, and beneath the pavements Boring Nos. B-2 and B-4, extending to depths of 4½ to 8½± feet below ground surface. The fill soils generally consist of medium dense silty fine sands, fine sandy silts, fine sands, clayey fine sands, very stiff fine sandy clays and occasional silty clays. The fill soils possess a disturbed and mottled appearance, and some samples possess appreciable artificial debris content, including asphaltic concrete and brick fragments, resulting in their classification as artificial fill.

Alluvium

Native alluvium was encountered beneath the fill soils at all boring locations, extending to at least the maximum depth explored of 50± feet below ground surface. The alluvium consists of very loose to dense fine sandy silts, silty fine sands and fine sands. Occasional layers of stiff silt and clayey silt were encountered. The alluvial soils generally possess little iron oxide staining. The alluvial soils generally possess trace to little iron oxide staining and varying amounts of calcareous deposits.

Groundwater

Free water was encountered during drilling at Boring Nos. B-1, B-3, and B-4 at depths of 23½ to 27± feet below existing site grades. Boring No. B-3 remained open for a delayed groundwater reading for 4½ hours, after the completion of drilling, before backfilling. A delayed groundwater

measurement was performed at Boring No. B-3 after the duration specified above, free ground water was measured at a depth of 23.9± feet below ground surface. At the completion of drilling Boring No. B-1 was converted into a temporary piezometer to establish the current groundwater elevation. Two (2) water depth measurements were performed within the piezometer approximately 1 day and 11 days after drilling. Free groundwater was measured at a depth of 26.9± feet below ground surface at this location during both of these delayed measurements.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the subject site. The primary reference used to determine the historic groundwater depth in this area is the California Geological Survey (CGS) Open File Report 98-19, the Seismic Hazard Zone Report for the Long Beach 7.5-Minute Quadrangle, which indicates that the historic high groundwater level for this site is located at a depth between 20 and 30± feet below the ground surface.

Recent water level data was obtained from the California State Water Resources Control Board website, https://geotracker.waterboards.ca.gov/. The nearest cluster of monitoring wells is located approximately 1,350± feet northwest of the site. Data from these monitoring wells indicates a recent high groundwater level of 26½± feet below the ground surface in August 2020.

5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

Classification

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. Field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring Logs and are periodically referenced throughout this report.

Density and Moisture Content

The density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring Logs.

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-7 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample was tested for its maximum dry density and optimum moisture content. The results were obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plate C-8 in Appendix C of this report. This test is generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829. The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample. The sample is initially remolded to 50± 1 percent saturation and then loaded with a surcharge

equivalent to 144 pounds per square foot. The sample is then inundated with water, and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The results of the EI testing are as follows:

Sample Identification	Expansion Index	Expansive Potential
B-3 @ 1 to 5 feet	41	Low
B-4 @ 1 to 5 feet	33	Low

Soluble Sulfates

Representative samples of the near-surface soils were submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	Soluble Sulfates (%)	ACI-318 Classification
B-3 @ 1 to 5 feet	0.1193	Moderate (S1)
B-4 @ 1 to 5 feet	0.0033	Not Applicable (S0)

Corrosivity Testing

Representative samples of the near-surface soils were submitted to a subcontracted corrosion engineering laboratory to identify potentially corrosive characteristics with respect to common construction materials. The corrosivity testing included a determination of the electrical resistivity, pH, and chloride and nitrate concentrations of the soils, as well as other tests. The results of some of these tests are presented below.

Sample Identification	Saturated Resistivity (ohm-cm)	pH	Chlorides (mg/kg)	Nitrates (mg/kg)	Sulfides (mg/kg)	Redox Potential (mV)
B-3 @ 1 to 5 feet	2,144	7.0	26.3	3.1	0.1	184
B-4 @ 1 to 5 feet	3,350	7.2	12.1	5.7	0.3	170

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations.

The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record. The recommendations are provided with the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to verify compliance with these recommendations. Maintaining Southern California Geotechnical, Inc., (SCG) as the geotechnical consultant from the beginning to the end of the project will provide continuity of services. The geotechnical engineering firm providing testing and observation services shall assume the responsibility of Geotechnical Engineer of Record.

The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed structure should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Furthermore, SCG did not identify any evidence of faulting during the geotechnical investigation. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

The 2022 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to

the subject site. Based on the adoption of the 2022 CBC on January 1, 2023, we expect that the proposed development will be designed in accordance with the 2022 CBC.

The 2022 CBC Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool, a web-based software application available at the website www.seismicmaps.org. This software application calculates seismic design parameters in accordance with several building code reference documents, including ASCE 7-16, upon which the 2022 CBC is based. The application utilizes a database of risk-targeted maximum considered earthquake (MCE_R) site accelerations at 0.01-degree intervals for each of the code documents. The table below was created using data obtained from the application. The output generated from this program is included as Plate E-1 in Appendix E of this report.

The 2022 CBC states that for Site Class D sites with a mapped S_1 value greater than 0.2, a site-specific ground motion analysis may be required in accordance with Section 11.4.8 of ASCE 7-16. Supplement 3 to ASCE 7-16 modifies Section 11.4.8 of ASCE 7-16 and states that "a ground motion hazard analysis is not required where the value of the parameter S_{M1} determined by Eq. (11.4-2) is increased by 50% for all applications of S_{M1} in this Standard. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in this Standard."

The seismic design parameters presented in the table below were calculated using the site coefficients (Fa and Fv) from Tables 1613.2.3(1) and 1613.2.3(2) presented in Section 16.4.4 of the 2022 CBC. It should be noted that the site coefficient Fv and the parameters SM1 and SD1 were not included in the SEAOC/OSHPD Seismic Design Maps Tool output for the ASCE 7-16 standard. We calculated these parameters-based on Table 1613.2.3(2) in Section 16.4.4 of the 2022 CBC using the value of S_1 obtained from the Seismic Design Maps Tool. **The values of S_{M1} and S_{D1} tabulated below** were evaluated using equations 11.4-2 and 11.4-4 of ASCE 7-16 (Equations 16-20 and 16-23, respectively, of the 2022 CBC) and **do not include a 50 percent increase.** As discussed above, if a ground motion hazard analysis has not been performed, S_{M1} and S_{D1} must be increased by 50 percent for all applications with respect to ASCE 7-16.

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.685
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.607
Site Class	---	D*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.685
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.032
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.123
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.688

*The 2022 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site *coefficients* are to be determined in accordance with Section 11.4.7 of ASCE 7-16. However, Section 20.3.1 of ASCE 7-16 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of the structure(s) is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the

subject site is underlain by potentially liquefiable soils. Therefore, if the proposed structures have a fundamental period greater than 0.5 seconds, a site-specific seismic hazards analysis will be required and additional subsurface exploration will be necessary.

Ground Motion Parameters

For the purposes of the liquefaction analysis performed for this study, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2022 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-16. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-16. The web-based software application SEAOC/OSHPD Seismic Design Maps Tool (described in the previous section) was used to determine PGA_M, which is 0.81g. A portion of the program output is included as Plate E-1 of this report. An associated earthquake magnitude was obtained from the USGS Unified Hazard Tool, Interactive Deaggregation application available on the USGS website. The deaggregated mean magnitude is 6.85, based on the peak ground acceleration and soil classification D.

Liquefaction

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and grain size characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Clayey (cohesive) soils or soils which possess clay particles (d<0.005mm) in excess of 20 percent (Seed and Idriss, 1982) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

Research of the Long Beach Quadrangle, California 7.5 Minute Seismic Hazard Zone Map, published by the California Geological Survey, indicates that the site is located in a designated liquefaction hazard zone. Therefore, the scope of this investigation included a detailed liquefaction evaluation in order to determine the site-specific liquefaction potential.

The liquefaction analysis was conducted in accordance with the requirements of Special Publication 117A (CDMG, 2008), and currently accepted practice (SCEC, 1997). The liquefaction potential of the subject site was evaluated using the empirical method developed by Boulanger and Idriss (Boulanger and Idriss, 2008, 2014). This method predicts the earthquake-induced liquefaction potential of the site based on a given design earthquake magnitude and peak ground acceleration at the subject site. This procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude). CRR is determined as a function of the corrected SPT N-value $(N_1)_{60\text{-}cs}$, adjusted for fines content and/or the corrected CPT tip stress, $q_{c1N\text{-}cs}$. The factor of safety against liquefaction is defined as CRR/CSR. Based on Special Publication 117A, a factor of safety of at

least 1.3 is required in order to demonstrate that a given soil stratum is non-liquefiable. Additionally, in accordance with Special Publication 117A, clayey soils which do not meet the criteria for liquefiable soils defined by Bray and Sancio (2006), loose soils with a plasticity index (PI) less than 12 and moisture content greater than 85 percent of the liquid limit, are considered to be insusceptible to liquefaction. Non-sensitive soils with a PI greater than 18 are also considered non-liquefiable.

The liquefaction potential for the on-site soils was evaluated using data obtained at the three (3) CPT locations. This data was analyzed using the computer program CLiq V3.5.2.10, which was developed by Geologismiki, copyright 2006. The analysis method is based on Boulanger and Idriss 2014. The liquefaction potential of the site was analyzed utilizing a PGA_M of 0.81g for a magnitude 6.85 seismic event. A copy of the program output is presented in Appendix G of this report. As part of the liquefaction evaluation, Boring Nos. B-1 and B-3 were extended to a depth of 50± feet in order to provide samples for laboratory testing and correlation with the results of the CPT soundings.

If liquefiable soils are identified, the potential settlements that could occur as a result of liquefaction are determined using the equation for volumetric strain due to post-cyclic reconsolidation (Yoshimine et. al, 2006). This procedure uses an empirical relationship between the induced cyclic shear strain and the cyclic resistance ratio to determine the expected volumetric strain of saturated sands subjected to earthquake shaking.

Conclusions and Recommendations

The results of the liquefaction analysis have identified potentially liquefiable soils at all three (3) of the CPT soundings performed at the site. Soils which are located above the historic groundwater table or possess factors of safety of at least 1.3 are considered non-liquefiable. Several clayey strata located below the ground water table are also considered to be non-liquefiable due to their cohesive characteristics and the results of the Atterberg limits testing with respect to the criteria of Bray and Sancio (2006). Settlement analyses were conducted for each of the potentially liquefiable strata. The results of the dynamic settlement analyses are included the CLiq program output in Appendix G and are presented below:

- CPT-1: 1.70± inches
- CPT-2: 2.81± inches
- CPT-3: 2.85± inches
- CPT-4: 2.02± inches

Based on these total settlements, differential settlements of up to 1½± inches should be expected to occur during a liquefaction inducing seismic event. The estimated differential settlement could be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of less than 0.002 inches per inch.

Based on our understanding of the proposed development, it is considered feasible to support the proposed structure on shallow foundations. Such a foundation system can be designed to resist the effects of the anticipated differential settlements, to the extent that the structure would not catastrophically fail. Designing the proposed structure to remain completely undamaged

during a seismic event that could occur once every 2475 years (the code-specified return period used in the liquefaction analysis) is not considered to be economically feasible. Based on this understanding, the use of a shallow foundation system is considered to be the most economical means of supporting the proposed structure.

In order to support the proposed structure on shallow foundations (such as spread footings) the structural engineer should verify that the structure would not catastrophically fail due to the predicted dynamic differential settlements. Any utility connections to the structure should be designed to withstand the estimated differential settlements. It should also be noted that minor to moderate repairs, including re-leveling, restoration of utility connections, repair of damaged drywall and stucco, etc., would likely be required after occurrence of the liquefaction-induced settlements.

The use of a shallow foundation system, as described in this report, is typical for buildings of this type, where they are underlain the extent of liquefiable soils encountered at this site. The post-liquefaction damage that could occur within the building proposed for this site will also be typical of similar buildings in the vicinity of this project. However, if the owner determines that this level of potential damage is not acceptable, other geotechnical and structural options are available, including the use of ground improvement or mat foundations.

6.2 Geotechnical Design Considerations

General

All of the borings encountered artificial fill materials, extending to depths of 4½ to 8½± feet. Based their strength characteristics and a lack of documentation regarding the placement and compaction of the existing fill materials, these soils are considered to consist of undocumented fill. Therefore, these materials are not suitable for the support of the foundation loads of the proposed building. In addition, the near surface fill and alluvium possess a potential for consolidation when exposed to load increases in the range of those that will be exerted by the foundations of the new structure. Additionally, significant disturbance the upper 3 to 4± feet of soil is expected to occur during demolition of the existing buildings. Based on these conditions, remedial grading is considered warranted within the proposed building area to completely remove the artificial fill soils and the upper portion of the near-surface native alluvium and replace these soils as compacted structural fill.

Settlement

The recommended remedial grading will remove the existing undocumented fill soils, as well as the potentially compressible near-surface native alluvium, and replace these materials as compacted structural fill. The native soils that will remain in place below the recommended depth of overexcavation possess generally favorable consolidation/collapse characteristics and will not be subject to significant load increases from the foundations of the new structure. Provided that the recommended remedial grading is completed, the post-construction static settlements of the proposed structure are expected to be within tolerable limits.

The recommended remedial grading, discussed below, will create a new layer of structural fill below the foundations and floor slabs of the new building. This layer of fill will help to mitigate any liquefaction-induced differential settlements.

Expansion

Laboratory testing performed on representative samples of the near surface soils indicates that these materials are possess low expansion potentials (EI = 33 and 41). Therefore, care should be given to proper moisture conditioning of all building pad subgrade soils to a moisture content of 2 to 4 percent above the ASTM D-1557 optimum during site grading. In addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintaining moisture content of these soils at 2 to 4 percent above the optimum moisture content.

Soluble Sulfates

The results of the soluble sulfate testing, as discussed in Section 5.0 of this report, indicate soluble sulfate concentrations ranging from 0.0033 to 0.1193 percent. Based on these test results, some of the on-site soils possess sulfate concentrations corresponding to Class S1 with respect to the American Concrete Institute (ACI) Publication 318-14 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Based on these concentrations, the ACI requires that all concrete which will come into contact with the on-site soils incorporate the following characteristics:

- Cement Type: II (Two)
- Minimum Compressive Strength (f'_c) = 4,000 lbs/in^2
- Maximum Water/Cement Ratio: 0.50

It is recommended that additional sulfate testing be performed at the completion of rough grading to verify the concentrations which are present in the actual building pad subgrade soils.

Corrosion Potential

The results of laboratory testing indicate that the tested samples of the on-site soils possess saturated resistivities of 2,144 and 3,350 ohm-cm, and pH values of 7.0 and 7.2. The soils possess redox potentials of 170 and 184 mV and sulfide concentrations of 0.1 and 0.3 mg/kg. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity, pH, sulfide concentration, redox potential, and moisture content are the five factors that enter into the evaluation procedure. Based on these factors, the on-site soils are considered to be mildly corrosive to ferrous pipes. **Therefore, corrosion protection may be required for cast iron or ductile iron pipes.**

Relatively low concentrations (12.1 to 26.3 mg/kg) of chlorides were detected in the samples submitted for corrosivity testing. In general, soils possessing chloride concentrations in excess of 500 parts per million (ppm) are considered to be corrosive with respect to steel reinforcement within reinforced concrete. Based on the lack of an external source of chlorides and the relatively

low chloride concentrations in the tested samples, the site is considered to have a C1 chloride exposure in accordance with the American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary. Therefore, a specialized concrete mix design for reinforced concrete for protection against chloride exposure is not considered warranted.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested samples possess nitrate concentrations of 3.1 and 5.7 mg/kg. Based on these test results, the on-site soils are not considered to be corrosive to copper pipe.

It should be noted that SCG does not practice in the field of corrosion engineering. Therefore, the client may wish to contact a corrosion engineer to provide a more thorough evaluation.

Shrinkage/Subsidence

Based on the results of the laboratory testing, removal and recompaction of the near-surface fill soils is estimated to result in an average shrinkage of 7 to 15 percent. However, the estimated shrinkage of the individual soil layers at the site is highly variable, locally ranging from a minimum shrinkage value of 5 percent to a maximum shrinkage of 19 percent at varying sample depths and locations. It should be noted that the potential shrinkage estimate is based on dry density testing performed on small-diameter samples taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1 feet.

These estimates are based on previous experience and the subsurface conditions encountered at the boring locations. The actual amount of subsidence is expected to be variable and will be dependent on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely.

Grading and Foundation Plan Review

No detailed grading or foundation plans were available at the time of this report. It is therefore recommended that we be provided with copies of the preliminary plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring locations and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide

Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

Site Stripping and Demolition

Demolition of the existing structures and pavements will be necessary in order to facilitate the construction of the proposed development. Demolition should include all foundations, floor slabs, pavements, septic systems, utilities and any other subsurface improvements that will not remain in place with the new development. Debris resultant from demolition should be disposed of off-site. Alternatively, concrete and asphalt debris may be pulverized to a maximum 1-inch particle size, well-mixed with sands, and incorporated into new structural fills or it may be processed to create crushed miscellaneous base (CMB), if desired. Concrete debris crushed to a 3 to 6-inch particle size may also be used for subgrade stabilization purposes.

Initial site stripping should include removal of any surficial vegetation and organic material within the landscaped areas of the site. The actual extent of site stripping should be determined by the geotechnical engineer at the time of grading, based on the organic content and the stability of the encountered materials.

Treatment of Existing Soils: Building Pad

It is recommended that remedial grading be performed within the proposed building area to remove the existing undocumented fill soils, and a portion of the underlying native alluvium. The undocumented fill soils, which extend to depths of up to 8½ ± feet at the boring locations should be overexcavated in their entirety, as well as any soils disturbed during demolition of the existing improvements. The proposed building areas should also be overexcavated to a depth of at least 6 feet below existing grade and to a depth of at least 4 feet below proposed pad grade.

Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade. The overexcavation should extend at least 5 feet beyond the building perimeters, and to an extent equal to the depth of new fill below the foundation bearing grade. If the proposed structure incorporates any exterior columns (such as for a building canopy or overhang) the overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the building area should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or loose, porous, or low density native soils are encountered at the base of the overexcavation.

Based on conditions encountered at the exploratory boring locations, moist to very moist soils will be encountered in localized areas of the site, at or near the base of the recommended overexcavation. Scarification and air drying of these materials may be sufficient to obtain a stable subgrade. However, if highly unstable soils are identified, and if the construction schedule does not allow for delays associated with drying, mechanical stabilization, usually consisting of coarse crushed stone and/or geotextile, may be necessary. Concrete and asphalt debris that is crushed to a 3- to 6-inch particle size may also be feasible to use as a subgrade stabilization material. If

unstable subgrade conditions are encountered, the geotechnical engineer should be contacted for supplementary recommendations.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, moisture conditioned to at least 2 to 4 percent above optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Flatwork, Parking and Drive Areas

Based on economic considerations, overexcavation of the existing fill and potentially compressible native soils in the new flatwork, parking and drive areas is not considered warranted, with the exception of areas where lower strength, or unstable, soils are identified by the geotechnical engineer during grading.

Subgrade preparation in the new flatwork, parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

The grading recommendations presented above for the proposed flatwork, parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within the proposed parking areas. The grading recommendations presented above do not completely mitigate the extent of existing undocumented fill or low strength native soils in the subgrade areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the flatwork, parking and drive areas should overexcavated to provide a new layer of structural fill at least 2 feet in thickness.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining walls and non-retaining site walls should be overexcavated to a depth of 3 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pad. Any undocumented fill soils within any of these foundation areas should be removed in their entirety. The overexcavation areas should extend at least 5 feet beyond the foundation perimeters, and to an extent equal to the depth of fill below the new foundations. Any erection re for tilt-up concrete walls are considered to be part of the foundation system. Therefore, these overexcavation recommendations are applicable to erection pads. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning to within 2 to 4 percent above the optimum moisture content, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

If the full lateral recommended remedial grading cannot be completed for any proposed non-retaining screen walls located along property lines, the foundations for those walls should be

designed using a reduced allowable bearing pressure. Furthermore, the contractor should take necessary precautions to protect the adjacent structures during rough grading. Specialized grading techniques, such as A-B-C slot cuts, could likely be required during remedial grading. The geotechnical engineer should be contacted if additional recommendations, such as shoring design recommendations, are required during grading.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer. All fill should conform with the recommendations presented in the Grading Guide Specifications, included as Appendix D. Some of the existing fill soils possess elevated moisture contents. Drying of these materials may be required prior to reuse as structural fill.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2022 CBC and the grading code of the city of Long Beach.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

All imported structural fill should consist of very low expansive (EI < 20), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). It is recommended that materials in excess of 3 inches in size not be used for utility trench backfill. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Long Beach. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

6.4 Construction Considerations

Excavation Considerations

The near-surface soils at this site generally consist of silty sands and sandy silts. These materials may be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. Temporary excavation slopes should be no steeper than 2h:1v. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

Some of the near surface soils possess appreciable silt and occasional clay content and may become unstable if exposed to significant moisture infiltration or disturbance by construction traffic. The base of the recommended building pad overexcavation will also be subject to subgrade instability, due the clay content of the native soils at this depth. Based on their granular content, some of the on-site soils will also be susceptible to erosion. The site should, therefore, be graded to prevent ponding of surface water and to prevent water from running into excavations.

As discussed in Section 6.3 of this report, unstable subgrade soils will likely be encountered at the base of the overexcavation within the proposed building area, in localized areas. The extent of unstable subgrade soils will to a large degree depend on methods used by the contractor to avoid adding additional moisture to these soils or disturbing soils which already possess high moisture contents. If grading occurs during a period of relatively wet weather, an increase in subgrade instability should also be expected. **If unstable subgrade conditions are encountered, it is recommended that only track mounted vehicles be used for fill placement and compaction.**

If the construction schedule dictates that site grading will occur during a period of wet weather, allowances should be made for costs and delays associated with drying the on-site soils or import of a less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pad areas as well as the need for and or the thickness of the crushed stone stabilization layer, discussed in Section 6.3 of this report.

Groundwater

The static groundwater table at this site is considered to currently exist at a depth in excess of 24 to 27± feet. Therefore, groundwater is not expected to impact grading or foundation construction activities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by structural fill soils extending to depths of at least 3 feet below foundation bearing

grade. Based on this subsurface profile, and based on the design considerations presented in Section 6.1 of this report, the proposed structure may be supported on conventional shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom), due to minor amounts of potential liquefaction-induced settlement and the presence of low expansive soils.
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 24 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressure presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements, as discussed in Section 6.1. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill compacted at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 2 to 4 percent above the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential static settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively, under static conditions. Differential movements are expected to occur over a 50-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch. These settlements are in addition to the liquefaction-induced settlements previously discussed in Section 6.1 of this report.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 275 lbs/ft^3
- Friction Coefficient: 0.28

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill soils. The maximum allowable passive pressure is 2,750 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support the new floor slab should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, and based on the design considerations presented in Section 6.1 of this report, the floor of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, extending to a depth of at least 4 feet below finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.

- Modulus of Subgrade Reaction: 125 psi/in.

- Minimum slab reinforcement: No. 3 bars at 18-inches on-center, in both directions, due to presence of potentially liquefiable soils and low expansive soils. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading, and the potential liquefaction induced settlements.

- Slab underlayment: If moisture sensitive floor coverings will be used then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire slab area where such moisture sensitive floor coverings are expected. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-

88. A polyolefin material such as 15 mil Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

- Moisture condition the floor slab subgrade soils to 2 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement.

6.7 Concrete Flatwork Design and Construction

Presented below are recommendations for flatwork which will be subject only to pedestrian traffic. Based on the results of the laboratory testing presented in Section 5.0 of this report, the on-site soils possess a low expansion potential. Based on the preceding grading recommendations, it is expected that the flatwork will be underlain by at least 12 inches of existing soils that have been scarified, thoroughly moisture conditioned, and recompacted. Based on these remedial grading recommendations, the concrete flatwork should incorporate the following characteristics:

- Minimum slab thickness: 4½ inches.

- Minimum slab reinforcement: No. 3 bars at 18 inches on center, in both directions.

- The flatwork at building entry areas should be structurally connected to the perimeter foundation that is recommended to span across the door opening. This recommendation is designed to reduce the potential for differential movement at this joint.

- Moisture condition the slab subgrade soils to at least 2 to 4 percent above optimum moisture content, to a depth of at least 12 inches. Adequate moisture conditioning should be verified by the geotechnical engineer 24 hours prior to concrete placement.

- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

- Control joints should be provided at a maximum spacing of 8 feet on center in two directions for slabs and at 6 feet on center for sidewalks. Control joints are intended to direct cracking. Minor cracking of exterior concrete slabs on grade should be expected.

- Where flatwork is immediately adjacent to landscape planters, a thickened edge should be utilized. This edge should extend to a depth of at least 12 inches and incorporate longitudinal reinforcement consisting of at least two No. 4 bars.

- Expansion or felt joints should be used at the interface of exterior slabs on grade and any fixed structures to permit relative movement.

These recommendations are contingent upon additional expansion index testing being conducted at the completion of rough grading, to verify the actual expansion potential of the flatwork subgrade soils.

6.8 Retaining Wall Design and Construction

Although not indicated on the site plan, some small (less than 3 to 4 feet in height) retaining walls may be required to facilitate the new site grades. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near-surface on-site soils generally consist of silty sands and sandy silts and occasional sandy clays. Based on their classification, these materials are expected to possess a friction angle of at least 30 degrees when compacted to at least 90 percent of the ASTM D-1557 maximum dry-density. **The sandy clays encountered at some of the borings are potentially expansive and are not recommend for use as retaining wall backfill**.

If desired, SCG could provide design parameters for an alternative select backfill material behind the retaining walls. The use of select backfill material could result in lower lateral earth pressures. In order to use the design parameters for the imported select fill, this material must be placed within the entire active failure wedge. This wedge is defined as extending from the heel of the retaining wall upwards at an angle of approximately 60° from horizontal. If select backfill material behind the retaining wall is desired, SCG should be contacted for supplementary recommendations.

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type: On-site Sands and Silts
Internal Friction Angle (ϕ)		30°
Unit Weight		120 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	40 lbs/ft^3
	Active Condition (2h:1v backfill)	64 lbs/ft^3
	At-Rest Condition (level backfill)	60 lbs/ft^3

The walls should be designed using a soil-footing coefficient of friction of 0.28 and an equivalent passive pressure of 275 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly.

Where the soils on the toe side of the retaining wall are not covered by a "hard" surface such as a structure or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Seismic Lateral Earth Pressures

In addition to the lateral earth pressures presented in the previous section, retaining walls which are more than 6 feet in height should be designed for a seismic lateral earth pressure, in accordance with the 2022 CBC. Based on the current site plan, it is not expected that any walls in excess of 6 feet in height will be required for this project. If any such walls are proposed, our office should be contacted for supplementary design recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 3 feet below proposed foundation bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

On-site sandy soils may be used to backfill the retaining walls. The silty clays and clayey silts encountered at depths of more than 5± feet are potentially expensive and are not recommend for use as retaining wall backfill. All backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

It is recommended that a minimum 1 foot thick layer of free-draining granular material (less than 5 percent passing the No. 200 sieve) be placed against the face of the retaining walls. This material should extend from the top of the retaining wall footing to within 1 foot of the ground surface on the back side of the retaining wall. This material should be approved by the geotechnical engineer. In lieu of the 1 foot thick layer of free-draining material, a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls, may be used. If the layer of free-draining material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The layer of free draining granular material should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

As previously indicated, the retaining wall design parameters are based upon drained backfill conditions. Consequently, some form of permanent drainage system will be necessary in conjunction with the appropriate backfill material. Subsurface drainage may consist of either:

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system.

6.9 Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the ***Site Grading Recommendations*** section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface on-site soils consist of a variety of materials, mainly comprising silty sands and sandy silts with occasional clayey sands and sandy clays. Based on their classification, these materials are expected to possess good pavement support characteristics with estimated R-values in the range of 30 to 50. Since R-value testing was outside the scope of services for this project, the subsequent pavement design is therefore based upon an assumed R-value of 30. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20-year design life, assuming six operational traffic days per week.

Traffic Index	No. of Heavy Trucks per Day
4.0	0
5.0	1
6.0	3
7.0	11
8.0	35
9.0	93

For the purpose of the traffic volumes indicated above, a truck is defined as a 5-axle tractor trailer unit with one 8-kip axle and two 32-kip tandem axles. All of the traffic indices allow for 1,000 automobiles per day.

ASPHALT PAVEMENTS (R = 30)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	6	8	10	11	13
Compacted Subgrade	12	12	12	12	12

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the Marshall maximum density, as determined by ASTM D-2726. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

The preparation of the subgrade soils within concrete pavement areas should be performed as previously described for proposed asphalt pavement areas. The minimum recommended thicknesses for the Portland Cement Concrete pavement sections are as follows:

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 30)				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. Any reinforcement within the PCC pavements should be determined by the project structural engineer. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness.

7.0 GENERAL COMMENTS

This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, civil engineer, and/or structural engineer. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The client(s)' reliance upon this report is subject to the Engineering Services Agreement, incorporated into our proposal for this project.

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between boring locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted.

The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

8.0 REFERENCES

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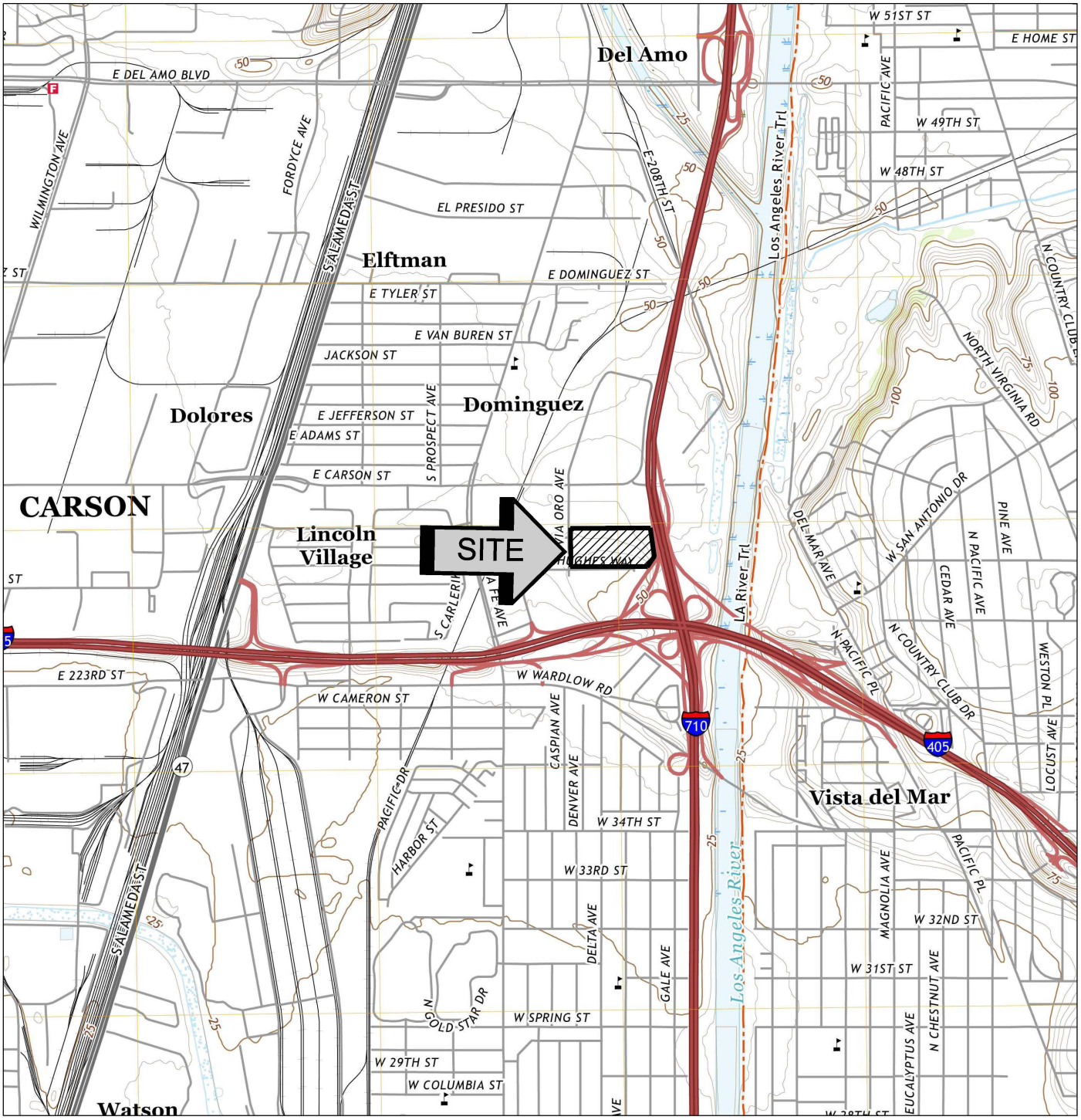
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APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAP OF THE LONG BEACH QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA, 2021.

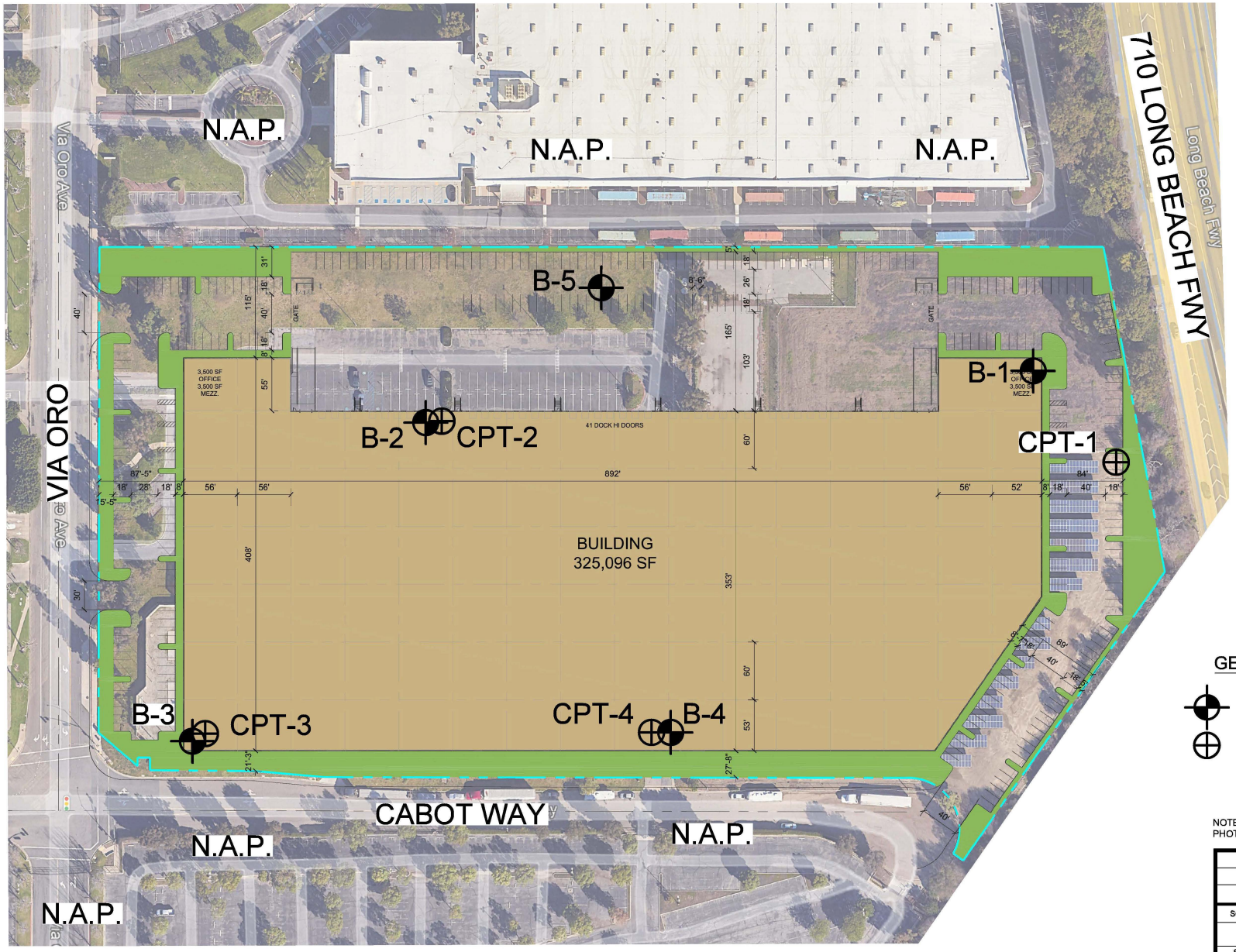
SITE LOCATION MAP

PROPOSED INDUSTRIAL BUILDING

LONG BEACH, CALIFORNIA

SCALE: 1" = 2000'	
DRAWN: MK CHKD: DN	
SCG PROJECT 24G159-1	
PLATE 1	

SOUTHERN CALIFORNIA GEOTECHNICAL



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

APPROXIMATE CPT LOCATION

NOTE: CONCEPTUAL SITE PLAN PROVIDED BY THE CLIENT. AERIAL PHOTO OBTAINED FROM GOOGLE EARTH.

BORING LOCATION PLAN

PROPOSED INDUSTRIAL BUILDING

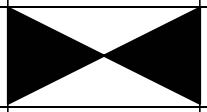
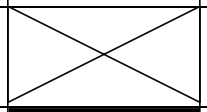
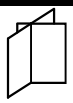
LONG BEACH, CALIFORNIA

SCALE: 1" = 100'	
DRAWN: MK CHKD: DN	
SCG PROJECT 24G159-1	
PLATE 2	

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

SAMPLE TYPE	GRAPHICAL SYMBOL	SAMPLE DESCRIPTION
AUGER		SAMPLE COLLECTED FROM AUGER CUTTINGS, NO FIELD MEASUREMENT OF SOIL STRENGTH. (DISTURBED)
CORE		ROCK CORE SAMPLE: TYPICALLY TAKEN WITH A DIAMOND-TIPPED CORE BARREL. TYPICALLY USED ONLY IN HIGHLY CONSOLIDATED BEDROCK.
GRAB		SOIL SAMPLE TAKEN WITH NO SPECIALIZED EQUIPMENT, SUCH AS FROM A STOCKPILE OR THE GROUND SURFACE. (DISTURBED)
CS		CALIFORNIA SAMPLER: 2-1/2 INCH I.D. SPLIT BARREL SAMPLER, LINED WITH 1-INCH HIGH BRASS RINGS. DRIVEN WITH SPT HAMMER. (RELATIVELY UNDISTURBED)
NSR		NO RECOVERY: THE SAMPLING ATTEMPT DID NOT RESULT IN RECOVERY OF ANY SIGNIFICANT SOIL OR ROCK MATERIAL.
SPT		STANDARD PENETRATION TEST: SAMPLER IS A 1.4 INCH INSIDE DIAMETER SPLIT BARREL, DRIVEN 18 INCHES WITH THE SPT HAMMER. (DISTURBED)
SH		SHELBY TUBE: TAKEN WITH A THIN WALL SAMPLE TUBE, PUSHED INTO THE SOIL AND THEN EXTRACTED. (UNDISTURBED)
VANE		VANE SHEAR TEST: SOIL STRENGTH OBTAINED USING A 4 BLADED SHEAR DEVICE. TYPICALLY USED IN SOFT CLAYS-NO SAMPLE RECOVERED.

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

BLOW COUNT: Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3" indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.

POCKET PEN.: Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.

GRAPHIC LOG: Graphic Soil Symbol as depicted on the following page.

DRY DENSITY: Dry density of an undisturbed or relatively undisturbed sample in lbs/ft^3.

MOISTURE CONTENT: Moisture content of a soil sample, expressed as a percentage of the dry weight.

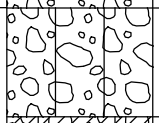
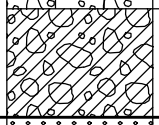
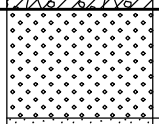
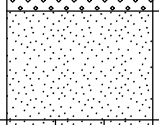
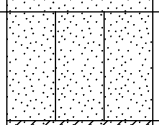
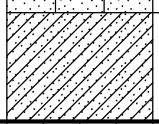
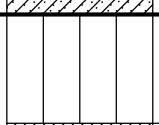
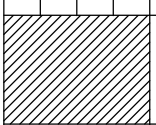
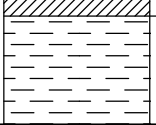
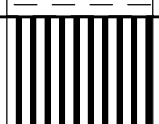
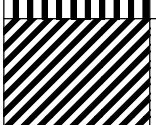
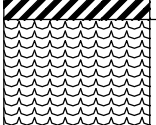
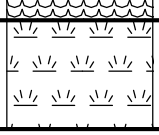
LIQUID LIMIT: The moisture content above which a soil behaves as a liquid.

PLASTIC LIMIT: The moisture content above which a soil behaves as a plastic.

PASSING #200 SIEVE: The percentage of the sample finer than the #200 standard sieve.

UNCONFINED SHEAR: The shear strength of a cohesive soil sample, as measured in the unconfined state.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 24G159-1
PROJECT: Proposed Industrial Building
LOCATION: Long Beach, California

DRILLING DATE: 9/12/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: 27 feet
CAVE DEPTH: N/A
READING TAKEN: 19.5 Hrs After Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 29.6 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
		26			FILL: Light Gray Brown fine Sand, little Silt, little medium to coarse Sand, little fine Gravel, trace fine root fibers, trace asphaltic concrete fragments, mottled, medium dense-damp	107	3					
		36			FILL: Brown Silty fine Sand, trace medium Sand, mottled, medium dense-damp	109	7					
5		21			FILL: Brown to Red Brown to Gray Clayey fine Sand, little Silt, little medium to coarse Sand, trace fine to coarse Gravel, trace asphaltic concrete fragments, mottled, medium dense-very moist	106	17					
		19				112	14					
		22			ALLUVIUM: Gray Brown fine Sandy Silt, trace medium Sand, little Iron oxide staining, medium dense-very moist	104	14					
10					Gray Brown Silty fine Sand, medium dense-moist							
		14					12					
15					Light Gray Brown fine Sand, 1-inch fine Sandy Silt lens, medium dense-damp							
		17					6			17		
20					Gray Brown Silty fine Sand, medium dense-very moist to wet							
		13					30			23		
25												
		15			@ 28½ feet, trace medium Sand, little Iron oxide staining		22			28		
					Gray Brown Silt, trace fine Sand, medium dense-wet		33			96		
30												
					Dark Gray Brown fine Sandy Silt, trace medium Sand, medium dense-wet							
		16					34			73		

TBL 24G159-1.GPJ SOCALGEO.GDT 10/21/24

TEST BORING LOG

PLATE B-1a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 24G159-1
PROJECT: Proposed Industrial Building
LOCATION: Long Beach, California

DRILLING DATE: 9/12/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: 27 feet
CAVE DEPTH: N/A
READING TAKEN: 19.5 Hrs After Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Dark Gray Brown fine Sandy Silt, trace medium Sand, medium dense-wet							
		44			Dark Gray Brown Silty fine Sand, trace medium Sand, dense-wet		22			42		
40					Dark Gray fine Sandy Silt, little Clay, medium dense-wet							
		24					21			53		
					Dark Gray Brown Silty fine Sand, trace medium Sand, medium dense-wet		24			15		
45					Gray Brown fine Sand, little Silt, medium dense-wet							
		16					16			8		
50					Boring Terminated at 50 feet							

TBL 24G159-1.GPJ SOCALGEO.GDT 10/21/24

TEST BORING LOG

PLATE B-1b

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 24G159-1
PROJECT: Proposed Industrial Building
LOCATION: Long Beach, California

DRILLING DATE: 9/12/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 31.9 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					3± inches Asphaltic concrete; 5± inches Aggregate Base							
		28			FILL: Dark Gray fine Sandy Silt, trace Clay, mottled, medium dense-very moist	107	14					
					FILL: Interbedded Light Brown Silty fine Sand and fine to medium Sand, medium dense-very moist							
		17	4.5		FILL: Gray fine Sandy Clay, little Silt, mottled, very stiff-very moist	113	13					
5		18			FILL: Gray fine Sandy Silt, little Clay, trace medium Sand, mottled, medium dense-very moist	112	13					
		33	2.0		FILL: Interbedded Dark Gray Brown fine Sandy Silt and Silty Clay, mottled, medium dense/stiff-very moist	105	17					
					FILL: Dark Brown Silty Clay, little fine Sand, very stiff-very moist							
		13			ALLUVIUM: Gray Brown Silty fine Sand, little Iron oxide staining, loose-damp	86	7					
10												
					Light Gray fine Sand, trace Silt, little medium Sand, medium dense-damp							
		19				91	3					
15												
		23				93	4					
20												
		31				94	4					
25					Boring Terminated at 25 feet							

TBL 24G159-1.GPJ SOCALGEO.GDT 10/21/24

TEST BORING LOG

PLATE B-2

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 24G159-1
PROJECT: Proposed Industrial Building
LOCATION: Long Beach, California

DRILLING DATE: 9/12/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: 23.5 feet
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 29.8 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					6± inches Open-graded Gravel							
		25			FILL: Gray Brown Silty fine Sand, trace to little Clay, trace medium Sand, mottled, medium dense-very moist	97	14					El=41 @ 1-5'
		26			FILL: Dark Gray Brown fine Sandy Clay, trace to little medium to coarse Sand, trace fine Gravel, trace AC fragments, trace brick fragments, medium dense-very moist	111	22					
5		22	4.5			112	14					
		18			ALLUVIUM: Gray Brown fine Sand, little Silt, trace to little Iron oxide staining, medium dense-damp to moist	88	9					
		14				87	6					
10												
		14					6					
15												
		14			Light Gray Brown fine Sand, trace medium Sand, 2-inch fine Sandy Silt lens, medium dense-moist		11			31		
20												
		2			Dark Brown fine Sandy Silt, very loose-wet		33			60		
25												
		14			Dark Brown Silty fine Sand, trace medium Sand, loose-wet	94	24			17		
		17			Dark Gray fine Sandy Silt, trace Clay, trace Iron oxide staining, medium dense-wet		25			77		
30												
		26			Dark Gray Brown Silty fine Sand, trace medium Sand, medium dense-wet		24			20		

TBL 24G159-1.GPJ SOCALGEO.GDT 10/21/24

TEST BORING LOG

PLATE B-3a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 24G159-1
PROJECT: Proposed Industrial Building
LOCATION: Long Beach, California

DRILLING DATE: 9/12/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: 23.5 feet
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Dark Gray Brown Silty fine Sand, trace medium Sand, medium dense-wet							
					Gray Brown fine Sand, trace to little Silt, loose to dense-wet							
		38					24			5		
40												
		5					26			8		
45												
		6					25			9		
50					Boring Terminated at 50 feet							

TBL 24G159-1.GPJ SOCALGEO.GDT 10/21/24

TEST BORING LOG

PLATE B-3b

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 24G159-1
PROJECT: Proposed Industrial Building
LOCATION: Long Beach, California

DRILLING DATE: 9/12/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: 24 feet
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 28.5 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					3± inches Asphaltic concrete; 4± inches Aggregate Base							
		26			FILL: Gray Brown Silty fine Sand, trace medium Sand, mottled, medium dense-very moist	107	13					El=33 @ 1-5'
		17			FILL: Brown fine Sandy Clay, little medium Sand, mottled, medium dense-moist	112	11					
					FILL: Dark Brown fine Sandy Silt, little Clay, mottled, medium dense-moist							
5		25			ALLUVIUM: Gray Brown Silty fine Sand, trace Iron oxide staining, medium dense-damp	102	7					
		18			Light Gray fine Sand, little medium Sand, trace to little Iron oxide staining, medium dense-damp to moist	89	4					
		18				88	9					
10												
		34				100	6					
15												
		23				95	6					
20												
		18			@ 24 feet, trace Silt, wet	95	24					Free Water @ 24 feet
25												
					Boring Terminated at 25 feet							

TBL 24G159-1.GPJ SOCALGEO.GDT 10/21/24

TEST BORING LOG

PLATE B-4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 24G159-1
PROJECT: Proposed Industrial Building
LOCATION: Long Beach, California

DRILLING DATE: 9/12/24
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: N/A
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 32 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
		24			FILL: Brown fine Sandy Silt, trace to little medium to coarse Sand, trace fine Gravel, trace fine root fibers, mottled, medium dense to dense-moist to very moist		11					
		36			@ 3½ feet, trace to little Clay		18					
5		37					10					
		16			ALLUVIUM: Gray Brown fine Sandy Silt, trace medium Sand, trace Iron oxide staining, trace Calcareous nodules, medium dense-very moist		21					
10					Gray Brown Clayey Silt, little fine Sand, trace Iron oxide staining, stiff-very moist							
		10	1.0		Gray Brown Silty fine Sand, 1-inch Clayey Silt lense, little fine Sand lens, medium dense-very moist		35 22					
15					Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, trace Iron oxide staining, medium dense-very moist							
		12					14					
20					Boring Terminated at 20 feet							

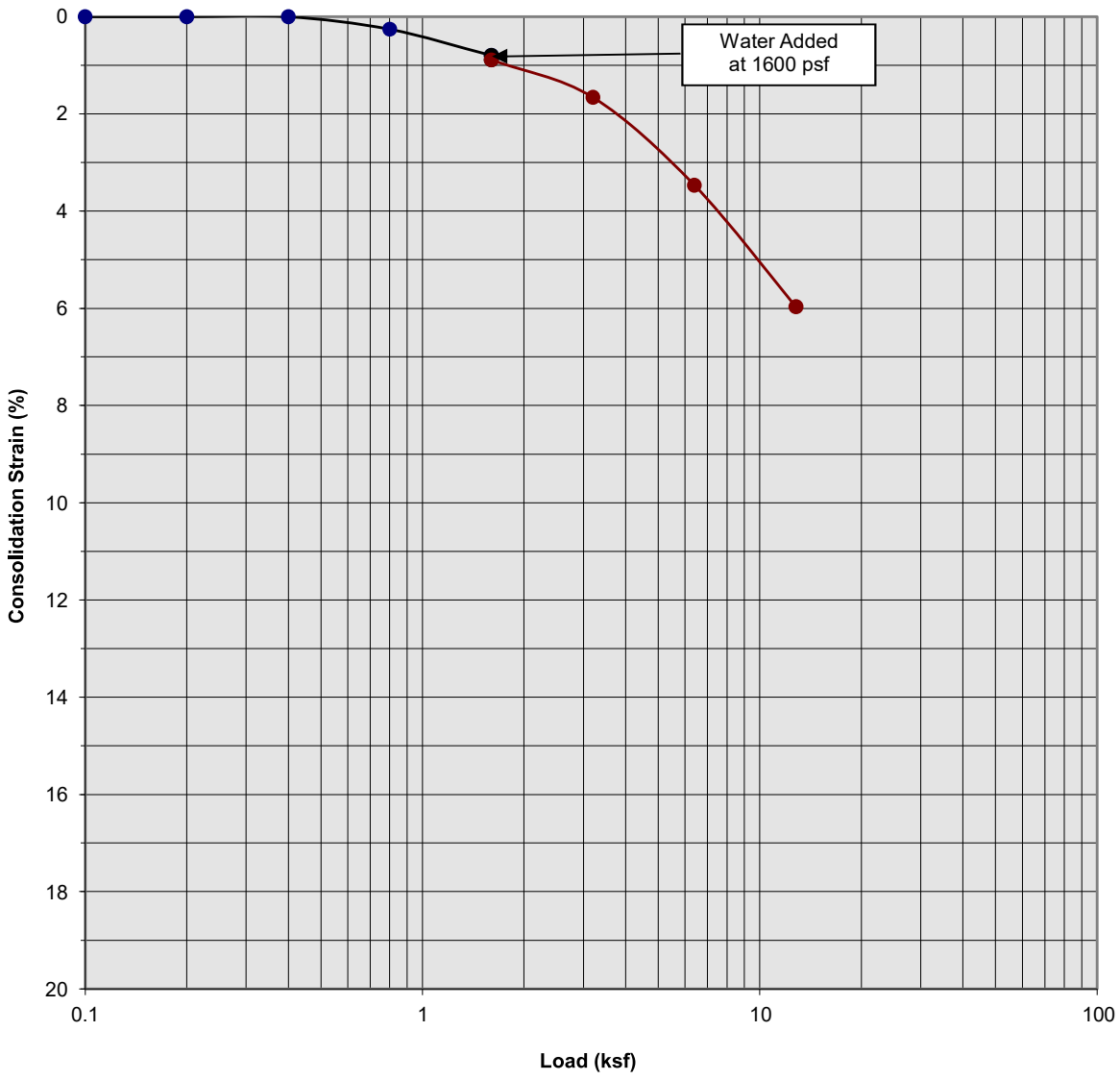
TBL 24G159-1.GPJ SOCALGEO.GDT 10/21/24

TEST BORING LOG

PLATE B-5

APPENDIX C

Consolidation/Collapse Test Results



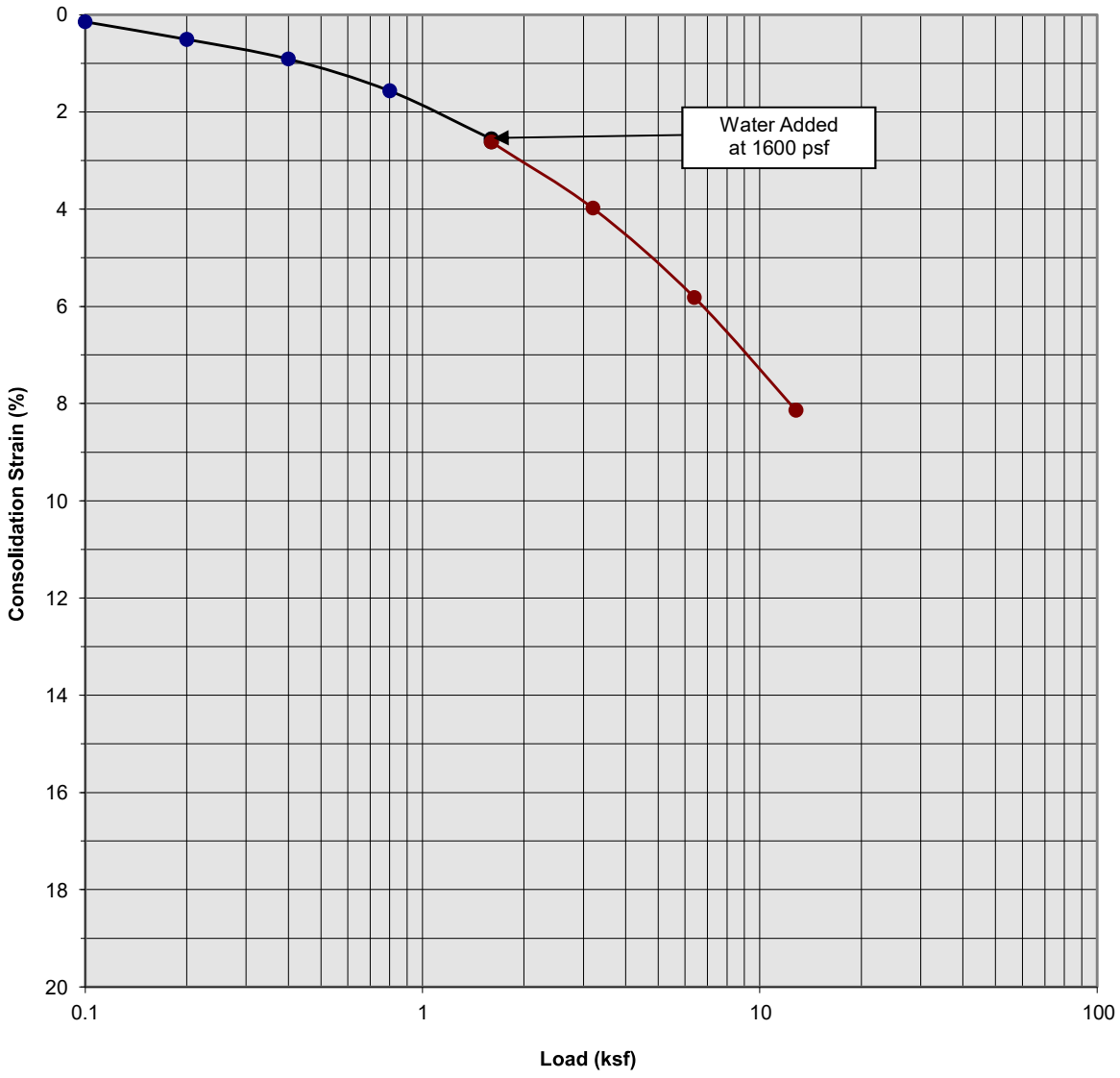
Classification: FILL: Interbedded Light Brown Silty fine Sand and fine to medium Sand

Boring Number:	B-2	Initial Moisture Content (%)	13
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	3 to 4	Initial Dry Density (pcf)	113.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	114.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.09

Proposed Industrial Building
Long Beach, California
Project No. 24G159-1
PLATE C- 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



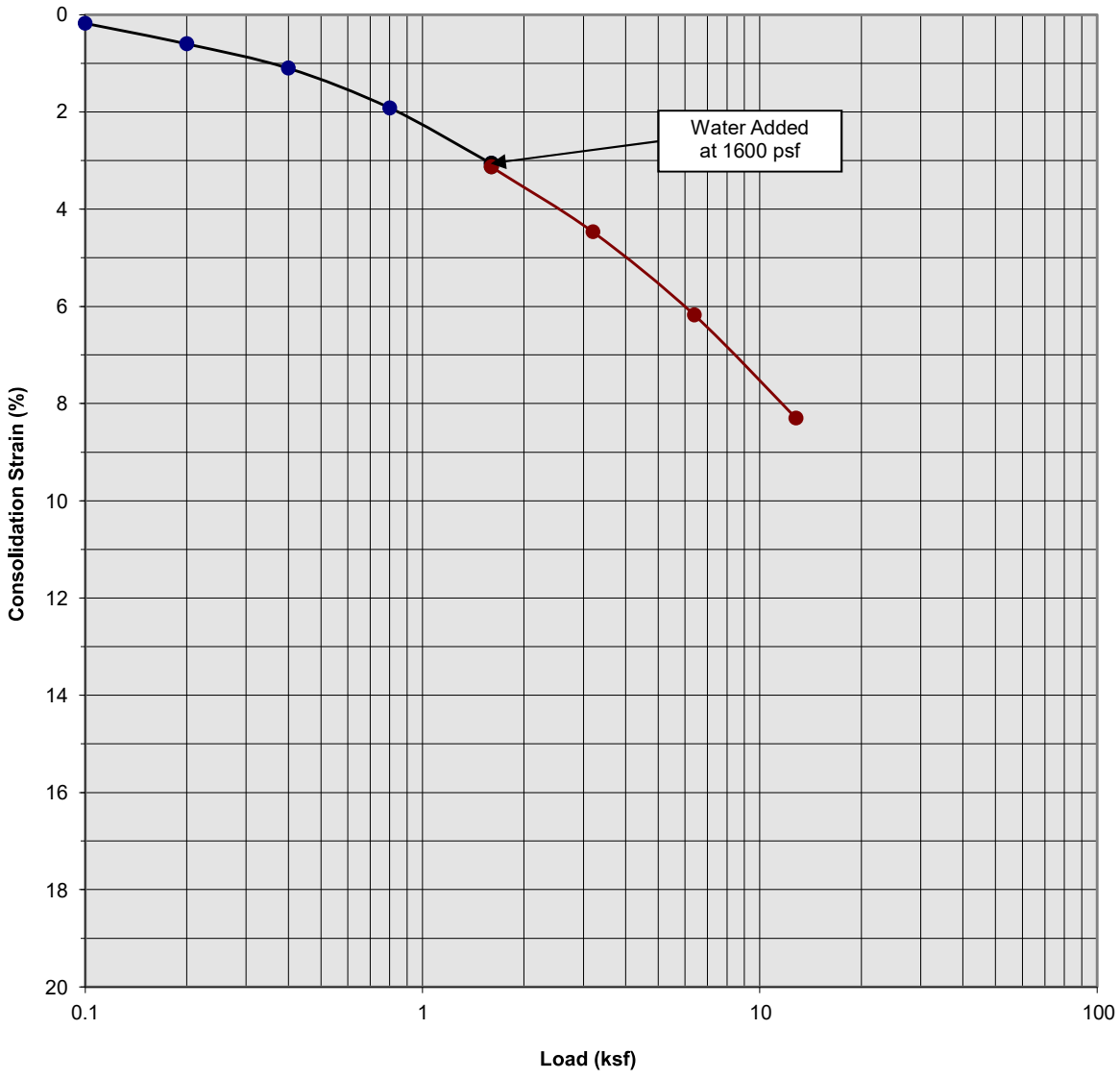
Classification: FILL: Gray fine Sandy Silt, little Clay, trace medium Sand

Boring Number:	B-2	Initial Moisture Content (%)	13
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	5 to 6	Initial Dry Density (pcf)	111.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	120.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.06

Proposed Industrial Building
Long Beach, California
Project No. 24G159-1
PLATE C- 2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



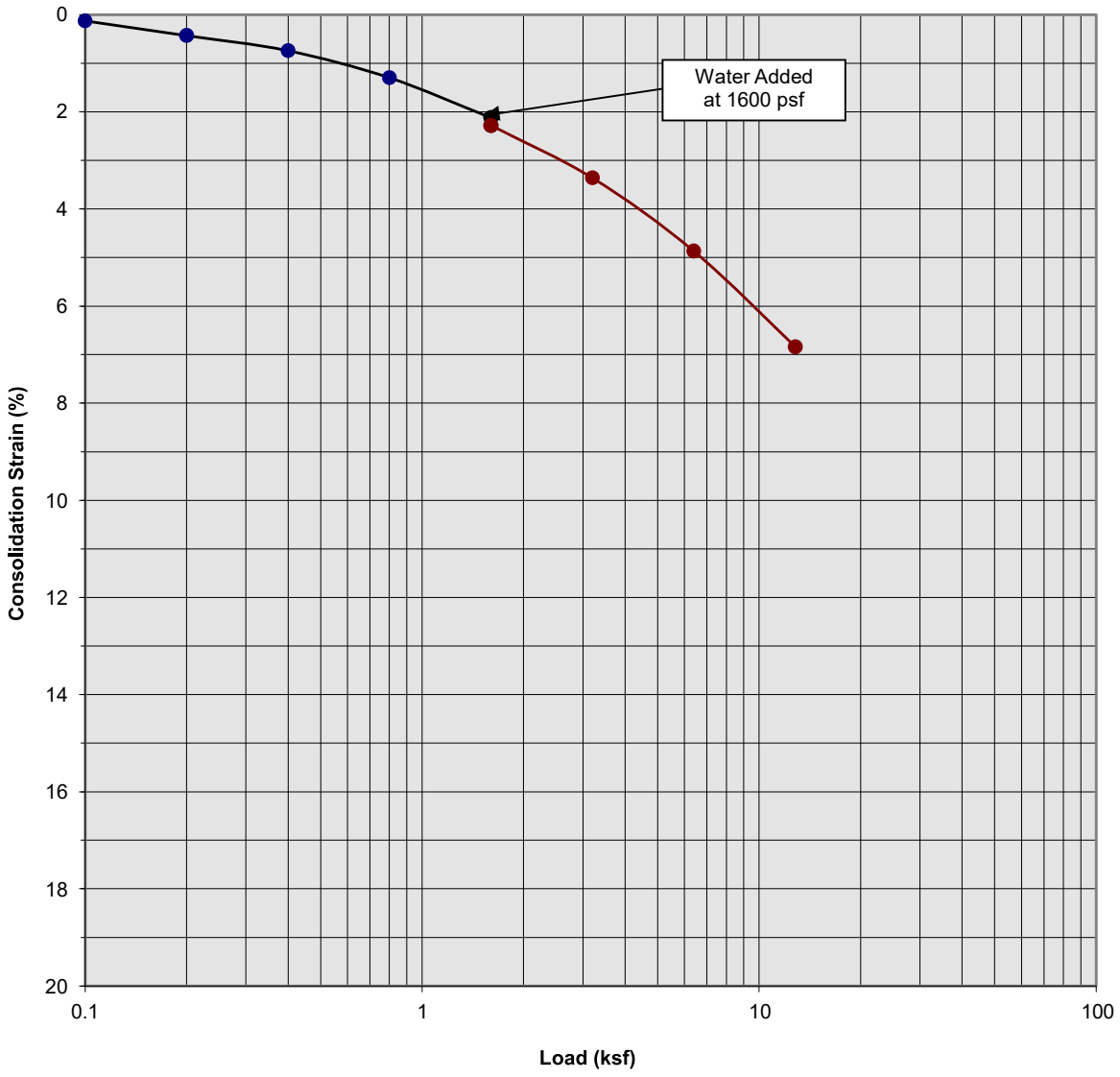
Classification: FILL: Dark Brown Silty Clay, little fine Sand

Boring Number:	B-2	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	7 to 8	Initial Dry Density (pcf)	106.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.07

Proposed Industrial Building
Long Beach, California
Project No. 24G159-1
PLATE C- 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



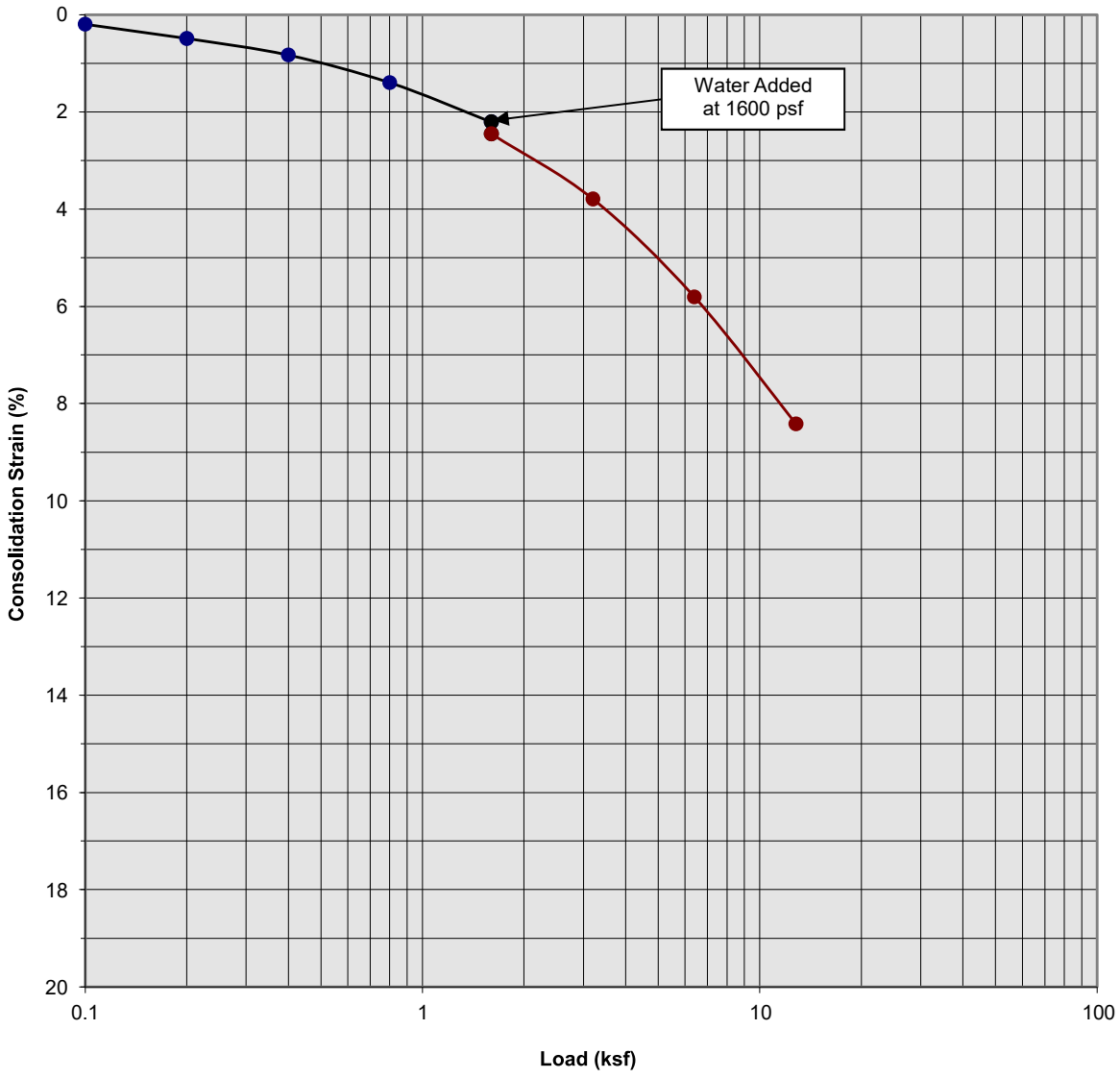
Classification: Gray Brown Silty fine Sand

Boring Number:	B-2	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	29
Depth (ft)	9 to 10	Initial Dry Density (pcf)	87.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	93.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.16

Proposed Industrial Building
Long Beach, California
Project No. 24G159-1
PLATE C- 4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



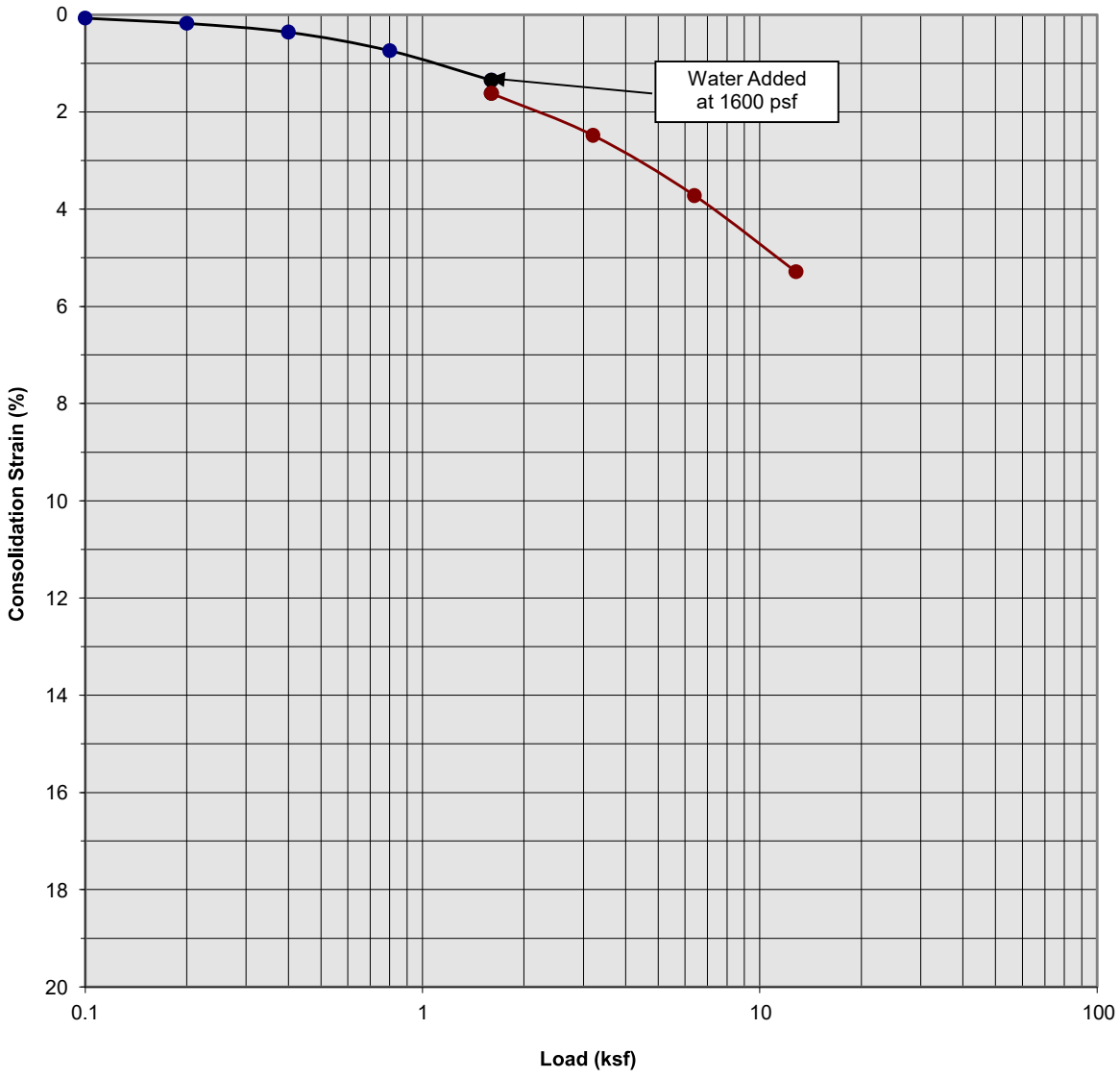
Classification: FILL: Brown fine Sandy Silt, little Clay

Boring Number:	B-4	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	3 to 4	Initial Dry Density (pcf)	111.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	130.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.24

Proposed Industrial Building
Long Beach, California
Project No. 24G159-1
PLATE C- 5

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



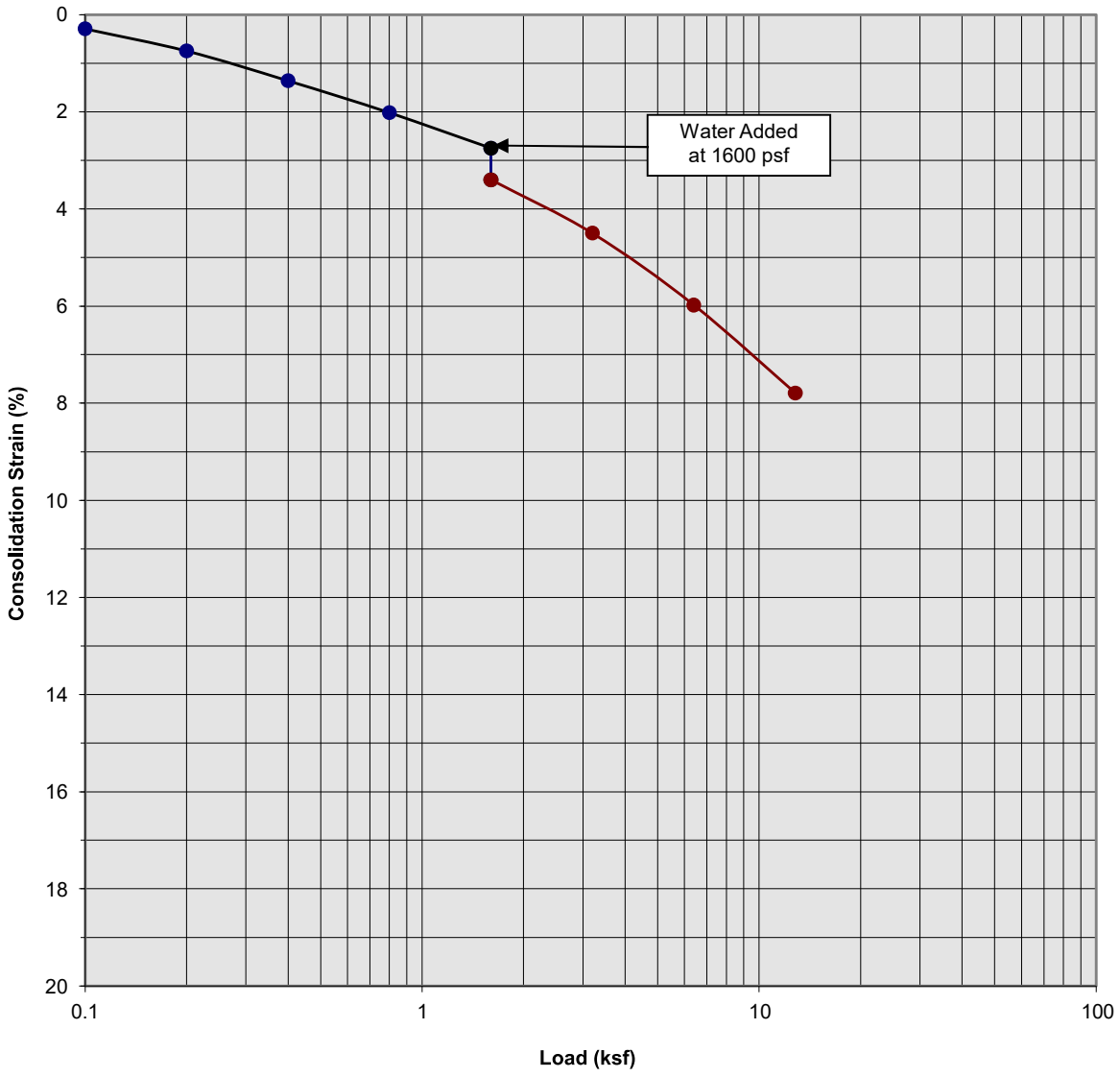
Classification: Gray Brown Silty fine Sand

Boring Number:	B-4	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	5 to 6	Initial Dry Density (pcf)	102.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.27

Proposed Industrial Building
Long Beach, California
Project No. 24G159-1
PLATE C- 6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



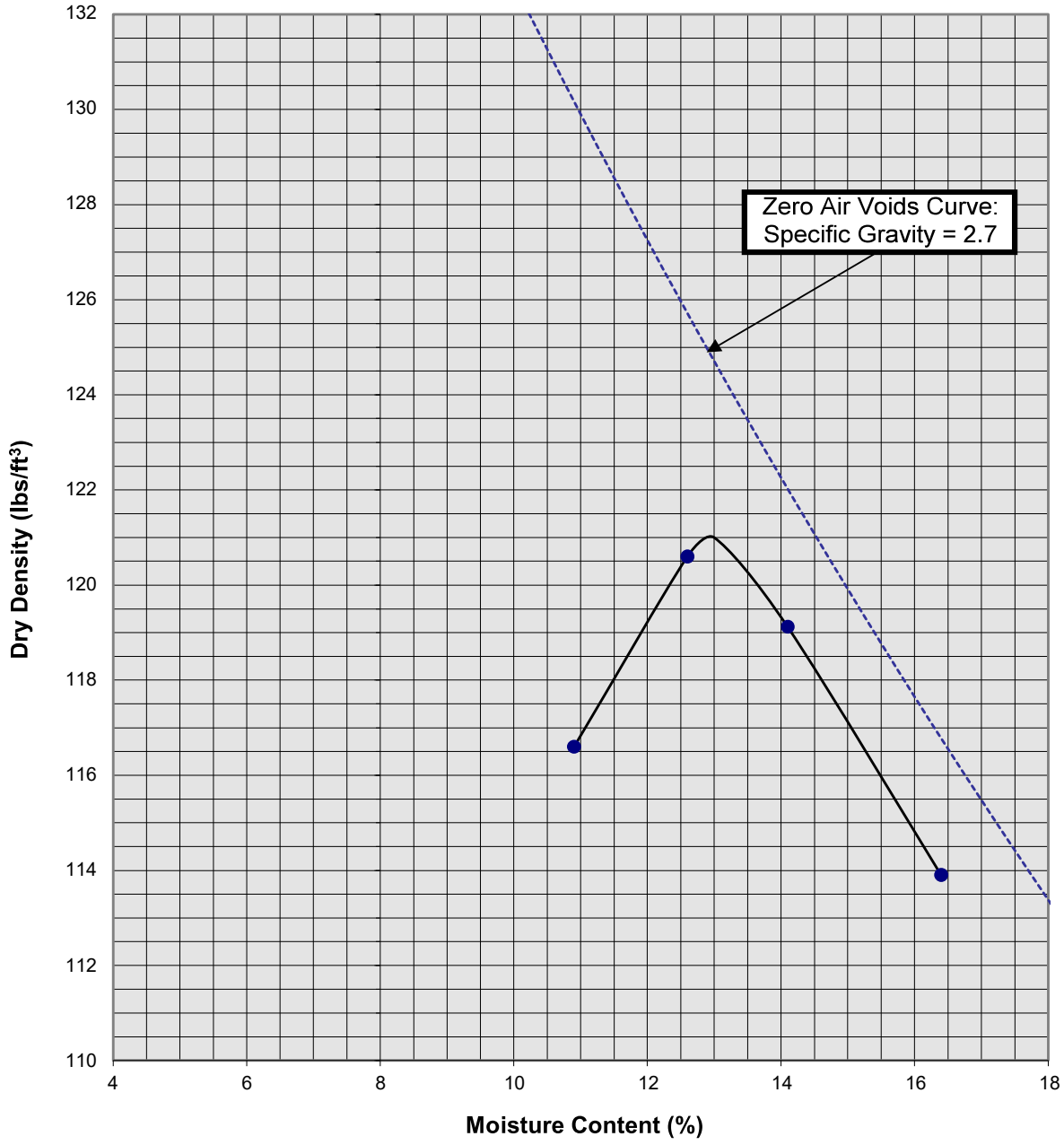
Classification: Light Gray fine Sand, little medium Sand

Boring Number:	B-4	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	25
Depth (ft)	7 to 8	Initial Dry Density (pcf)	90.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	98.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.65

Proposed Industrial Building
Long Beach, California
Project No. 24G159-1
PLATE C- 7

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
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Moisture/Density Relationship
ASTM D-1557



Soil ID Number	B-3 @ 1-5'
Optimum Moisture (%)	13
Maximum Dry Density (pcf)	121
Soil Classification	Dark Brown Clayey fine Sand to fine Sandy Clay, trace fine Gravel

Proposed Industrial Building
Long Beach, California
Project No. 24G159-1

PLATE C-8

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

These grading guide specifications are intended to provide typical procedures for grading operations. They are intended to supplement the recommendations contained in the geotechnical investigation report for this project. Should the recommendations in the geotechnical investigation report conflict with the grading guide specifications, the more site specific recommendations in the geotechnical investigation report will govern.

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.

- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.

- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.

- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.

- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.

- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.

- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

- Major vegetation should be stripped and disposed of off-site. This includes trees, brush, heavy grasses and any materials considered unsuitable by the Geotechnical Engineer.

- Underground structures such as basements, cesspools or septic disposal systems, mining shafts, tunnels, wells and pipelines should be removed under the inspection of the Geotechnical Engineer and recommendations provided by the Geotechnical Engineer and/or city, county or state agencies. If such structures are known or found, the Geotechnical Engineer should be notified as soon as possible so that recommendations can be formulated.

- Any topsoil, slopewash, colluvium, alluvium and rock materials which are considered unsuitable by the Geotechnical Engineer should be removed prior to fill placement.

- Remaining voids created during site clearing caused by removal of trees, foundations basements, irrigation facilities, etc., should be excavated and filled with compacted fill.

- Subsequent to clearing and removals, areas to receive fill should be scarified to a depth of 10 to 12 inches, moisture conditioned and compacted

- The moisture condition of the processed ground should be at or slightly above the optimum moisture content as determined by the Geotechnical Engineer. Depending upon field conditions, this may require air drying or watering together with mixing and/or discing.

Compacted Fills

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated," and shall be very low to non-expansive with a maximum expansion index (EI) of 20. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.

- All soils should be evaluated and tested by the Geotechnical Engineer. Materials with high expansion potential, low strength, poor gradation or containing organic materials may require removal from the site or selective placement and/or mixing to the satisfaction of the Geotechnical Engineer.

- Rock fragments or rocks less than 6 inches in their largest dimensions, or as otherwise determined by the Geotechnical Engineer, may be used in compacted fill, provided the distribution and placement is satisfactory in the opinion of the Geotechnical Engineer.

- Rock fragments or rocks greater than 12 inches should be taken off-site or placed in accordance with recommendations and in areas designated as suitable by the Geotechnical Engineer. These materials should be placed in accordance with Plate D-8 of these Grading Guide Specifications and in accordance with the following recommendations:

    - Rocks 12 inches or more in diameter should be placed in rows at least 15 feet apart, 15 feet from the edge of the fill, and 10 feet or more below subgrade. Spaces should be left between each rock fragment to provide for placement and compaction of soil around the fragments.

    - Fill materials consisting of soil meeting the minimum moisture content requirements and free of oversize material should be placed between and over the rows of rock or

concrete. Ample water and compactive effort should be applied to the fill materials as they are placed in order that all of the voids between each of the fragments are filled and compacted to the specified density.

- Subsequent rows of rocks should be placed such that they are not directly above a row placed in the previous lift of fill. A minimum 5-foot offset between rows is recommended.

- To facilitate future trenching, oversized material should not be placed within the range of foundation excavations, future utilities or other underground construction unless specifically approved by the soil engineer and the developer/owner representative.

- Fill materials approved by the Geotechnical Engineer should be placed in areas previously prepared to receive fill and in evenly placed, near horizontal layers at about 6 to 8 inches in loose thickness, or as otherwise determined by the Geotechnical Engineer for the project.

- Each layer should be moisture conditioned to optimum moisture content, or slightly above, as directed by the Geotechnical Engineer. After proper mixing and/or drying, to evenly distribute the moisture, the layers should be compacted to at least 90 percent of the maximum dry density in compliance with ASTM D-1557-78 unless otherwise indicated.

- Density and moisture content testing should be performed by the Geotechnical Engineer at random intervals and locations as determined by the Geotechnical Engineer. These tests are intended as an aid to the Earthwork Contractor, so he can evaluate his workmanship, equipment effectiveness and site conditions. The Earthwork Contractor is responsible for compaction as required by the Geotechnical Report(s) and governmental agencies.

- Fill areas unused for a period of time may require moisture conditioning, processing and recompaction prior to the start of additional filling. The Earthwork Contractor should notify the Geotechnical Engineer of his intent so that an evaluation can be made.

- Fill placed on ground sloping at a 5-to-1 inclination (horizontal-to-vertical) or steeper should be benched into bedrock or other suitable materials, as directed by the Geotechnical Engineer. Typical details of benching are illustrated on Plates D-2, D-4, and D-5.

- Cut/fill transition lots should have the cut portion overexcavated to a depth of at least 3 feet and rebuilt with fill (see Plate D-1), as determined by the Geotechnical Engineer.

- All cut lots should be inspected by the Geotechnical Engineer for fracturing and other bedrock conditions. If necessary, the pads should be overexcavated to a depth of 3 feet and rebuilt with a uniform, more cohesive soil type to impede moisture penetration.

- Cut portions of pad areas above buttresses or stabilizations should be overexcavated to a depth of 3 feet and rebuilt with uniform, more cohesive compacted fill to impede moisture penetration.

- Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure that excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls must be properly tested and approved by the Geotechnical Engineer with consideration of the lateral earth pressure used in the design.

Foundations

- The foundation influence zone is defined as extending one foot horizontally from the outside edge of a footing, and proceeding downward at a ½ horizontal to 1 vertical (0.5:1) inclination.

- Where overexcavation beneath a footing subgrade is necessary, it should be conducted so as to encompass the entire foundation influence zone, as described above.

- Compacted fill adjacent to exterior footings should extend at least 12 inches above foundation bearing grade. Compacted fill within the interior of structures should extend to the floor subgrade elevation.

Fill Slopes

- The placement and compaction of fill described above applies to all fill slopes. Slope compaction should be accomplished by overfilling the slope, adequately compacting the fill in even layers, including the overfilled zone and cutting the slope back to expose the compacted core

- Slope compaction may also be achieved by backrolling the slope adequately every 2 to 4 vertical feet during the filling process as well as requiring the earth moving and compaction equipment to work close to the top of the slope. Upon completion of slope construction, the slope face should be compacted with a sheepsfoot connected to a sideboom and then grid rolled. This method of slope compaction should only be used if approved by the Geotechnical Engineer.

- Sandy soils lacking in adequate cohesion may be unstable for a finished slope condition and therefore should not be placed within 15 horizontal feet of the slope face.

- All fill slopes should be keyed into bedrock or other suitable material. Fill keys should be at least 15 feet wide and inclined at 2 percent into the slope. For slopes higher than 30 feet, the fill key width should be equal to one-half the height of the slope (see Plate D-5).

- All fill keys should be cleared of loose slough material prior to geotechnical inspection and should be approved by the Geotechnical Engineer and governmental agencies prior to filling.

- The cut portion of fill over cut slopes should be made first and inspected by the Geotechnical Engineer for possible stabilization requirements. The fill portion should be adequately keyed through all surficial soils and into bedrock or suitable material. Soils should be removed from the transition zone between the cut and fill portions (see Plate D-2).

Cut Slopes

- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.

- Cut slopes exposing loose, cohesionless sands should be reported to the Geotechnical Engineer for possible stabilization recommendations.

- All stabilization excavations should be cleared of loose slough material prior to geotechnical inspection. Stakes should be provided by the Civil Engineer to verify the location and dimensions of the key. A typical stabilization fill detail is shown on Plate D-5.

- Stabilization key excavations should be provided with subdrains. Typical subdrain details are shown on Plates D-6.

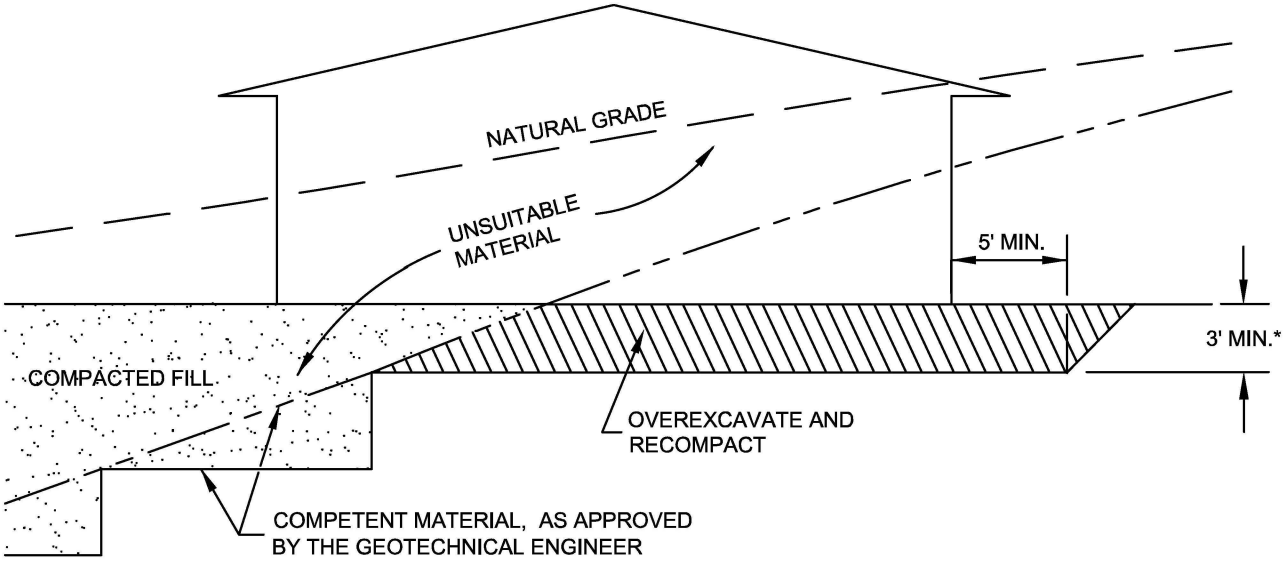
Subdrains

- Subdrains may be required in canyons and swales where fill placement is proposed. Typical subdrain details for canyons are shown on Plate D-3. Subdrains should be installed after approval of removals and before filling, as determined by the Soils Engineer.

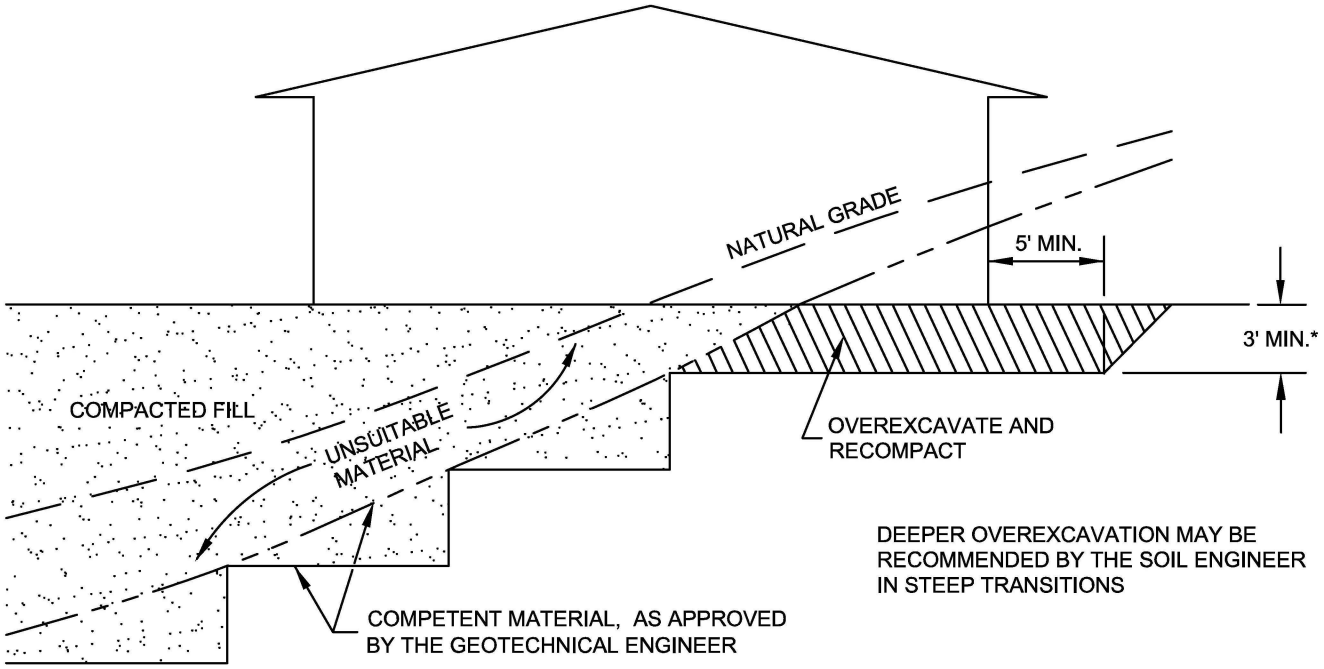
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- Filter material for subdrains should conform to CALTRANS Specification 68-1.025 or as approved by the Geotechnical Engineer for the specific site conditions. Clean ¾-inch crushed rock may be used provided it is wrapped in an acceptable filter cloth and approved by the Geotechnical Engineer. Pipe diameters should be 6 inches for runs up to 500 feet and 8 inches for the downstream continuations of longer runs. Four-inch diameter pipe may be used in buttress and stabilization fills.

CUT LOT



CUT/FILL LOT (TRANSITION)



*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION. ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

TRANSITION LOT DETAIL

GRADING GUIDE SPECIFICATIONS

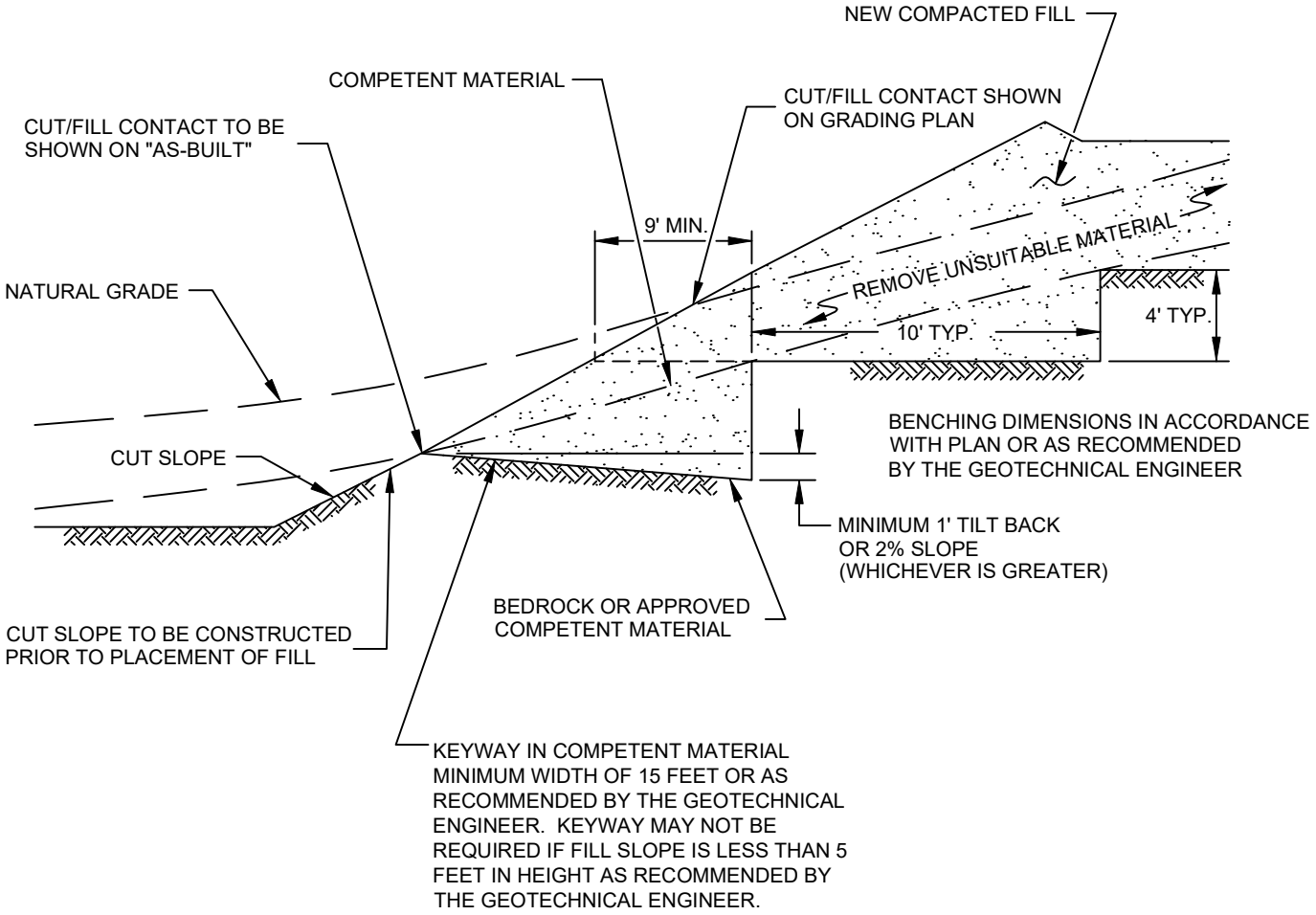
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

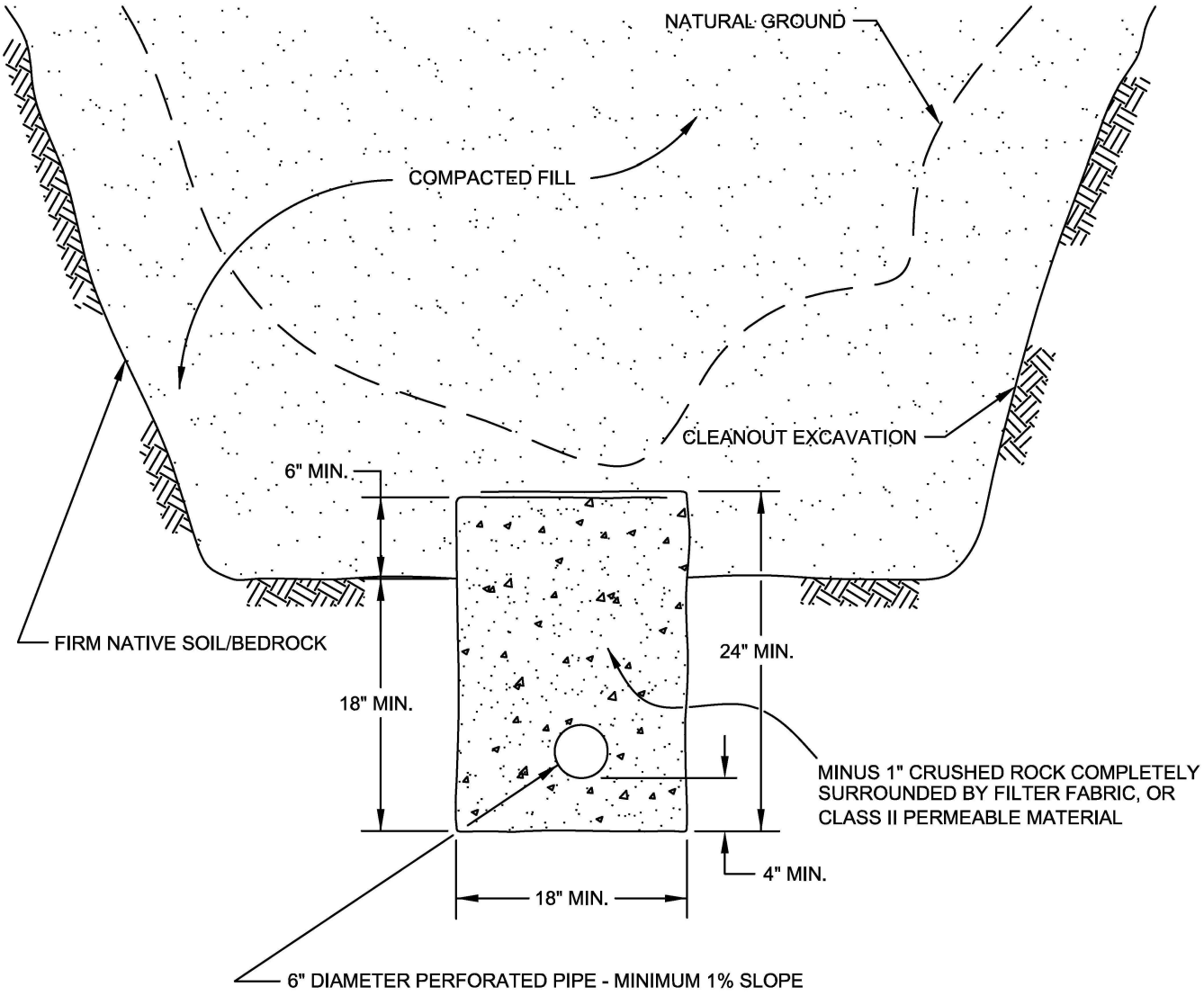
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



PIPE MATERIAL	DEPTH OF FILL OVER SUBDRAIN
ADS (CORRUGATED POLETHYLENE)	8
TRANSITE UNDERDRAIN	20
PVC OR ABS: SDR 35	35
SDR 21	100

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

GRADING GUIDE SPECIFICATIONS

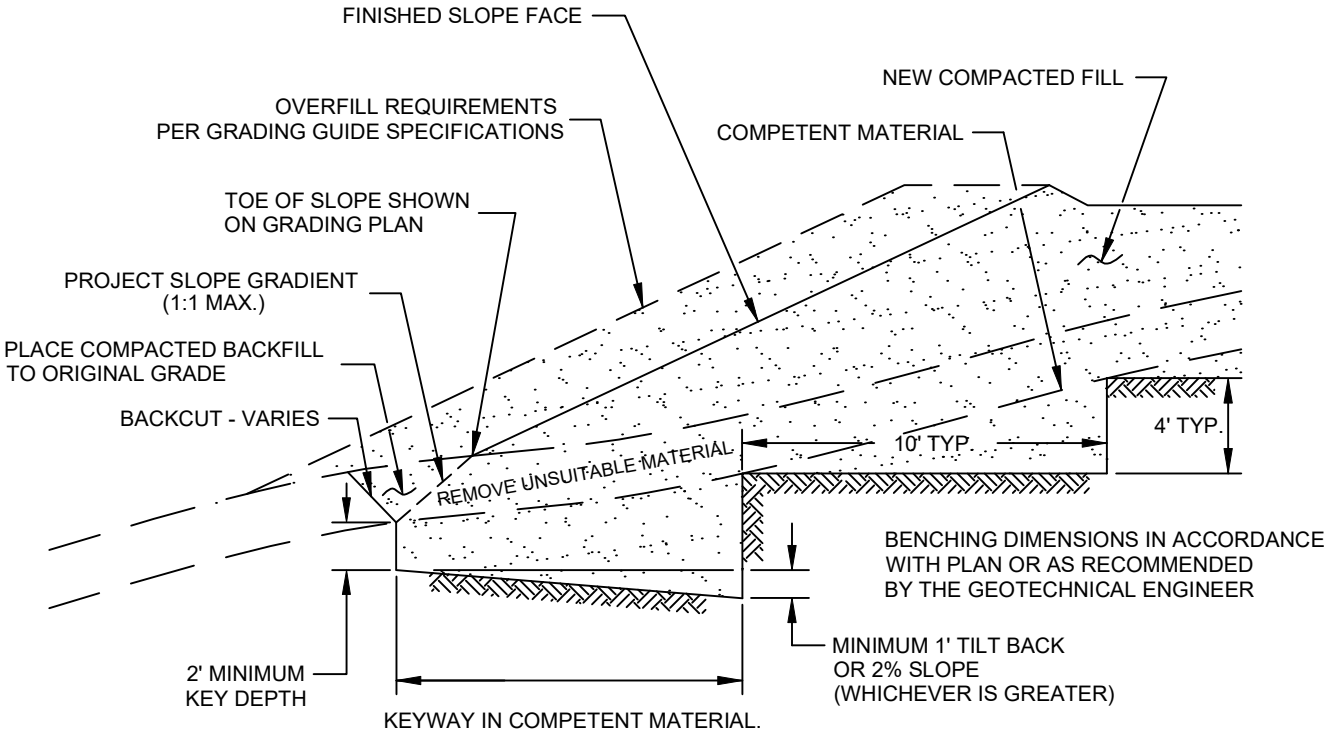
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



NOTE:
BENCHING SHALL BE REQUIRED
WHEN NATURAL SLOPES ARE
EQUAL TO OR STEEPER THAN 5:1
OR WHEN RECOMMENDED BY
THE GEOTECHNICAL ENGINEER.

FILL ABOVE NATURAL SLOPE DETAIL

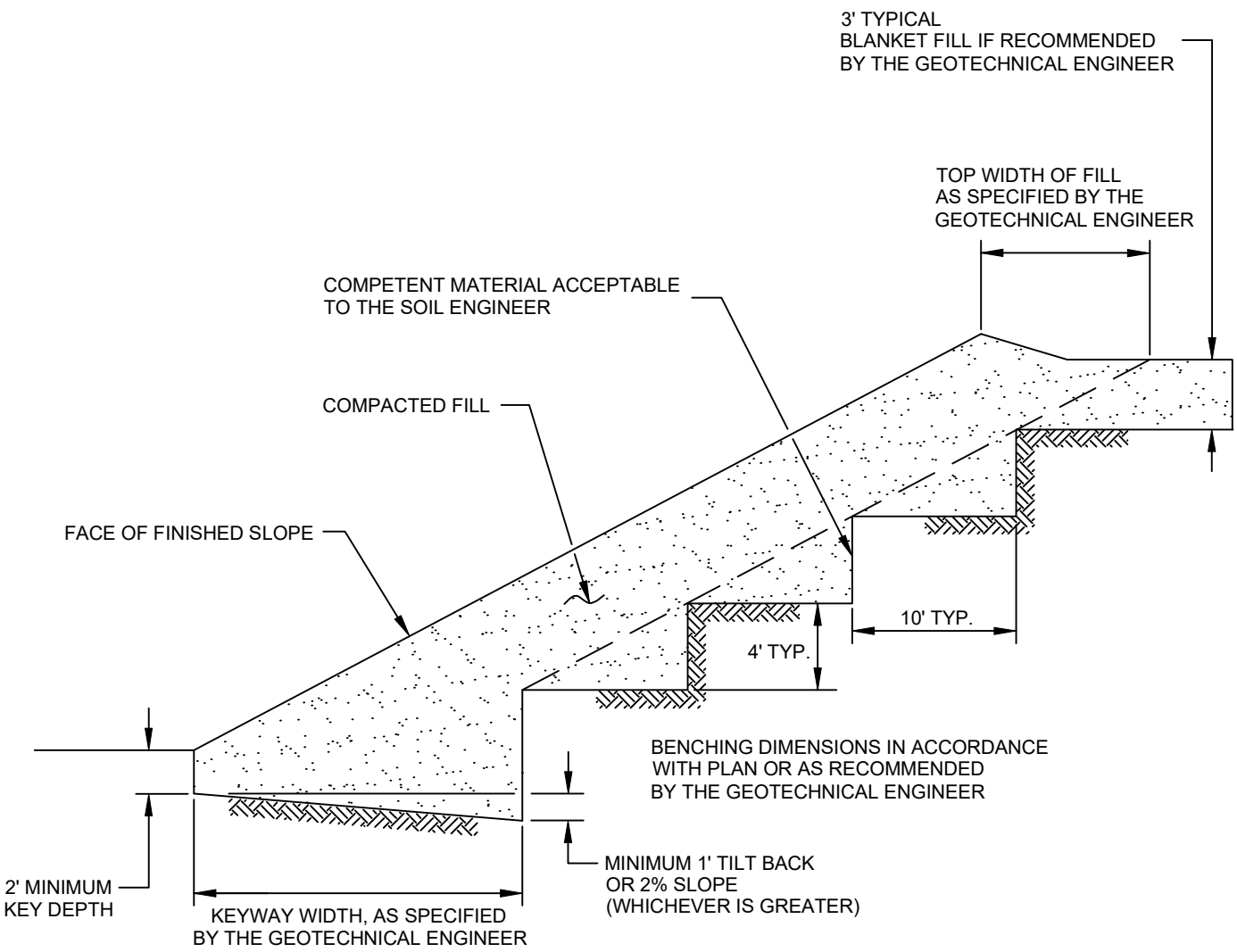
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

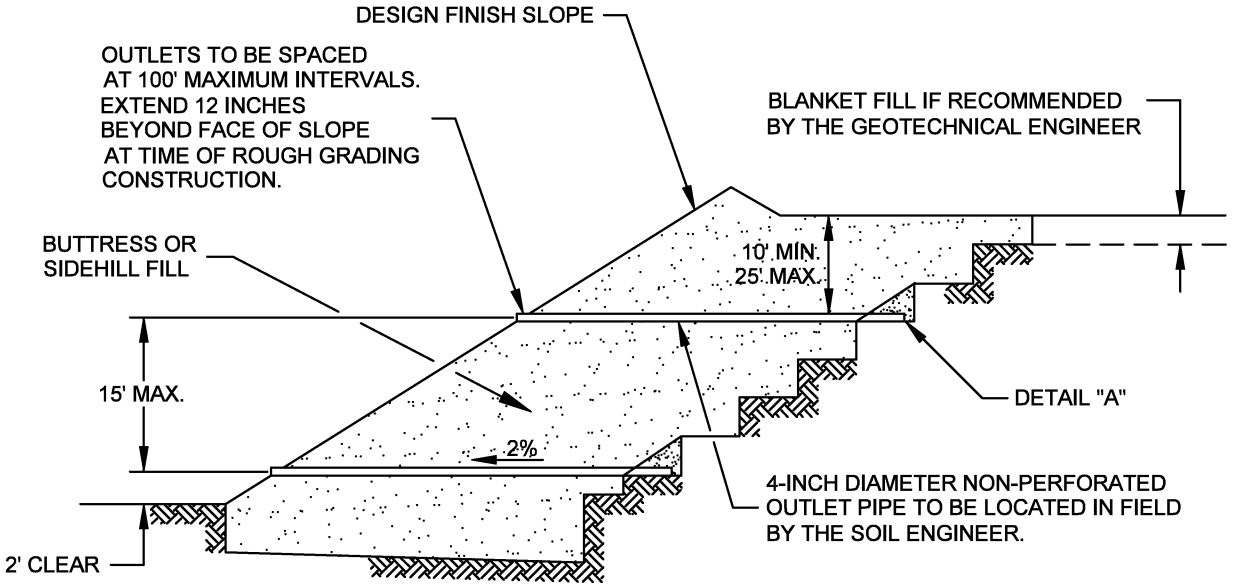
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



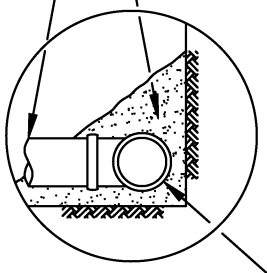
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50



DETAIL "A"

NOTES:
1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

SLOPE FILL SUBDRAINS

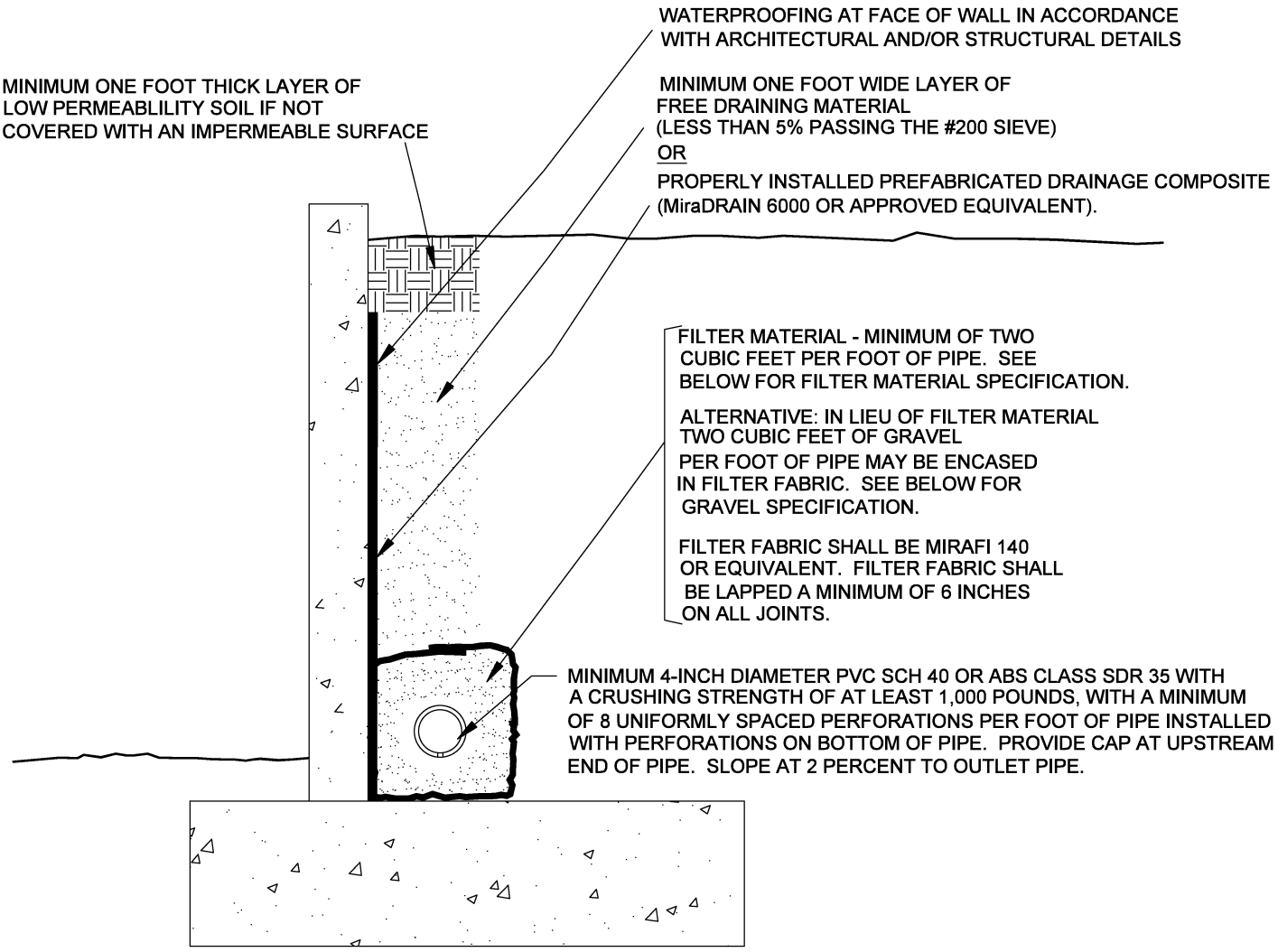
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

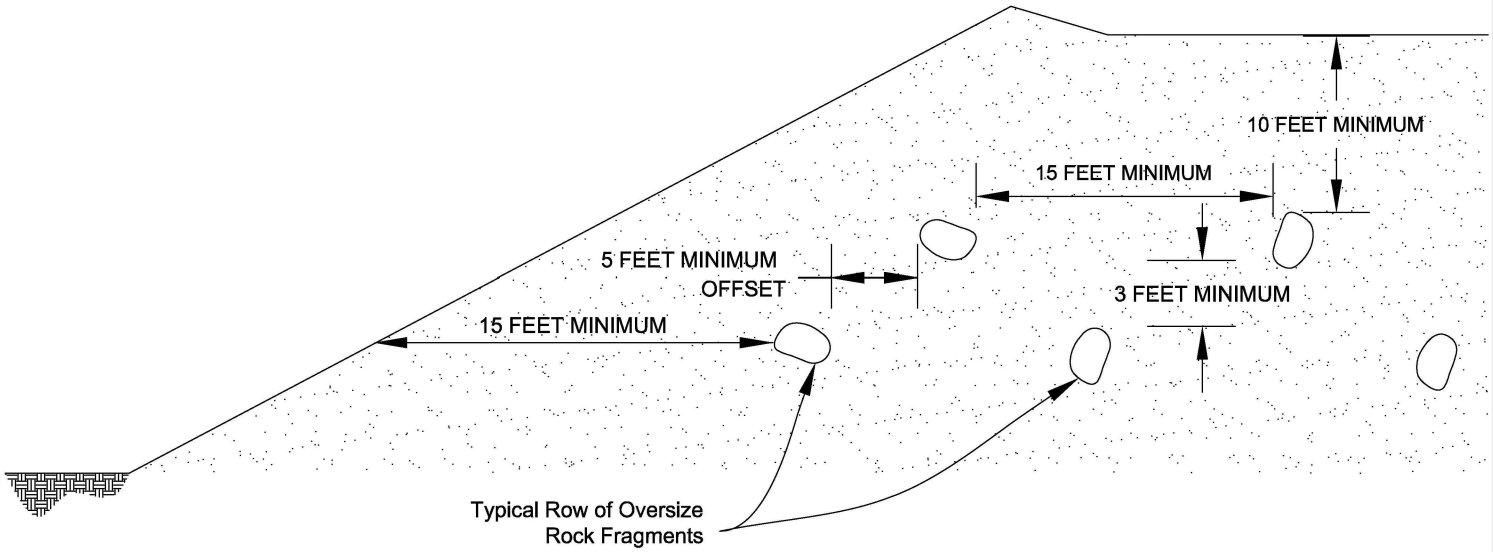
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

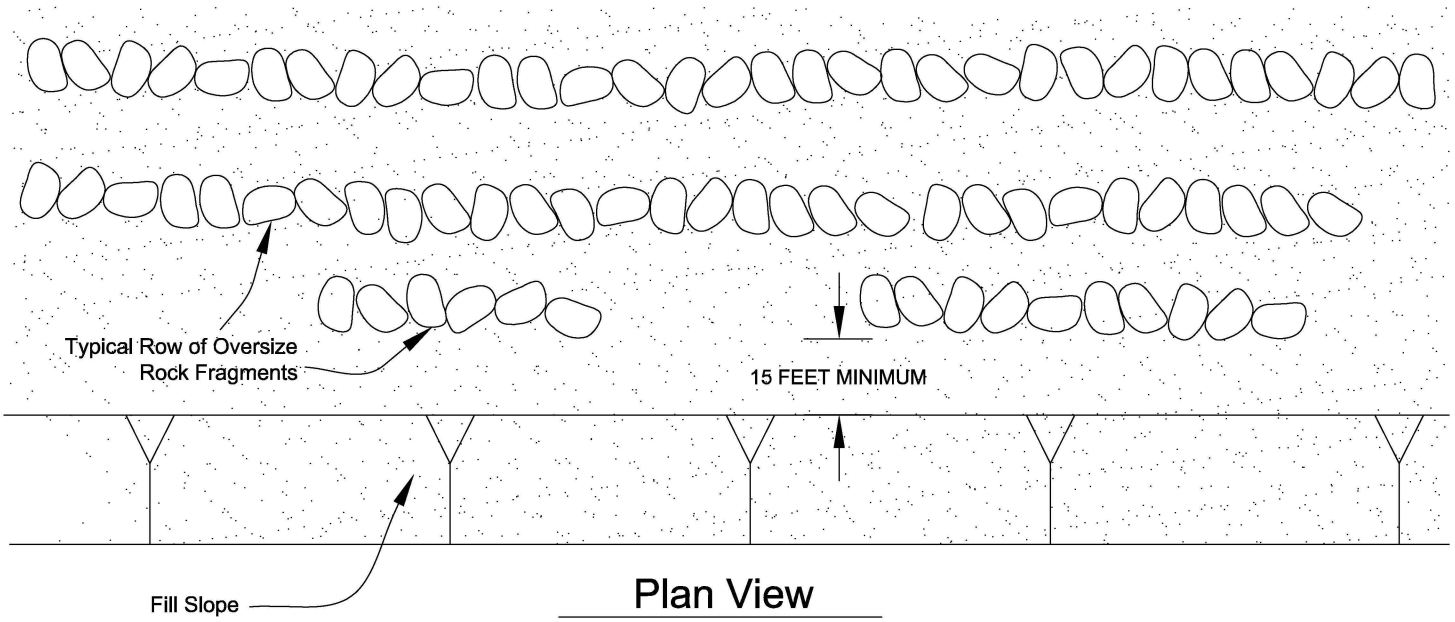
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

OSHPD

Latitude, Longitude: 33.82931100, -118.21069760



Date	9/19/2024, 1:24:59 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.685	MCE_R ground motion. (for 0.2 second period)
S_1	0.607	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.685	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.123	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.736	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.81	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.685	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.869	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.423	Factored deterministic acceleration value. (0.2 second)
S1RT	0.607	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.675	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.835	Factored deterministic acceleration value. (1.0 second)
PGAd	0.987	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.736	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.901	Mapped value of the risk coefficient at short periods
C_{R1}	0.899	Mapped value of the risk coefficient at a period of 1 s
C_V	1.437	Vertical coefficient

SOURCE: SEAOC/OSHPD Seismic Design Maps Tool
<https://seismicmaps.org/>

SEISMIC DESIGN PARAMETERS - 2022 CBC
PROPOSED INDUSTRIAL BUILDING
LONG BEACH, CALIFORNIA

DRAWN: MK
CHKD: DN

SCG PROJECT
24G159-1

PLATE E-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

Proposed Industrial Building
3800 Via Oro Avenue
Long Beach, CA
September 12, 2024

Prepared for:

Mr. Daryl Kas
Southern California Geotechnical, Inc.
22885 E. Savi Ranch Parkway, Ste E
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Office (714) 685-1115 / Fax (714) 685-1118

Prepared by:



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Huntington Beach, CA 92649-1518
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www.kehoetesting.com

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1. **INTRODUCTION**
2. **SUMMARY OF FIELD WORK**
3. **FIELD EQUIPMENT & PROCEDURES**
4. **CONE PENETRATION TEST DATA & INTERPRETATION**

APPENDIX

- CPT Plots
- CPT Classification/Soil Behavior Chart
- Pore Pressure Dissipation Graphs
- CPT Data Files (sent via email)

SUMMARY
OF
CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the Proposed Industrial Building project located at 3800 Via Oro Avenue in Long Beach, California. The work was performed by Kehoe Testing & Engineering (KTE) on September 12, 2024. The scope of work was performed as directed by Southern California Geotechnical, Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at four locations to determine the soil lithology. A summary is provided in **TABLE 2.1**.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	50	
CPT-2	62	
CPT-3	50	
CPT-4	51	

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 cm^2 cone with a cone net area ratio of 0.83. The following parameters were recorded 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)

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for up to 2 years 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 behavior type on the CPT plots is derived from the attached CPT SBT plot (Robertson, "Interpretation of Cone Penetration Test…", 2009) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), 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.

The CPT data files have also been provided. These files can be imported in CPeT-IT (software by GeoLogismiki) and other programs to calculate various geotechnical parameters.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs 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



Steven P. Kehoe
President

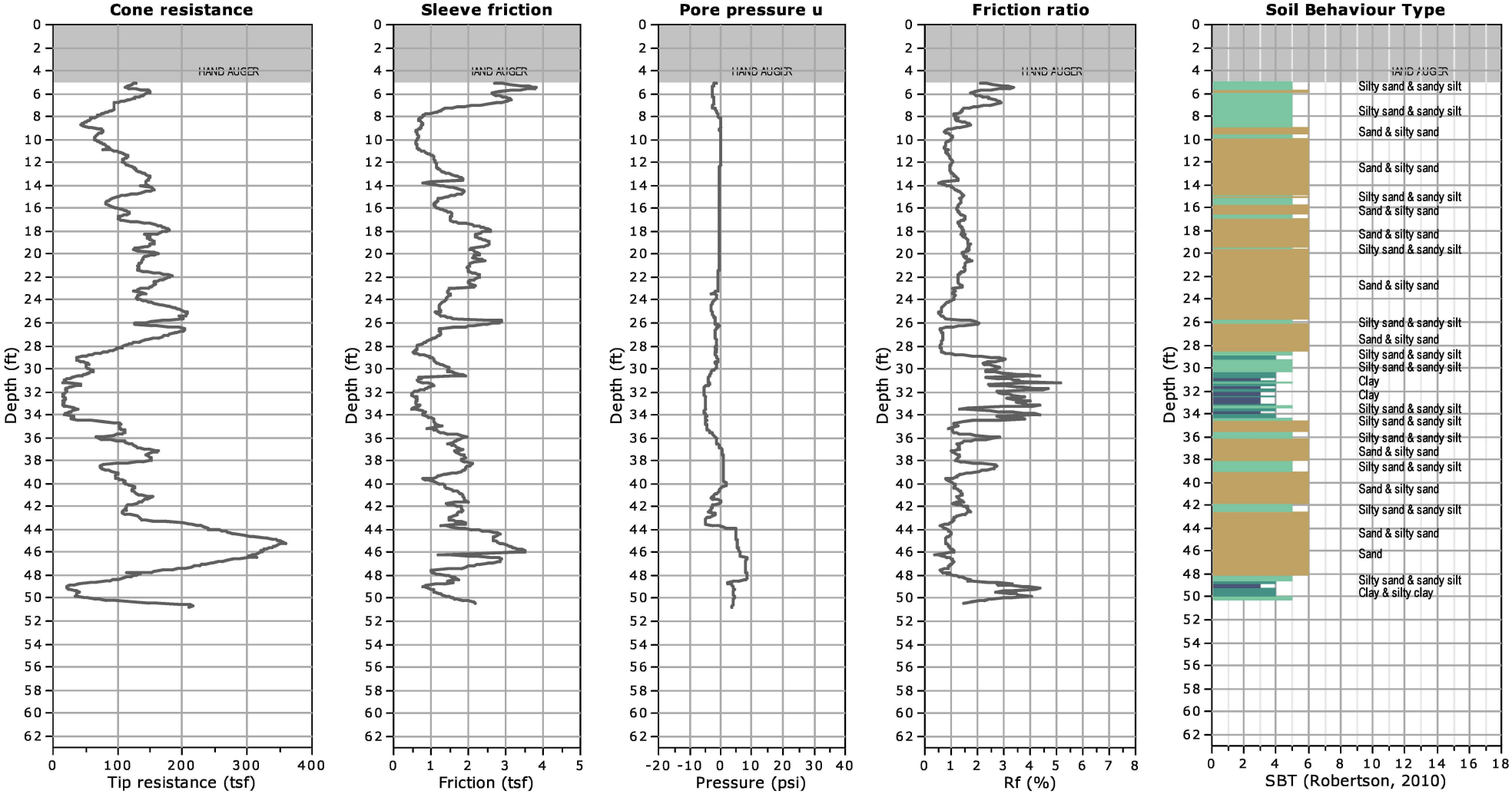
09/17/24-eb-6835

APPENDIX

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

Project: **Southern California Geotechnical / Proposed Industrial Building**
Location: **3800 Via Oro Ave, Long Beach, CA**

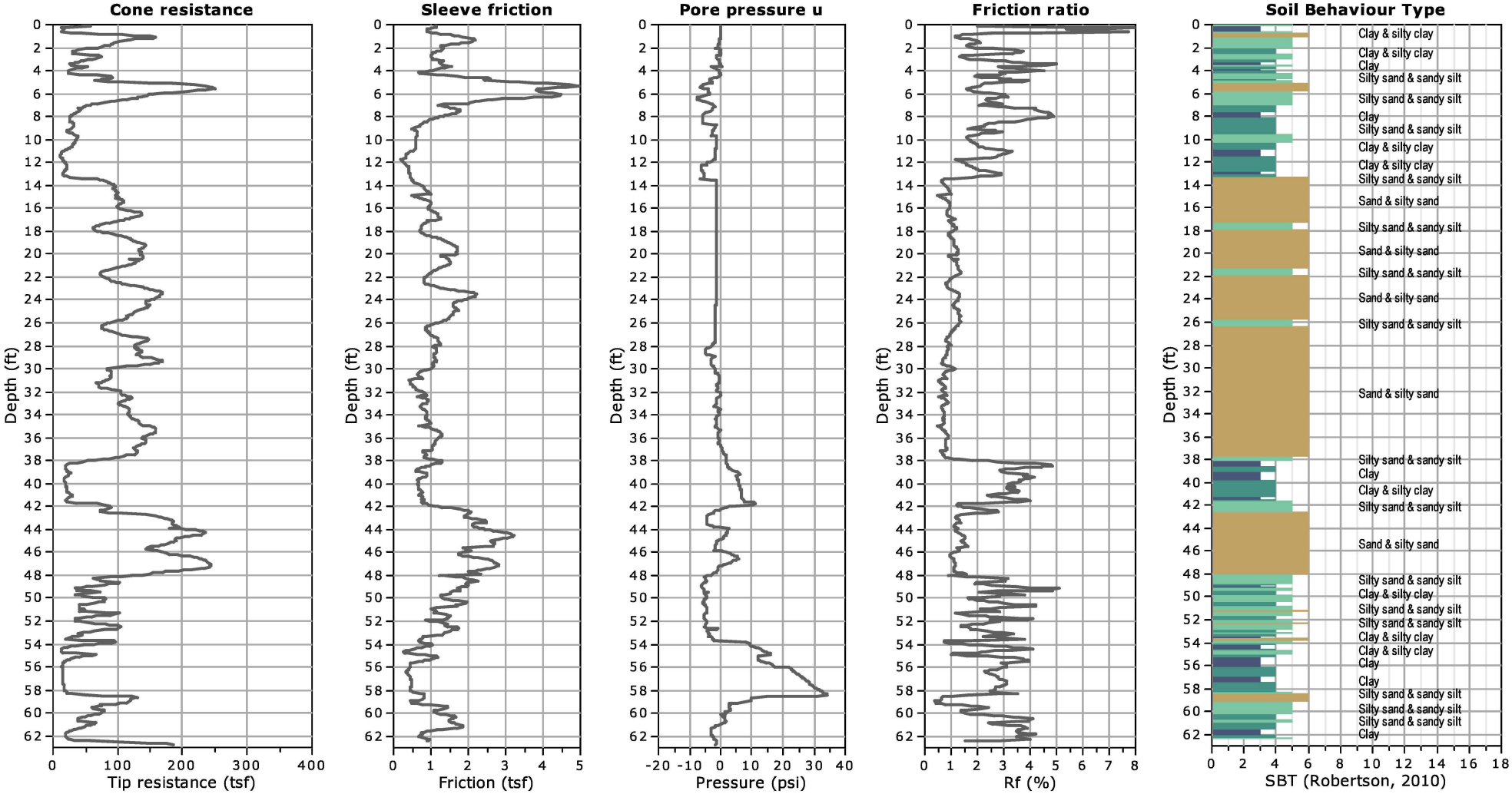
CPT-1
Total depth: 50.73 ft, Date: 9/12/2024



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

Project: **Southern California Geotechnical / Proposed Industrial Building**
Location: **3800 Via Oro Ave, Long Beach, CA**

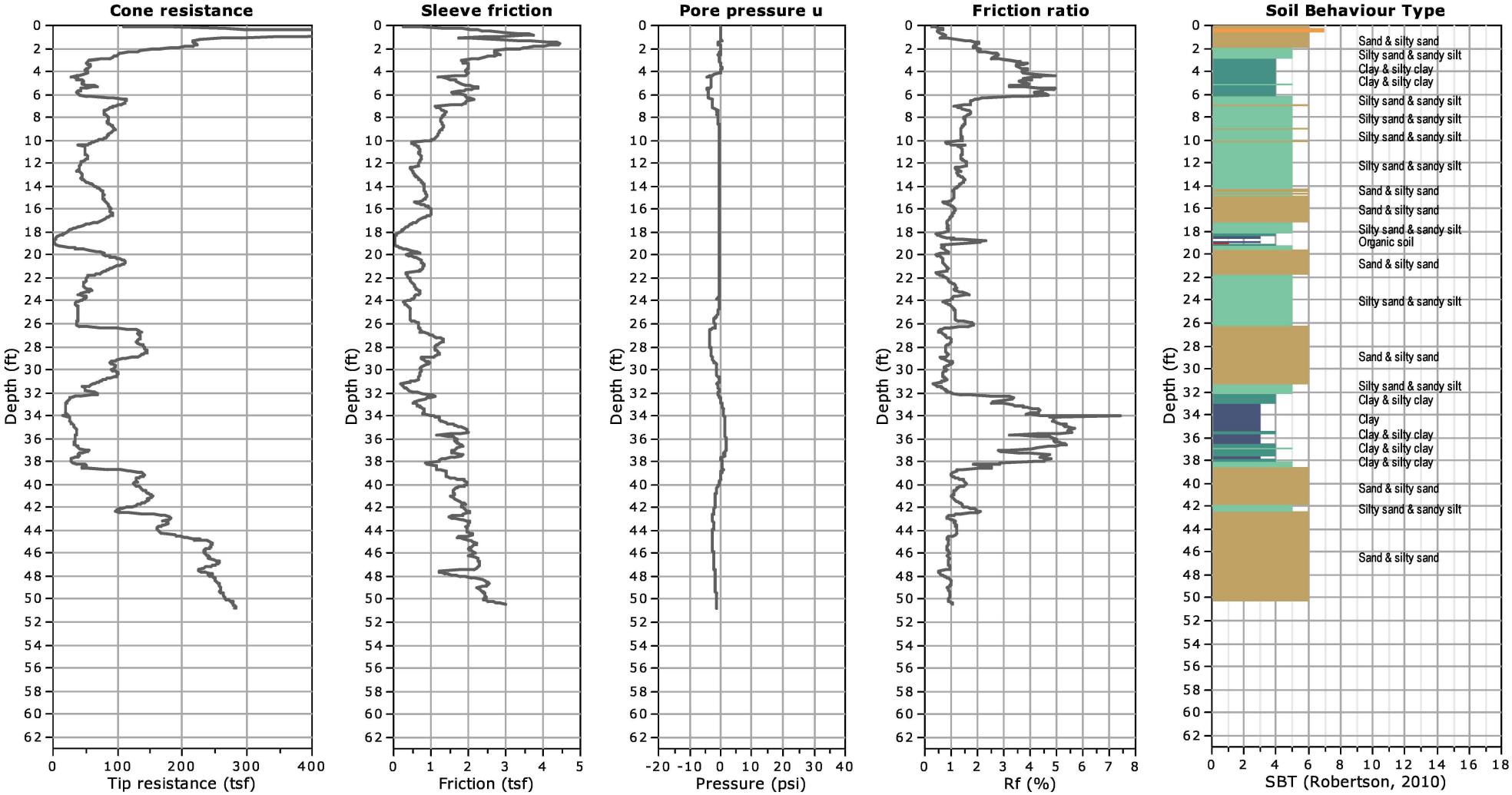
CPT-2
Total depth: 62.77 ft, Date: 9/12/2024



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

Project: Southern California Geotechnical / Proposed Industrial Building
Location: 3800 Via Oro Ave, Long Beach, CA

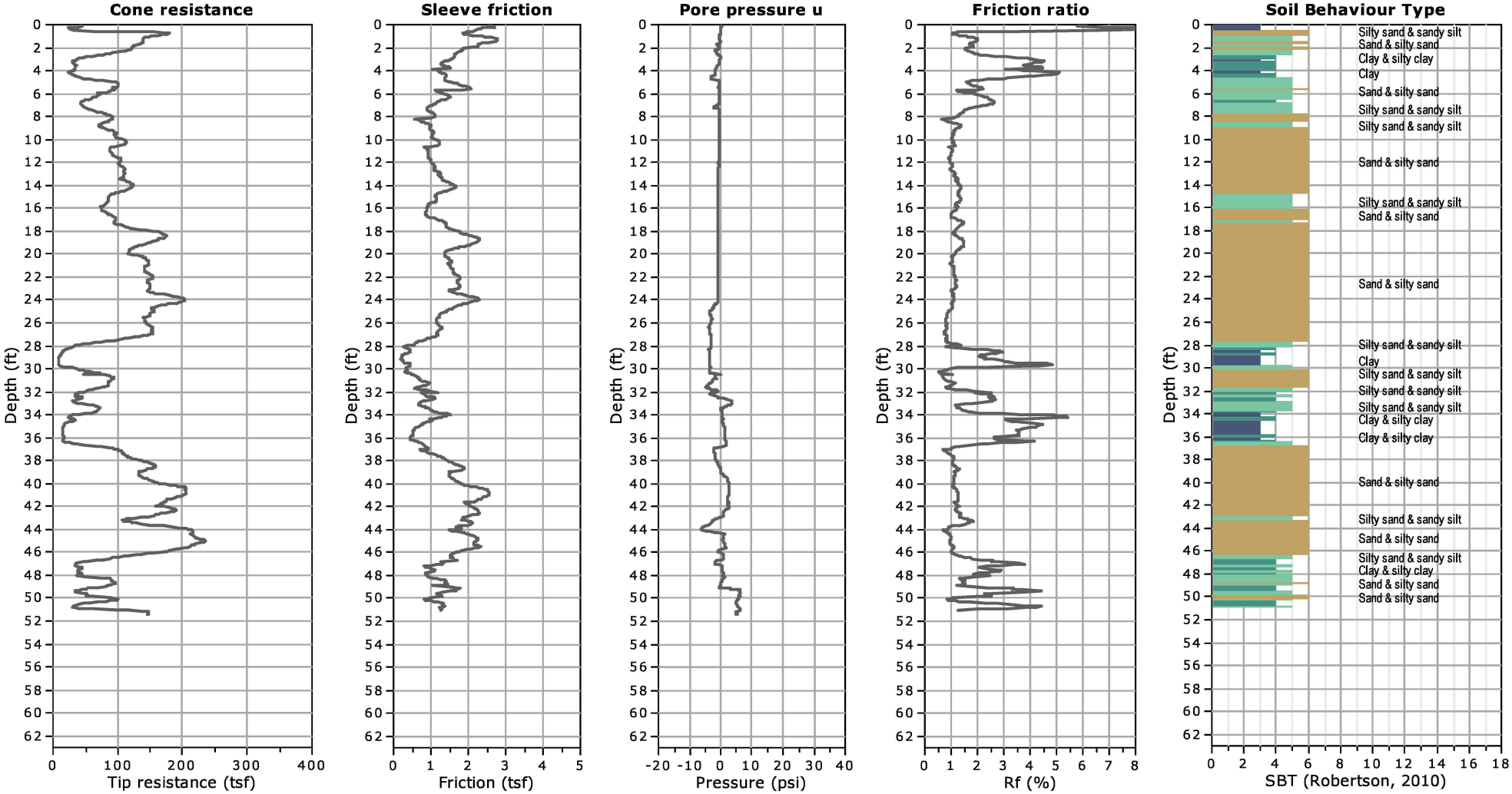
CPT-3
Total depth: 50.80 ft, Date: 9/12/2024



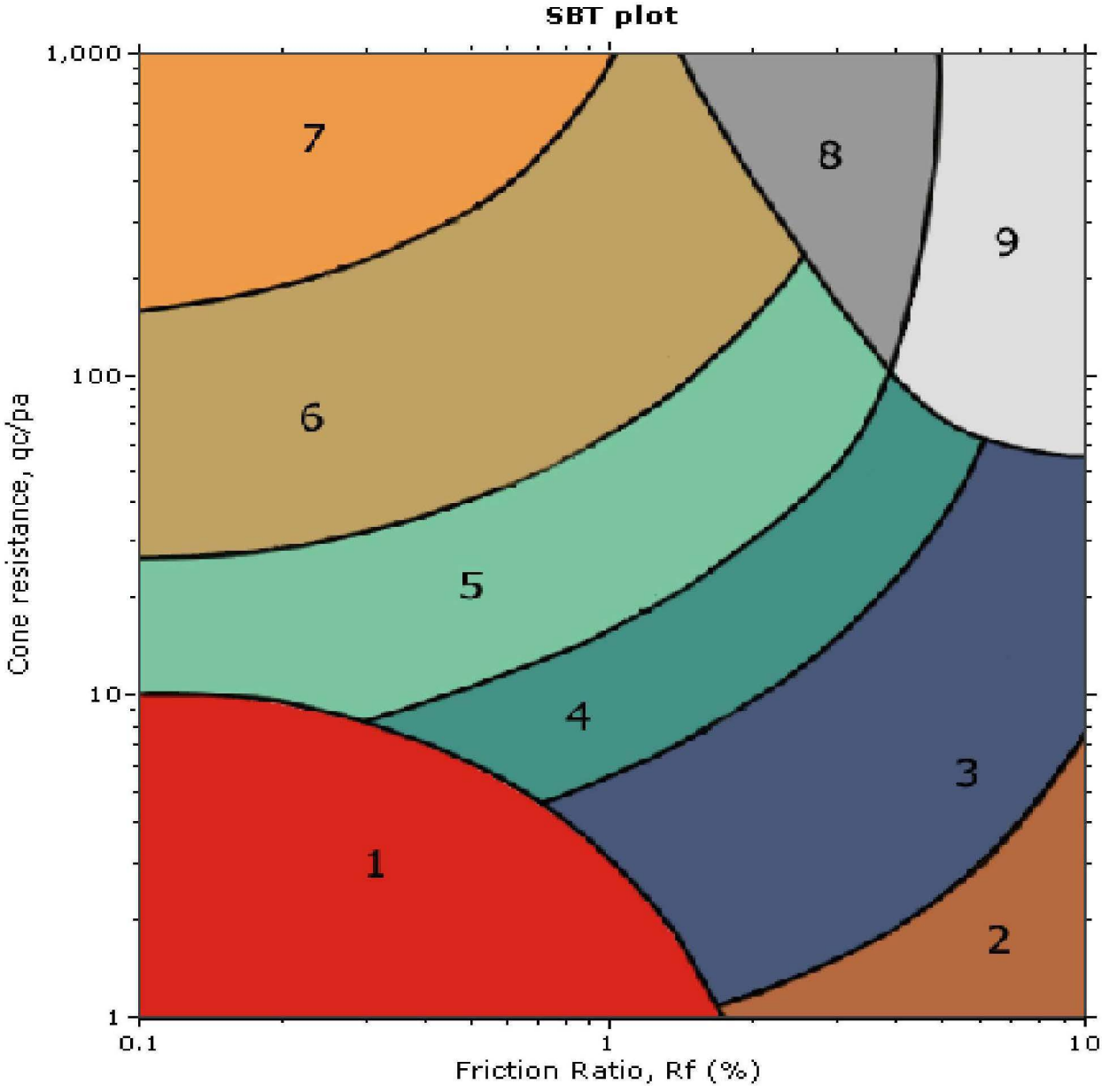
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **Southern California Geotechnical / Proposed Industrial Building**
Location: **3800 Via Oro Ave, Long Beach, CA**

CPT-4
Total depth: 51.39 ft, Date: 9/12/2024



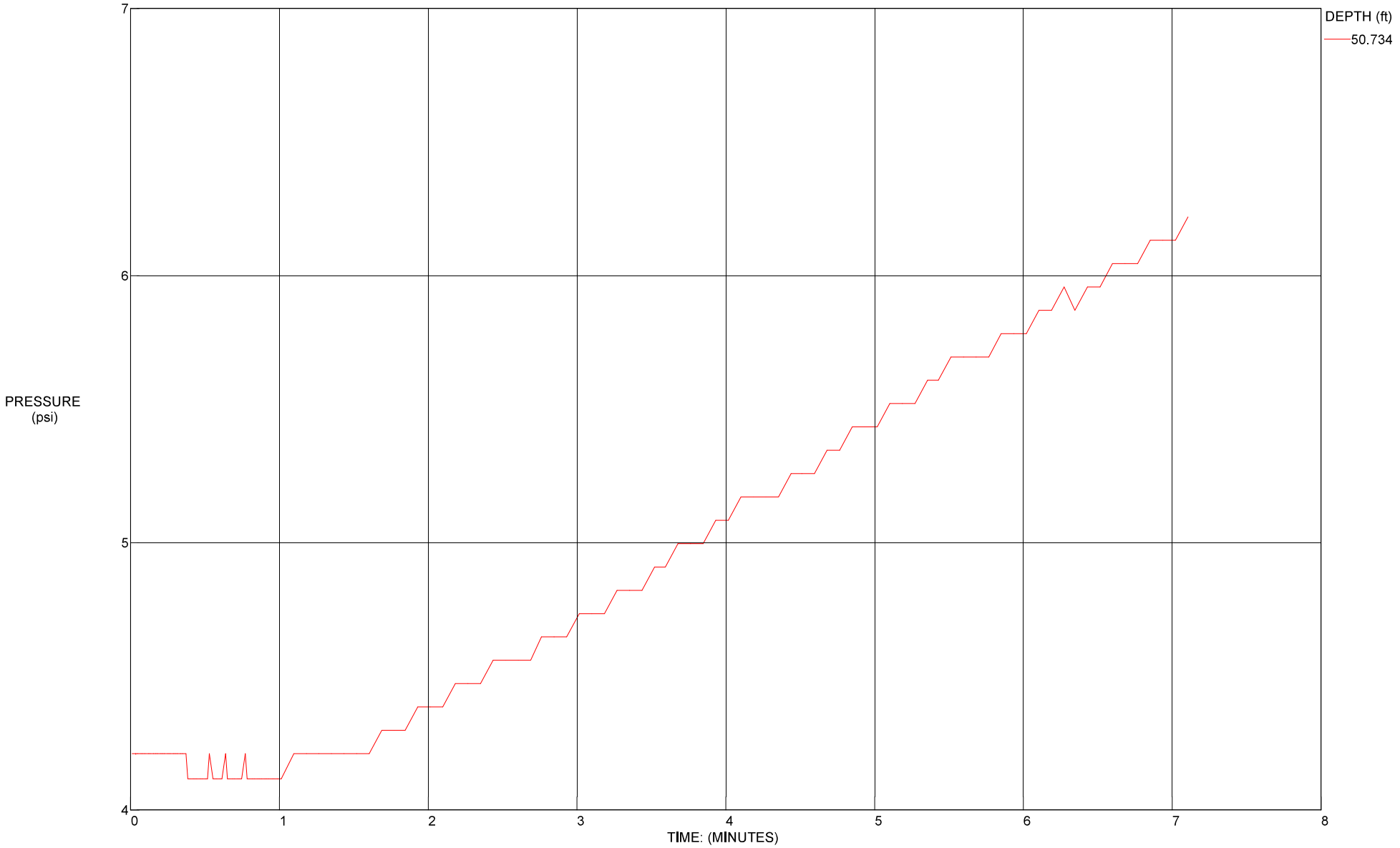
Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com



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

TEST ID: CPT-1



APPENDIX G

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Summary data report 1
Vertical settlements summary report 2

CPT-2 results
Summary data report 3
Vertical settlements summary report 4

CPT-3 results
Summary data report 5
Vertical settlements summary report 6

CPT-4 results
Summary data report 7
Vertical settlements summary report 8

Southern California Geotechnical, Inc.
www.socalgeo.com

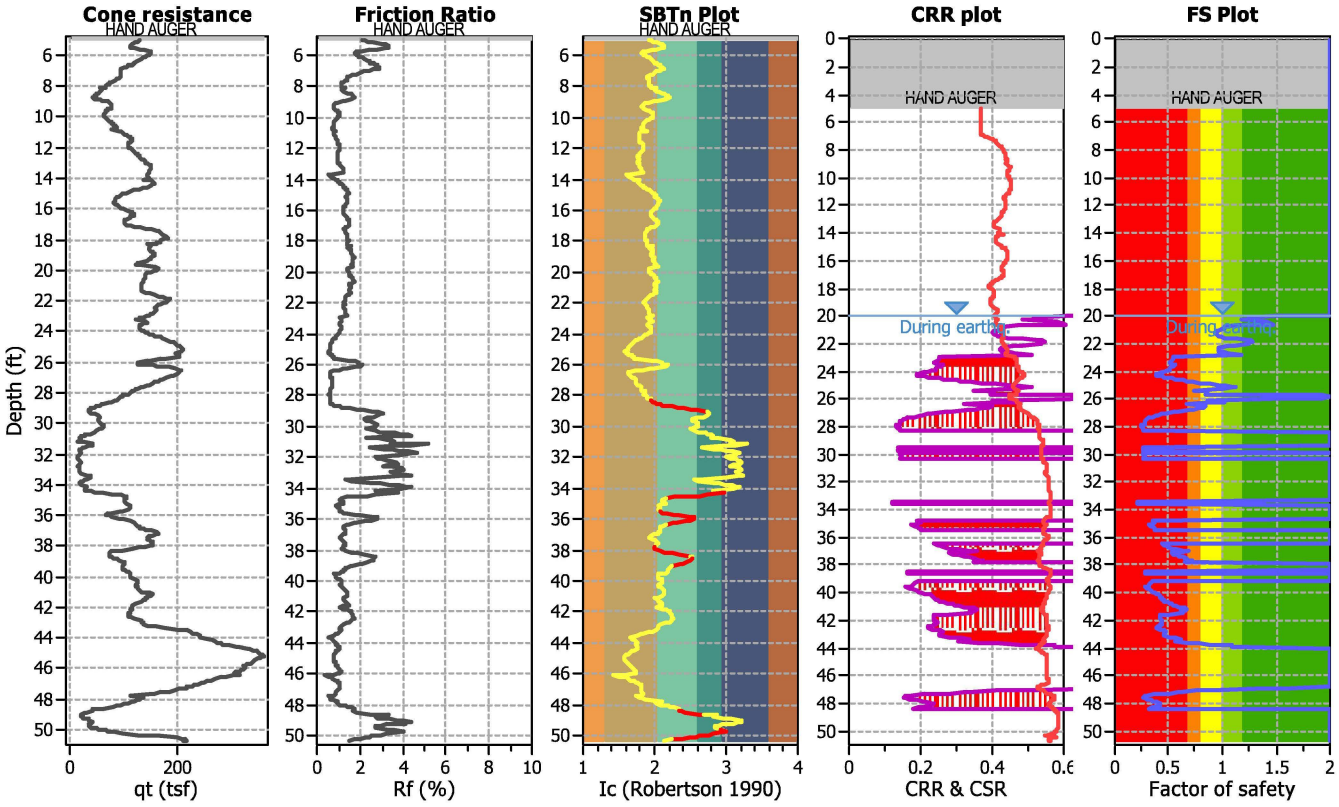
LIQUEFACTION ANALYSIS REPORT

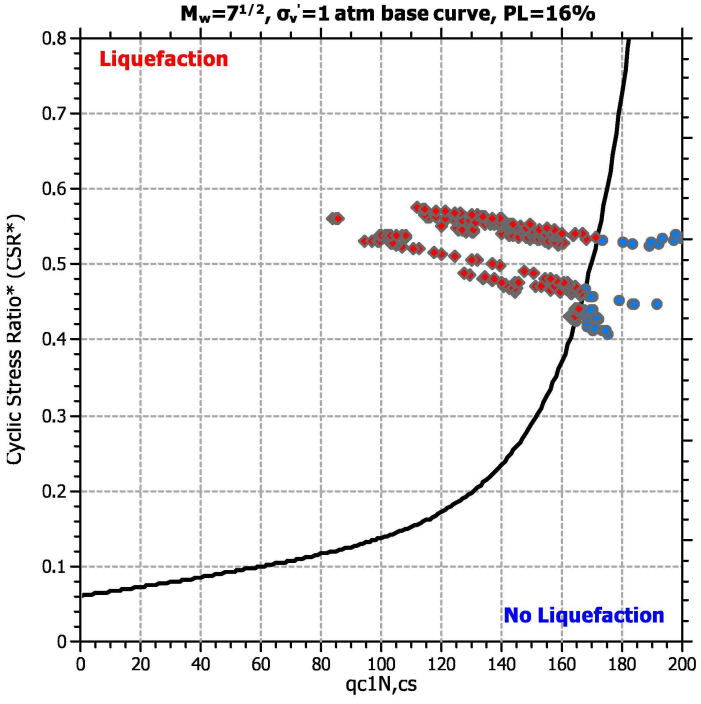
Project title : Proposed Industrial Building **Location : Long Beach, CA**

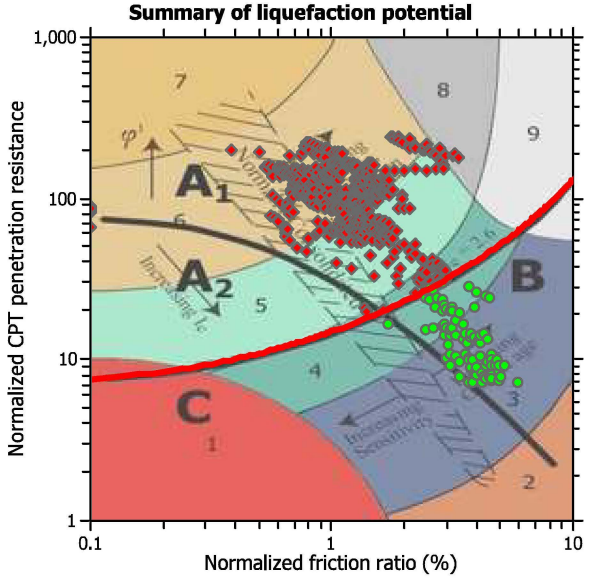
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	27.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.85	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.81	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

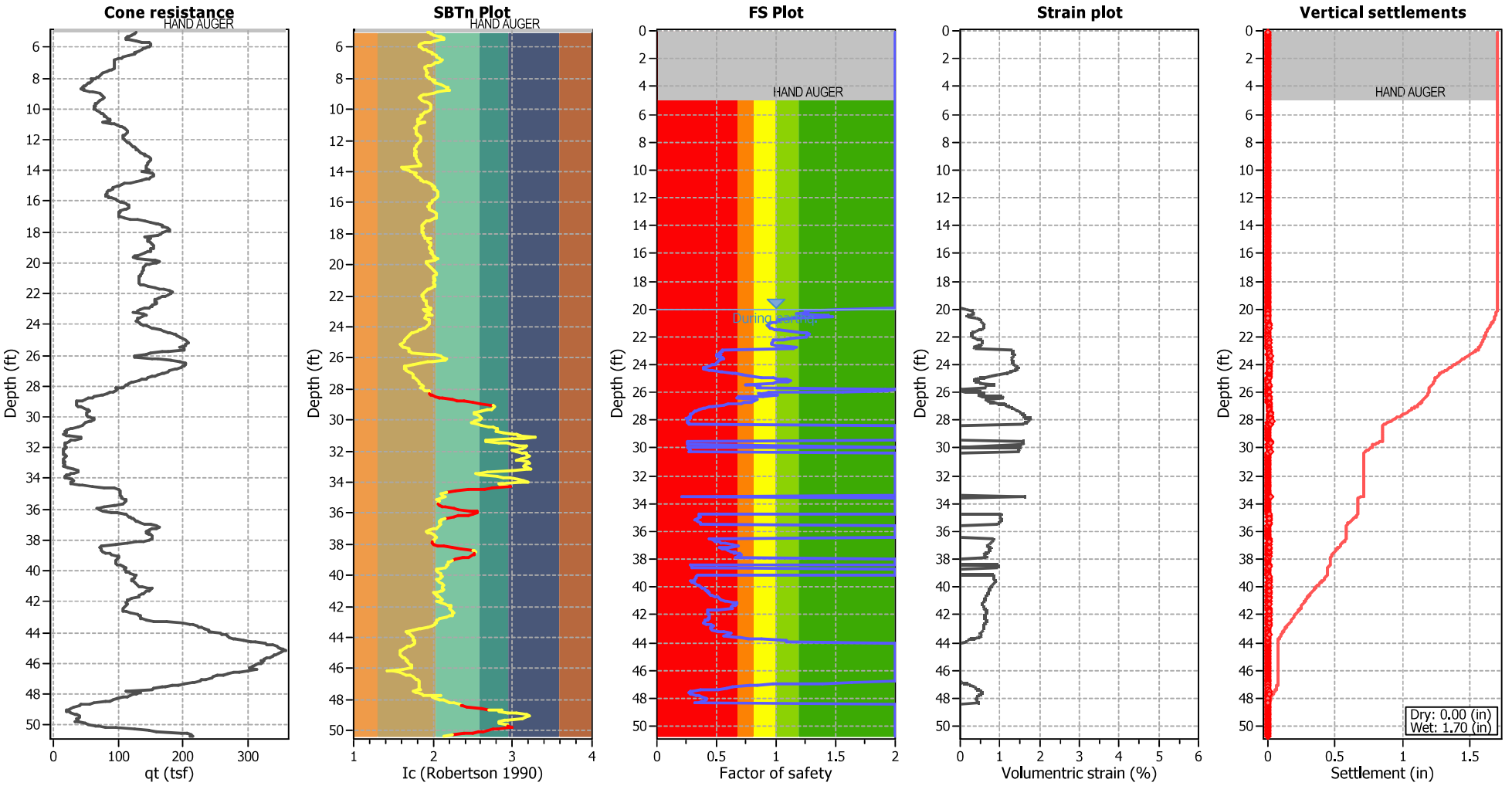






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

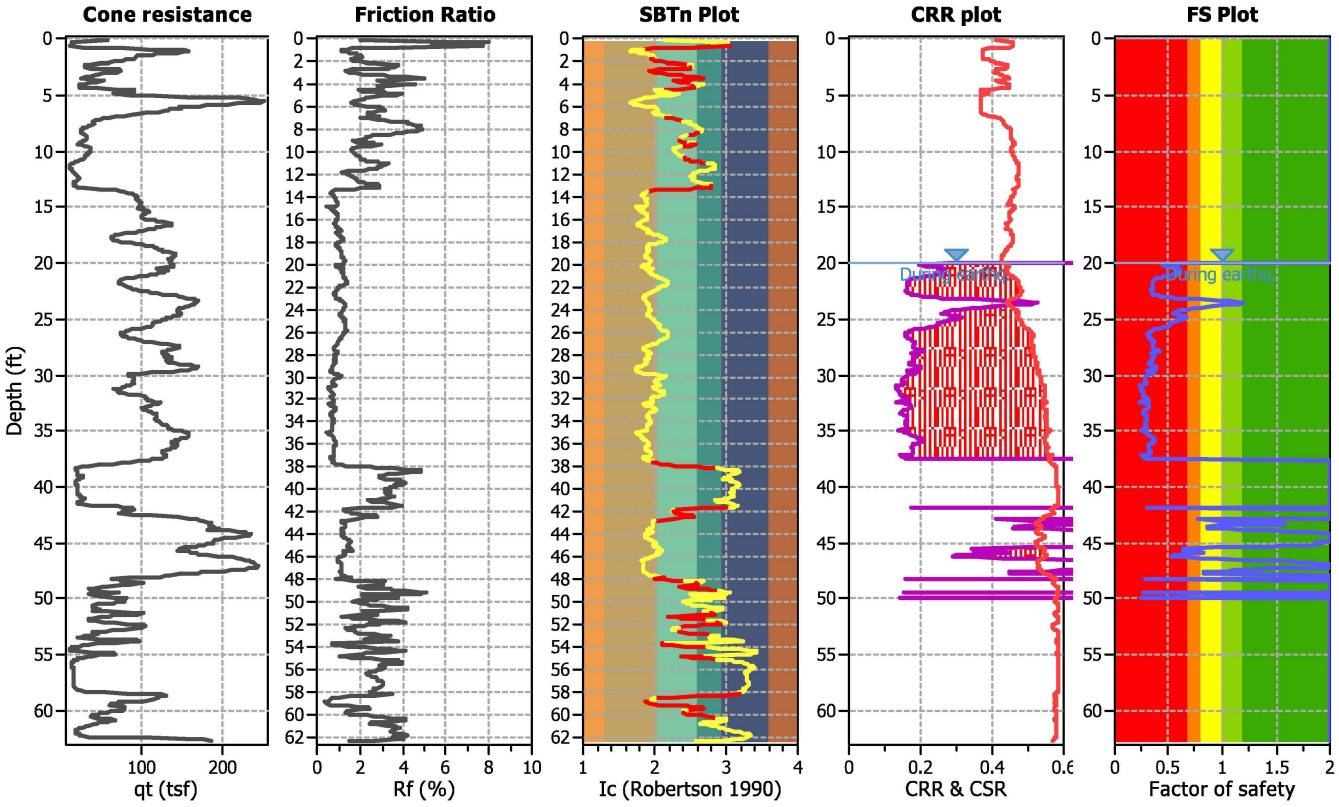
LIQUEFACTION ANALYSIS REPORT

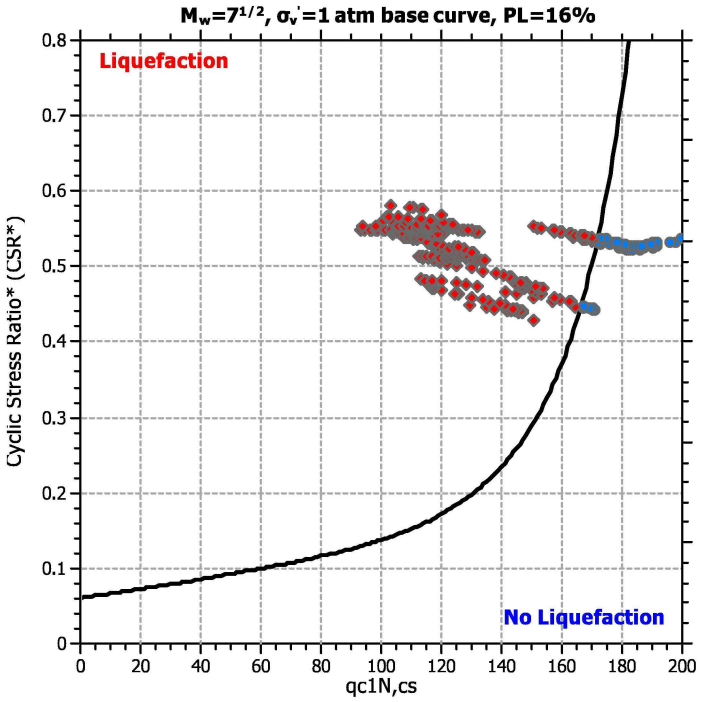
Project title : Proposed Industrial Building **Location : Long Beach, CA**

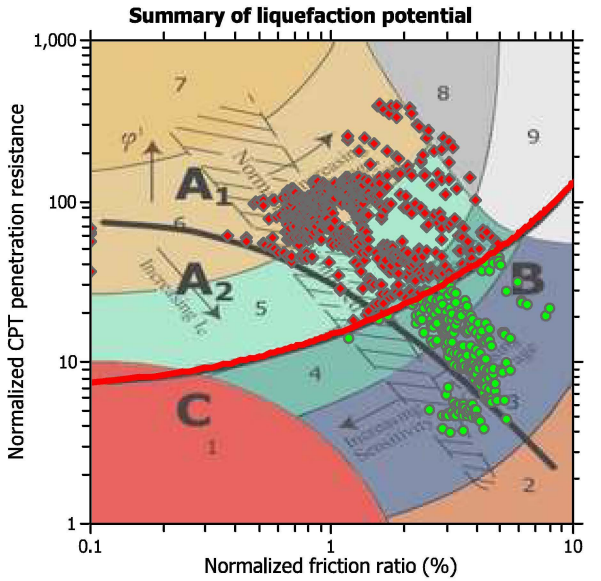
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	26.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.85	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.81	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

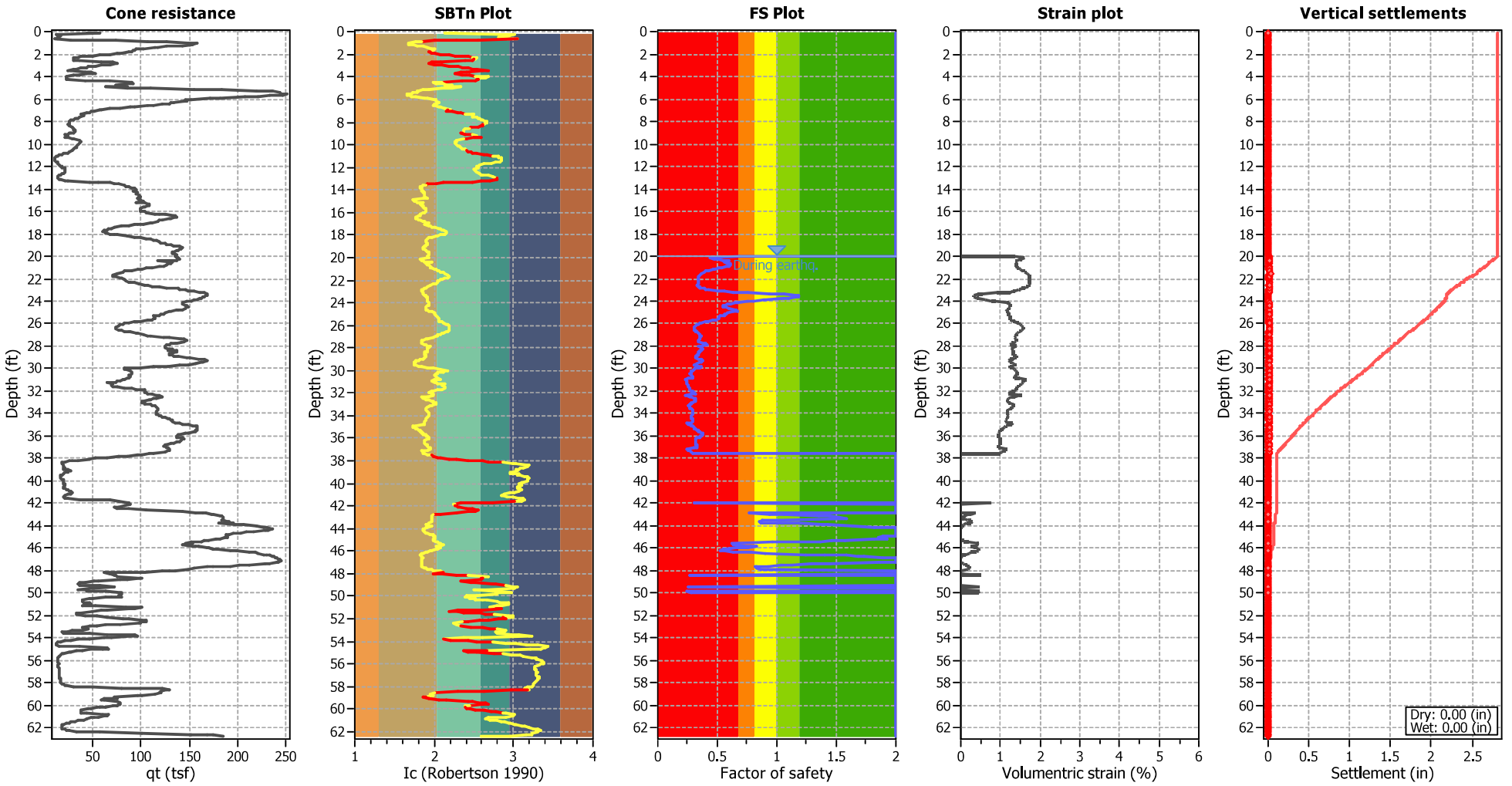






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

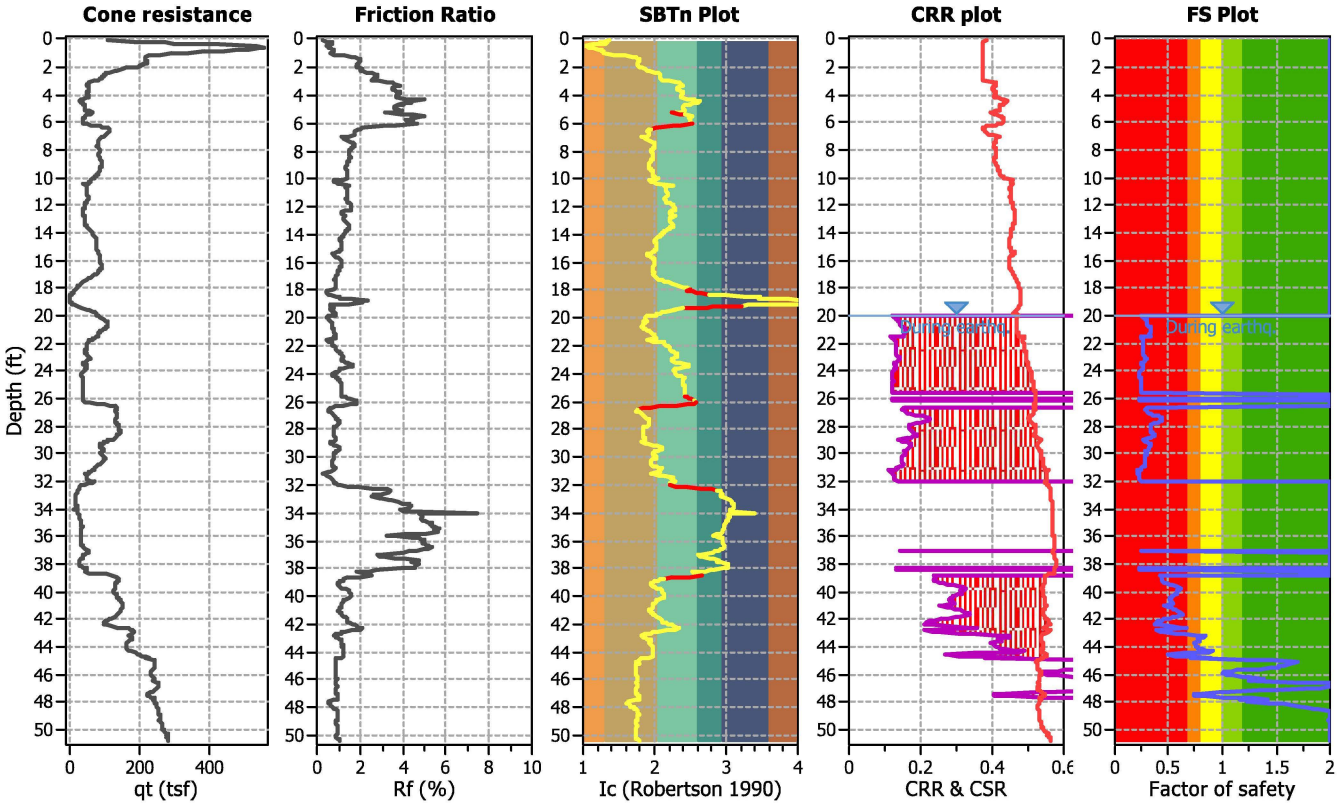
LIQUEFACTION ANALYSIS REPORT

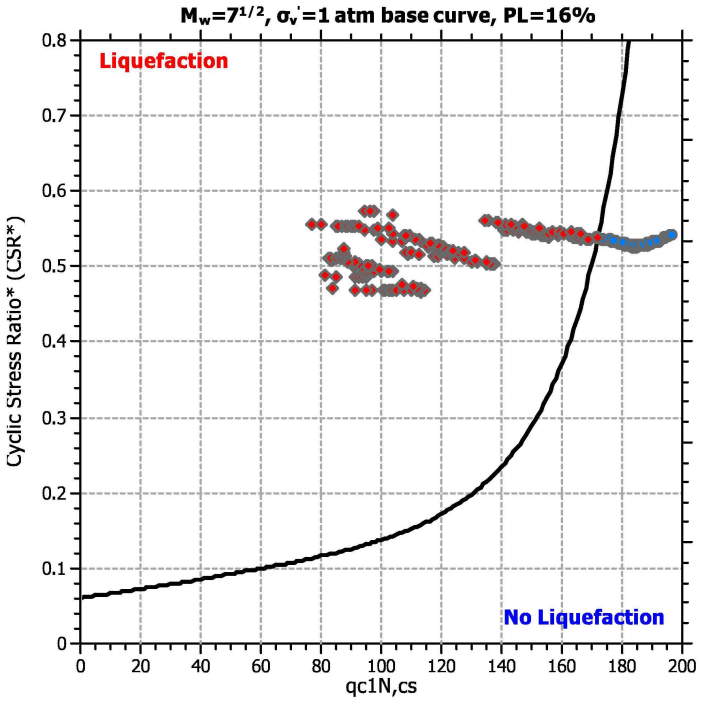
Project title : Proposed Industrial Building **Location : Long Beach, CA**

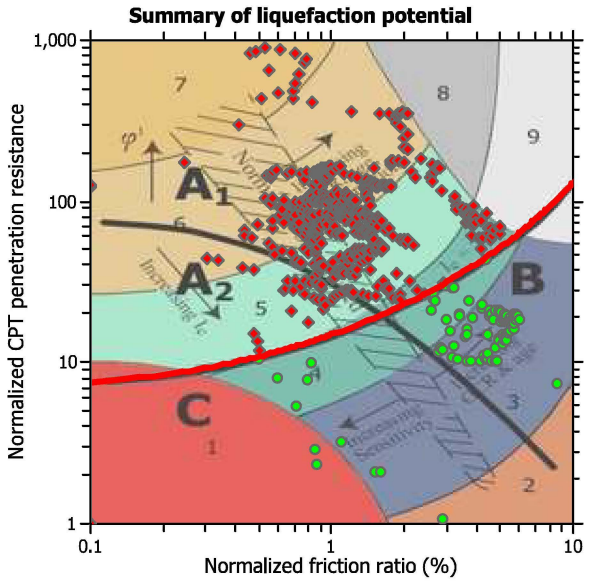
CPT file : CPT-3

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	23.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.85	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.81	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

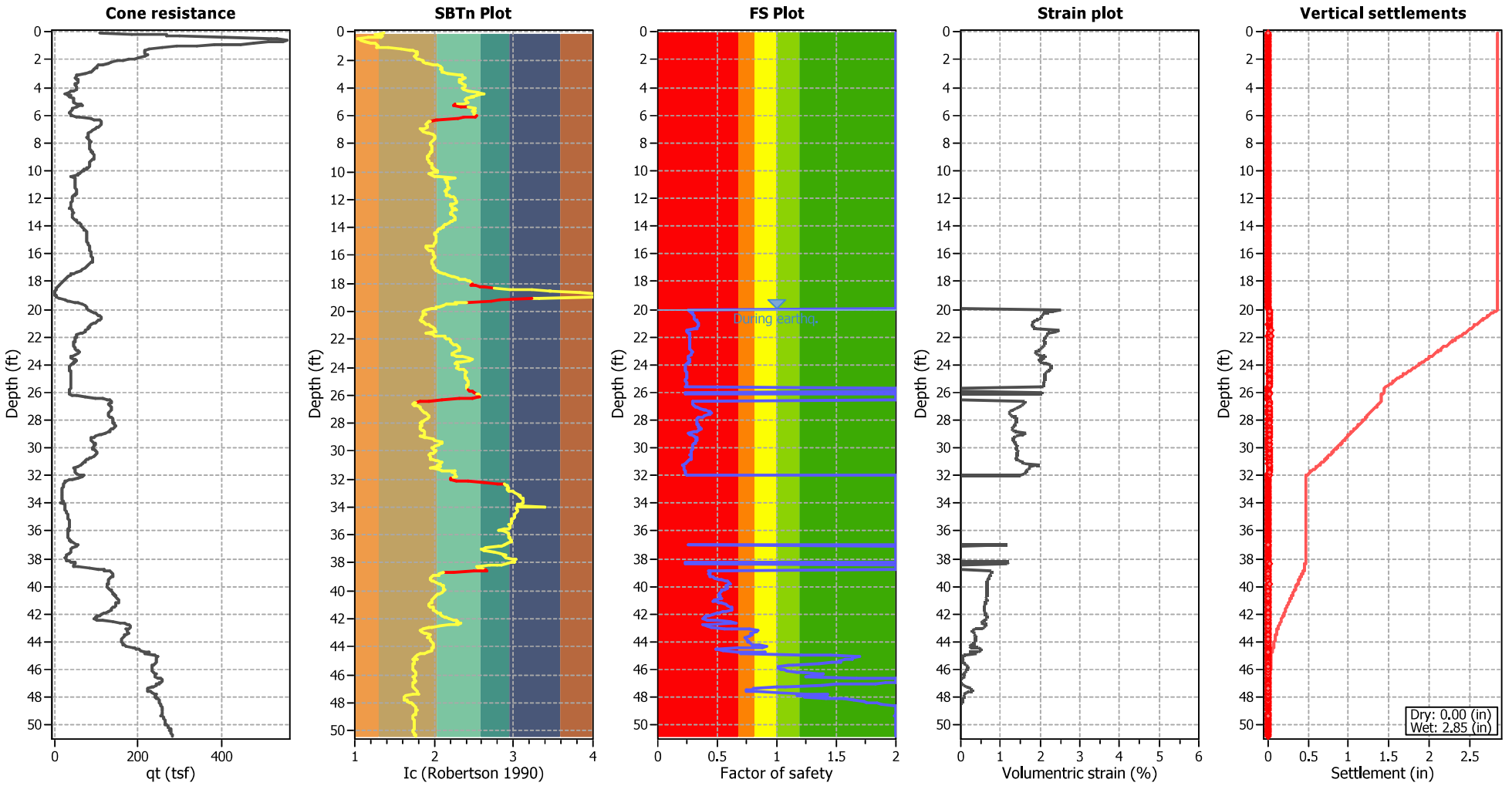






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
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Estimation of post-earthquake settlements



Abbreviations

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I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

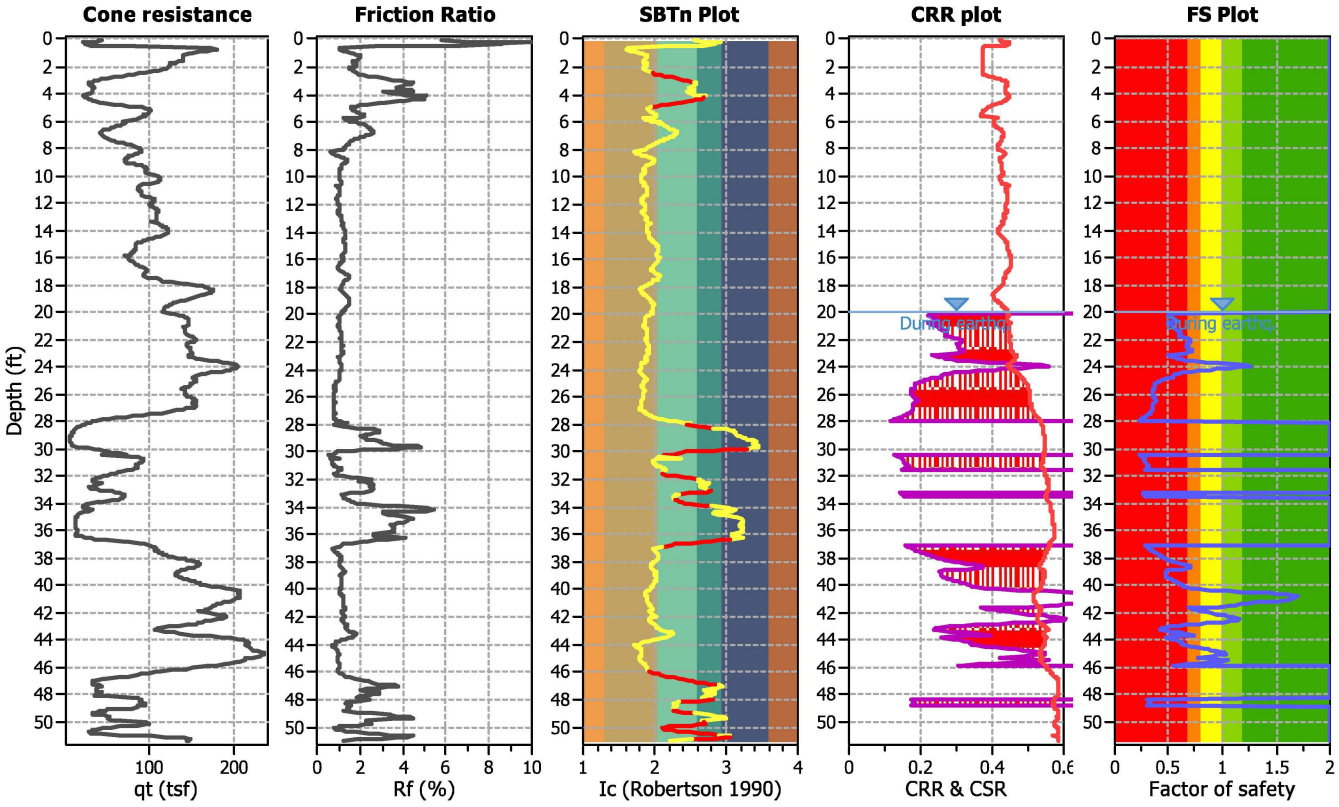
LIQUEFACTION ANALYSIS REPORT

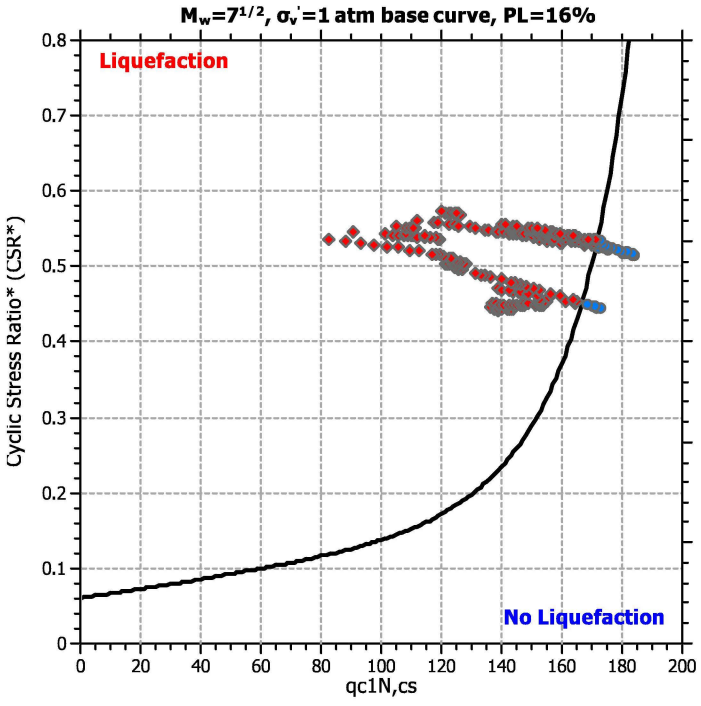
Project title : Proposed Industrial Building **Location : Long Beach, CA**

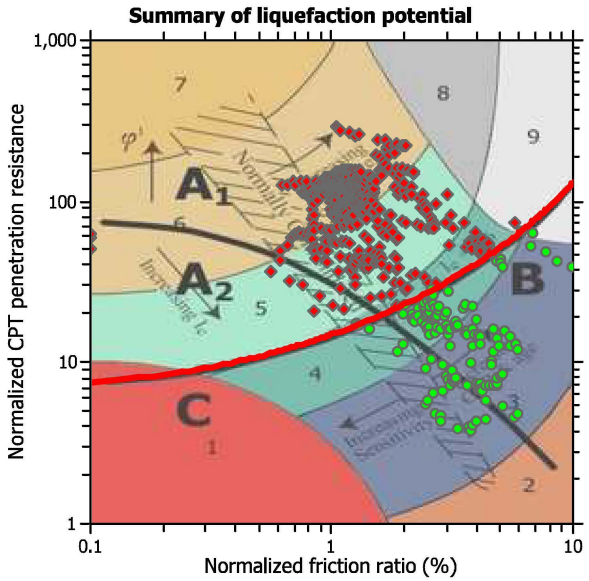
CPT file : CPT-4

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	24.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth applied:	Yes
Earthquake magnitude M_w:	6.85	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	50.00 ft
Peak ground acceleration:	0.81	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

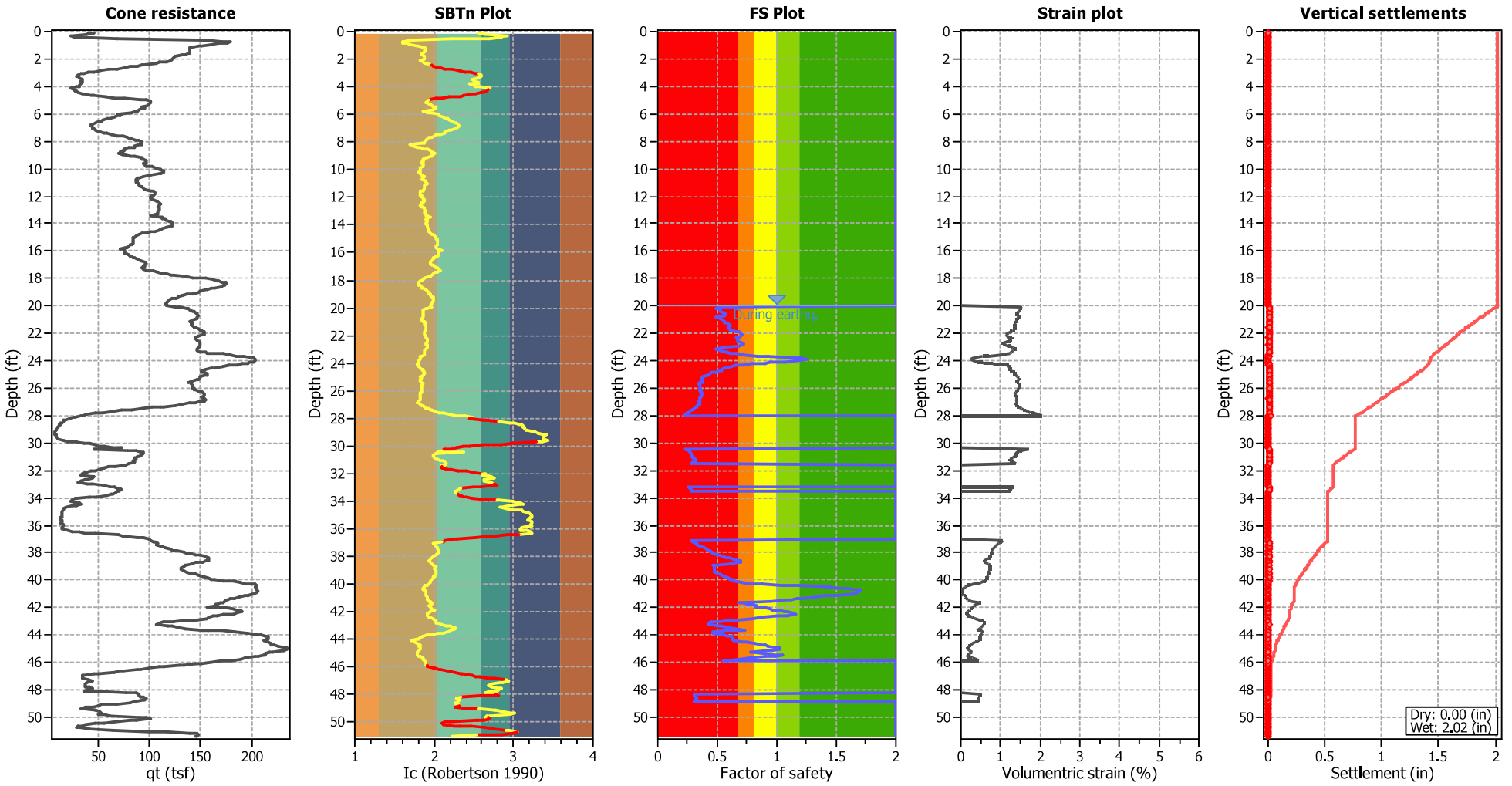






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
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Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements

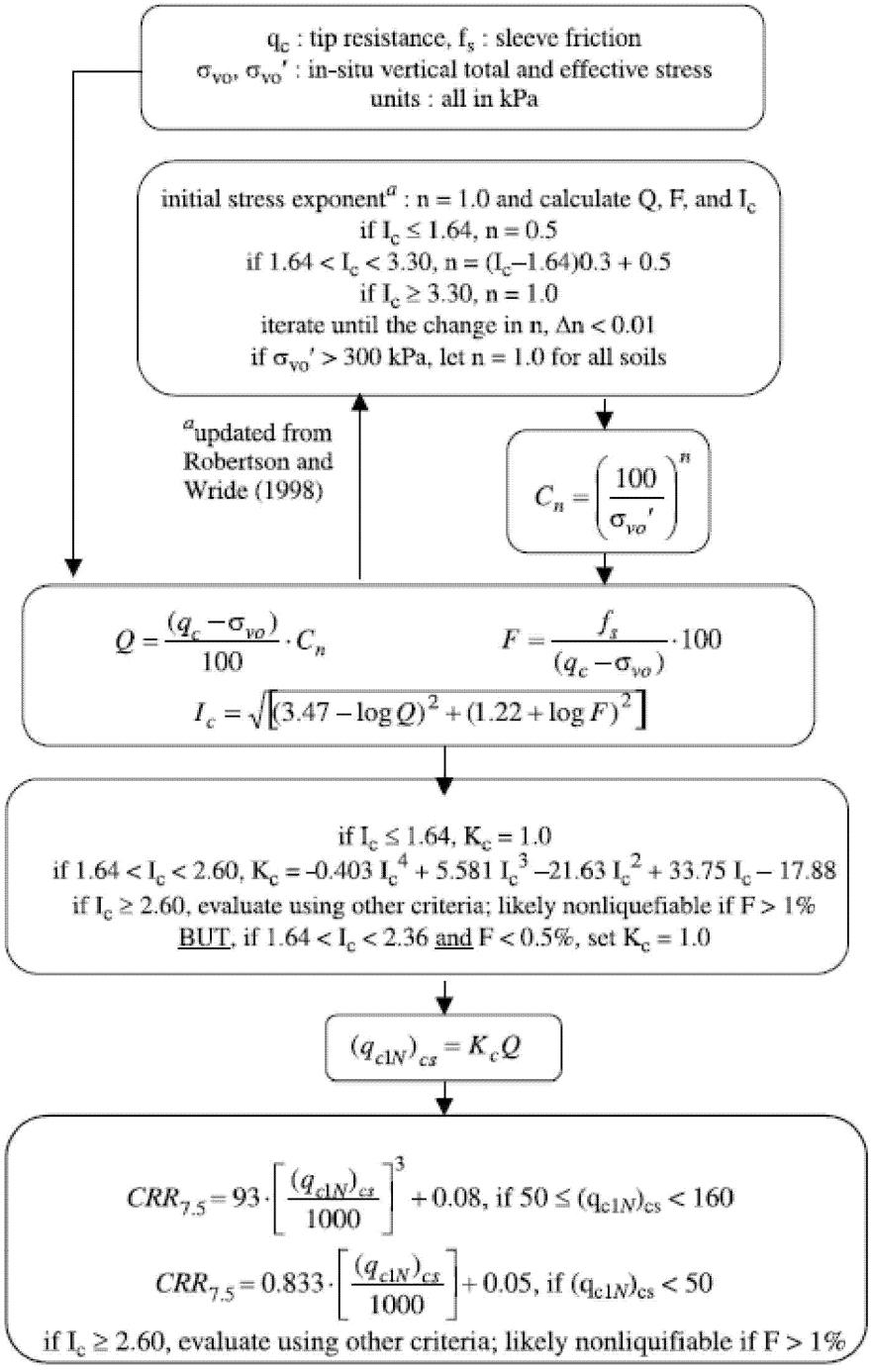


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

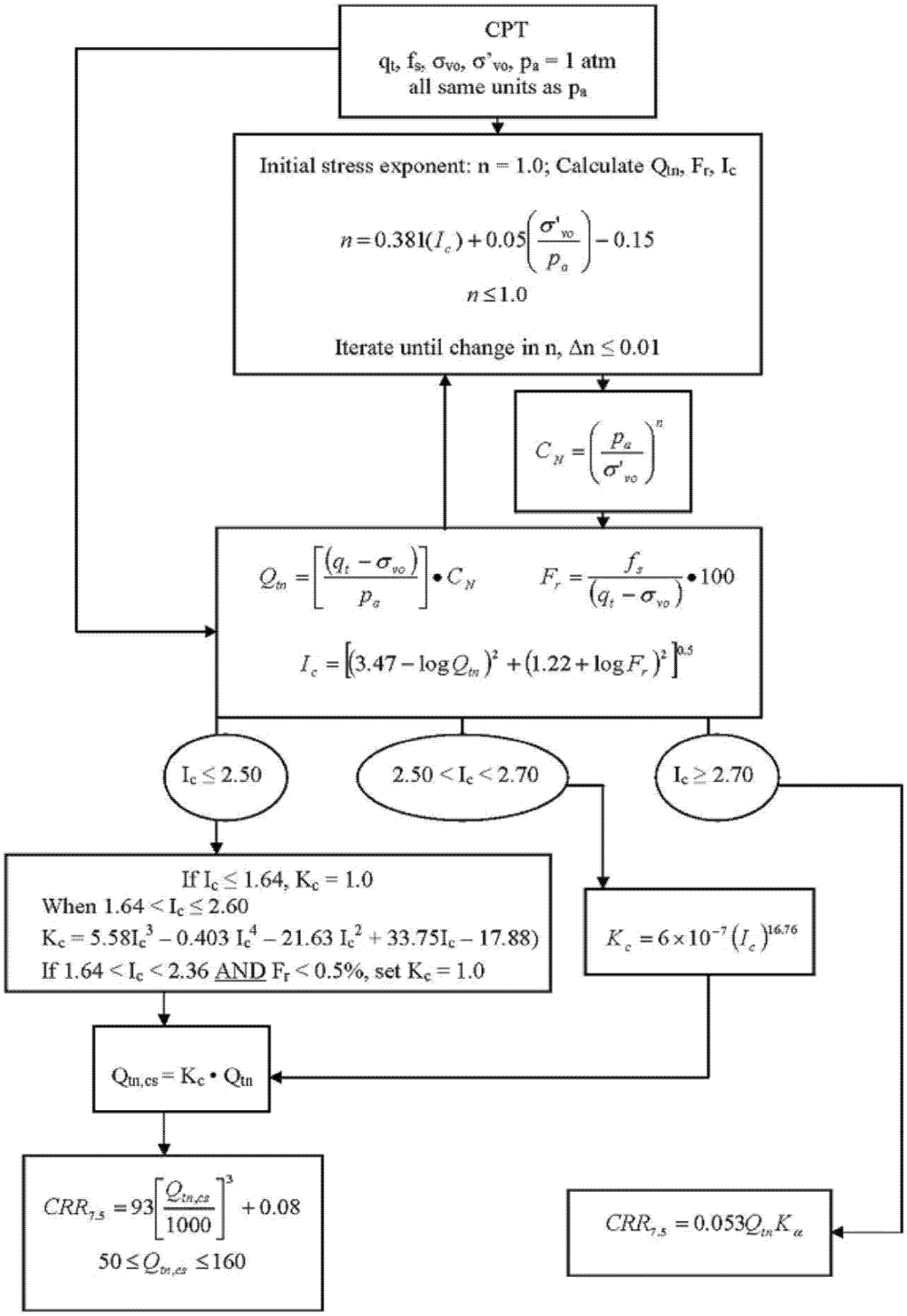
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

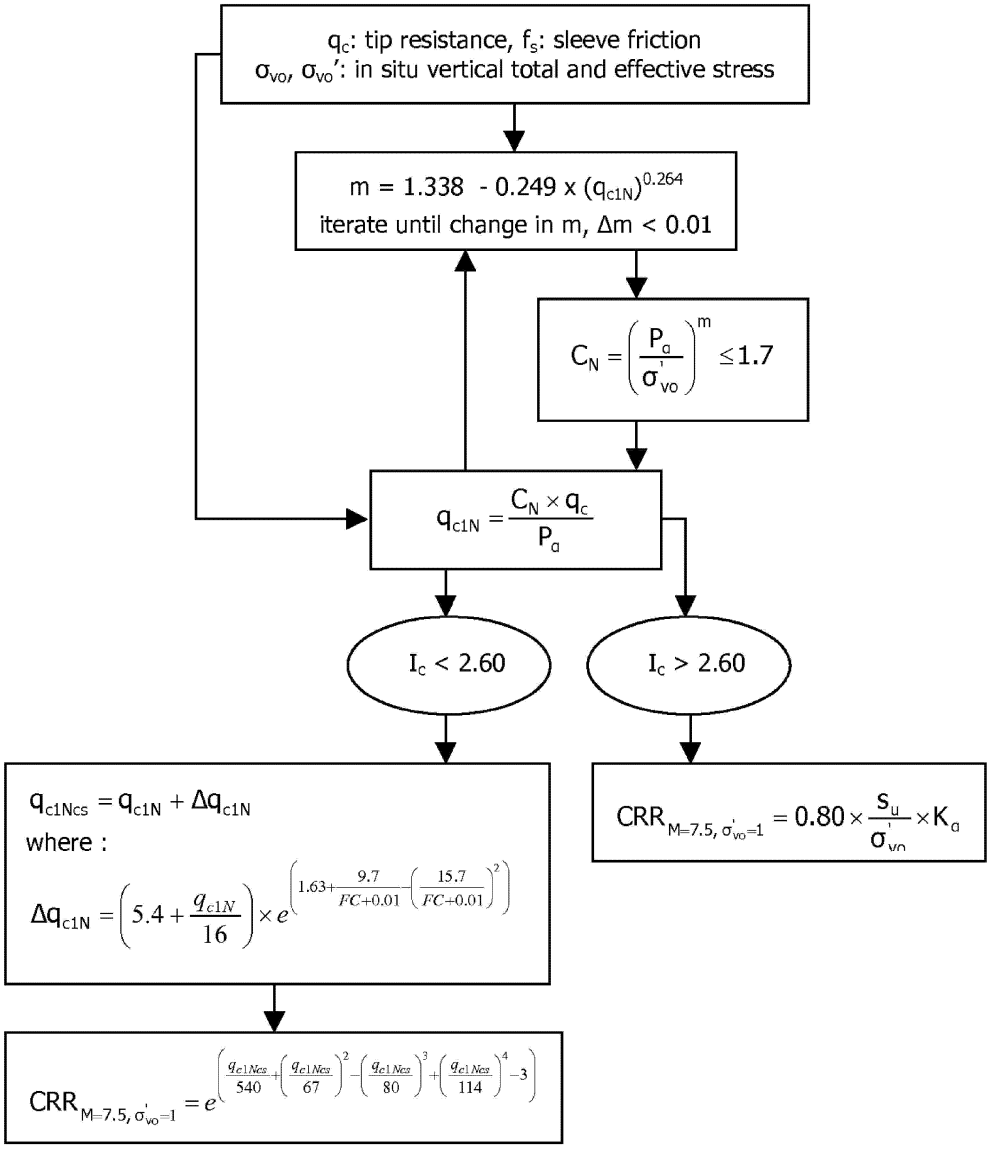
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:

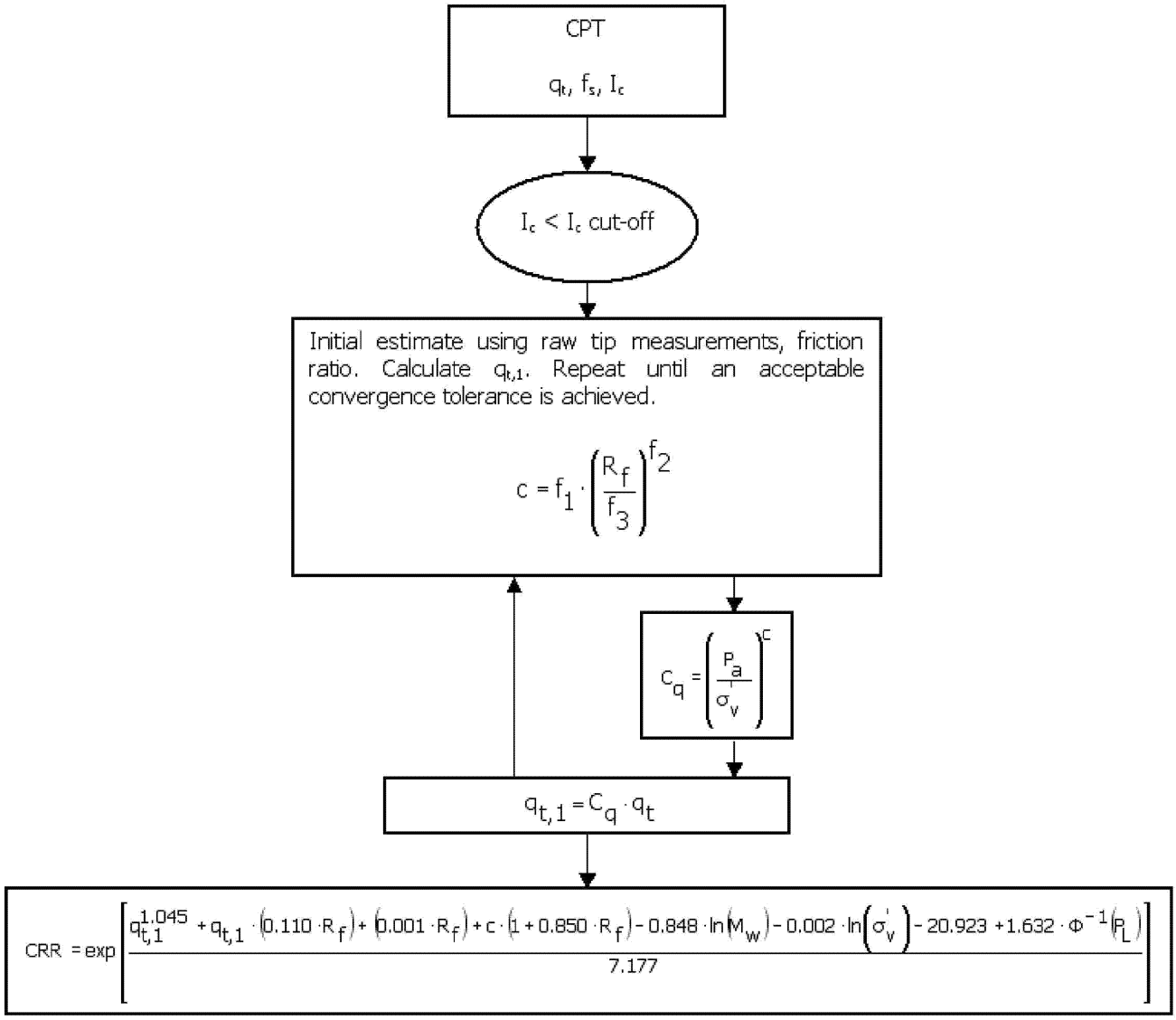


[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)



Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)



Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)

$$FS_{liq} = \frac{CRR_{M=7.5,\sigma'_v=1atm}}{CSR_{M=7.5,\sigma'_v=1atm}}$$

$$CSR_{M=7.5,\sigma'_v=1atm} = 0.65 \frac{\sigma_v}{\sigma'_v} \frac{a_{max}}{g} r_d \frac{1}{MSF} \frac{1}{K_\sigma}$$

$$r_d = \exp\left[\alpha(z) + \beta(z) \cdot M\right]$$

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right)$$

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right)$$

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_v}{P_a}\right) \le 1.1$$

$$C_\sigma = \frac{1}{37.3 - 8.27\left(q_{c1Ncs}\right)^{0.264}} \le 0.3$$

$$MSF = 1 + \left(MSF_{max} - 1\right)\left(8.64 \exp\left(\frac{-M}{4}\right) - 1.325\right)$$

$$MSF_{max} = 1.09 + \left(\frac{q_{c1Ncs}}{180}\right)^3 \le 2.2$$

σ'_v *and* σ_v *at start of earthquake shaking*

$$CRR_{M=7.5,\sigma'_v=1atm} = \exp\left(\frac{q_{c1Ncs}}{113} + \left(\frac{q_{c1Ncs}}{1000}\right)^2 - \left(\frac{q_{c1Ncs}}{140}\right)^3 + \left(\frac{q_{c1Ncs}}{137}\right)^4 - 2.80\right)$$

$$q_{c1Ncs} = q_{c1N} + \Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(11.9 + \frac{q_{c1N}}{14.6}\right) \exp\left(1.63 - \frac{9.7}{FC+2} - \left(\frac{15.7}{FC+2}\right)^2\right)$$

$$q_{c1N} = C_{Ne} \frac{q_c}{P_a}$$

$$C_{Ne} = \left(\frac{P_a}{\sigma'_v}\right)^m \le 1.7$$

$$m = 1.338 - 0.249\left(q_{c1Ncs}\right)^{0.264} \quad with \quad 0.264 \le m \le 0.782$$

$$FC = 80\left(I_C + C_{FC}\right) - 137 \quad with \quad 0\% \le FC \le 100\%$$

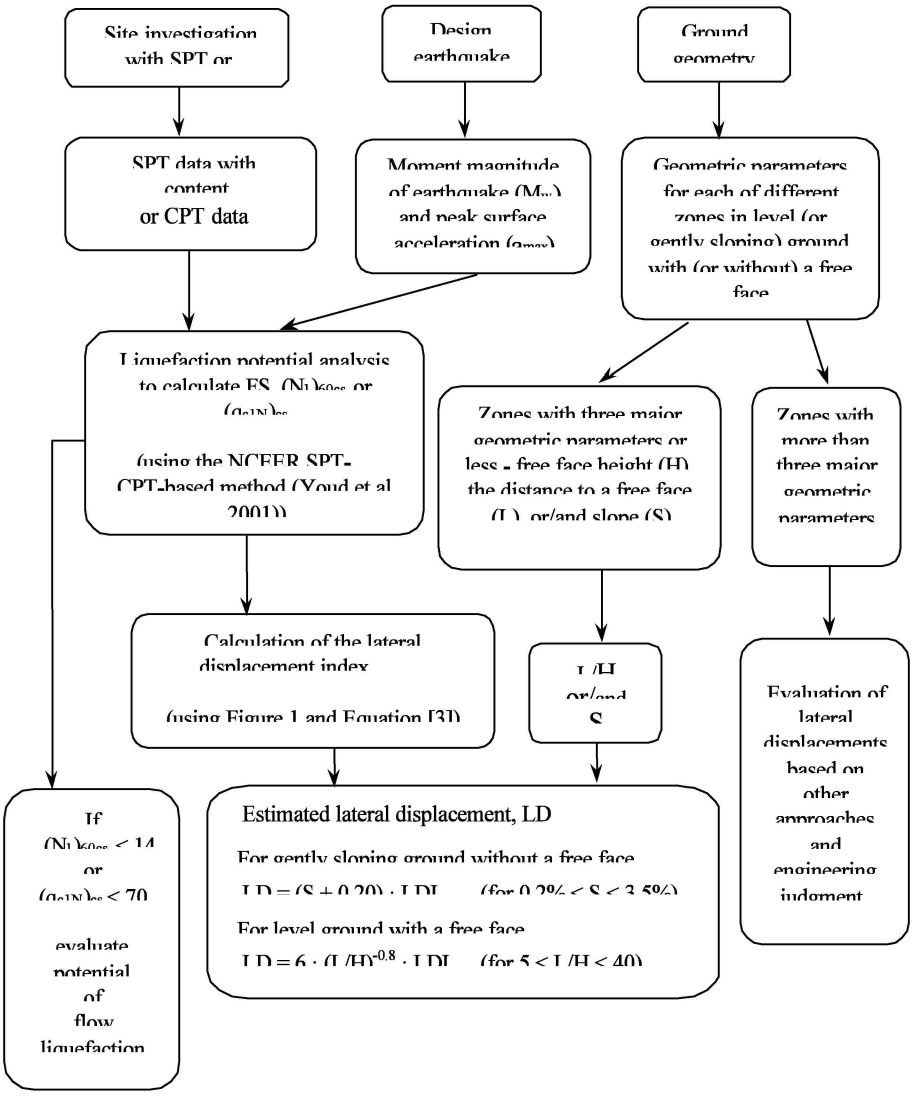
$$I_c = \left[\left(3.47 - \log\left(Q\right)\right)^2 + \left(1.22 + \log\left(F\right)\right)^2\right]^{0.5}$$

$$Q = \left(\frac{q_c - \sigma_{vc}}{P_a}\right)\left(\frac{P_a}{\sigma'_{vc}}\right)^n \; with \; 0.5 \le n \le 1.0 \; per \; Robertson \; \& \; Wride \; (1998)$$

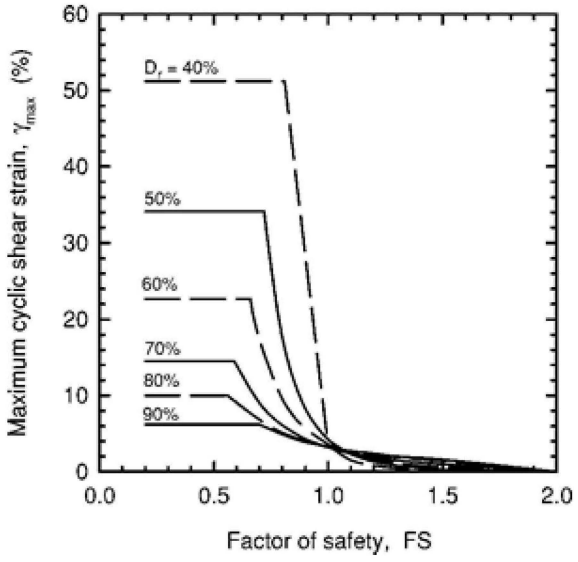
$$F = \left(\frac{f_s}{q_c - \sigma_{vc}}\right) \cdot 100\%$$

σ'_v *at time of CPT sounding*

Procedure for the evaluation of liquefaction-induced lateral spreading displacements



[1] Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach



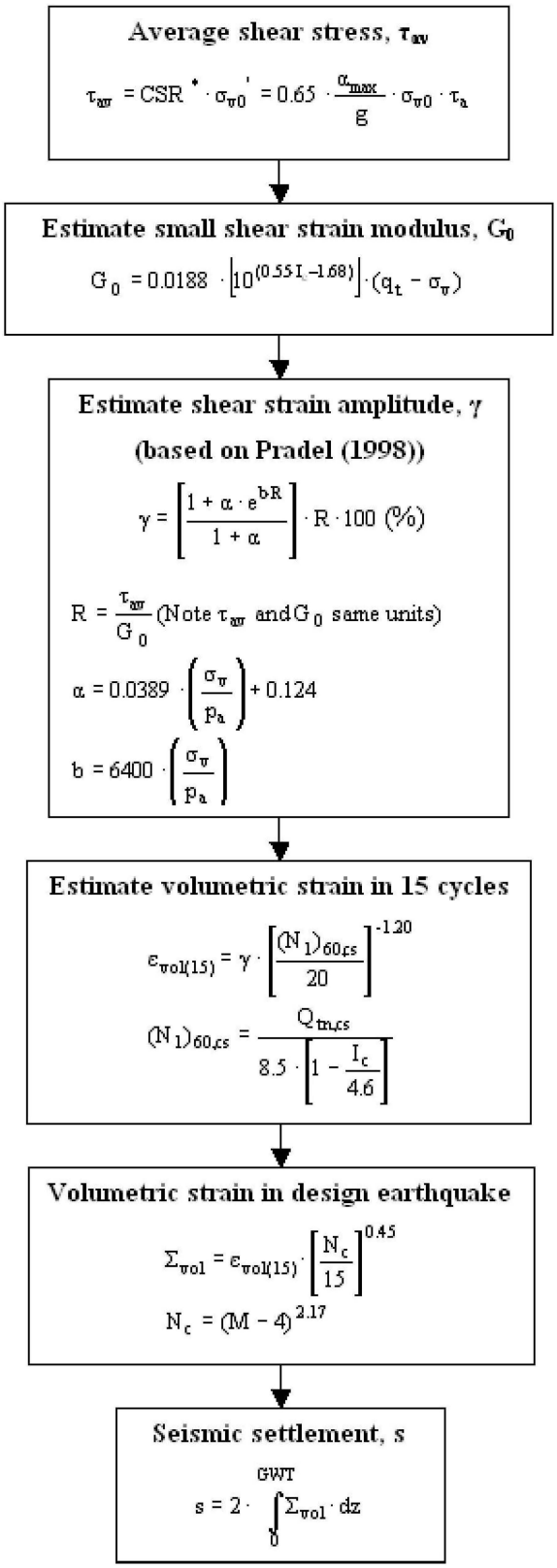
[1] **Figure 1**

$$LDI = \int_0^{z_{max}} \gamma_{max} dz$$

[1] **Equation [3]**

[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the estimation of seismic induced settlements in dry sands



Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT" FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methology developed by Iwasaki (1982) and is adopted by AFPS.

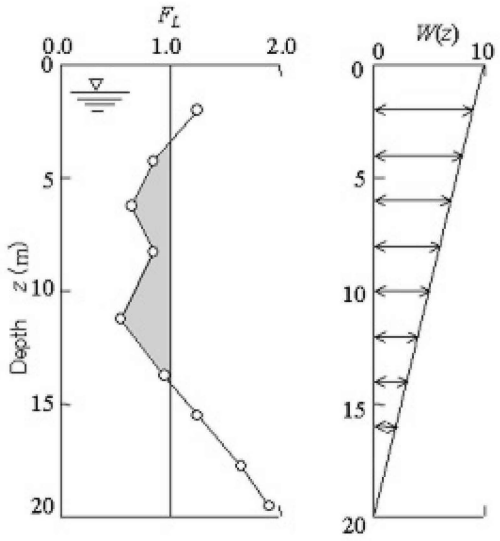
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\mathbf{LPI} = \int_0^{20} (10 - 0{,}5z) \times F_L \times dz$$

where:
F_L = 1 - F.S. when F.S. less than 1
F_L = 0 when F.S. greater than 1
z depth of measurment in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- 0 < LPI <= 5 : Liquefaction risk is low
- 5 < LPI <= 15 : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high



Graphical presentation of the LPI calculation procedure

Shear-Induced Building Settlement (Ds) calculation procedure

The shear-induced building settlement (Ds) due to liquefaction below the building can be estimated using the relationship developed by Bray and Macedo (2017):

$$Ln(Ds) = c1 + c2 * LBS + 0.58 * Ln\left(Tanh\left(\frac{HL}{6}\right)\right) + 4.59 * Ln(Q) - 0.42 * Ln(Q)^2 - 0.02 * B + 0.84 * Ln(CAVdp) + 0.41 * Ln(Sa1) + \varepsilon$$

where Ds is in the units of mm, c1= -8.35 and c2= 0.072 for LBS ≤ 16, and c1= -7.48 and c2= 0.014 otherwise. Q is the building contact pressure in units of kPa, HL is the cumulative thickness of the liquefiable layers in the units of m, B is the building width in the units of m, CAVdp is a standardized version of the cumulative absolute velocity in the units of g-s, Sa1 is 5%-damped pseudo-acceleration response spectral value at a period of 1 s in the units of g, and ε is a normal random variable with zero mean and 0.50 standard deviation in Ln units. The liquefaction-induced building settlement index (LBS) is:

$$LBS = \sum W * \frac{\varepsilon_{shear}}{z} dz$$

where z (m) is the depth measured from the ground surface > 0, W is a foundation-weighting factor wherein W = 0.0 for z less than Df, which is the embedment depth of the foundation, and W = 1.0 otherwise. The shear strain parameter (ε_shear) is the liquefaction-induced free-field shear strain (in %) estimated using Zhang et al. (2004). It is calculated based on the estimated Dr of the liquefied soil layer and the calculated safety factor against liquefaction triggering (FSL).

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