GEOTECHNICAL INVESTIGATION
PROPOSED WAREHOUSE

16323 Shoemaker Avenue
Cerritos, California
For
Duke Realty

SoCalGeo
SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

November 5, 2021

Duke Realty
200 Spectrum Center Drive, Suite 1600
Irvine, California 92618

Attention: Mr. Michael Weber
Senior Development Services Manager

Project No.: **21G233-1**

Subject: **Geotechnical Investigation**
Proposed Warehouse
16323 Shoemaker Avenue
Cerritos, California

Gentlemen:

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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Senior Engineer

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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 borings encountered artificial fill soils underlain by native alluvium. The fill soils extend to depths of 3 to 4½± feet at the boring locations and are considered to consist of undocumented fill soils. Most of the native alluvial soils possess low strengths and densities with some loose soils within the upper 12± feet.
- All of the borings encountered groundwater at depths of 15 to 20± feet.
- The subject site is located within an area mapped as a liquefaction hazard zone by the state of California. Our site-specific liquefaction evaluation included four (4) CPT soundings advanced to depths of 50± feet. Potentially liquefiable soils were encountered at all of the CPT locations.
- The potential liquefaction-induced settlements at the CPT locations range between 3.76 and 5.07± inches.
- The potentially liquefiable soils were encountered at depths ranging between 8 and 50± feet. Some of the potentially liquefiable strata are present at relatively shallow depths of about 8 to 15± feet. The foundation loads of the new structures are expected to influence the potentially liquefiable soils present at these depths. Without mitigation, dynamic total and differential settlements are expected to be in excess of tolerable limits for conventional shallow foundations.
- The most feasible method of mitigating potential static and dynamic settlements at this site is considered to be remedial grading of the near surface soils in conjunction with ground improvement of the loose and potentially liquefiable soils located within the zones of influence for the new building foundations.
- Improvement of the loose and potentially liquefiable soils within the foundation influence zones may consist of overexcavation and recompaction throughout the depths of soils significantly influenced by the foundations. However, this would require some dewatering and may not be feasible. Specialized ground improvement techniques may be used to improve the existing soils within the influence zones of the new foundations.
- Some remedial grading should be performed throughout the entire building pad area to remove the undocumented fill soils and any soils disturbed during demolition of the existing building and associated improvements.
- This report presents recommendations for the use conventional shallow foundations, assuming that remedial grading and/or ground improvement is performed within the zones of influence of new building foundations, as defined above.

Site Preparation

- Demolition of the existing structures, pavements, and associated improvements (that will not remain with the proposed development) will be required in order to facilitate construction of the new building. Demolition should also include all utilities and any other subsurface improvements that will not remain in place for use with the new development. Debris resultant from demolition should be disposed of offsite. Concrete and asphalt debris may be pulverized to a maximum 2-inch particle size, well mixed with the on-site sandy soils, and incorporated into new structural fills, or it may be crushed into miscellaneous base (CMB). Alternatively, concrete and asphalt debris may be crushed to particles sizes of 2 to 4 inches and used to stabilize overexcavation subgrades.

- Initial site preparation should include stripping of the existing grass, trees, and weed growth present in some areas the site. Stripping should also include removal of any tree root masses. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.
- Mitigation of loose and potentially liquefiable soils should extend to a minimum depth of at least 20± feet and to a depth equal to at least 2 times the width of the foundation below the foundation bearing grade for square column foundations. The depth of improvement should extend to a depth of at least 3 times the width of continuous footings below foundation bearing grades. This mitigation may consist of remedial grading and/or specialized ground improvement techniques.
- At a minimum, remedial grading should be performed within the new building pad area to remove the existing fill soils (which extend to depths of 3 to 4½± feet at the boring locations) in their entirety and any soils disturbed during demolition of the existing improvements. Additionally, overexcavation should extend to a minimum depth of 3 feet below the proposed building pad grade.
- After overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated, moisture conditioned (or air dried) to within 2 to 4 percent of the ASTM D-1557 maximum dry density, 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.
- The new parking area subgrade soils are recommended to be scarified to a depth of 12± inches, thoroughly moisture conditioned 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, assuming that onsite soils are overexcavated and recompacted within the depths of foundation influence. A greater foundation bearing pressure may be allowed based on the ground improvement technique used.
- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom) due to the presence of potentially liquefiable and low expansive soils. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade, 6 inches thick.
- Modulus of Subgrade Reaction: k = 100 psi/in
- Reinforcement consisting of No. 3 rebars at 18 inches on center in both directions due to the presence of potentially liquefiable and low expansive soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed slab loading.

Pavements

ASPHALT PAVEMENTS (R = 15)					
Materials	**Thickness (inches)**				
	Automobile Parking (TI = 4.0)	Automobile Drive Lanes (TI = 5.0)	Truck Traffic		
			(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	6	9	11	13	15
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 15)			
Materials	**Thickness (inches)**		
	Automobile and Light Truck Traffic (TI = 5.0 & 6.0)	Truck Traffic	
		(TI = 7.0)	(TI = 8.0)
PCC	5	5½	7
Compacted Subgrade (95% minimum compaction)	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 21P422, dated September 3, 2021. 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 this 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 site is located at the street address of 16323 Shoemaker Avenue in Cerritos, California. The site is bounded to the north by a railroad easement, to west by an existing commercial/industrial building, to the south by Moore Street, and to the east by Shoemaker Avenue. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in of this report.

The subject site consists of a rectangular-shaped parcel, 7± acres in size. The site is currently developed with a commercial/industrial building along with several other maintenance and storage buildings. The main building is approximately 64,000 ft^2 in size and is located in the central area of the site. The smaller maintenance and storage buildings, 500 to 3,900± ft^2 in size, are located north and west of the main building. The existing buildings are one- to two-story structures of concrete tilt-up and metal construction. Structural plans for the existing buildings have not been provided to our office. We assume that the existing structures are supported on conventional shallow foundation systems. Ground surface cover surrounding the buildings generally consists of asphaltic concrete (AC) pavements and landscaped areas planted with grass and medium-sized to large trees. The existing pavements appear to be in fair condition, with moderate cracking throughout.

Detailed topographic information was not available at the time of this report. Based on the elevations obtained from Google Earth and visual observations made at the time of the subsurface investigation, the site slopes gently to the south at a gradient of less than 1± percent.

3.2 Proposed Development

A site plan, prepared by HPA, was provided to our office by the client. Based on this plan, the subject site will be developed with a warehouse with a footprint area of 159,870± ft^2, located in the central portion of the site. Dock-high doors will be constructed along a portion of the western building wall. The proposed building is expected to be surrounded by AC pavements in the parking and drive areas, Portland Cement Concrete (PCC) pavements in the loading dock area, and concrete flatwork and landscaped planters throughout the site.

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, supported on a conventional shallow foundation system 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 and basements or crawl spaces, are expected to be included in the proposed development. Based on the existing topography, and assuming a relatively balanced site, cuts and fills of up to 2 to 4± feet are expected to be necessary to achieve the proposed building pad grades. It should be noted that this estimate does not include any remedial grading recommendations which are presented in a subsequent section of this report.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of four (4) borings (identified as Boring Nos. B-1 through B-4) advanced to depths of 25 to 50± feet below the existing site grades. Two (2) of the borings were drilled to a depth of 50± feet as part of the liquefaction evaluation. 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 (identified as CPT-1 through CPT-4) were advanced to a depth of 50± feet as part of the liquefaction evaluation. All of the boring and CPT locations were cleared by a private geophysical testing company prior to our subsurface exploration.

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. In-situ 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 an SCG engineer. 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 in the CPT report, dated September 29, 2021, enclosed in Appendix F of this report.

The data generated by the cone penetrometer equipment has been interpreted by KTE using CPeT-IT, V2.3.18, published by Geologismiki Geotechnical Software. The CPeT-IT program output as well as more details regarding the interpretation procedure are presented in the aforementioned reports prepared by KTE.

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. The results of the CPT soundings are presented in the report prepared by KTE, included in Appendix F of this report.

4.2 Geotechnical Conditions

Pavements

Asphaltic concrete pavements were encountered at the ground surface at all of the boring locations. The pavement sections at the boring locations consists of 2 to 3± inches of asphaltic concrete (AC), underlain by 0 to 4± inches of aggregate base.

Artificial Fill

Artificial fill soils were encountered beneath the pavements at all of the boring locations, extending to depths of 3 to 4½± feet below ground surface. The fill soils encountered at the boring locations generally consist of very stiff to hard clayey silts and medium dense silty fine sands and fine sandy silts. The fill soils possess a disturbed and mottled appearance, resulting in their classification as artificial fill. Some of the samples of the fill soils possess traces of fine gravel.

Alluvium

Native alluvium was encountered beneath the fill soils at all of the boring locations, extending to at least the maximum depth explored of 50± feet below ground surface. The alluvial soils generally consist of interbedded layers of loose to dense fine sands, silty fine sands, fine sandy silts and occasional strata of medium stiff to very stiff sandy clays, silty clays, and fine sandy silts. Trace iron oxide staining was observed on several of the samples of alluvium.

Groundwater

Free water was encountered during the drilling at all four of the boring locations at depths between 15 and 20± feet below existing site grades. Delayed groundwater measurements were not practical due to caving within the open boreholes.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the historic groundwater depths in this area is the California Geological Survey (CGS) Open File Report 98-28, the Seismic Hazard Zone Report for the Whittier 7.5-Minute Quadrangle, which indicates that the historic high groundwater level for the site was about 8± feet below the ground surface.

Recent water level data was obtained from the California State Water Resources Control Board, GeoTracker, website, https://geotracker.waterboards.ca.gov/. Four monitoring wells in this database are located in the northwest portion of the site. The highest water level readings within these monitoring wells range between depths of 10 to 12½± feet below the ground surface.

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. The 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.

Dry 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-4 in Appendix C of this report.

Soluble Sulfates

A representative sample of the near-surface soils was 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 (%)	Severity	Exposure Class
B-2 @ 0 to 5 feet	0.100	Moderate	S1

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829 as required by the California Building Code (CBC). 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-2 @ 0 to 5 feet	23	Low
B-4 @ 0 to 5 feet	35	Low

Corrosivity Testing

Representative bulk samples of the near-surface soils were submitted to a subcontracted analytical laboratory for determination of electrical resistivity, pH, and chloride concentrations. The resistivity of the soils is a measure of their potential to attack buried metal improvements such as utility lines. The results of the resistivity and pH testing are presented below:

Sample Identification	Resistivity (ohm-cm)	pH	Chlorides (mg/kg)	Nitrates (mg/kg)
B-2 @ 0 to 5 feet	260	8.4	892	237

Maximum Dry Density and Optimum Moisture Content

One representative bulk sample was tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil type or soil mixes may be necessary at a later date. The results of this test are plotted on Plate C-5 in Appendix C of this report.

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 structures 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.

The potential for other geologic hazards such as seismically induced settlement, tsunamis, inundation, seiches, flooding, and subsidence affecting the site is considered low.

Seismic Design Parameters

The 2019 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 standards in place at the time of this report, the proposed development is expected to be designed in accordance with the requirements of the 2019 edition of the California Building Code (CBC), which was adopted on January 1, 2020.

The 2019 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 2019 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 tables below were 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 2019 CBC requires that a site-specific ground motion study be performed in accordance with Section 11.4.8 of ASCE 7-16 for Site Class D sites with a mapped S_1 value greater than 0.2. However, Section 11.4.8 of ASCE 7-16 also indicates an exception to the requirement for a site-specific ground motion hazard analysis for certain structures on Site Class D sites. The commentary for Section 11 of ASCE 7-16 (Page 534 of Section C11 of ASCE 7-16) indicates that "In general, this exception effectively limits the requirements for site-specific hazard analysis to very tall and or flexible structures at Site Class D sites." **Based on our understanding of the proposed development, the seismic design parameters presented below were calculated assuming that the exception in Section 11.4.8 applies to the proposed structure at this site. However, the structural engineer should verify that this exception is applicable to the proposed structure.** Based on the exception, the spectral response accelerations presented below were calculated using the site coefficients (F_a and F_v) from Tables 1613.2.3(1) and 1613.2.3(2) presented in Section 16.4.4 of the 2019 CBC.

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped MCE_R Acceleration at 0.2 sec Period	S_S	1.572
Mapped MCE_R Acceleration at 1.0 sec Period	S_1	0.561
Site Class	---	D*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.572
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.976
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.048
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.650

*The 2019 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 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 structure has a fundamental period greater than 0.5 seconds, a site-specific seismic hazards analysis will be required and additional subsurface exploration will be necessary.

It should be noted that the site coefficient F_v and the parameters S_{M1} and S_{D1} were not included in the SEAOC/OSHPD Seismic Design Maps Tool output for the 2019 CBC. We calculated these parameters-based on Table 1613.2.3(2) in Section 16.4.4 of the 2019 CBC using the value of S_1 obtained from the Seismic Design Maps Tool, assuming that a site-specific ground motion hazards analysis is not required for the proposed buildings at this site.

Ground Motion Parameters

For the preliminary liquefaction evaluation, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2019 CBC. The peak ground acceleration (PGA_M) 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, based on ASCE 7-16 as the building code reference document. A portion of the program output is included as Plate E-1 in Appendix E of this report. As indicated on Plate E-1, the PGA_M for this site is 0.739g. 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 6.81, based on the peak ground acceleration and soil classification D.

Liquefaction

Research of the Whittier 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.

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 plasticity 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). Non-sensitive clayey (cohesive) soils which possess a plasticity index of at least 18 (Bray and Sancio, 2006) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

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 V3.3.2.9, 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 0.739g for a magnitude 6.81 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-4 were extended to depths of 50± feet in order to provide samples for laboratory testing and correlation with the results of the adjacent 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.

<u>Conclusions and Recommendations</u>

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: 4.73± inches
- CPT-2: 5.07± inches

- CPT-3: 3.76± inches
- CPT-4: 3.77± inches

Based on these total settlements, differential settlements of up to 2½± 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 50 feet, indicating a maximum angular distortion of about 0.004 to 0.005 inches per inch. Based on Special Publication 117A, dynamic settlements greater than 4 inches exceed the threshold for which structural mitigation is allowed for liquefaction mitigation. Additionally, we understand that the potential angular distortion may be in excess of the structural tolerances for the proposed concrete tilt-up structures. Therefore, conventional shallow foundations should not be used at this site without some form of ground improvement.

Based on the potential total and differential dynamic settlements and with the presence of shallow liquefiable layers, discussed in the subsequent section, we recommend that ground improvement and/or overexcavation of the existing site soils be performed to a depth of at least 20± feet below the existing site grades in order to reduce the potential liquefaction settlements to less than 4 inches, and to improve the soils within the foundation influence of the new structure.

Shallow Liquefiable Layers

Liquefaction induced settlement is projected to occur at all of the CPT locations at various depths of between 8 and 50± feet during the design level earthquake. Based on these considerations, we expect that liquefiable soils will be present within the influence zones of new foundations. Additionally, based on Ishihara's criteria, liquefaction of the near surface-soils could result in surface manifestations, including sand boils.

The consequences of soil liquefaction occurring within the zone of influence of a foundation can result in the loss of bearing capacity and/or punching failure. An isolated column footing with typical structural loads could settle rapidly during a liquefaction inducing seismic event. The magnitude of the settlement below a loaded column can be much higher than the dynamic settlements presented above for free-field conditions.

Based on the presence of shallow liquefiable soil layers, we do not recommend that the new buildings be supported on conventional shallow foundations without mitigation of the liquefaction potential of the near surface soils within the proposed building area. Therefore, we recommend that ground improvement be performed within the proposed building area to improve the near surface soils present within the zone of influence of any foundation elements. This improvement may consist of conventional remedial grading (with dewatering) or specialized ground improvement techniques.

The recommended mitigation measures will not completely eliminate the potential for liquefaction induced settlements. Designing the proposed buildings to remain completely undamaged during a major seismic event is not considered to be economically feasible. The ground improvement program should be designed to mitigate potentially liquefiable soil layers within the depths that will be significantly influenced by the new foundation loads. Additional geotechnical design considerations regarding the recommended ground improvement, such as the presence of very loose soils below the ground water table, are discussed in a subsequent section of this report.

General

Any utility connections to the structures should be designed to withstand the estimated dynamic settlements. It should also be noted that minor to moderate repairs, including releveling, restoration of utility connections, repair of damaged drywall and stucco, etc., would likely be required after occurrence of a major earthquake.

The use of shallow foundation systems in conjunction with the recommend ground improvement, as described in this report, is typical for buildings of the proposed type, where they are underlain by the extent of liquefiable soils encountered at this site. The post-liquefaction damage that could occur within the building is expected to be typical of similar buildings in the vicinity of this project. Other geotechnical and structural options are available, including the use of deep foundations such as driven piles, and drilled piers, but are considered to be less economically feasible.

Lateral Spreading

No significant slopes or free faces are present at within several hundred feet the subject site. Based on the fact no significant slopes are present near the subject, lateral spreading is not considered to be a significant design concern for this project.

6.2 Geotechnical Design Considerations

General

The borings encountered artificial fill soils, extending to depths of 3 to 4½± feet at the boring locations. These soils possess variable densities and strengths and some of these soils possess a disturbed, mottled appearance. Additionally, no documentation regarding the placement and compaction of the existing fill soil soils has been provided to our office. The fill soils are therefore considered to be undocumented fill. The fill soils are underlain by native alluvium which possesses variable strengths and composition. Based on the results of laboratory testing, the near-surface native alluvial soils with in the upper 50± feet generally possess loose to medium dense relative densities with loose soils present as deep as 12± feet.

The results of our site-specific liquefaction evaluation indicate that some potentially liquefiable soil layers are present between depths of 8 and 50± feet. Some of these liquefiable layers are expected to be located within the zones of influence for conventional shallow foundations for the new building. As discussed in the previous section, liquefaction can result in a loss of bearing capacity and excessive settlements of foundation elements supported on liquefiable soils. Therefore, liquefaction potential of soil layers present within the influence zone of any new building foundations should mitigated if conventional shallow foundations will be used for the proposed building at this site.

Based on the presence of undocumented fill materials and low strength, loose and potentially liquefiable native alluvial soils, the near surface soils, in their present condition, are not considered suitable for support of the foundations and floor slab of the new structure.

Specialized ground improvement techniques and/or overexcavation and recompaction (with some dewatering, necessary between depths of 15 to 20± feet) should be implemented to mitigate the liquefaction potential of liquefiable soils within the foundation influence zones and reduce potential dynamic settlements to within tolerable limits. The grading and foundation design parameters provided in the subsequent sections of this report assume that the soils in at least the upper 20± feet have been improved using ground improvement techniques and/or remedial grading.

Settlement

The recommended ground improvement measures (or remedial grading) will improve the very loose fill and native alluvial soils as well as the liquefiable soils present within the foundation influence zones of the new buildings. The native soils that will remain in place below the recommended depth of ground improvement will not be subject to significant load increases from the foundations of the new structures. Provided that the ground improvement and/or recommended remedial grading is completed, the post-construction settlements of the proposed structures are expected to be within tolerable limits for conventional shallow foundations.

Soluble Sulfates

The results of the soluble sulfate testing, as discussed in Section 5.0 of this report, indicate a soluble sulfate concentration of 0.100 percent. This test result indicates that the concentration of soluble sulfates within the selected sample of the on-site soils corresponds to Class S1 with respect to the American Concrete Institute (ACI) Publication 318-05 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. The highest concentration of soluble sulfates constitutes a moderate exposure of soluble sulfates to concrete in contact with the soil (Exposure Category S1), according to American Concrete Institute (ACI) Publication 318 - Building Code Requirements for Structural Concrete and Commentary indicates that concrete in contact with the on-site soils should possess the following characteristics:

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

It is also recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the new pavement subgrade soils.
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Corrosion Potential

The results of the electrical resistivity and pH testing indicate that the tested sample of the on-site soils has a saturated resistivity of 260 ohm-cm and a pH value of 8.4. 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 and pH are two of the five factors that enter into the evaluation procedure. Relative soil moisture content as well as redox potential and sulfides are also included. Although redox potential and sulfide testing were not part of the scope of services for this project, we have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH and moisture content. Based on these factors, and utilizing the DIPRA procedure, **some of the on-site soils are considered to be severely corrosive to ductile iron pipes and other buried metal improvements. Therefore, it is expected that polyethylene encasement will be required for iron pipes.** If a more detailed evaluation is desired, redox potential and sulfide content should be determined for the on-site soils. Since SCG does not practice in the area of corrosion engineering, it is recommended that the client contact a corrosion engineer to provide a more thorough evaluation.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested sample possesses a nitrate concentration of 237 mg/kg. **Based on this test result, the on-site soils are considered to be corrosive to copper pipe. Since SCG does not practice in the area of corrosion engineering, we recommend that the client contact a corrosion engineer to provide recommendations for the protection of copper tubing/pipe in contact with the on-site soils.**

Based on American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary, reinforced concrete that is exposed to external sources of chlorides requires corrosion protection for the steel reinforcement contained within the concrete. ACI 318 defines concrete exposed to moisture and an external source of chlorides as "severe" or exposure category C2. For exposure category C2, ACI 318 prescribes the use of concrete with a compressive strength of 5,000 psi and a maximum water cement ratio of 0.4. ACI 318 does not clearly define a specific chloride concentration at which contact with the adjacent soil will constitute a "C2" or severe exposure. However, the Caltrans Memo to Designers 10-5, Protection of Reinforcement Against Corrosion Due to Chlorides, Acids and Sulfates, dated June 2010, indicates that soils possessing chloride concentrations greater than 500 mg/kg are considered to be corrosive to reinforced concrete.

Based on the results of the corrosivity testing and our understanding of the criteria for a "severe" (C2) chloride exposure, soils that can constitute a potentially corrosive exposure are present at the tested sample location.

Since SCG does not practice in the area of corrosion engineering, the client should consult with a corrosion engineer to provide any further recommendations the chloride exposure for this site. In accordance with the requirements of ACI 318 for severe or C2 chloride exposure, any reinforced concrete in contact with the on-site soils will require a minimum compressive strength of 5,000 lbs/in^2 and a maximum water cement ratio of 0.40.

Expansion

The near surface fill soils at this site generally consist of clayey silts and sandy silts with trace to little clay content. Laboratory testing performed on representative samples of these materials indicate that they possess a low expansion potential (EI = 23 and 35). Based on the presence of potentially expansive soils, special care should be taken to properly moisture condition and maintain adequate moisture content within all subgrade soils as well as newly placed fill soils. The foundation and floor slab design recommendations contained within this report are made in consideration of the expansion index test results. We recommend that additional expansion index testing be conducted at the completion of rough grading to verify the expansion potential of the as-graded building pad.

Shrinkage/Subsidence

Removal and recompaction of the near-surface fill and native alluvial soils is estimated to result in an average shrinkage of 10 to 18 percent. However, the estimated shrinkage of the individual soil layers at the site is highly variable, locally ranging from 4 to 22 percent shrinkage. 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

Grading and foundation plans for the proposed development were not available at the time of this report. It is therefore recommended that we be provided with copies of the 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.

Demolition and Site Stripping

The proposed development will require demolition of the existing pavements and structures. This should include all foundations, floor slabs, utilities, and any other subsurface improvements associated with the existing structures. The existing pavements are not expected to be reused with the new development. Additionally, any existing improvements that will not remain in place for use with the new development should be removed in their entirety. This should include all utilities, and any other subsurface improvements associated with the existing pavements. Existing improvements which are to remain in place with the new development should be protected from damage by construction traffic.

Debris resultant from demolition should be disposed of offsite. Concrete and asphalt debris may be re-used within compacted fills, provided they are pulverized to a maximum particle size of less than 2 inches, and thoroughly mixed with on-site sandy soils. Concrete and asphalt debris should not be blended with clayey soils. Existing asphalt and concrete materials may also be crushed into miscellaneous base (CMB) and re-used at the site. Alternatively, concrete and asphalt debris may be crushed to particle sizes of 2 to 4 inches and used to stabilize unstable overexcavation subgrades.

Demolition of some landscape planters is also expected to be required. Any vegetation or organic soils within these planters should be disposed of off-site. Turf grass and other grass and weed growth should be stripped from the site in its entirety. Removal of some trees may also be required. Where trees are removed, the removal should also include any associated root masses. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

<u>Treatment of Existing Soils: Building Pad</u>

Undocumented artificial fill soils were encountered within the upper 3 to 4½± feet, and loose native alluvial soils are present within the upper 8 and 12± feet, at the boring locations. Potentially liquefiable soils are present within the anticipated depths of the influence zones of the foundations for the proposed structure (and at greater depths). Based on these conditions, remedial grading and/or ground improvement techniques will be necessary to remove the artificial fill soils in their entirety from the proposed building pad area, and to remediate the loose and potentially liquefiable soils in the foundation influence zones.

In order to mitigate the potential for bearing capacity loss in the areas of the proposed building foundations, the depth of ground improvement should extend to a minimum of 20 feet below the existing site grades, and extend to a depth equal to two times the width of square footings and at least 3 times the width of continuous foundations.

Remedial grading is considered to be an acceptable means of improving the soils present within the foundation influence zones of the new structure. However, some dewatering will be required (at depths greater than 15 feet based on the conditions encountered at the time of drilling) to perform the recommended remedial grading to the depths recommended above. Therefore, it is anticipated that specialized ground improvement techniques will be more economical to mitigate the liquefiable soils and loose to very loose alluvium present within the foundation influence zones of the new structure. At a minimum, remedial grading should be performed to remove the undocumented fill soils from the proposed building pad area.

Remedial grading should be performed within the proposed building pad area in order to remove any soils disturbed during demolition of the existing improvements and the undocumented fill soils (which are present within the upper 3 to 4½± feet below the existing site grades at the boring locations). As discussed above, remedial grading may also be performed to remediate the loose and potentially liquefiable native alluvial soils within the foundation influence zones. If specialized ground improvement measures will be employed at this site, then the ground improvement contractor should determine whether or not the overexcavation should be performed before the ground improvement measures are implemented. At a minimum, it is recommended that the overexcavation extend to a depth of at least 3 feet below the proposed building pad grade and to a depth sufficient to remove the undocumented fill soils and any soils disturbed during demolition. If specialized ground improvement measures are not used at this site, then the overexcavation should extend to a depth equal to at least 2 times the width of square column footings (below the foundation bearing grade), to a depth of at least 3 times the width of continuous foundations (below the foundation bearing grade), and to a minimum depth of 20 feet below the existing site grades.

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

Following completion of the overexcavation, the subgrade soils within the overexcavation areas 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, overly moist, or low-density native soils are encountered at the base of the overexcavation.

Based on conditions encountered at the exploratory boring locations, very moist to wet soils may be encountered at or near the base of the recommended overexcavation. Stabilization of the exposed overexcavation subgrade soils may be necessary. 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 or geotextile, could be necessary. In this event, the geotechnical engineer should be contacted for supplementary recommendations. Typically, an unstable subgrade can be stabilized using a suitable geotextile fabric, such as Mirafi 580I, HP 570 or HP 270, and/or a 12 to 18-inch thick layer of coarse (2 to 4 inch particle size) crushed stone. Crushed asphalt and concrete debris resultant from demolition could also be used as a subgrade stabilization material. Other options, including lime treatment, are also available.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, and moisture conditioned to 2 to 4 percent above optimum moisture content. The subgrade soils 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, provided that they are dried to within 2 to 4 percent above the optimum moisture content. The use of an imported select fill material may be desirable if the construction schedule does not allow for drying of the on-site soils. `

Ground Improvement

As discussed above, it may not be practical to overexcavate and recompact all of the very loose to loose and potentially liquefiable native alluvial soils within the foundation influence zones of the new structures, due to the presence of groundwater at depths as shallow as 15± feet at the boring locations. If remedial grading is not considered practical due to the necessary dewatering, specialized ground improvement techniques will be necessary to mitigate the loose to very loose and potentially liquefiable soils present in the influence zones of the new building foundations. Based on the groundwater levels and the soil conditions, techniques such as deep soil mixing, rammed aggregate piers, or grout injection are considered to be applicable for ground improvement. Other methods may also be viable at this site. A specialty contractor should be contacted for specifics of design-build ground improvement methods. Ground improvement should be designed to mitigate potentially liquefiable soil layers and loose native alluvial soils within at least the upper 20 feet below existing site grades and should also extend to a depth of at least 2 times the width of the proposed square column footings (below the foundation bearing grade) and to a depth of at least 3 times the foundation width below continuous footings. The actual design of the ground improvement method should be performed by the design-build contractor who is specialized and experienced with these methods. Ground improvement methods are designed and implemented by specialty contractors on a design–build basis where the contractors are ultimately fully responsible for the effectiveness of their mitigation measures over the life of the project.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of proposed retaining and non-retaining site walls should be overexcavated to a depth of at least 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. Erection pads are considered to be part of the foundation system, and therefore these overexcavation recommendations apply to erection pads also. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils, as discussed for the building area. The previously excavated soils may then be replaced as compacted structural fill.

If the full lateral extent of overexcavation is not achievable for the proposed walls, the foundations elements must be redesigned using a lower allowable bearing pressure. If the vertical extent of the overexcavation cannot be completed due to the presence of groundwater, ground improvement may be necessary for these retaining walls. The geotechnical engineer of record should be contacted for recommendations pertaining to either of these conditions.

Treatment of Existing Soils: Parking Areas

Based on economic considerations, overexcavation of the existing soils in the new 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 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. Any such materials should be removed to a level of firm and unyielding soil. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to at least 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed parking area 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 the existing fill soils and low strength alluvium in the parking 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 parking and drive areas should be mitigated in a manner similar to that described for the building pad.

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. Drying of some the onsite soils may be required before placement and compaction as fill.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2019 CBC and the grading code of the city of Cerritos.
- 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. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Cerritos. 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 in the upper 5± feet generally consist of sandy silts and silty clays. Some of these materials will likely be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, temporary excavation slopes should be made no steeper than 2:1v. The contractor should take all necessary precautions during grading and foundation construction to prevent damage to structures and improvements which are adjacent to the proposed development. 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. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

The near surface soils possess appreciable silt and clay content and will become unstable if exposed to significant moisture infiltration or disturbance by construction traffic. If grading occurs during a period of relatively wet weather, an increase in subgrade instability should also be expected. 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 are likely to be encountered at the base of the overexcavations within the proposed building area. 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 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 drier, less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pad area 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

Based on the conditions encountered in the borings, the groundwater table is considered to have been present at depths of 15 to 20± feet at the time of subsurface exploration. Therefore, based on the current groundwater depths, excavations extending to depths of 15 feet or more may encounter the groundwater table. Dewatering will likely be required in excavations extending to depths of 15 feet or more below the existing site grades. It should be noted that groundwater depths fluctuate and that the historic high groundwater level is about 8 feet below the existing site grades.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad area will be underlain by newly placed structural fill soils extending to depths of at least 3 feet below the proposed building pad grade. Additionally, the loose and potentially liquefiable soils located within the zones of foundation influence will either mitigated using specialized ground improvement techniques or will be recompacted as structural fill. Based on this subsurface profile, the proposed structure may be supported on 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 (assuming that on-site soils are recompacted within the foundation influence zones). A greater foundation bearing pressure may be allowed based on the ground improvement technique used.

- 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 the presence of potentially liquefiable and low expansive soils.

- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 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.

- It is recommended that any isolated column footings be structurally connected to adjacent columns and/or the perimeter foundations in both perpendicular directions using grade beams. The grade beam system should be designed by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind loads. **However, based on the presence of shallow liquefiable soils,**

we do not recommend an increase in the allowable bearing capacity for seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations. 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 30-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. However, the likelihood of these two settlements combining is considered remote. The static settlements are expected to occur in a relatively short period of time after the building loads being applied to the foundations, during and immediately subsequent to construction. Additionally, the use of ground improvement techniques and/or remedial grading within the foundation influence zones is expected to significantly reduce the potential total and differential seismic settlements.

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: 250 lbs/ft^3
- Friction Coefficient: 0.29

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. The maximum allowable passive pressure is 2,500 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs 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 3 feet below proposed finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches
- Modulus of Subgrade Reaction: k = 100 psi/in.
- Minimum slab reinforcement: Minimum slab reinforcement: No. 3 bars at 18 inches on-center, in both directions, due to the presence of potentially liquefiable and low expansive soils at the site. The actual floor slab reinforcement should be determined by the structural engineer, based on 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 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 slabs should be completed by the structural engineer to verify adequate thickness and reinforcement. The steel reinforcement recommendations presented above are based on standard geotechnical practice, given the magnitude of predicted liquefaction-induced settlements, and the structure type proposed for the site. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements discussed in Section 6.1.

6.7 Retaining Wall Design and Construction

Although not indicated on the site plan, some small (less than 6 feet in height) retaining walls may be required to facilitate the new site grades and in loading docks. 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 soils generally consist of sandy silts and clayey silts. We do not recommend that the on-site clayey silts be used as retaining wall backfill, based on their expansion potential Based on their composition, the on-site sandy silts are expected to possess a friction angle of at least 29 degrees when compacted to at least 90 percent of the ASTM D-1557 maximum dry density.

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 Sandy Silts
Internal Friction Angle (ϕ)		29°
Unit Weight		125 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	44 lbs/ft^3
	Active Condition (2h:1v backfill)	73 lbs/ft^3
	At-Rest Condition (level backfill)	65 lbs/ft^3

The walls should be designed using a soil-footing coefficient of friction of 0.29 and an equivalent passive pressure of 250 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 2019 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 the proposed 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

Retaining wall backfill soils should consist of on-site sands, silty sands or sandy silts. 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 2-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 10-foot on-center spacing. Alternatively, 4-inch diameter holes at an approximate 20-foot on-center spacing can be used for this type of drainage system. In addition, 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. The actual design of this type of system should be determined by the civil engineer to verify that the drainage system possesses the adequate capacity and slope for its intended use.

Weep holes or a footing drain will not be required for building stem walls.

6.8 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 soils generally consist of sandy silts and clayey silts. These soils are generally considered to possess fair to good pavement support characteristics with an estimated R-values ranging from 15 to 30. The subsequent pavement design is therefore based upon an assumed R-value of 15. 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.

<u>Asphaltic Concrete</u>

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

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 = 15)					
Materials	**Thickness (inches)**				
	Automobile Parking (TI = 4.0)	Automobile Drive Lanes (TI = 5.0)	Truck Traffic		
			(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	6	9	11	13	15
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" <u>Standard Specifications for Public Works Construction</u>.

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			
Materials	**Thickness (inches)**		
	Automobile and Light Truck Traffic (TI = 5.0 & 6.0)	Truck Traffic	
		(TI = 7.0)	(TI = 8.0)
PCC	5	5½	7
Compacted Subgrade (95% minimum compaction)	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. 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

California Division of Mines and Geology (CDMG), "Guidelines for Evaluating and Mitigating Seismic Hazards in California," State of California, Department of Conservation, Division of Mines and Geology, Special Publication 117A, 2008.

Idriss, I. M. and Boulanger, R.W., "Soil Liquefaction During Earthquakes", Earthquake Engineering Research Institute, 2008.

National Research Council (NRC), "Liquefaction of Soils During Earthquakes," Committee on Earthquake Engineering, National Research Council, Washington D. C., Report No. CETS-EE-001, 1985.

Seed, H. B., and Idriss, I. M., "Simplified Procedure for Evaluating Soil Liquefaction Potential using field Performance Data," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, September 1971, pp. 1249-1273.

Sadigh, K., Chang, C. –Y., Egan, J. A., Makdisi. F., Youngs, R. R., "*Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data*", Seismological Research Letters, Seismological Society of America, Volume 68, Number 1, January/ February 1997, pp. 180-189.

Southern California Earthquake Center (SCEC), University of Southern California, "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California," Committee formed 1997.

Tokimatsu K., and Seed, H. B., "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of the Geotechnical Engineering Division, American society of Civil Engineers, Volume 113, No. 8, August 1987, pp. 861-878.

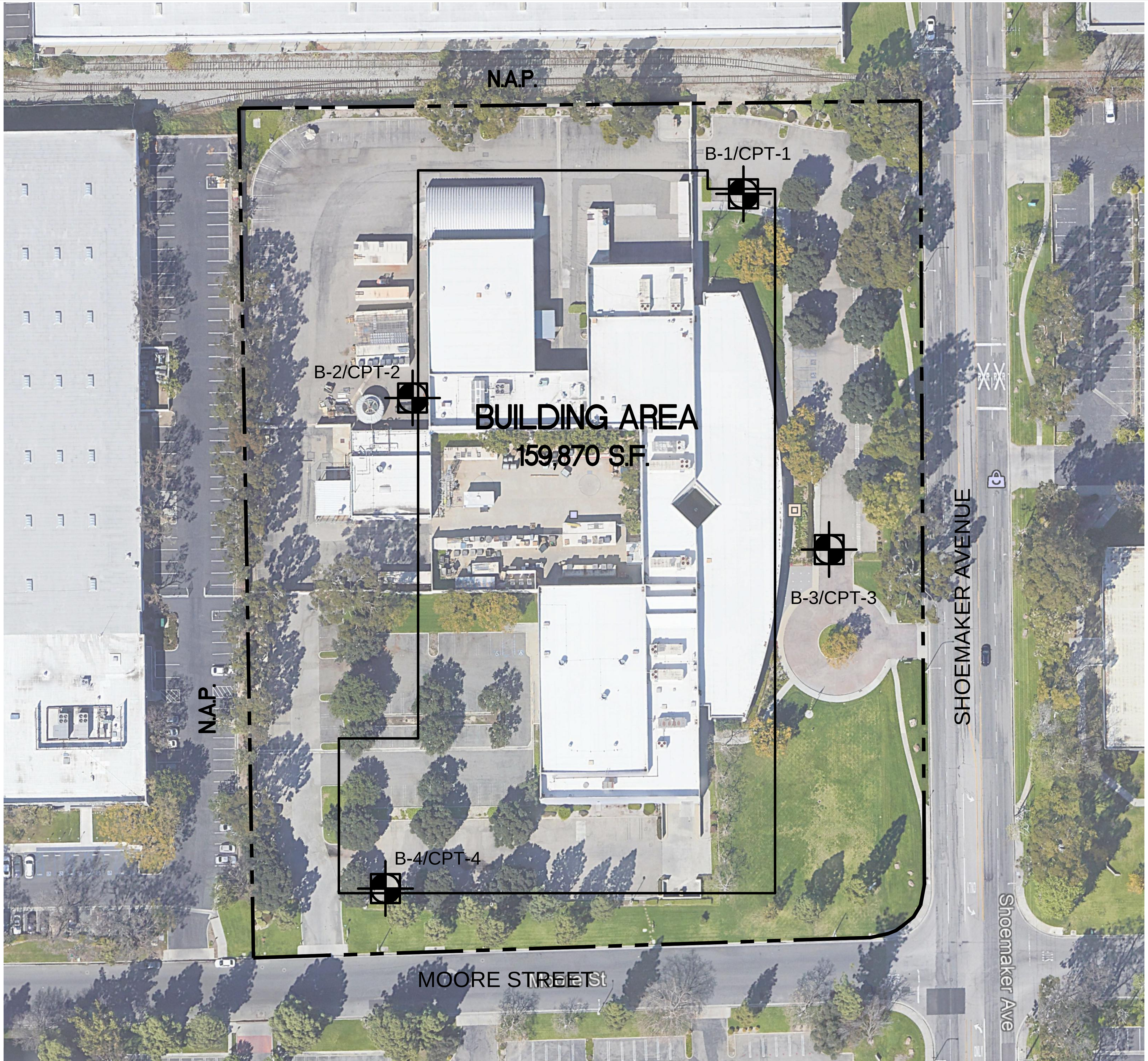
Tokimatsu, K. and Yoshimi, Y., "*Empirical Correlations of Soil Liquefaction Based on SPT N-value and Fines Content,*" Seismological Research Letters, Eastern Section Seismological Society Of America, Volume 63, Number 1, p. 73.

Youd, T. L. and Idriss, I. M. (Editors), "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," Salt Lake City, UT, January 5-6 1996, NCEER Technical Report NCEER-97-0022, Buffalo, NY.

APPENDIX A

SOURCE: USGS TOPOGRAPHIC MAP OF THE WHITTIER QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA, 2018.

SITE LOCATION MAP	
PROPOSED WAREHOUSE	
CERRITOS, CALIFORNIA	
SCALE: 1" = 2000'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: OS CHKD: DWN	
SCG PROJECT 21G233-1	
PLATE 1	



GEOTECHNICAL LEGEND

APPROXIMATE BORING AND CPT LOCATION (APPROXIMATELY 5 FEET APART)

NOTE: CONCEPTUAL SITE PLAN (SCHEME 1) PREPARED BY HPA, INC.
AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH (2018)

BORING AND CPT LOCATION PLAN

PROPOSED WAREHOUSE

CERRITOS, CALIFORNIA

SCALE: 1" = 80'

DRAWN: OS
CHKD: DWN

SCG PROJECT
21G233-1

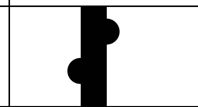
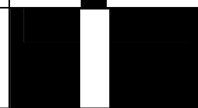
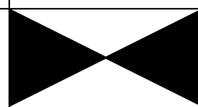
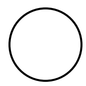
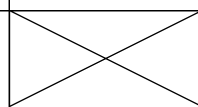
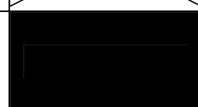
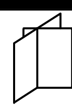
PLATE 2

SoCalGeo

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.: 21G233-1	DRILLING DATE: 10/4/21	WATER DEPTH: 20 feet
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 21 feet
LOCATION: Cerritos, California	LOGGED BY: Oscar Sandoval	READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
				2± inches Asphaltic Concrete; 4± inches Aggregate Base							
	X	18	2.0	FILL: Light Gray Brown Clayey Silt, trace to little fine Sand, trace fine root fibers, very stiff-very moist		25					
	X	15		ALLUVIUM: Light Brown Silty fine Sand, trace fine root fibers, medium dense-damp to moist		7					
5	X	8		Dark Brown Silty fine Sand to fine Sandy Silt, trace Clay, trace fine root fibers, loose-very moist		16					
	X	7				15			42		
10				Light Gray Brown fine Sand, trace Silt, trace Iron oxide staining, medium dense-damp							
	X	13				6			4		
15				Gray Brown to Dark Gray Silty Clay, trace fine Sand, medium stiff-very moist to wet							
	X	7	1.5			26	37	14	81		
20											Groundwater Encountered During Drilling
				Gray Brown to Dark Brown fine Sandy Silt, medium dense-wet							
	X	11				27			70		
25											
	X	25		@ 28½ to 30 feet, trace Clay		31			63		
30				Gray Silty fine Sand, medium dense-wet							
	X	13				22			18		

TBL 21G233-1.GPJ SOCALGEO.GDT 11/9/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 21G233-1
PROJECT: Proposed Warehouse
LOCATION: Cerritos, California

DRILLING DATE: 10/4/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Oscar Sandoval

WATER DEPTH: 20 feet
CAVE DEPTH: 21 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION (Continued)	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG		DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					Gray Silty fine Sand, medium dense-wet							
40	X	20					26			18		
					Gray Brown fine to medium Sand, little Silt, medium dense to dense-wet							
45	X	30					27					
50	X	29					26			8		
					Boring Terminated at 50'							

TBL 21G233-1.GPJ SOCALGEO.GDT 11/9/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 21G233-1
PROJECT: Proposed Warehouse
LOCATION: Cerritos, California

DRILLING DATE: 10/4/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Oscar Sandoval

WATER DEPTH: 20 feet
CAVE DEPTH: 19 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					3± inches Asphaltic Concrete; 4± inches Aggregate Base							
	X	34	4.5		FILL: Dark Gray Brown Clayey Silt, trace fine Sand, mottled, very stiff to hard-very moist	106	17					EI = 35 @ 0 to 5'
	X	21	4.0			104	20					
5	X	19			ALLUVIUM: Dark Gray Brown fine Sandy Silt, medium dense-very moist	85	32					
	X	14			Gray Brown to Dark Gray Brown Silty fine Sand, trace Iron oxide staining, loose-damp to moist	75	7					
10	X	14				98	11					
					Gray Brown fine Sand, little SIlt, medium dense-damp							
15	X	29				97	6					
					Gray Brown Silty Clay, little fine Sand, stiff-wet							
20	X	18	2.0			99	27					Groundwater Encountered During Drilling
					Gray Brown fine Sand, trace Silt, medium dense-wet							
25	X	26				102	23					
					Boring Terminated at 25'							

TBL 21G233-1.GPJ SOCALGEO.GDT 11/9/21

BORING NO. B-3

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 21G233-1
PROJECT: Proposed Warehouse
LOCATION: Cerritos, California

DRILLING DATE: 10/4/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Oscar Sandoval

WATER DEPTH: 19 feet
CAVE DEPTH: 19 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					2± inches Asphaltic Concrete; 3± inches Aggregate Base							
	X	23			FILL: Light Brown to Dark Gray Brown fine Sandy Silt, trace Clay, trace fine Gravel, mottled, medium dense-very moist	99	18					
	X	18				102	16					
5	X	16			ALLUVIUM: Light Brown fine Sand, trace Silt, medium dense-dry	98	2					
	X	17			Dark Gray Brown Silty fine Sand to fine Sandy Silt, trace Iron oxide staining, medium dense-moist to very moist	103	14					
10	X	9			Dark Gray Brown fine Sandy Silt, trace Clay, loose-very moist	107	20					
					Light Gray Brown fine Sand, little Silt, trace Iron oxide staining, medium dense-damp							
15	X	25				97	7					
					Dark Gray Brown fine Sandy Clay to Silty Clay, stiff-wet							
20	X	15	2.5			99	26					Groundwater Encountered
					Brown to Dark Gray Brown Silty fine Sand, medium dense-wet							
25	X	19				109	20					

Boring Terminated at 25'

TBL 21G233-1.GPJ SOCALGEO.GDT 11/9/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 21G233-1	DRILLING DATE: 10/4/21	WATER DEPTH: 15 feet
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Cerritos, California	LOGGED BY: Oscar Sandoval	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					2± inches Asphaltic Concrete; No discernible Aggregate Base							
		36			FILL: Brown to Dark Brown fine Sandy Silt, little Clay, medium dense-moist to very moist	119	11					EI = 23 @ 0 to 5'
		40				105	16					
5		20			ALLUVIUM: Light Brown fine Sand, medium dense-dry	88	2					
		12			Dark Brown fine Sandy Silt, trace Clay, micaceous, loose-very moist	108	18					
		10	3.0		Dark Brown fine Sandy Clay, trace Iron oxide staining, stiff-very moist	100	22	29	9	62		
10												
					Dark Brown SIlty fine Sand to fine Sandy Silt, medium dense-wet							
		13					30			53		
15												Groundwater Encountered During Drilling
					Dark Gray Brown fine Sandy Clay, soft to medium stiff-wet							
		4	2.0				26	32	12	73		
20												
					Dark Brown Silty fine Sand, medium dense to dense-wet							
		16					22			13		
25												
		45					22					
30												
		21					23			26		

TBL 21G233-1.GPJ SOCALGEO.GDT 11/9/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 21G233-1
PROJECT: Proposed Warehouse
LOCATION: Cerritos, California

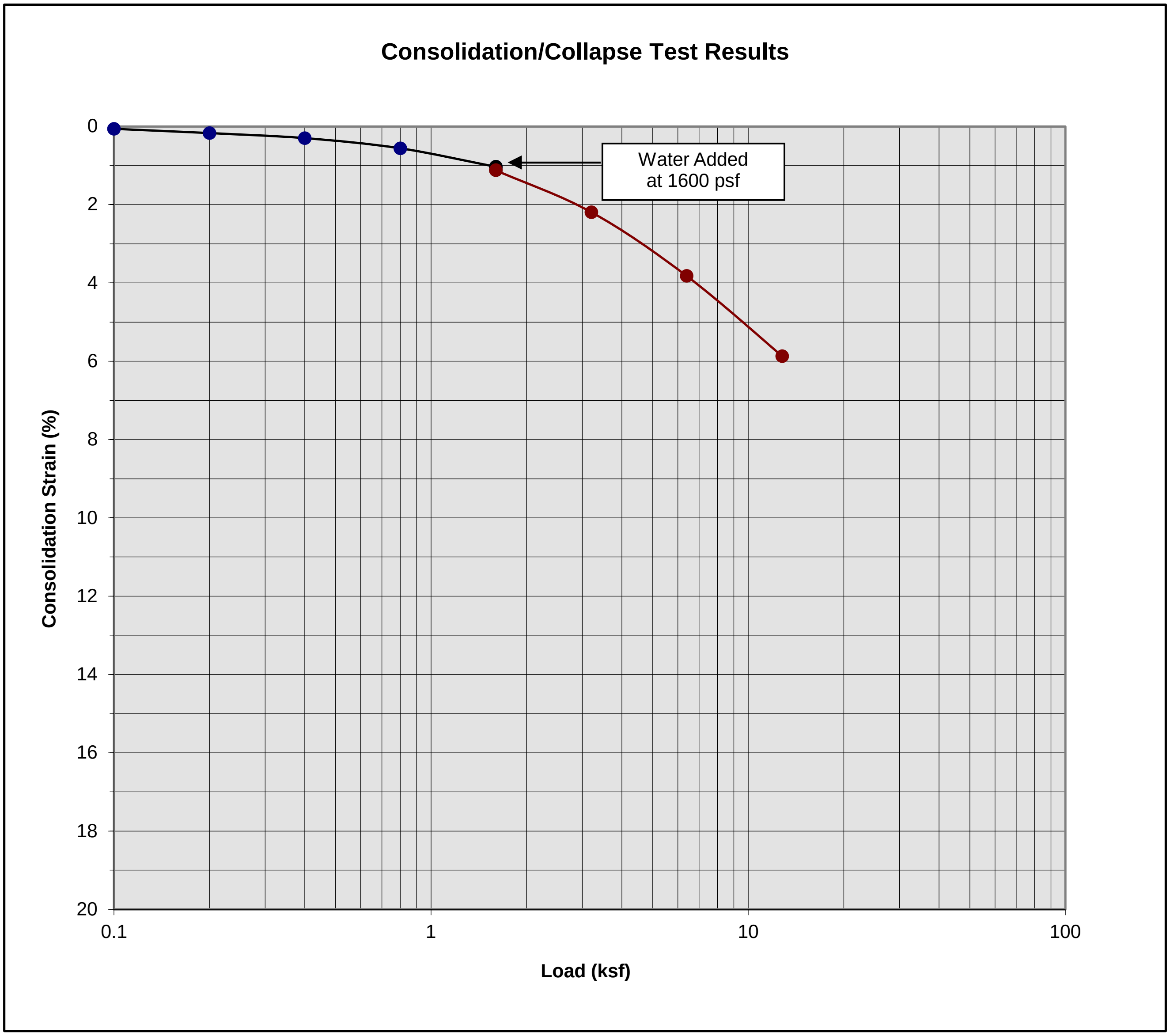
DRILLING DATE: 10/4/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Oscar Sandoval

WATER DEPTH: 15 feet
CAVE DEPTH: 17 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	(Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					Dark Brown Silty fine Sand, medium dense to dense-wet							
	X	40					17					
40												
					Gray Brown Clayey fine Sand, dense-wet							
	X	31					24					
45												
					Dark Gray Brown SIlty Clay to Clayey Silt, little fine Sand, very stiff-wet							
	X	24	3.0				33			92		
50												
					Boring Terminated at 50'							

TBL 21G233-1.GPJ SOCALGEO.GDT 11/9/21

APPENDIX C



Classification: FILL: Light Brown to Brown fine Sandy Silt, trace Clay

Boring Number:	B-3	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	3 to 4	Initial Dry Density (pcf)	102.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	105.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.09

Proposed Warehouse
Cerritos, CA
Project No. 21G233-1
PLATE C- 1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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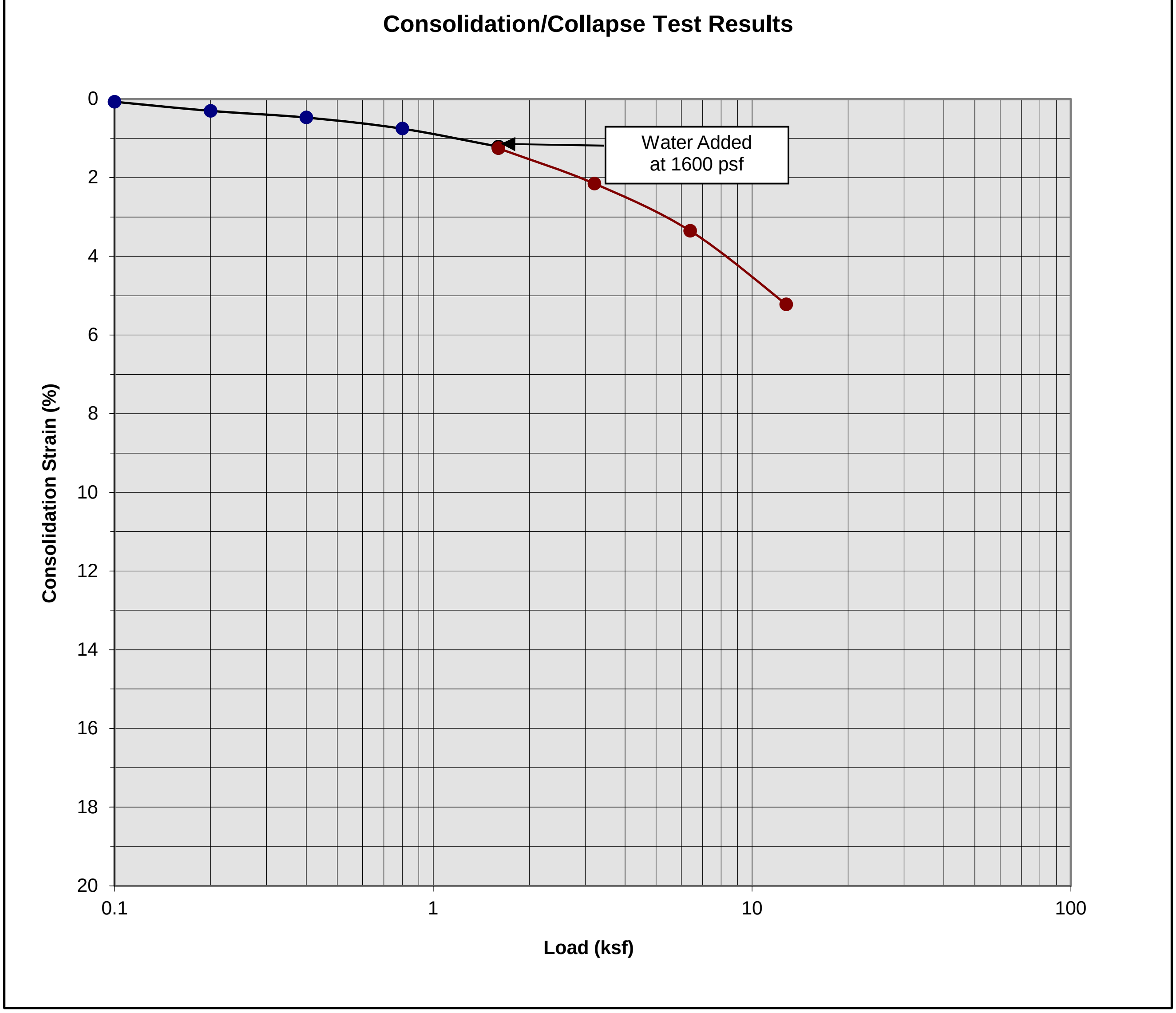

Classification: Light Brown fine Sand, trace Silt

Boring Number:	B-3	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	5 to 6	Initial Dry Density (pcf)	97.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	122.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.04

SOUTHERN CALIFORNIA GEOTECHNICAL
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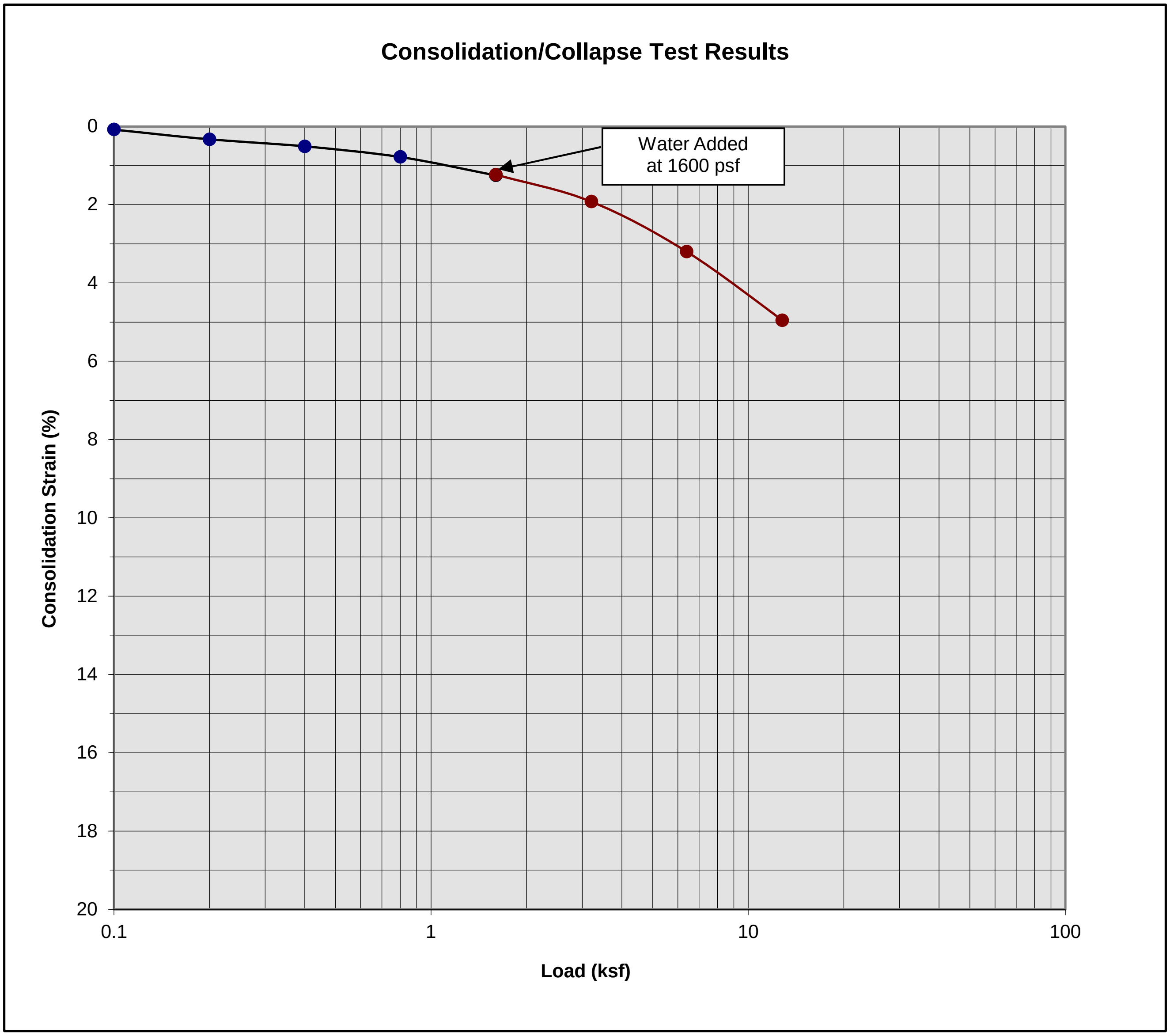

Classification: Dark Gray Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-3	Initial Moisture Content (%)	13
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	7 to 8	Initial Dry Density (pcf)	102.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.02

Proposed Warehouse
Cerritos, CA
Project No. 21G233-1
PLATE C- 3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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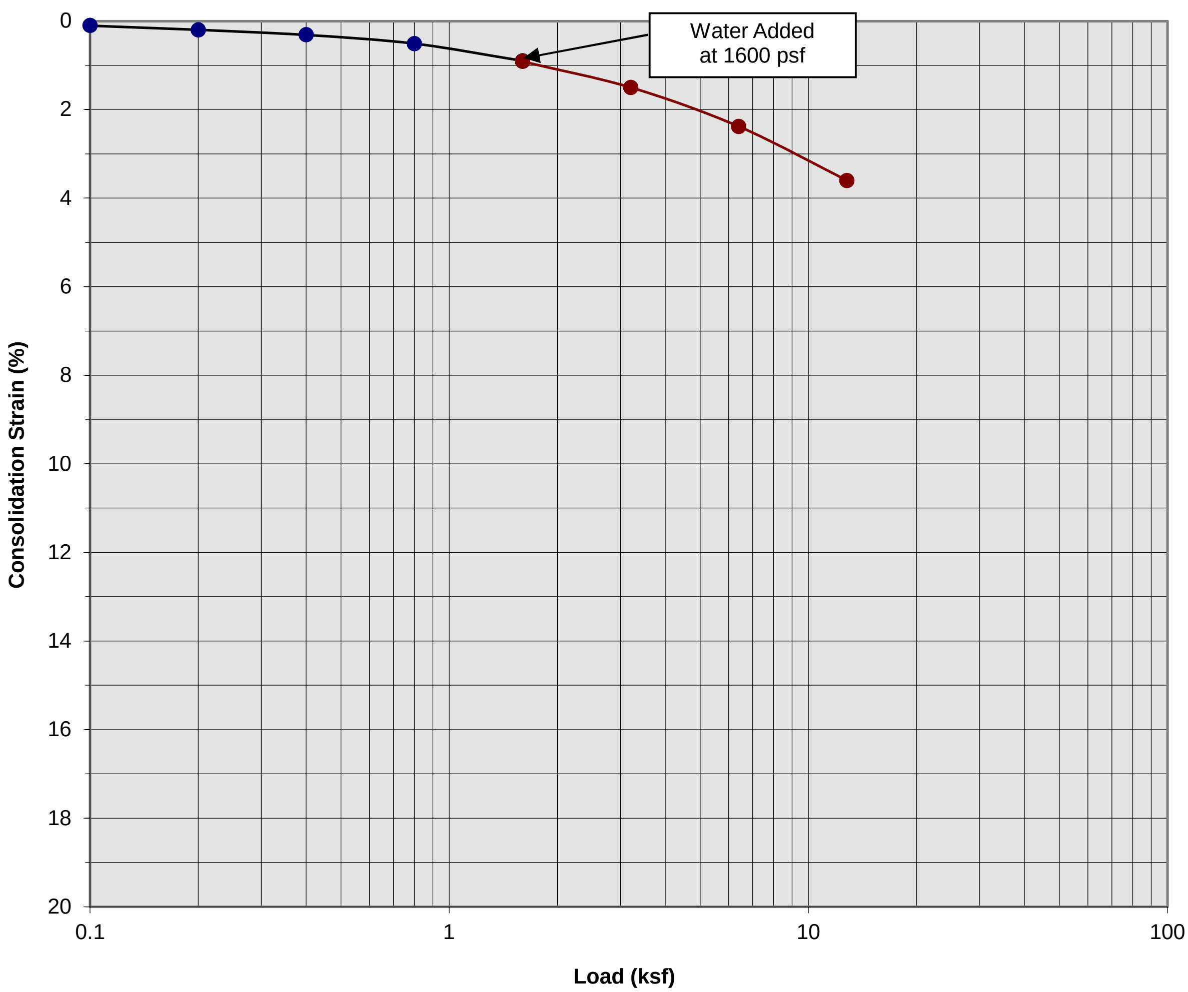
Consolidation/Collapse Test Results

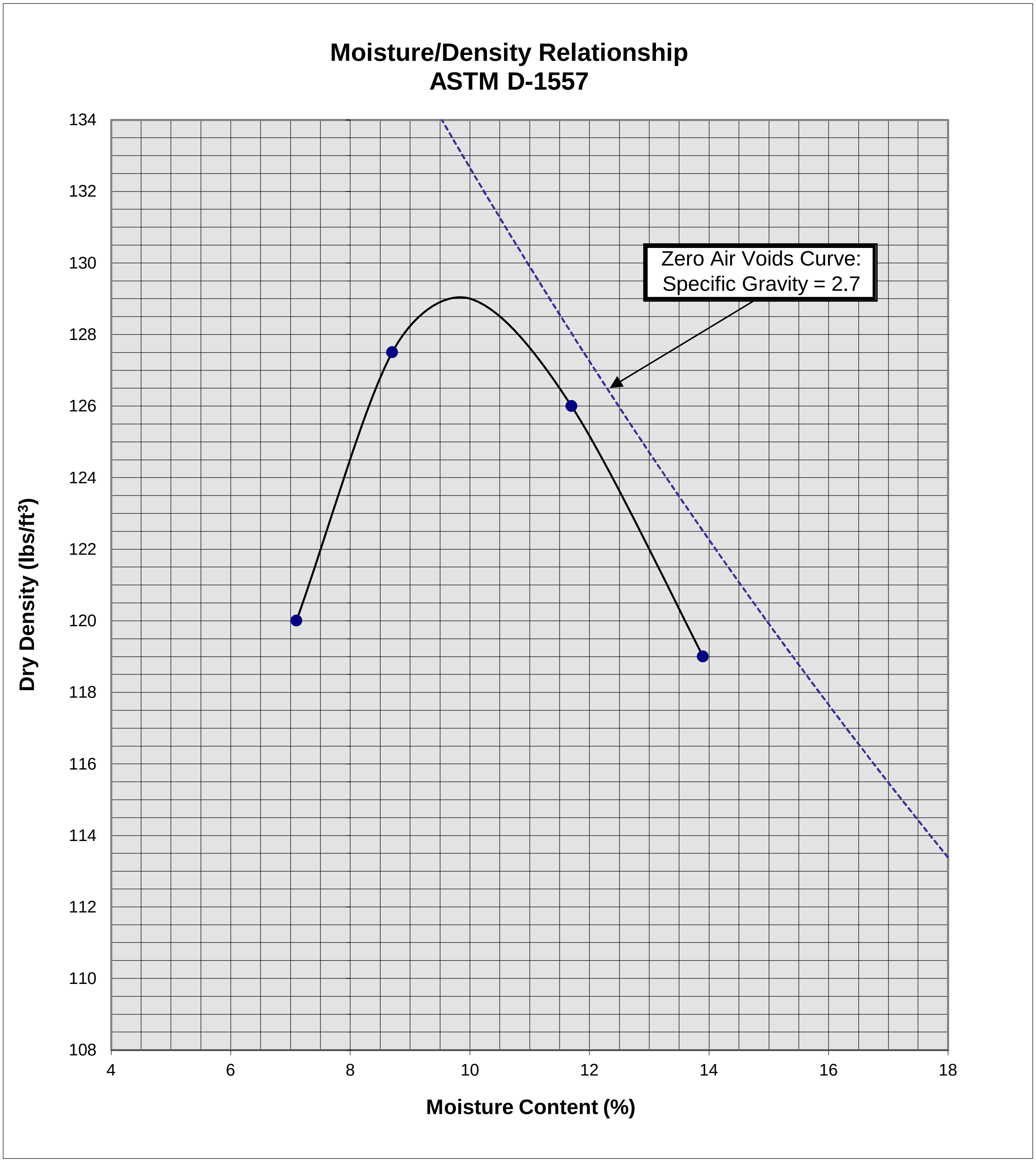


Classification: Dark Gray Brown fine Sandy Silt

Boring Number:	B-3	Initial Moisture Content (%)	20
Sample Number:	---	Final Moisture Content (%)	23
Depth (ft)	9 to 10	Initial Dry Density (pcf)	107.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.01

Proposed Warehouse
Cerritos, CA
Project No. 21G233-1
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number		B-4 @ 0 to 5'
Optimum Moisture (%)		10
Maximum Dry Density (pcf)		129
Soil Classification	Dark Brown Silty fine Sand, trace fine Gravel	

Proposed Warehouse
Cerritos, CA
Project No. 21G233-1
PLATE C-5

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 50. 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.

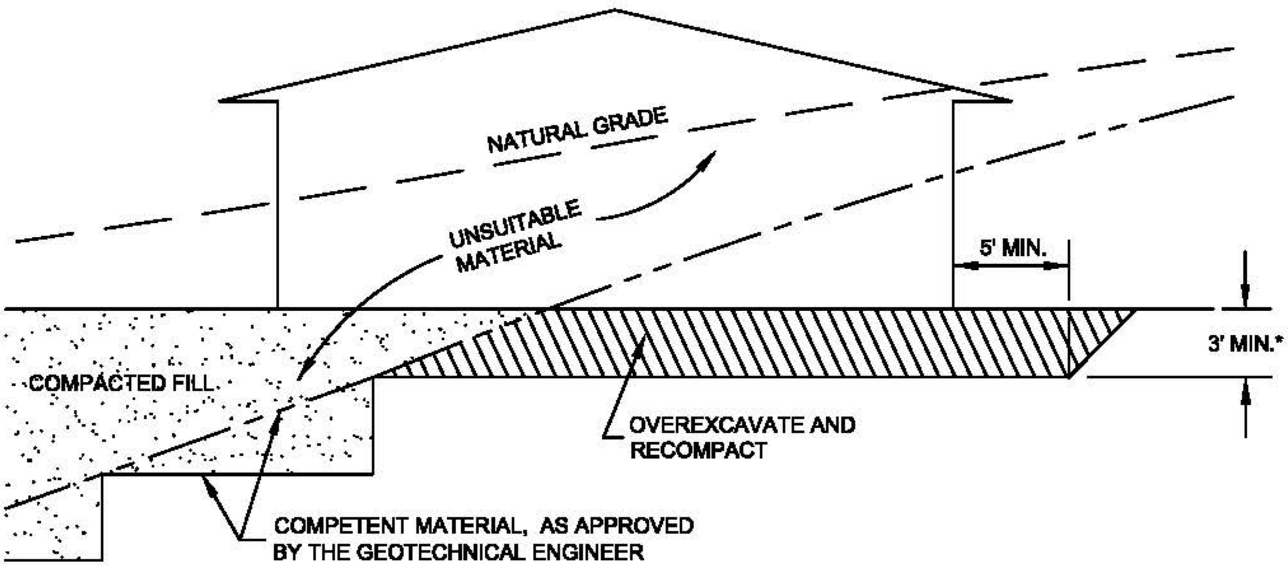
Subdrains

- 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.

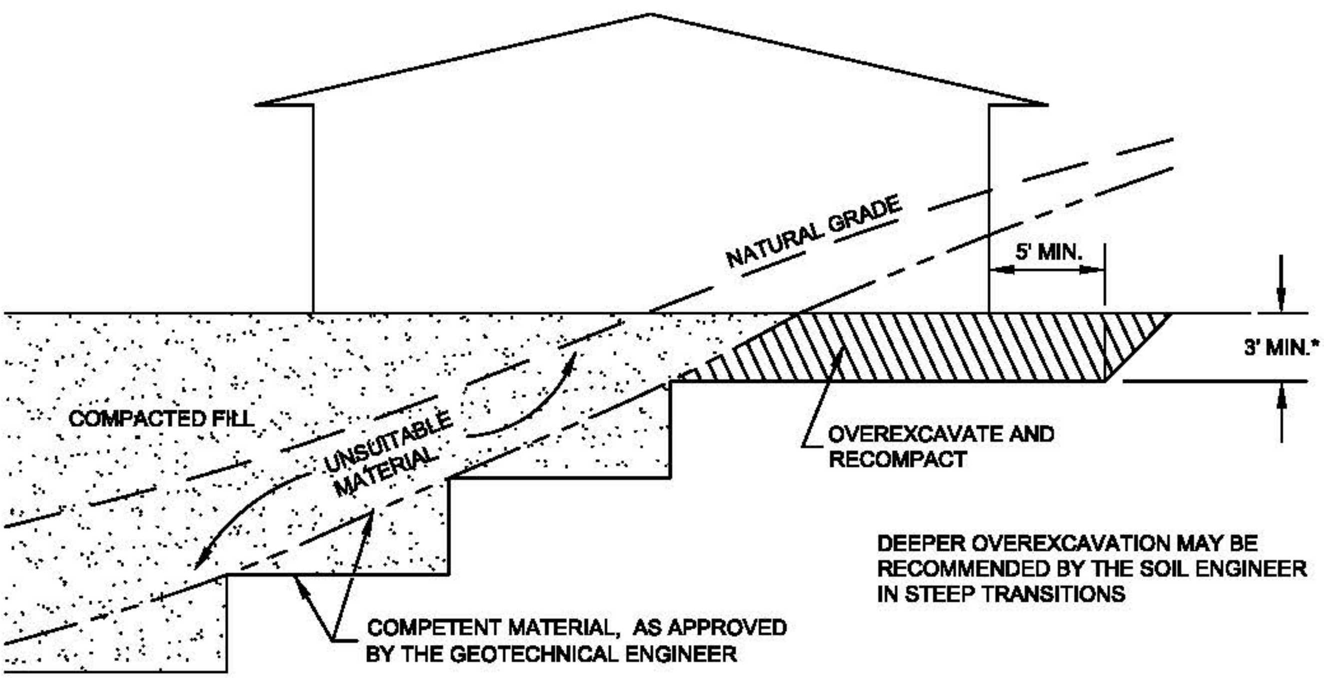
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- 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)



DEEPER OVEREXCAVATION MAY BE RECOMMENDED BY THE SOIL ENGINEER IN STEEP TRANSITIONS

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

TRANSITION LOT DETAIL

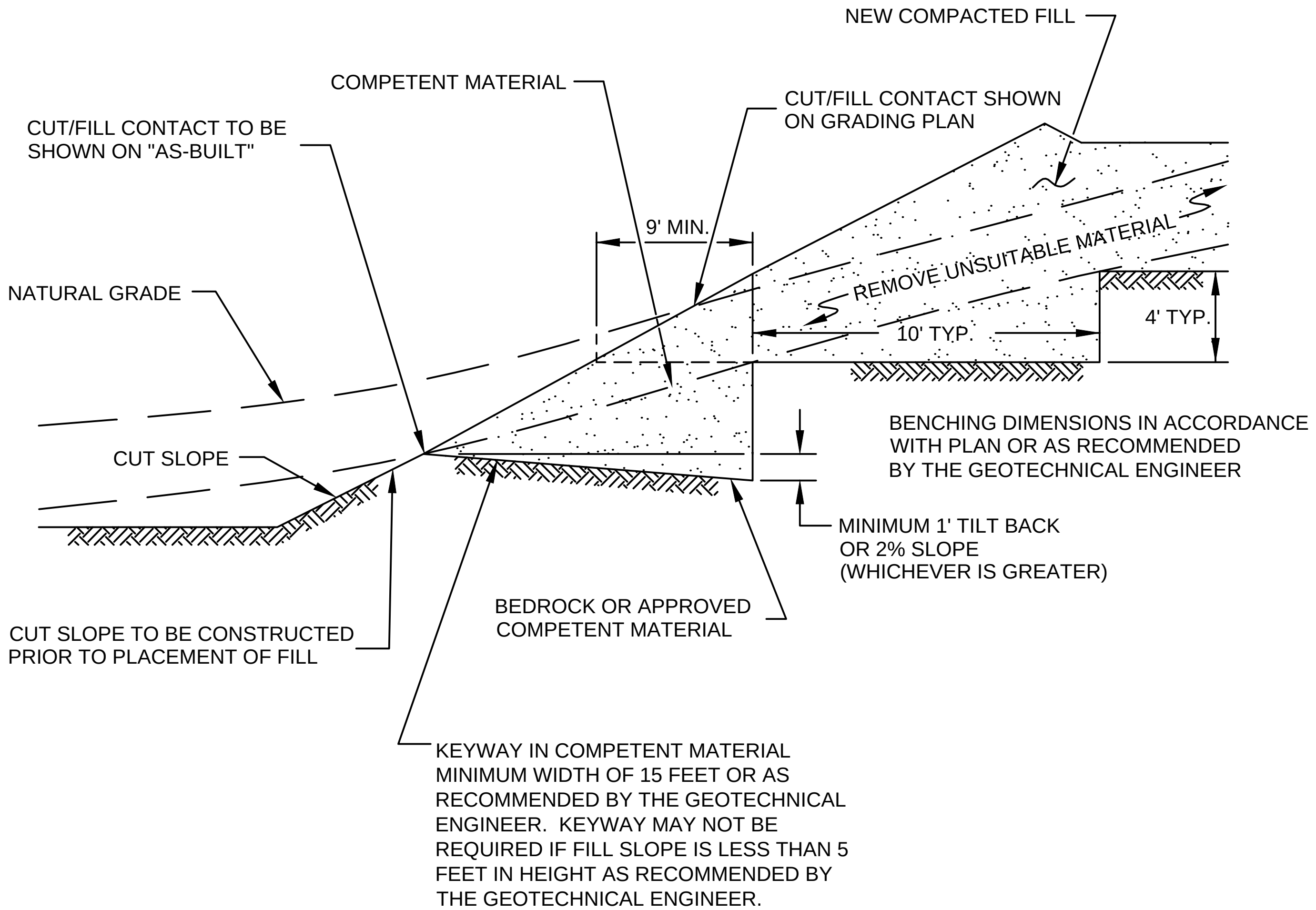
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

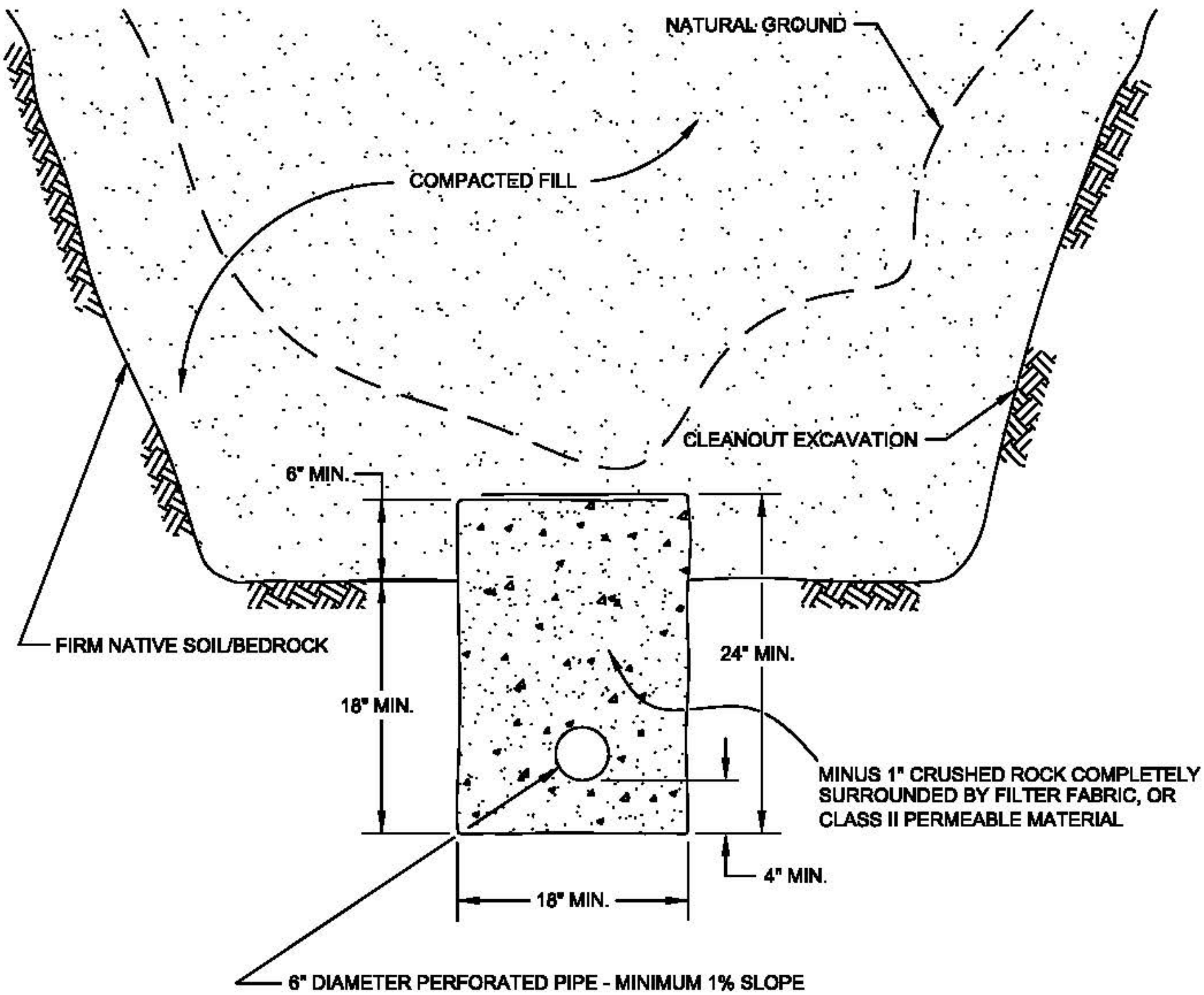
GRADING GUIDE SPECIFICATIONS

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

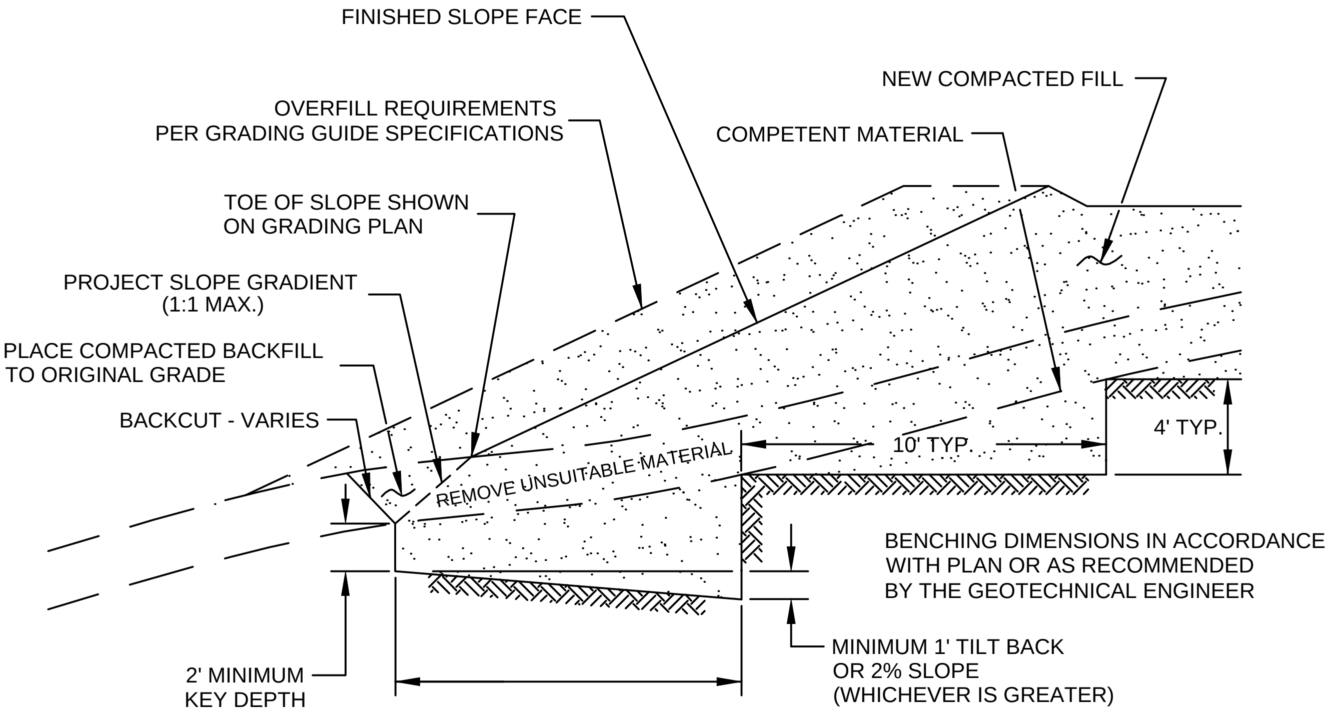
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



KEYWAY IN COMPETENT MATERIAL. MINIMUM WIDTH OF 15 FEET OR AS RECOMMENDED BY THE GEOTECHNIAL ENGINEER. KEYWAY MAY NOT BE REQUIRED IF FILL SLOPE IS LESS THAN 5' IN HEIGHT AS RECOMMENDED BY THE GEOTECHNICAL ENGINEER.

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

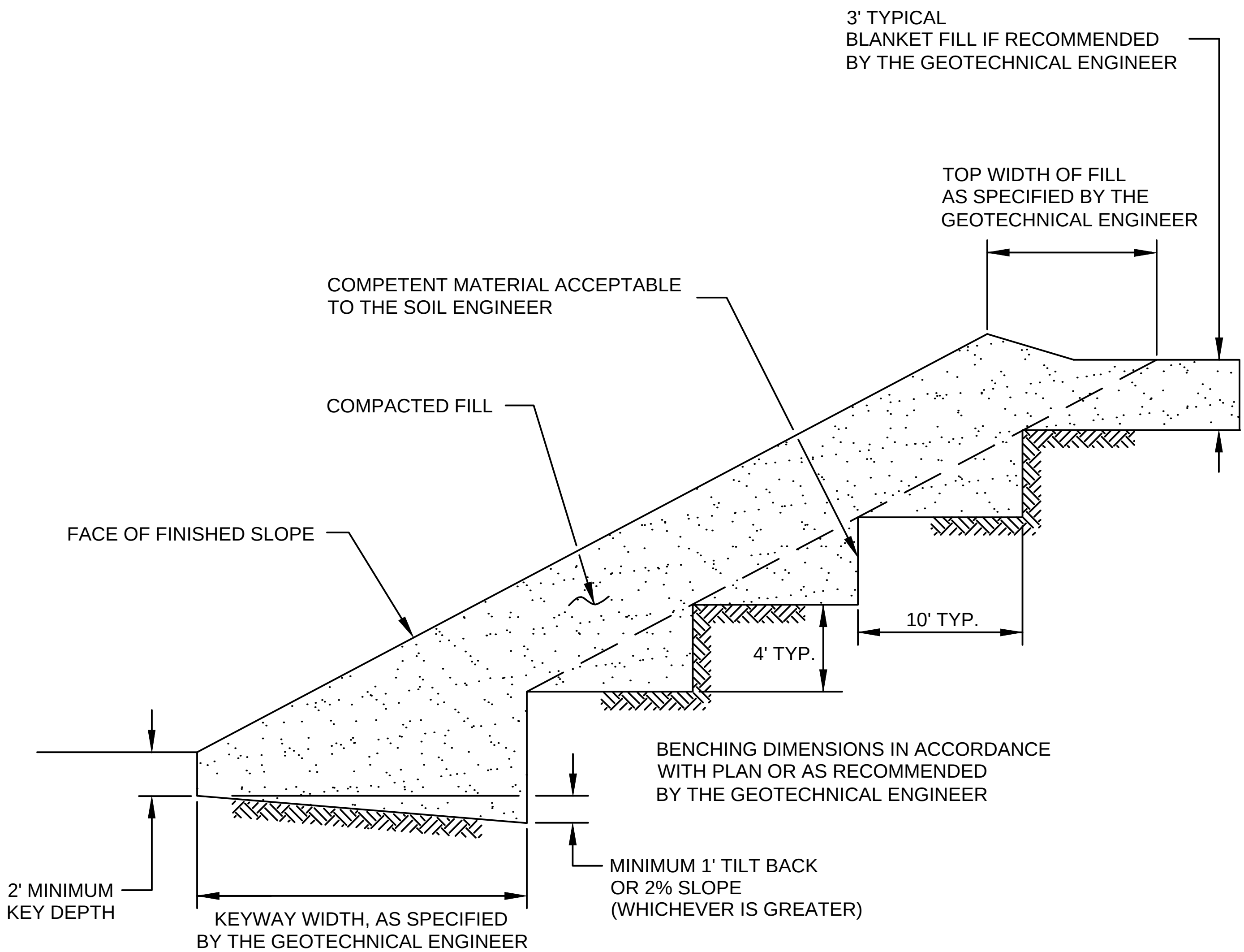
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

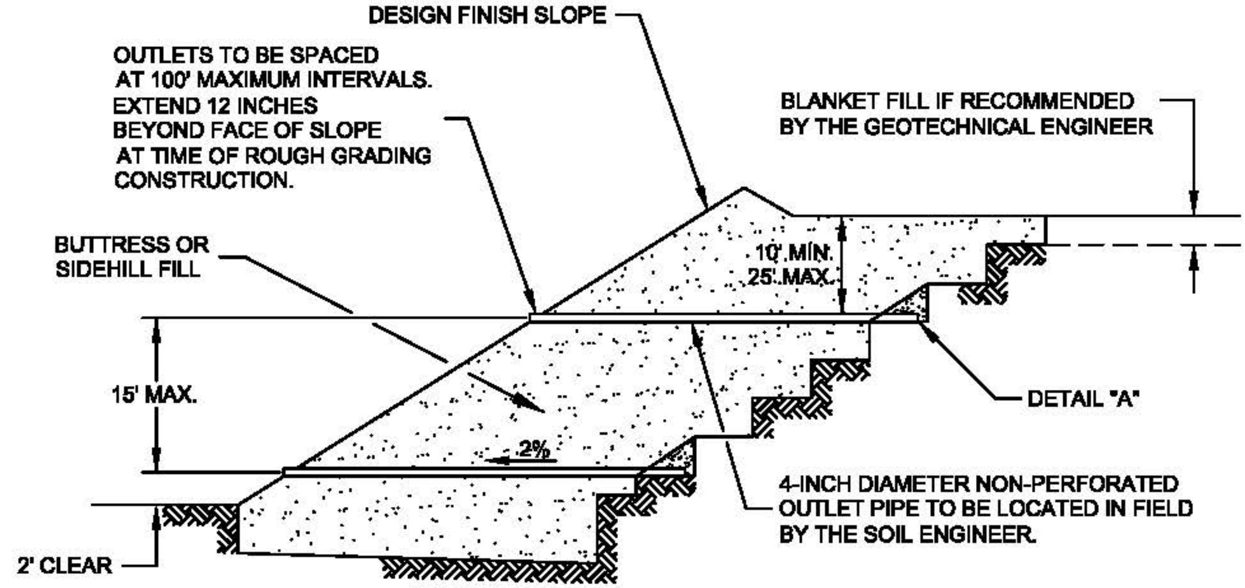
DRAWN: JAS
CHKD: GKM

PLATE D-4

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-5	



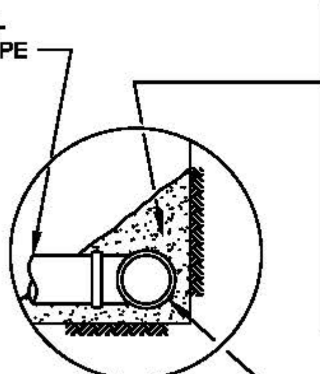
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

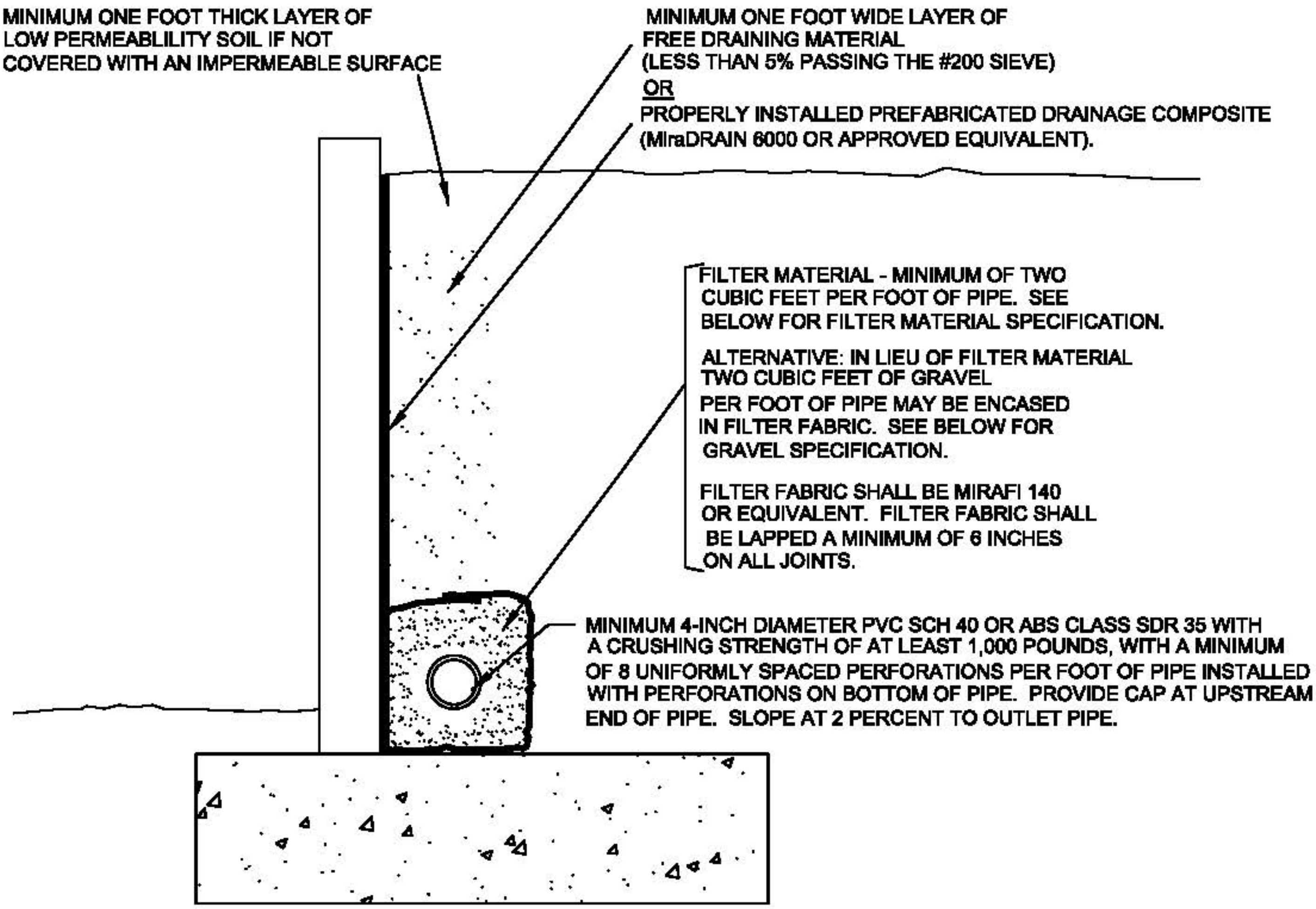
GRADING GUIDE SPECIFICATIONS

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

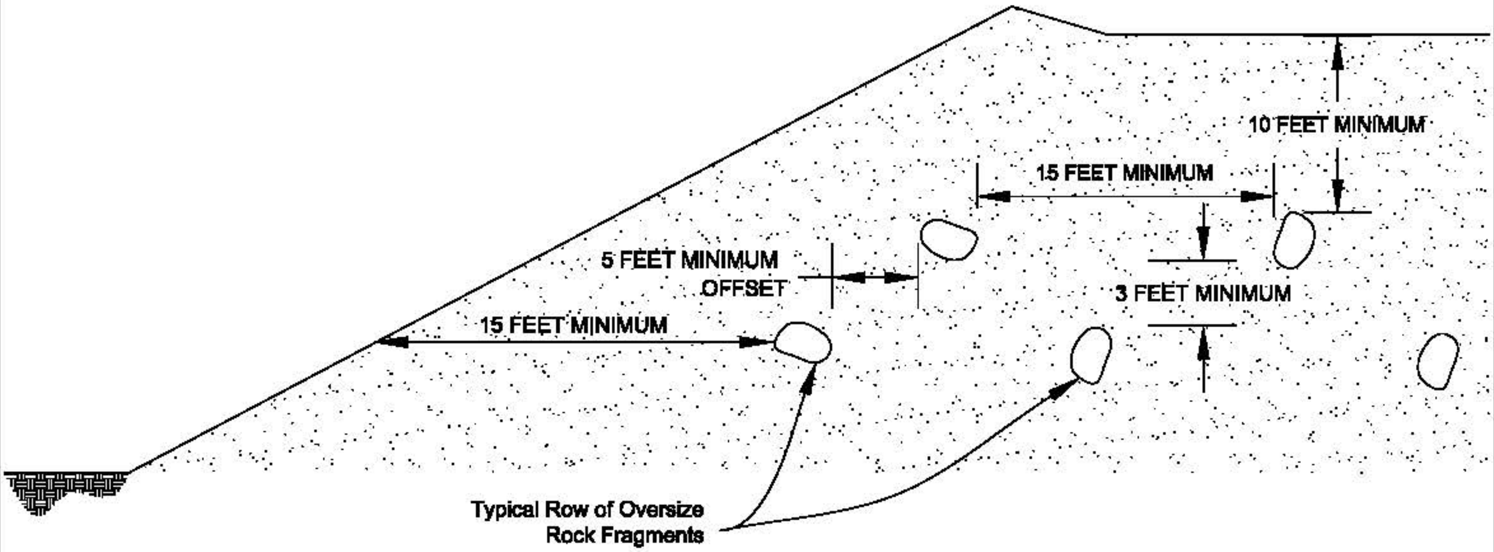
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

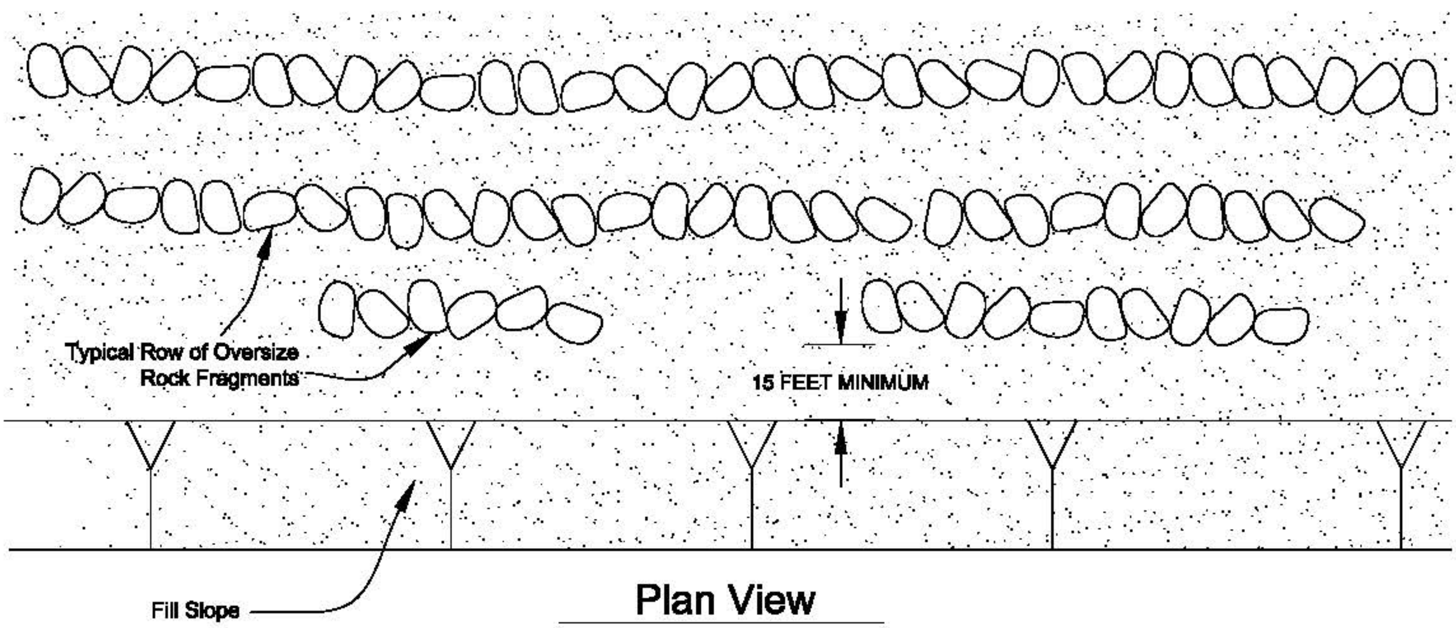
PLATE D-7

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

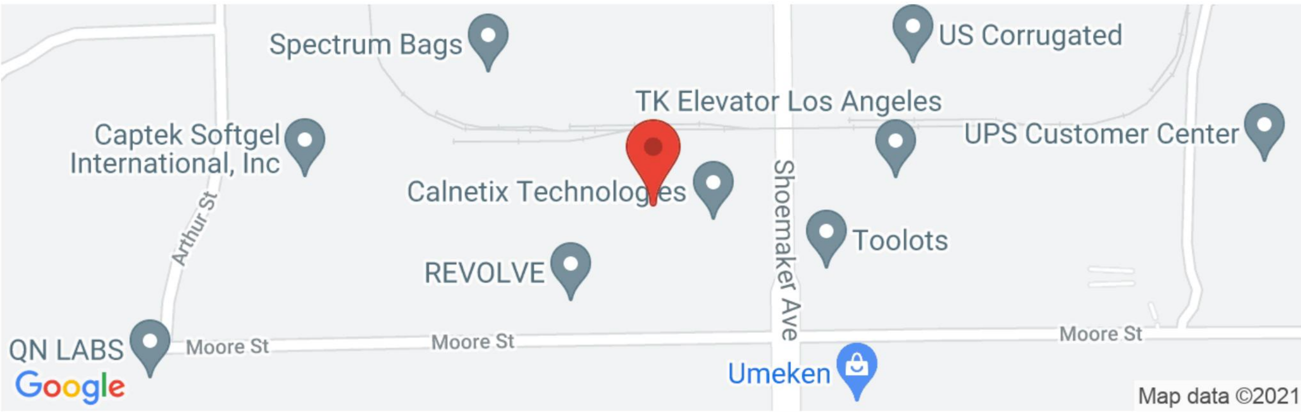
DRAWN: PM
CHKD: GKM

PLATE D-8

SOUTHERN
CALIFORNIA
GEOTECHNICAL

APPENDIX E

Latitude, Longitude: 33.883499, -118.056705



Date	10/25/2021, 1:55:09 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.572	MCE_R ground motion. (for 0.2 second period)
S_1	0.561	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.572	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.048	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	0.672	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.739	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.572	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.729	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.424	Factored deterministic acceleration value. (0.2 second)
S1RT	0.561	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.619	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.813	Factored deterministic acceleration value. (1.0 second)
PGAd	0.978	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.91	Mapped value of the risk coefficient at short periods
C_{R1}	0.906	Mapped value of the risk coefficient at a period of 1 s

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

SEISMIC DESIGN PARAMETERS - 2019 CBC

PROPOSED WAREHOUSE

CERRITOS, CALIFORNIA

DRAWN: OS
CHKD: DWN

SCG PROJECT
21G233-1

PLATE E-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

16323 Shoemaker Avenue
Cerritos, CA
September 29, 2021

Prepared for:

Mr. Daryl Kas
Southern California Geotechnical, Inc.
22885 E. Savi Ranch Parkway, Ste E
Yorba Linda, CA 92887
Office (714) 685-1115 / Fax (714) 685-1118

Prepared by:

KTE

KEHOE TESTING & ENGINEERING
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Huntington Beach, CA 92649-1518
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www.kehoetesting.com

TABLE OF CONTENTS

SUMMARY

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at 16323 Shoemaker Avenue in Cerritos, California. The work was performed by Kehoe Testing & Engineering (KTE) on September 29, 2021. 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	50	
CPT-3	50	
CPT-4	50	

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

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

10/06/21-wt-3400

APPENDIX

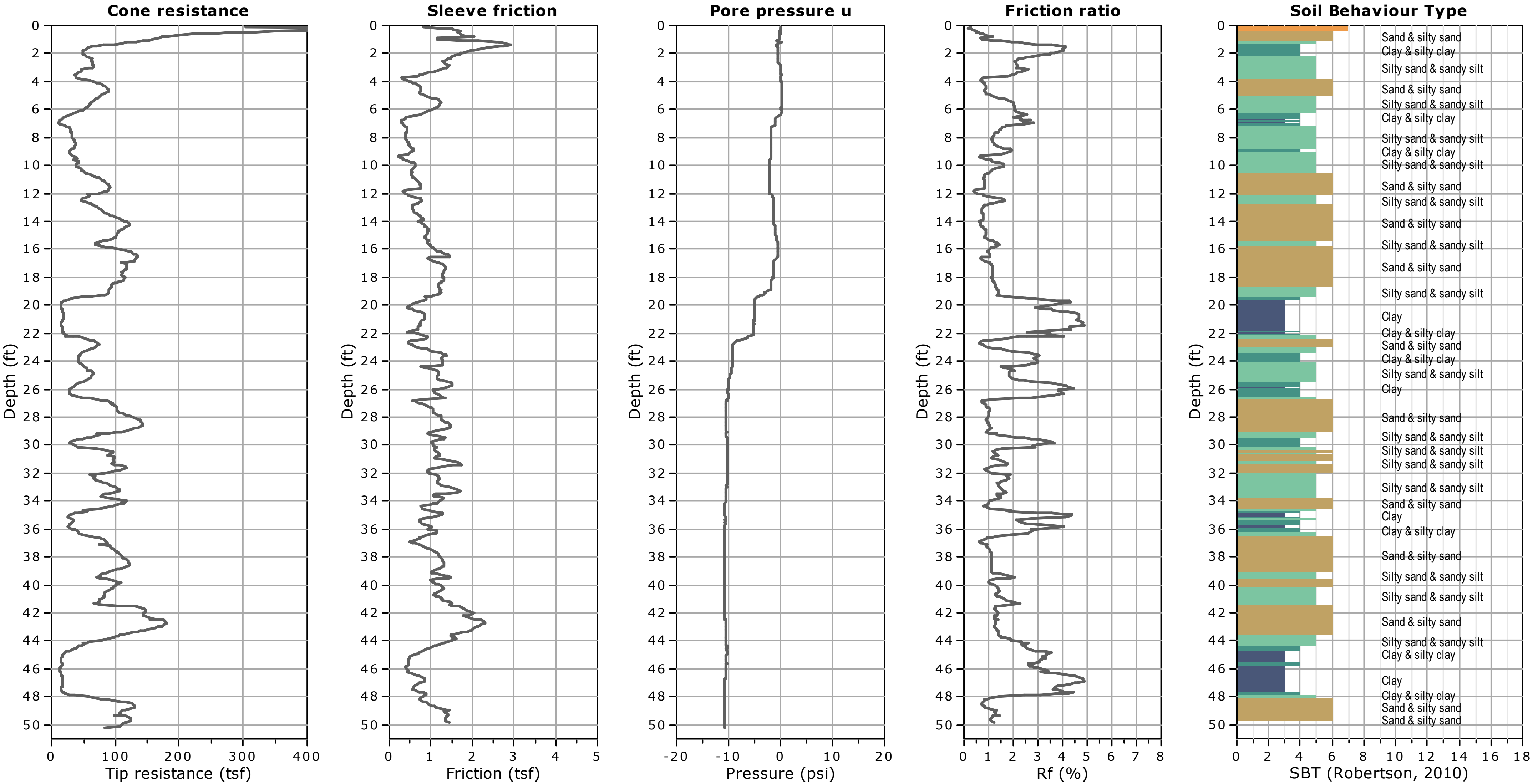
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Southern California Geotechnical

Location: 16323 Shoemaker Ave, Cerritos, CA

CPT-1

Total depth: 50.20 ft, Date: 9/29/2021

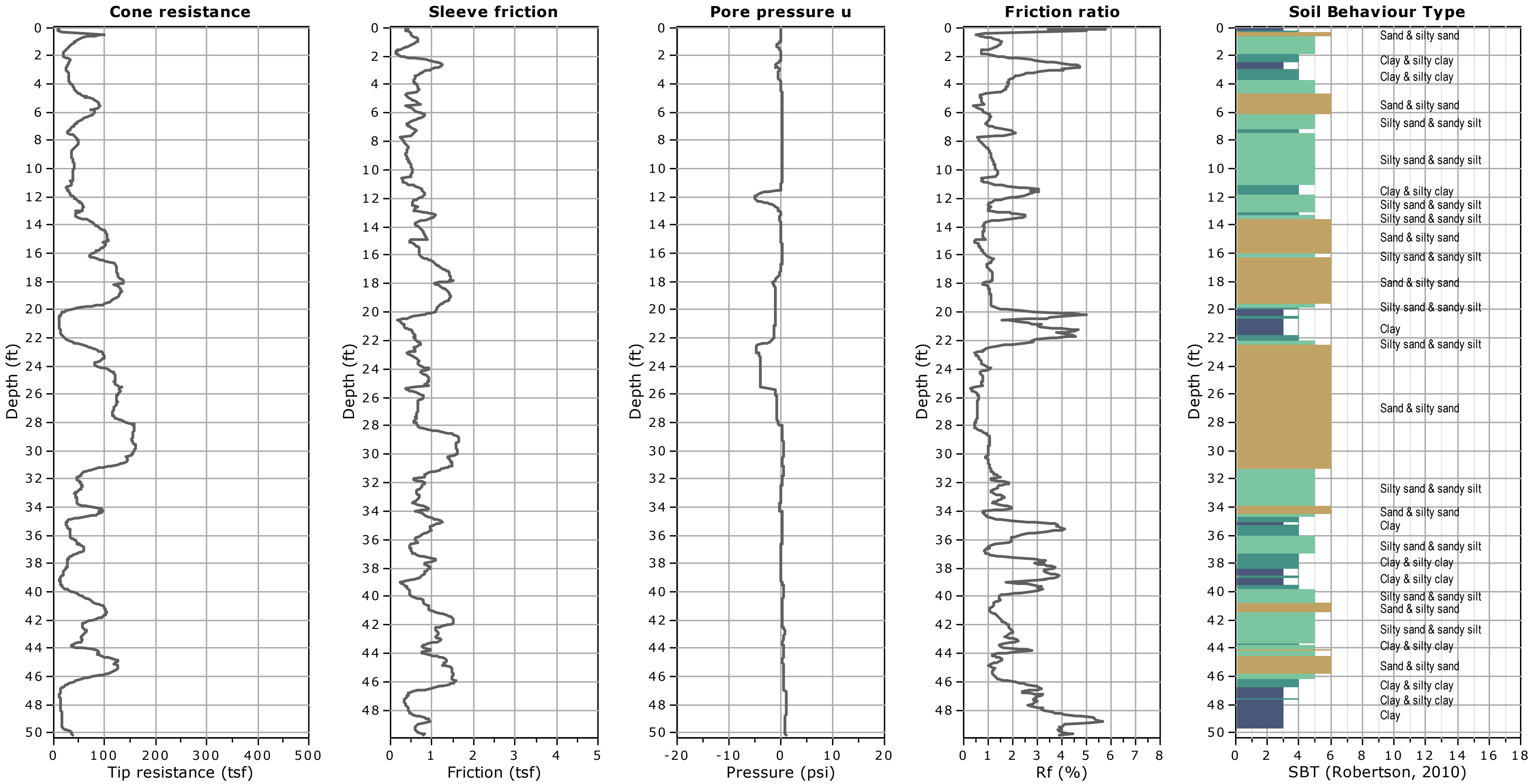


Project file: C:\CPT Project Data\SoCalGeotech-Cerritos9-21\CPT Report\CPeT.cpt

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

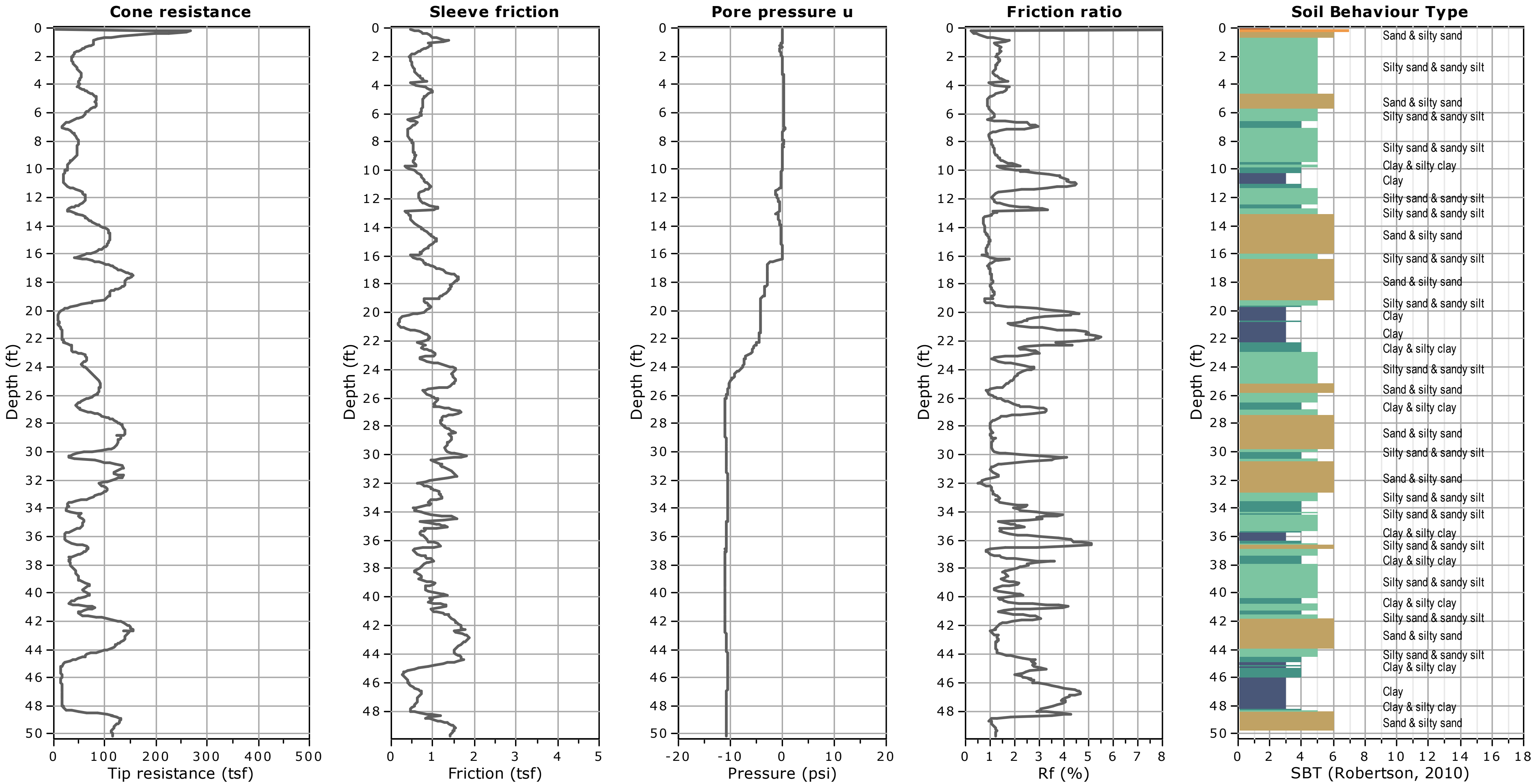
Project: **Southern California Geotechnical** **CPT-2**

Location: **16323 Shoemaker Ave, Cerritos, CA** Total depth: 50.17 ft, Date: 9/29/2021



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

Project: **Southern California Geotechnical** | **CPT-3**
Location: **16323 Shoemaker Ave, Cerritos, CA** | Total depth: 50.15 ft, Date: 9/29/2021



Kehoe Testing and Engineering
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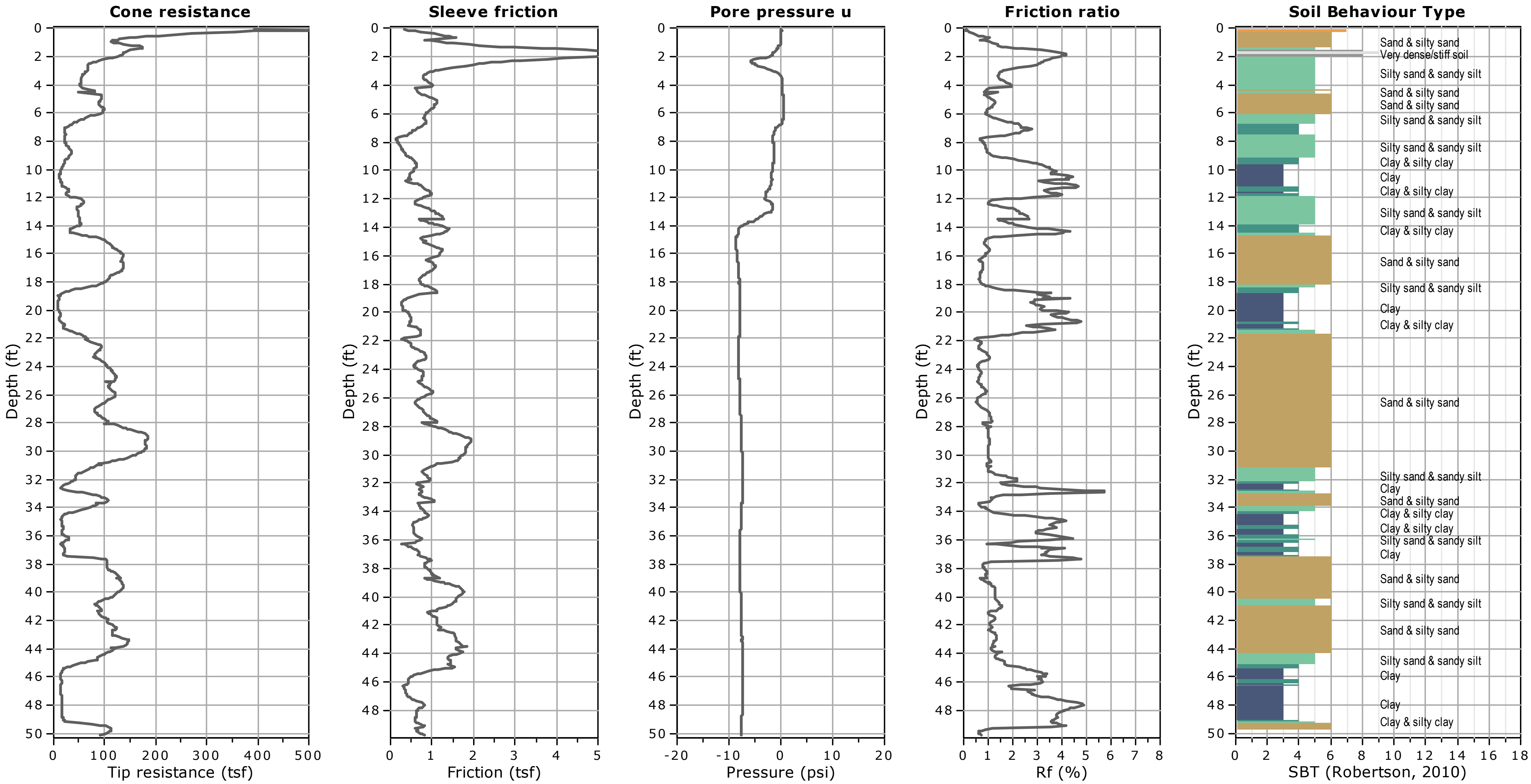
www.kehoetesting.com

Project: Southern California Geotechnical
Location: 16323 Shoemaker Ave, Cerritos, CA

CPT-4
Total depth: 50.14 ft, Date: 9/29/2021



Kehoe Testing & Engineering
714-901-7270
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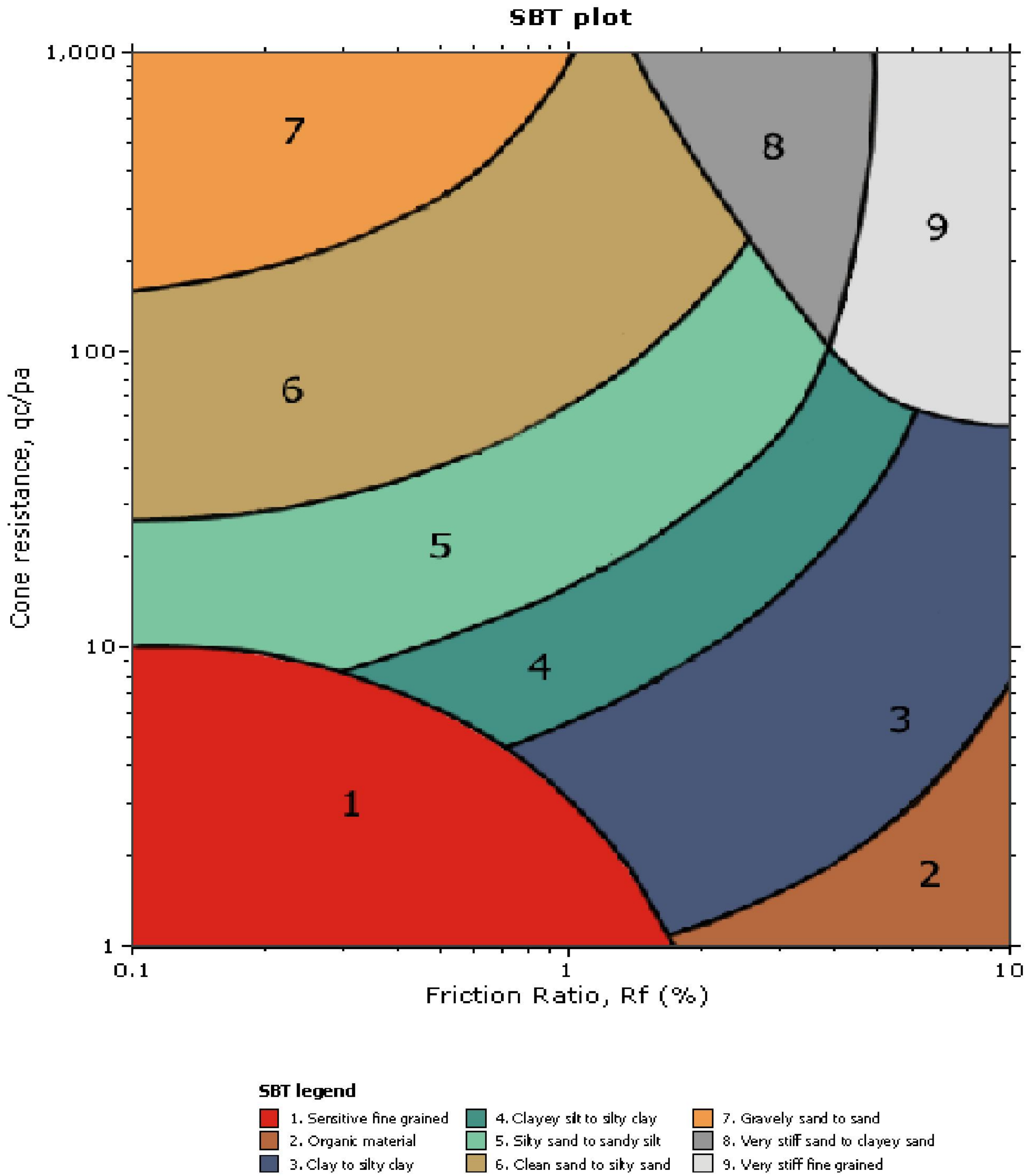
www.kehoetesting.com



APPENDIX G

T A B L E   O F   C O N T E N T S

Southern California Geotechnical
info@socalgeo.com
www.socalgeo.com

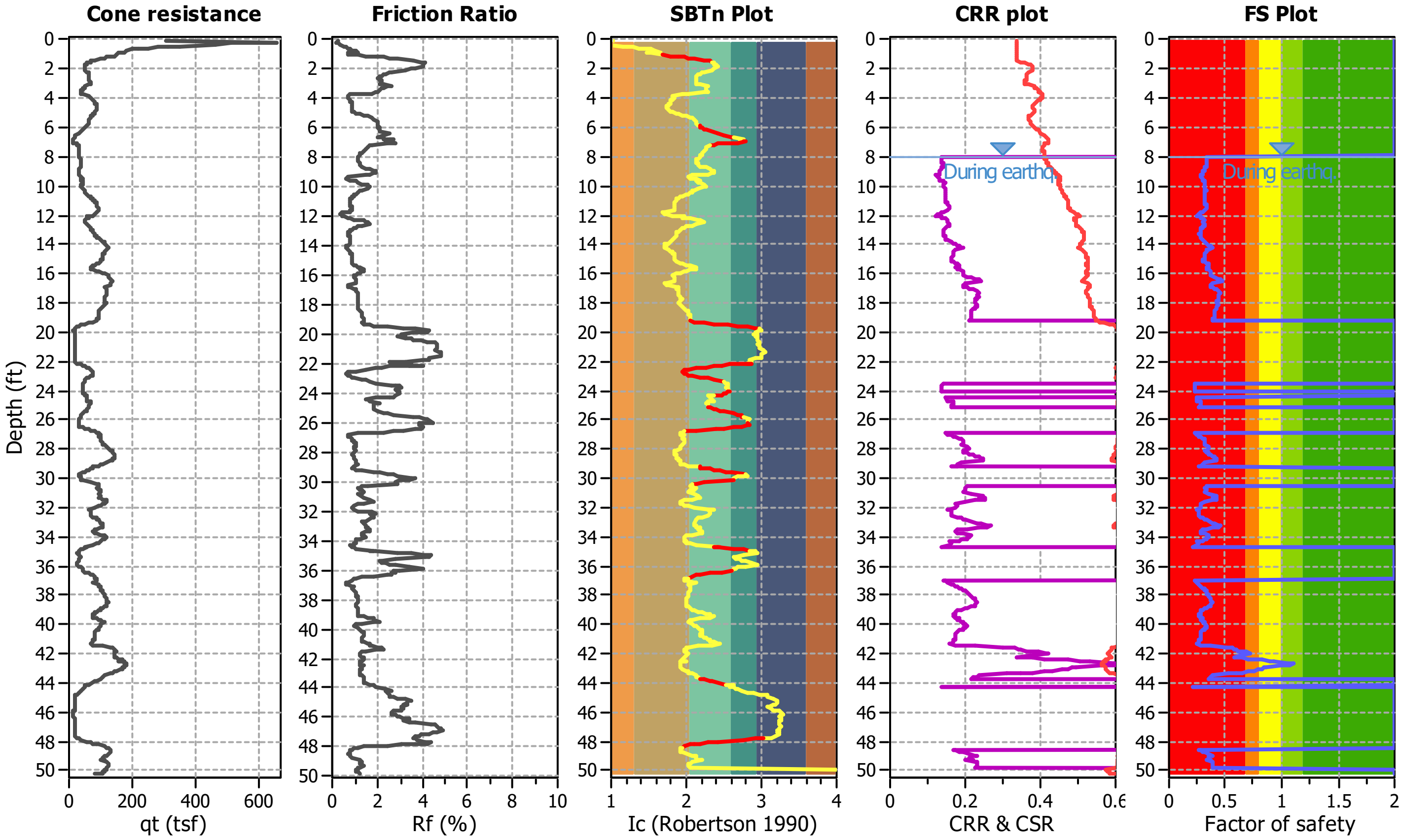
LIQUEFACTION ANALYSIS REPORT

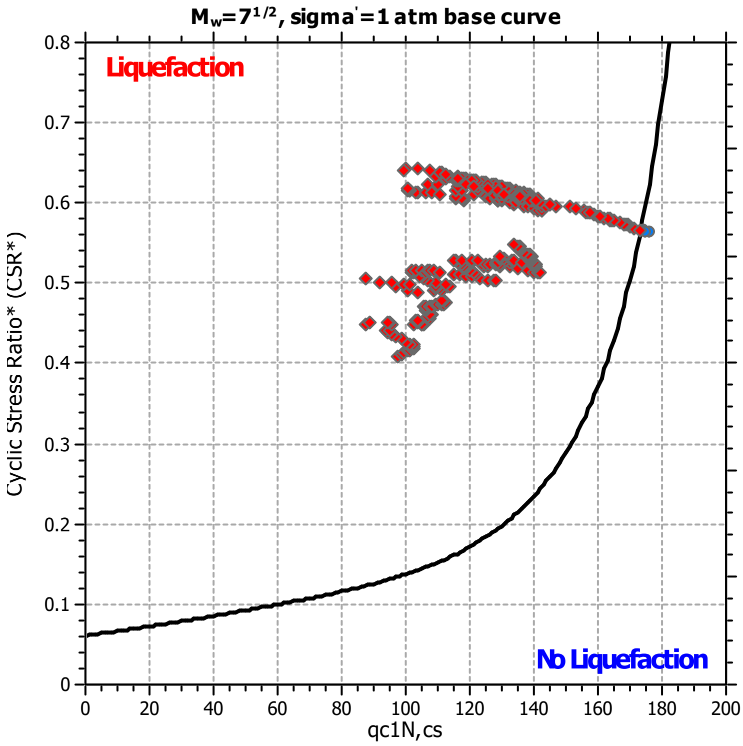
Project title : Proposed Warehouse **Location : Cerritos, CA**

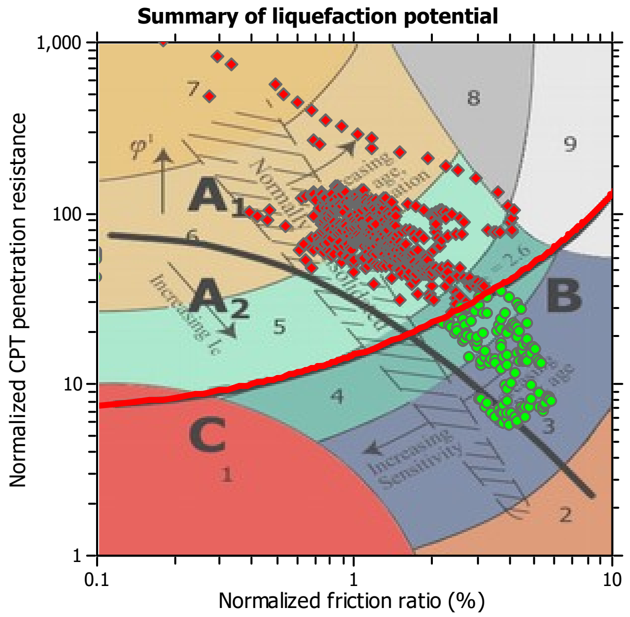
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.81	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

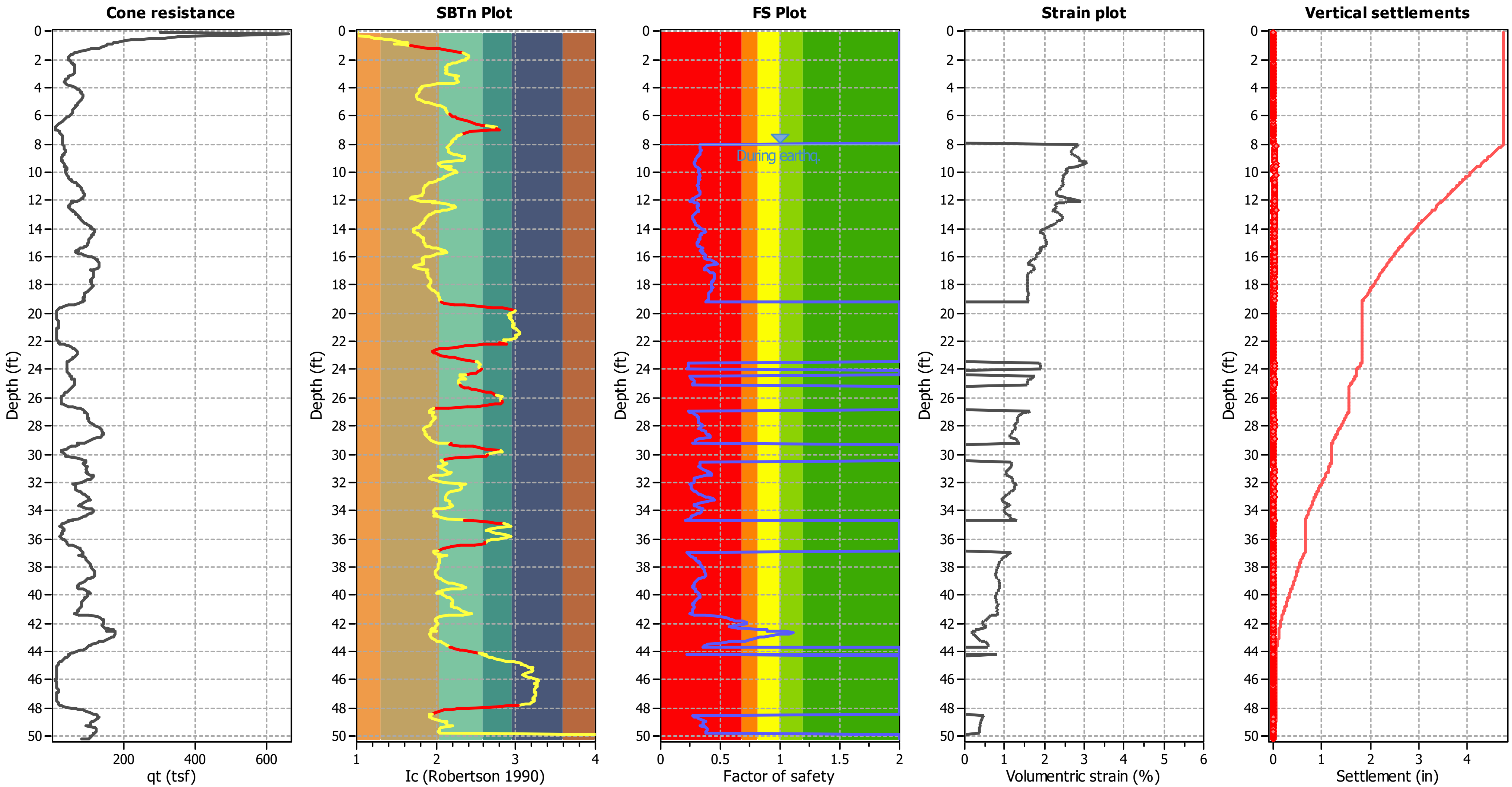






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and 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)
8.01	97.52	0.33	2.85	0.86	0.02	8.14	98.79	0.33	2.80	0.86	0.05
8.22	99.68	0.33	2.77	0.86	0.03	8.28	100.46	0.33	2.75	0.86	0.02
8.36	101.37	0.33	2.72	0.86	0.03	8.43	101.99	0.34	2.70	0.86	0.02
8.49	102.49	0.34	2.68	0.86	0.02	8.55	102.55	0.34	2.68	0.86	0.02
8.63	102.20	0.33	2.68	0.85	0.02	8.70	101.54	0.33	2.70	0.85	0.02
8.76	100.28	0.32	2.73	0.85	0.02	8.85	98.84	0.32	2.76	0.85	0.03
8.91	96.91	0.31	2.82	0.85	0.02	8.98	95.39	0.30	2.86	0.85	0.03
9.04	94.36	0.30	2.89	0.85	0.02	9.12	93.70	0.29	2.90	0.85	0.03
9.20	95.31	0.30	2.85	0.84	0.03	9.26	87.69	0.27	3.09	0.84	0.02
9.39	88.88	0.28	3.04	0.84	0.05	9.54	95.32	0.29	2.83	0.84	0.05
9.57	94.98	0.29	2.84	0.84	0.01	9.60	94.62	0.29	2.85	0.84	0.01
9.70	102.61	0.31	2.61	0.84	0.03	9.77	104.86	0.32	2.55	0.83	0.02
9.82	105.61	0.32	2.53	0.83	0.01	9.87	104.85	0.32	2.55	0.83	0.01
9.92	103.76	0.32	2.57	0.83	0.02	9.99	103.33	0.31	2.58	0.83	0.02
10.04	104.02	0.31	2.56	0.83	0.02	10.20	106.98	0.32	2.48	0.83	0.05
10.27	107.68	0.33	2.46	0.83	0.02	10.35	106.67	0.32	2.48	0.82	0.02
10.39	106.83	0.32	2.47	0.82	0.01	10.48	107.98	0.32	2.44	0.82	0.03
10.53	107.83	0.32	2.44	0.82	0.01	10.57	107.58	0.32	2.45	0.82	0.01
10.70	106.47	0.31	2.47	0.82	0.04	10.76	106.49	0.31	2.46	0.82	0.02
10.84	106.00	0.31	2.47	0.82	0.02	10.90	105.84	0.31	2.47	0.82	0.02
10.97	106.45	0.31	2.45	0.81	0.02	11.04	107.61	0.31	2.42	0.81	0.02
11.10	108.38	0.32	2.40	0.81	0.02	11.19	109.57	0.32	2.37	0.81	0.02
11.24	110.17	0.32	2.35	0.81	0.02	11.32	111.23	0.33	2.33	0.81	0.02
11.38	111.98	0.33	2.31	0.81	0.02	11.46	112.44	0.33	2.29	0.81	0.02
11.52	112.56	0.33	2.29	0.80	0.02	11.59	112.17	0.33	2.29	0.80	0.02
11.67	111.52	0.32	2.30	0.80	0.02	11.73	103.92	0.29	2.47	0.80	0.02
11.81	100.92	0.28	2.55	0.80	0.02	11.89	96.88	0.27	2.65	0.80	0.03
11.95	91.75	0.25	2.79	0.80	0.02	12.04	87.21	0.24	2.93	0.80	0.03
12.06	95.53	0.26	2.68	0.80	0.01	12.09	99.21	0.27	2.58	0.80	0.01
12.14	101.45	0.28	2.51	0.79	0.02	12.21	108.57	0.30	2.34	0.79	0.02
12.31	111.07	0.31	2.28	0.79	0.03	12.35	110.79	0.31	2.28	0.79	0.01
12.41	109.50	0.31	2.31	0.79	0.02	12.48	109.50	0.31	2.31	0.79	0.02
12.65	113.23	0.32	2.22	0.79	0.05	12.70	113.65	0.32	2.21	0.78	0.01
12.75	112.37	0.31	2.23	0.78	0.01	12.80	109.49	0.30	2.29	0.78	0.02
12.85	107.04	0.29	2.34	0.78	0.01	12.91	105.64	0.29	2.37	0.78	0.02
12.96	104.80	0.28	2.39	0.78	0.02	13.01	104.43	0.28	2.40	0.78	0.02
13.10	102.32	0.27	2.44	0.78	0.03	13.16	101.96	0.27	2.45	0.78	0.02
13.21	101.95	0.27	2.44	0.78	0.02	13.29	101.99	0.27	2.44	0.77	0.02
13.37	103.38	0.28	2.40	0.77	0.02	13.41	104.94	0.28	2.36	0.77	0.01
13.46	106.27	0.28	2.33	0.77	0.01	13.54	106.72	0.29	2.32	0.77	0.02
13.60	107.47	0.29	2.30	0.77	0.02	13.65	108.52	0.29	2.27	0.77	0.01
13.72	110.44	0.30	2.23	0.77	0.02	13.81	114.82	0.31	2.13	0.77	0.02
13.88	117.00	0.32	2.09	0.76	0.02	13.93	118.63	0.33	2.05	0.76	0.01
13.98	120.86	0.34	2.01	0.76	0.01	14.04	123.16	0.35	1.97	0.76	0.01
14.12	125.75	0.37	1.92	0.76	0.02	14.19	127.43	0.38	1.89	0.76	0.02
14.25	128.20	0.38	1.87	0.76	0.01	14.36	121.79	0.34	1.98	0.76	0.02
14.42	121.29	0.34	1.98	0.76	0.01	14.47	119.73	0.33	2.01	0.75	0.01
14.55	118.58	0.33	2.03	0.75	0.02	14.61	118.46	0.32	2.03	0.75	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)
14.68	118.40	0.32	2.02	0.75	0.02	14.73	118.50	0.32	2.02	0.75	0.01
14.78	118.26	0.32	2.02	0.75	0.01	14.86	117.33	0.32	2.03	0.75	0.02
14.93	116.68	0.31	2.04	0.75	0.02	14.99	116.00	0.31	2.05	0.75	0.02
15.06	115.39	0.31	2.06	0.74	0.02	15.13	115.22	0.31	2.06	0.74	0.02
15.22	117.42	0.31	2.02	0.74	0.02	15.27	120.54	0.33	1.96	0.74	0.01
15.35	124.07	0.35	1.90	0.74	0.02	15.40	125.51	0.35	1.87	0.74	0.01
15.48	123.73	0.34	1.90	0.74	0.02	15.54	120.85	0.33	1.94	0.74	0.01
15.62	124.76	0.35	1.87	0.74	0.02	15.70	122.79	0.34	1.90	0.73	0.02
15.75	126.22	0.36	1.84	0.73	0.01	15.83	128.41	0.37	1.81	0.73	0.02
15.89	129.60	0.38	1.79	0.73	0.01	15.97	129.38	0.37	1.78	0.73	0.02
16.04	128.26	0.37	1.80	0.73	0.01	16.11	128.62	0.37	1.79	0.73	0.02
16.18	132.54	0.39	1.73	0.73	0.02	16.24	134.78	0.41	1.70	0.72	0.01
16.32	138.08	0.44	1.65	0.72	0.02	16.38	140.14	0.46	1.62	0.72	0.01
16.45	141.10	0.47	1.60	0.72	0.02	16.51	141.97	0.48	1.59	0.72	0.01
16.59	132.46	0.39	1.71	0.72	0.02	16.66	131.25	0.38	1.73	0.72	0.02
16.73	130.50	0.38	1.74	0.72	0.01	16.81	129.49	0.37	1.75	0.72	0.02
16.91	129.46	0.37	1.74	0.71	0.02	16.95	133.09	0.39	1.69	0.71	0.01
17.02	134.03	0.40	1.68	0.71	0.02	17.08	136.42	0.42	1.64	0.71	0.01
17.17	139.14	0.44	1.60	0.71	0.02	17.26	140.67	0.46	1.58	0.71	0.02
17.30	140.63	0.45	1.58	0.71	0.01	17.34	140.39	0.45	1.58	0.71	0.01
17.42	139.91	0.45	1.58	0.70	0.01	17.47	139.88	0.45	1.58	0.70	0.01
17.54	139.94	0.45	1.58	0.70	0.01	17.62	139.73	0.44	1.58	0.70	0.01
17.66	139.51	0.44	1.58	0.70	0.01	17.75	138.60	0.43	1.59	0.70	0.02
17.79	139.00	0.44	1.58	0.70	0.01	17.87	138.65	0.43	1.58	0.70	0.02
17.91	138.35	0.43	1.58	0.70	0.01	18.01	138.10	0.43	1.58	0.69	0.02
18.05	138.24	0.43	1.58	0.69	0.01	18.14	138.78	0.43	1.57	0.69	0.02
18.19	139.11	0.43	1.56	0.69	0.01	18.27	138.75	0.43	1.56	0.69	0.02
18.33	138.47	0.43	1.57	0.69	0.01	18.38	137.98	0.42	1.57	0.69	0.01
18.45	137.03	0.41	1.58	0.69	0.01	18.54	135.34	0.40	1.60	0.69	0.02
18.59	135.31	0.40	1.60	0.68	0.01	18.66	135.27	0.40	1.59	0.68	0.01
18.80	135.54	0.40	1.58	0.68	0.03	18.86	135.64	0.40	1.58	0.68	0.01
18.91	135.57	0.40	1.58	0.68	0.01	18.97	135.45	0.40	1.58	0.68	0.01
19.07	135.82	0.40	1.57	0.68	0.02	19.13	135.22	0.39	1.58	0.68	0.01
19.17	133.83	0.38	1.59	0.68	0.01	19.24	129.66	2.00	0.00	0.67	0.00
19.30	126.34	2.00	0.00	0.67	0.00	19.36	117.24	2.00	0.00	0.67	0.00
19.43	103.63	2.00	0.00	0.67	0.00	19.51	93.66	2.00	0.00	0.67	0.00
19.56	26.65	2.00	0.00	0.67	0.00	19.64	21.34	2.00	0.00	0.67	0.00
19.69	17.75	2.00	0.00	0.67	0.00	19.79	15.40	2.00	0.00	0.66	0.00
19.82	14.68	2.00	0.00	0.66	0.00	19.92	13.77	2.00	0.00	0.66	0.00
20.04	14.28	2.00	0.00	0.66	0.00	20.09	14.45	2.00	0.00	0.66	0.00
20.14	14.08	2.00	0.00	0.66	0.00	20.16	14.53	2.00	0.00	0.66	0.00
20.22	14.34	2.00	0.00	0.66	0.00	20.30	15.38	2.00	0.00	0.66	0.00
20.35	15.99	2.00	0.00	0.66	0.00	20.45	16.42	2.00	0.00	0.65	0.00
20.49	16.67	2.00	0.00	0.65	0.00	20.58	17.36	2.00	0.00	0.65	0.00
20.67	17.43	2.00	0.00	0.65	0.00	20.72	17.33	2.00	0.00	0.65	0.00
20.76	17.32	2.00	0.00	0.65	0.00	20.81	17.31	2.00	0.00	0.65	0.00
20.89	17.72	2.00	0.00	0.65	0.00	20.94	17.54	2.00	0.00	0.65	0.00
21.04	16.89	2.00	0.00	0.64	0.00	21.08	16.27	2.00	0.00	0.64	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)
21.18	15.37	2.00	0.00	0.64	0.00	21.24	14.83	2.00	0.00	0.64	0.00
21.36	14.45	2.00	0.00	0.64	0.00	21.42	14.35	2.00	0.00	0.64	0.00
21.56	15.03	2.00	0.00	0.63	0.00	21.70	15.00	2.00	0.00	0.63	0.00
21.78	15.07	2.00	0.00	0.63	0.00	21.83	15.41	2.00	0.00	0.63	0.00
21.91	15.91	2.00	0.00	0.63	0.00	21.97	17.04	2.00	0.00	0.63	0.00
22.04	18.76	2.00	0.00	0.63	0.00	22.11	21.79	2.00	0.00	0.63	0.00
22.18	20.29	2.00	0.00	0.62	0.00	22.18	91.89	2.00	0.00	0.62	0.00
22.25	98.98	2.00	0.00	0.62	0.00	22.33	104.23	2.00	0.00	0.62	0.00
22.40	108.09	2.00	0.00	0.62	0.00	22.47	107.46	2.00	0.00	0.62	0.00
22.55	102.95	2.00	0.00	0.62	0.00	22.59	99.03	2.00	0.00	0.62	0.00
22.71	96.77	2.00	0.00	0.62	0.00	22.80	102.58	2.00	0.00	0.61	0.00
22.84	105.00	2.00	0.00	0.61	0.00	22.94	107.81	2.00	0.00	0.61	0.00
22.98	110.38	2.00	0.00	0.61	0.00	23.08	113.20	2.00	0.00	0.61	0.00
23.13	113.19	2.00	0.00	0.61	0.00	23.18	111.74	2.00	0.00	0.61	0.00
23.28	107.05	2.00	0.00	0.61	0.00	23.42	103.97	2.00	0.00	0.60	0.00
23.46	103.44	2.00	0.00	0.60	0.00	23.51	103.60	0.23	1.86	0.60	0.01
23.58	103.23	0.23	1.87	0.60	0.02	23.64	101.38	0.23	1.90	0.60	0.01
23.78	101.33	0.23	1.89	0.60	0.03	23.83	101.23	0.23	1.89	0.60	0.01
23.90	100.90	0.22	1.89	0.59	0.02	23.95	100.48	0.22	1.90	0.59	0.01
24.00	100.47	0.22	1.90	0.59	0.01	24.06	100.78	2.00	0.00	0.59	0.00
24.17	103.23	2.00	0.00	0.59	0.00	24.22	104.86	2.00	0.00	0.59	0.00
24.27	106.45	2.00	0.00	0.59	0.00	24.33	105.63	2.00	0.00	0.59	0.00
24.39	104.50	2.00	0.00	0.59	0.00	24.43	106.34	0.24	1.77	0.59	0.01
24.48	108.32	0.24	1.73	0.59	0.01	24.67	110.61	0.25	1.68	0.58	0.04
24.70	115.87	0.27	1.60	0.58	0.01	24.75	118.03	0.28	1.57	0.58	0.01
24.83	118.42	0.28	1.56	0.58	0.02	24.87	118.43	0.28	1.56	0.58	0.01
24.97	117.98	0.28	1.56	0.58	0.02	25.00	117.24	0.27	1.57	0.58	0.01
25.10	115.50	0.27	1.59	0.57	0.02	25.14	114.53	2.00	0.00	0.57	0.00
25.27	111.13	2.00	0.00	0.57	0.00	25.33	110.01	2.00	0.00	0.57	0.00
25.37	108.72	2.00	0.00	0.57	0.00	25.42	107.31	2.00	0.00	0.57	0.00
25.58	37.60	2.00	0.00	0.57	0.00	25.63	35.23	2.00	0.00	0.57	0.00
25.71	32.58	2.00	0.00	0.56	0.00	25.76	31.39	2.00	0.00	0.56	0.00
25.81	30.04	2.00	0.00	0.56	0.00	25.95	25.65	2.00	0.00	0.56	0.00
26.02	24.79	2.00	0.00	0.56	0.00	26.07	24.36	2.00	0.00	0.56	0.00
26.13	24.26	2.00	0.00	0.56	0.00	26.18	24.41	2.00	0.00	0.56	0.00
26.25	24.39	2.00	0.00	0.56	0.00	26.33	24.12	2.00	0.00	0.55	0.00
26.38	25.42	2.00	0.00	0.55	0.00	26.46	29.05	2.00	0.00	0.55	0.00
26.51	35.34	2.00	0.00	0.55	0.00	26.60	105.04	2.00	0.00	0.55	0.00
26.66	114.03	2.00	0.00	0.55	0.00	26.73	114.67	2.00	0.00	0.55	0.00
26.79	104.71	2.00	0.00	0.55	0.00	26.87	104.87	2.00	0.00	0.54	0.00
26.94	106.74	0.24	1.63	0.54	0.01	27.00	109.69	0.24	1.58	0.54	0.01
27.05	115.65	0.26	1.50	0.54	0.01	27.10	117.53	0.27	1.47	0.54	0.01
27.18	121.02	0.28	1.42	0.54	0.01	27.27	124.69	0.30	1.37	0.54	0.01
27.32	126.39	0.31	1.35	0.54	0.01	27.40	128.71	0.32	1.32	0.54	0.01
27.45	128.79	0.32	1.32	0.53	0.01	27.49	128.84	0.32	1.31	0.53	0.01
27.59	128.48	0.32	1.31	0.53	0.02	27.66	128.18	0.32	1.31	0.53	0.01
27.75	128.14	0.32	1.31	0.53	0.01	27.87	133.08	0.35	1.25	0.53	0.02
27.94	133.63	0.35	1.24	0.53	0.01	28.07	129.71	0.33	1.28	0.52	0.02

:: 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)
28.15	129.00	0.32	1.28	0.52	0.01	28.21	129.88	0.33	1.27	0.52	0.01
28.28	132.75	0.34	1.24	0.52	0.01	28.34	134.40	0.35	1.22	0.52	0.01
28.42	136.97	0.37	1.19	0.52	0.01	28.48	138.42	0.38	1.17	0.52	0.01
28.56	139.99	0.40	1.16	0.52	0.01	28.62	141.25	0.41	1.14	0.51	0.01
28.69	142.29	0.42	1.13	0.51	0.01	28.77	142.44	0.42	1.13	0.51	0.01
28.82	142.56	0.42	1.12	0.51	0.01	28.96	127.50	0.31	1.27	0.51	0.02
29.04	123.46	0.29	1.31	0.51	0.01	29.11	122.92	0.29	1.31	0.51	0.01
29.18	122.62	0.29	1.31	0.51	0.01	29.21	116.53	0.26	1.38	0.51	0.00
29.28	120.02	2.00	0.00	0.50	0.00	29.38	117.26	2.00	0.00	0.50	0.00
29.42	113.92	2.00	0.00	0.50	0.00	29.47	109.41	2.00	0.00	0.50	0.00
29.54	103.80	2.00	0.00	0.50	0.00	29.60	36.21	2.00	0.00	0.50	0.00
29.75	24.52	2.00	0.00	0.50	0.00	29.82	23.70	2.00	0.00	0.49	0.00
29.86	25.52	2.00	0.00	0.49	0.00	29.91	29.65	2.00	0.00	0.49	0.00
30.03	32.33	2.00	0.00	0.49	0.00	30.09	31.83	2.00	0.00	0.49	0.00
30.15	33.82	2.00	0.00	0.49	0.00	30.29	117.33	2.00	0.00	0.49	0.00
30.35	124.81	2.00	0.00	0.49	0.00	30.44	128.52	2.00	0.00	0.48	0.00
30.49	130.44	2.00	0.00	0.48	0.00	30.57	131.79	0.33	1.16	0.48	0.01
30.63	131.01	0.33	1.16	0.48	0.01	30.70	130.03	0.32	1.17	0.48	0.01
30.76	130.57	0.33	1.16	0.48	0.01	30.84	130.64	0.33	1.16	0.48	0.01
30.89	129.64	0.32	1.16	0.48	0.01	31.03	134.78	0.35	1.11	0.47	0.02
31.24	142.62	0.41	1.03	0.47	0.03	31.32	140.42	0.39	1.05	0.47	0.01
31.38	143.47	0.42	1.02	0.47	0.01	31.46	143.99	0.43	1.01	0.47	0.01
31.52	138.51	0.38	1.06	0.47	0.01	31.59	130.26	0.32	1.13	0.46	0.01
31.73	120.48	0.28	1.22	0.46	0.02	31.80	121.59	0.28	1.21	0.46	0.01
31.86	122.29	0.28	1.20	0.46	0.01	31.94	122.27	0.28	1.19	0.46	0.01
32.00	120.72	0.28	1.21	0.46	0.01	32.10	112.63	0.25	1.29	0.46	0.02
32.11	109.53	0.24	1.33	0.46	0.00	32.20	116.66	0.26	1.24	0.45	0.01
32.24	116.16	0.26	1.25	0.45	0.01	32.33	115.51	0.26	1.25	0.45	0.01
32.37	115.41	0.26	1.25	0.45	0.01	32.43	116.09	0.26	1.24	0.45	0.01
32.49	116.91	0.26	1.23	0.45	0.01	32.64	122.14	0.28	1.16	0.45	0.02
32.68	124.41	0.29	1.14	0.45	0.01	32.73	126.53	0.30	1.12	0.45	0.01
32.81	129.15	0.32	1.09	0.44	0.01	32.86	131.21	0.33	1.07	0.44	0.01
33.03	138.81	0.38	1.00	0.44	0.02	33.09	141.59	0.40	0.97	0.44	0.01
33.17	144.95	0.43	0.94	0.44	0.01	33.24	146.58	0.45	0.93	0.44	0.01
33.30	145.18	0.44	0.94	0.44	0.01	33.36	140.74	0.39	0.97	0.43	0.01
33.43	131.89	0.33	1.04	0.43	0.01	33.52	123.74	0.29	1.11	0.43	0.01
33.57	120.79	0.28	1.14	0.43	0.01	33.65	120.41	0.28	1.14	0.43	0.01
33.71	123.48	0.29	1.10	0.43	0.01	33.79	129.84	0.32	1.04	0.43	0.01
33.86	135.42	0.35	0.99	0.43	0.01	33.92	134.97	0.35	0.99	0.43	0.01
34.01	133.77	0.34	1.00	0.42	0.01	34.06	133.01	0.34	1.00	0.42	0.01
34.14	132.20	0.33	1.01	0.42	0.01	34.20	127.99	0.31	1.04	0.42	0.01
34.28	124.84	0.29	1.07	0.42	0.01	34.35	114.01	0.25	1.17	0.42	0.01
34.41	115.73	0.26	1.15	0.42	0.01	34.48	115.68	0.26	1.15	0.42	0.01
34.55	113.16	0.25	1.17	0.41	0.01	34.65	99.93	0.21	1.33	0.41	0.02
34.67	106.40	2.00	0.00	0.41	0.00	34.71	104.75	2.00	0.00	0.41	0.00
34.81	34.32	2.00	0.00	0.41	0.00	34.87	29.17	2.00	0.00	0.41	0.00
34.94	25.35	2.00	0.00	0.41	0.00	34.99	23.07	2.00	0.00	0.41	0.00
35.07	21.61	2.00	0.00	0.41	0.00	35.13	20.16	2.00	0.00	0.40	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)
35.25	23.66	2.00	0.00	0.40	0.00	35.33	27.03	2.00	0.00	0.40	0.00
35.38	27.38	2.00	0.00	0.40	0.00	35.47	26.23	2.00	0.00	0.40	0.00
35.52	25.02	2.00	0.00	0.40	0.00	35.60	23.80	2.00	0.00	0.40	0.00
35.65	22.51	2.00	0.00	0.40	0.00	35.74	21.08	2.00	0.00	0.39	0.00
35.87	20.01	2.00	0.00	0.39	0.00	36.00	24.61	2.00	0.00	0.39	0.00
36.07	28.57	2.00	0.00	0.39	0.00	36.14	32.58	2.00	0.00	0.39	0.00
36.22	34.45	2.00	0.00	0.39	0.00	36.27	34.20	2.00	0.00	0.39	0.00
36.36	34.09	2.00	0.00	0.38	0.00	36.42	98.02	2.00	0.00	0.38	0.00
36.49	105.65	2.00	0.00	0.38	0.00	36.56	109.81	2.00	0.00	0.38	0.00
36.62	110.93	2.00	0.00	0.38	0.00	36.70	111.75	2.00	0.00	0.38	0.00
36.76	111.67	2.00	0.00	0.38	0.00	36.84	107.55	2.00	0.00	0.38	0.00
36.91	99.52	2.00	0.00	0.37	0.00	36.97	103.66	0.22	1.16	0.37	0.01
37.03	107.65	0.23	1.11	0.37	0.01	37.13	111.18	0.24	1.07	0.37	0.01
37.15	110.78	0.24	1.07	0.37	0.00	37.24	116.19	0.26	1.01	0.37	0.01
37.29	118.47	0.27	0.99	0.37	0.01	37.38	121.75	0.28	0.96	0.37	0.01
37.43	123.49	0.29	0.94	0.37	0.01	37.49	125.15	0.29	0.93	0.36	0.01
37.60	127.82	0.31	0.90	0.36	0.01	37.64	128.76	0.31	0.89	0.36	0.00
37.69	129.94	0.32	0.88	0.36	0.01	37.73	131.27	0.33	0.87	0.36	0.00
37.83	132.58	0.33	0.86	0.36	0.01	37.88	133.32	0.34	0.85	0.36	0.01
37.99	134.64	0.35	0.83	0.36	0.01	38.04	135.23	0.35	0.83	0.36	0.00
38.09	135.94	0.36	0.82	0.35	0.00	38.22	137.15	0.36	0.81	0.35	0.01
38.28	137.61	0.37	0.80	0.35	0.01	38.35	138.21	0.37	0.80	0.35	0.01
38.49	139.00	0.38	0.79	0.35	0.01	38.55	139.00	0.38	0.78	0.35	0.01
38.62	138.61	0.37	0.78	0.35	0.01	38.69	137.41	0.37	0.79	0.34	0.01
38.75	135.83	0.35	0.80	0.34	0.01	38.83	132.80	0.34	0.81	0.34	0.01
38.88	130.05	0.32	0.83	0.34	0.01	38.97	126.62	0.30	0.85	0.34	0.01
39.03	123.62	0.29	0.87	0.34	0.01	39.11	121.53	0.28	0.88	0.34	0.01
39.17	121.36	0.28	0.88	0.34	0.01	39.24	120.52	0.27	0.89	0.33	0.01
39.32	118.83	0.27	0.89	0.33	0.01	39.37	121.90	0.28	0.87	0.33	0.01
39.46	117.95	0.26	0.90	0.33	0.01	39.51	121.34	0.28	0.87	0.33	0.01
39.59	124.37	0.29	0.84	0.33	0.01	39.65	123.55	0.29	0.84	0.33	0.01
39.72	125.40	0.30	0.83	0.33	0.01	39.80	127.65	0.31	0.81	0.33	0.01
39.86	129.34	0.32	0.79	0.32	0.01	39.91	129.03	0.31	0.79	0.32	0.01
39.92	129.10	0.31	0.79	0.32	0.00	39.97	130.84	0.32	0.78	0.32	0.01
40.06	132.11	0.33	0.77	0.32	0.01	40.14	131.85	0.33	0.77	0.32	0.01
40.19	131.03	0.32	0.77	0.32	0.00	40.23	129.79	0.32	0.78	0.32	0.00
40.30	128.29	0.31	0.78	0.32	0.01	40.41	125.33	0.30	0.80	0.32	0.01
40.51	122.30	0.28	0.82	0.31	0.01	40.58	121.29	0.28	0.82	0.31	0.01
40.62	120.36	0.27	0.82	0.31	0.00	40.69	120.03	0.27	0.82	0.31	0.01
40.76	120.63	0.28	0.82	0.31	0.01	40.80	121.20	0.28	0.81	0.31	0.00
40.86	121.61	0.28	0.80	0.31	0.01	40.93	121.63	0.28	0.80	0.31	0.01
40.98	120.17	0.27	0.81	0.31	0.00	41.06	117.47	0.26	0.83	0.30	0.01
41.12	117.73	0.26	0.82	0.30	0.01	41.20	119.11	0.27	0.81	0.30	0.01
41.27	116.18	0.26	0.83	0.30	0.01	41.33	112.59	0.25	0.85	0.30	0.01
41.41	120.63	0.28	0.79	0.30	0.01	41.47	138.43	0.37	0.68	0.30	0.00
41.55	151.43	0.50	0.61	0.30	0.01	41.60	155.47	0.56	0.59	0.29	0.00
41.69	156.93	0.58	0.56	0.29	0.01	41.73	157.57	0.59	0.55	0.29	0.00
41.82	158.94	0.62	0.52	0.29	0.01	41.88	162.13	0.68	0.47	0.29	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)
41.96	163.93	0.72	0.43	0.29	0.00	42.02	164.17	0.72	0.43	0.29	0.00
42.16	160.06	0.64	0.49	0.29	0.01	42.22	156.32	0.57	0.56	0.28	0.00
42.30	160.60	0.65	0.48	0.28	0.00	42.35	163.52	0.71	0.43	0.28	0.00
42.44	168.05	0.82	0.32	0.28	0.00	42.49	170.18	0.89	0.27	0.28	0.00
42.51	170.15	0.89	0.27	0.28	0.00	42.52	171.54	0.93	0.25	0.28	0.00
42.59	174.66	1.05	0.19	0.28	0.00	42.67	176.08	1.11	0.17	0.28	0.00
42.75	175.17	1.07	0.18	0.28	0.00	42.80	173.32	1.00	0.21	0.27	0.00
42.93	169.31	0.86	0.28	0.27	0.00	42.97	168.77	0.84	0.29	0.27	0.00
43.01	168.13	0.83	0.31	0.27	0.00	43.11	167.00	0.80	0.33	0.27	0.00
43.19	165.16	0.75	0.37	0.27	0.00	43.24	163.29	0.70	0.41	0.27	0.00
43.28	160.64	0.65	0.45	0.27	0.00	43.33	157.29	0.59	0.50	0.27	0.00
43.38	153.17	0.53	0.54	0.26	0.00	43.52	141.02	0.39	0.58	0.26	0.01
43.57	138.73	0.38	0.59	0.26	0.00	43.63	137.18	0.37	0.60	0.26	0.00
43.68	135.77	0.36	0.60	0.26	0.00	43.73	134.84	2.00	0.00	0.26	0.00
43.77	134.09	2.00	0.00	0.26	0.00	43.86	128.38	2.00	0.00	0.26	0.00
43.91	122.32	2.00	0.00	0.26	0.00	44.08	106.56	2.00	0.00	0.25	0.00
44.13	103.82	2.00	0.00	0.25	0.00	44.18	36.04	2.00	0.00	0.25	0.00
44.25	99.63	0.21	0.81	0.25	0.01	44.33	34.19	2.00	0.00	0.25	0.00
44.38	31.54	2.00	0.00	0.25	0.00	44.43	29.26	2.00	0.00	0.25	0.00
44.51	27.13	2.00	0.00	0.25	0.00	44.56	26.42	2.00	0.00	0.24	0.00
44.65	22.64	2.00	0.00	0.24	0.00	44.72	19.87	2.00	0.00	0.24	0.00
44.75	17.33	2.00	0.00	0.24	0.00	44.86	14.59	2.00	0.00	0.24	0.00
44.91	14.31	2.00	0.00	0.24	0.00	44.95	13.56	2.00	0.00	0.24	0.00
45.05	11.94	2.00	0.00	0.24	0.00	45.13	11.26	2.00	0.00	0.24	0.00
45.18	10.25	2.00	0.00	0.23	0.00	45.28	10.27	2.00	0.00	0.23	0.00
45.35	10.27	2.00	0.00	0.23	0.00	45.44	10.30	2.00	0.00	0.23	0.00
45.50	10.89	2.00	0.00	0.23	0.00	45.57	11.62	2.00	0.00	0.23	0.00
45.62	12.15	2.00	0.00	0.23	0.00	45.67	12.08	2.00	0.00	0.23	0.00
45.79	11.07	2.00	0.00	0.22	0.00	45.84	10.40	2.00	0.00	0.22	0.00
45.89	10.00	2.00	0.00	0.22	0.00	45.94	9.40	2.00	0.00	0.22	0.00
46.01	9.19	2.00	0.00	0.22	0.00	46.11	9.25	2.00	0.00	0.22	0.00
46.16	9.58	2.00	0.00	0.22	0.00	46.23	9.77	2.00	0.00	0.22	0.00
46.28	9.97	2.00	0.00	0.22	0.00	46.33	10.02	2.00	0.00	0.21	0.00
46.45	10.81	2.00	0.00	0.21	0.00	46.50	11.26	2.00	0.00	0.21	0.00
46.56	11.79	2.00	0.00	0.21	0.00	46.70	12.44	2.00	0.00	0.21	0.00
46.74	12.44	2.00	0.00	0.21	0.00	46.78	12.37	2.00	0.00	0.21	0.00
46.85	12.22	2.00	0.00	0.21	0.00	46.90	11.96	2.00	0.00	0.21	0.00
46.99	11.55	2.00	0.00	0.20	0.00	47.03	11.35	2.00	0.00	0.20	0.00
47.08	11.28	2.00	0.00	0.20	0.00	47.17	11.20	2.00	0.00	0.20	0.00
47.23	11.13	2.00	0.00	0.20	0.00	47.28	11.13	2.00	0.00	0.20	0.00
47.32	10.86	2.00	0.00	0.20	0.00	47.43	10.85	2.00	0.00	0.20	0.00
47.46	10.85	2.00	0.00	0.20	0.00	47.51	10.97	2.00	0.00	0.19	0.00
47.66	12.34	2.00	0.00	0.19	0.00	47.70	13.13	2.00	0.00	0.19	0.00
47.75	14.52	2.00	0.00	0.19	0.00	47.80	16.71	2.00	0.00	0.19	0.00
47.84	20.06	2.00	0.00	0.19	0.00	47.92	24.92	2.00	0.00	0.19	0.00
48.01	99.79	2.00	0.00	0.19	0.00	48.05	105.03	2.00	0.00	0.19	0.00
48.12	109.15	2.00	0.00	0.18	0.00	48.19	112.59	2.00	0.00	0.18	0.00
48.23	114.88	2.00	0.00	0.18	0.00	48.34	117.26	2.00	0.00	0.18	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)
48.41	118.21	2.00	0.00	0.18	0.00	48.50	118.01	2.00	0.00	0.18	0.00
48.55	118.83	0.27	0.48	0.18	0.00	48.61	121.25	0.28	0.46	0.18	0.00
48.67	125.51	0.30	0.44	0.18	0.00	48.72	128.47	0.31	0.43	0.17	0.00
48.94	138.67	0.38	0.39	0.17	0.01	48.98	138.80	0.38	0.38	0.17	0.00
49.03	134.83	0.35	0.40	0.17	0.00	49.10	134.55	0.35	0.39	0.17	0.00
49.15	134.94	0.35	0.39	0.17	0.00	49.20	135.70	0.36	0.39	0.17	0.00
49.25	136.55	0.36	0.38	0.17	0.00	49.30	135.54	0.36	0.38	0.16	0.00
49.31	130.63	0.33	0.40	0.16	0.00	49.38	138.62	0.38	0.37	0.16	0.00
49.42	139.19	0.38	0.37	0.16	0.00	49.51	139.50	0.39	0.36	0.16	0.00
49.55	139.42	0.38	0.36	0.16	0.00	49.65	139.14	0.38	0.36	0.16	0.00
49.69	138.94	0.38	0.36	0.16	0.00	49.75	138.50	0.38	0.36	0.16	0.00
49.82	140.42	0.39	0.35	0.16	0.00	49.87	87.52	2.00	0.00	0.15	0.00
49.95	84.60	2.00	0.00	0.15	0.00	50.01	82.24	2.00	0.00	0.15	0.00
50.09	80.31	2.00	0.00	0.15	0.00	50.15	78.28	2.00	0.00	0.15	0.00
50.20	60.94	2.00	0.00	0.15	0.00						

Total estimated settlement: 4.73

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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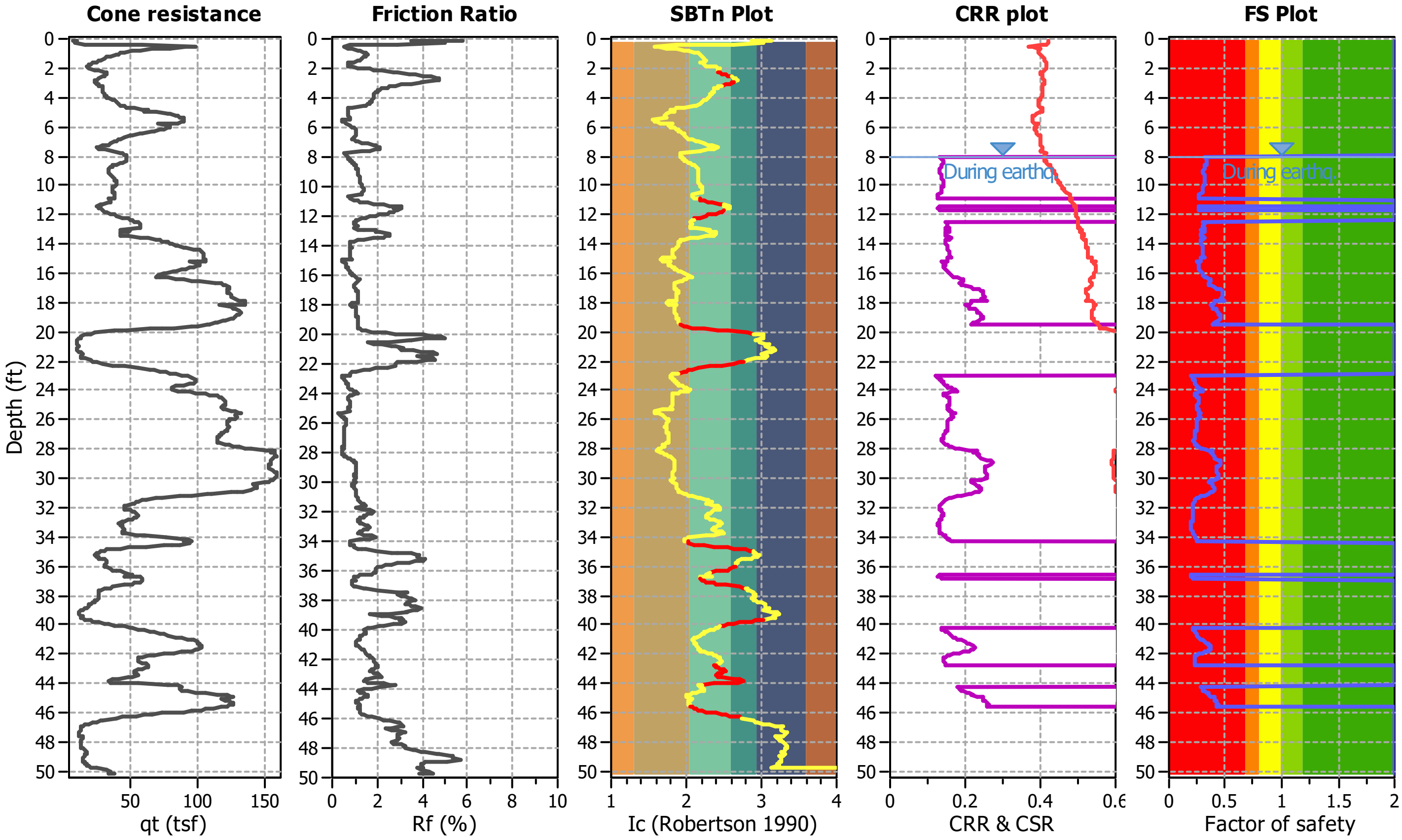
LIQUEFACTION ANALYSIS REPORT

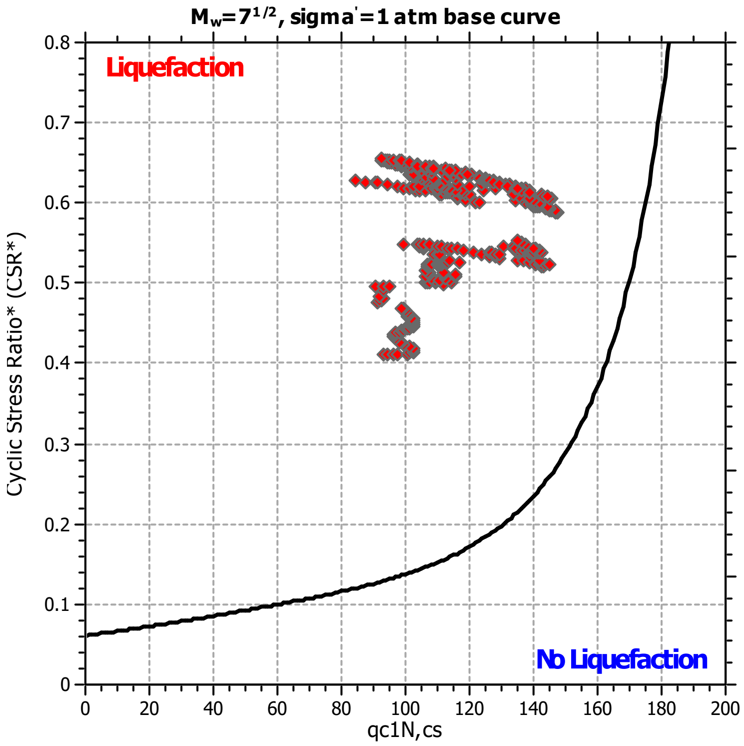
Project title : Proposed Warehouse **Location : Cerritos, CA**

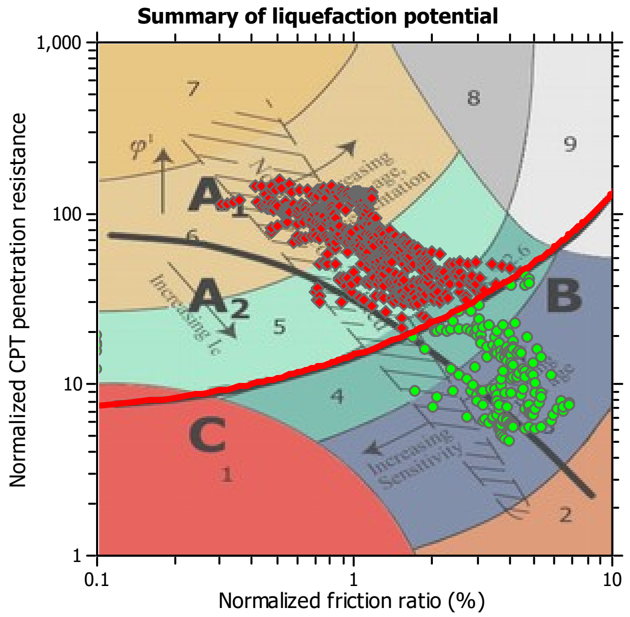
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.81	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

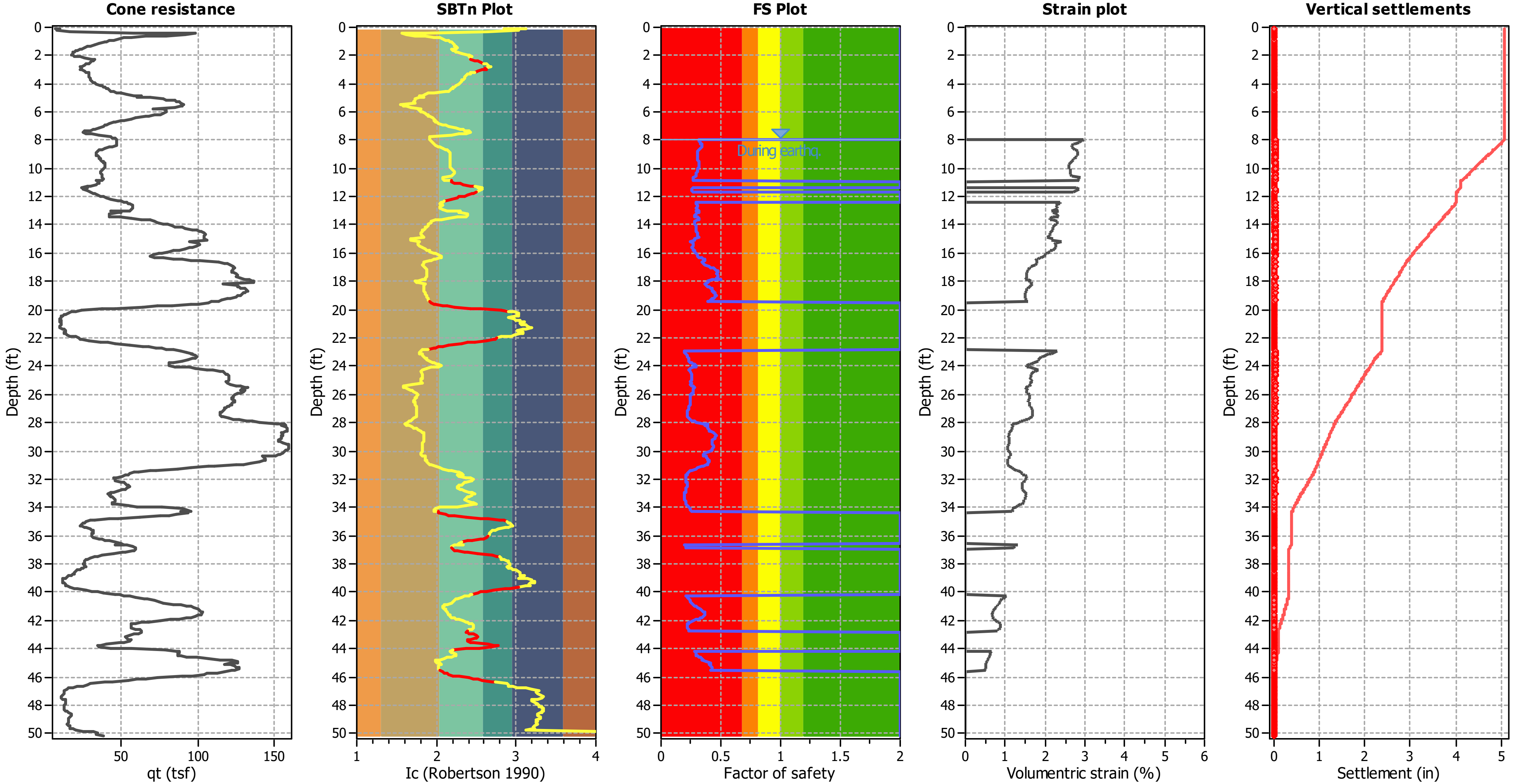






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and 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)
8.00	93.24	0.31	2.98	0.86	0.03	8.06	94.66	0.32	2.93	0.86	0.02
8.09	96.08	0.32	2.89	0.86	0.01	8.16	97.65	0.33	2.84	0.86	0.02
8.24	100.42	0.33	2.75	0.86	0.03	8.28	101.56	0.34	2.72	0.86	0.01
8.36	102.75	0.34	2.68	0.86	0.02	8.45	102.77	0.34	2.68	0.86	0.03
8.49	102.31	0.34	2.69	0.86	0.01	8.57	100.99	0.33	2.72	0.85	0.02
8.65	99.31	0.32	2.76	0.85	0.03	8.69	98.64	0.32	2.78	0.85	0.01
8.73	98.01	0.32	2.79	0.85	0.02	8.82	97.06	0.31	2.82	0.85	0.03
8.87	96.74	0.31	2.82	0.85	0.02	8.95	96.45	0.31	2.83	0.85	0.03
9.00	96.44	0.31	2.83	0.85	0.01	9.09	96.75	0.31	2.81	0.85	0.03
9.13	97.17	0.31	2.80	0.85	0.01	9.22	98.07	0.31	2.77	0.84	0.03
9.26	98.56	0.31	2.75	0.84	0.01	9.35	99.51	0.31	2.72	0.84	0.03
9.40	99.98	0.31	2.70	0.84	0.02	9.48	100.80	0.31	2.67	0.84	0.03
9.52	101.27	0.31	2.66	0.84	0.01	9.62	101.97	0.31	2.64	0.84	0.03
9.66	102.29	0.31	2.63	0.84	0.01	9.74	102.50	0.31	2.62	0.83	0.03
9.79	102.70	0.31	2.61	0.83	0.02	9.86	102.74	0.31	2.60	0.83	0.02
9.92	102.77	0.31	2.60	0.83	0.02	10.01	102.23	0.31	2.61	0.83	0.03
10.06	101.60	0.31	2.62	0.83	0.02	10.15	100.75	0.30	2.64	0.83	0.03
10.19	100.52	0.30	2.64	0.83	0.01	10.24	100.38	0.30	2.65	0.83	0.01
10.32	100.39	0.30	2.64	0.83	0.03	10.42	100.02	0.30	2.65	0.82	0.03
10.48	99.69	0.29	2.65	0.82	0.02	10.51	98.77	0.29	2.67	0.82	0.01
10.62	90.98	0.27	2.90	0.82	0.04	10.68	92.22	0.27	2.86	0.82	0.02
10.72	92.01	0.27	2.86	0.82	0.01	10.77	91.92	0.27	2.86	0.82	0.02
10.86	93.17	0.27	2.82	0.82	0.03	10.90	91.86	0.26	2.85	0.82	0.01
10.98	95.44	2.00	0.00	0.81	0.00	11.03	96.51	2.00	0.00	0.81	0.00
11.11	98.20	2.00	0.00	0.81	0.00	11.16	97.89	2.00	0.00	0.81	0.00
11.25	94.89	2.00	0.00	0.81	0.00	11.29	92.83	2.00	0.00	0.81	0.00
11.39	89.34	2.00	0.00	0.81	0.00	11.43	93.18	0.26	2.78	0.81	0.01
11.51	90.57	0.25	2.86	0.80	0.03	11.57	93.32	0.26	2.77	0.80	0.02
11.65	94.95	0.27	2.72	0.80	0.02	11.69	95.44	2.00	0.00	0.80	0.00
11.78	96.75	2.00	0.00	0.80	0.00	11.82	97.75	2.00	0.00	0.80	0.00
11.91	99.69	2.00	0.00	0.80	0.00	11.95	100.64	2.00	0.00	0.80	0.00
12.03	101.78	2.00	0.00	0.80	0.00	12.08	102.24	2.00	0.00	0.80	0.00
12.17	103.39	2.00	0.00	0.79	0.00	12.29	104.84	2.00	0.00	0.79	0.00
12.32	105.24	2.00	0.00	0.79	0.00	12.38	105.64	2.00	0.00	0.79	0.00
12.44	106.24	0.29	2.38	0.79	0.02	12.48	106.67	0.29	2.37	0.79	0.01
12.53	107.52	0.30	2.35	0.79	0.01	12.65	111.58	0.31	2.25	0.79	0.03
12.71	109.47	0.30	2.30	0.78	0.02	12.75	109.17	0.30	2.30	0.78	0.01
12.85	108.18	0.30	2.32	0.78	0.03	12.91	110.81	0.30	2.26	0.78	0.01
12.97	114.69	0.32	2.17	0.78	0.02	12.99	106.26	0.29	2.35	0.78	0.01
13.09	113.59	0.31	2.19	0.78	0.03	13.13	112.81	0.31	2.20	0.78	0.01
13.22	107.69	0.29	2.31	0.78	0.03	13.27	107.28	0.29	2.32	0.78	0.01
13.35	106.84	0.29	2.32	0.77	0.02	13.41	106.16	0.28	2.33	0.77	0.02
13.47	111.70	0.30	2.21	0.77	0.02	13.54	115.44	0.32	2.13	0.77	0.02
13.61	115.51	0.32	2.13	0.77	0.02	13.67	112.17	0.30	2.19	0.77	0.02
13.76	107.23	0.28	2.29	0.77	0.02	13.81	107.03	0.28	2.29	0.77	0.01
13.91	106.72	0.28	2.30	0.76	0.03	13.95	107.35	0.28	2.28	0.76	0.01
13.98	108.03	0.28	2.26	0.76	0.01	14.07	109.79	0.29	2.22	0.76	0.02
14.11	110.22	0.29	2.21	0.76	0.01	14.19	110.56	0.29	2.20	0.76	0.02

:: 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)
14.25	111.04	0.29	2.19	0.76	0.02	14.34	110.95	0.29	2.18	0.76	0.02
14.38	111.06	0.29	2.18	0.76	0.01	14.44	111.63	0.29	2.17	0.76	0.02
14.51	112.52	0.30	2.14	0.75	0.02	14.60	113.35	0.30	2.12	0.75	0.02
14.69	113.58	0.30	2.11	0.75	0.02	14.78	113.54	0.30	2.11	0.75	0.02
14.88	117.03	0.31	2.04	0.75	0.03	14.95	108.85	0.28	2.20	0.75	0.02
15.00	109.06	0.28	2.19	0.75	0.01	15.07	110.07	0.28	2.17	0.74	0.02
15.13	110.80	0.29	2.15	0.74	0.02	15.23	99.37	0.25	2.40	0.74	0.03
15.31	104.40	0.26	2.28	0.74	0.02	15.36	105.72	0.27	2.24	0.74	0.01
15.42	105.32	0.27	2.25	0.74	0.02	15.49	104.66	0.26	2.26	0.74	0.02
15.58	103.90	0.26	2.27	0.74	0.02	15.63	104.38	0.26	2.26	0.74	0.02
15.71	105.66	0.26	2.23	0.73	0.02	15.78	107.28	0.27	2.19	0.73	0.02
15.85	109.84	0.28	2.13	0.73	0.02	15.89	110.98	0.28	2.11	0.73	0.01
15.97	113.36	0.29	2.06	0.73	0.02	16.02	114.66	0.30	2.03	0.73	0.01
16.12	116.04	0.30	2.00	0.73	0.02	16.22	118.13	0.31	1.96	0.73	0.03
16.29	121.16	0.32	1.90	0.72	0.01	16.33	123.98	0.34	1.85	0.72	0.01
16.38	126.46	0.35	1.81	0.72	0.01	16.44	128.36	0.36	1.78	0.72	0.01
16.51	129.08	0.37	1.77	0.72	0.01	16.55	128.39	0.36	1.78	0.72	0.01
16.64	126.02	0.35	1.81	0.72	0.02	16.70	126.57	0.35	1.80	0.72	0.01
16.77	127.85	0.36	1.77	0.72	0.01	16.82	129.09	0.36	1.75	0.71	0.01
16.99	135.12	0.41	1.66	0.71	0.03	17.04	136.75	0.42	1.64	0.71	0.01
17.08	138.98	0.44	1.61	0.71	0.01	17.14	140.53	0.45	1.58	0.71	0.01
17.22	141.89	0.47	1.56	0.71	0.01	17.26	142.55	0.47	1.56	0.71	0.01
17.33	142.92	0.48	1.55	0.71	0.01	17.40	142.72	0.47	1.55	0.71	0.01
17.44	142.41	0.47	1.55	0.70	0.01	17.52	142.74	0.47	1.54	0.70	0.01
17.65	143.70	0.48	1.53	0.70	0.02	17.75	141.85	0.46	1.55	0.70	0.02
17.82	145.15	0.50	1.50	0.70	0.01	17.88	138.91	0.43	1.58	0.70	0.01
17.93	134.78	0.40	1.63	0.70	0.01	18.01	134.90	0.40	1.62	0.69	0.02
18.06	134.91	0.40	1.62	0.69	0.01	18.13	133.93	0.39	1.63	0.69	0.01
18.20	130.84	0.37	1.67	0.69	0.01	18.34	133.54	0.38	1.63	0.69	0.03
18.36	134.38	0.39	1.62	0.69	0.00	18.41	136.11	0.40	1.59	0.69	0.01
18.45	137.49	0.41	1.57	0.69	0.01	18.54	138.84	0.43	1.55	0.69	0.02
18.58	139.14	0.43	1.55	0.69	0.01	18.67	139.67	0.43	1.54	0.68	0.02
18.71	140.11	0.44	1.53	0.68	0.01	18.80	141.09	0.44	1.52	0.68	0.02
18.85	141.58	0.45	1.51	0.68	0.01	18.93	142.18	0.45	1.50	0.68	0.02
18.98	142.52	0.46	1.49	0.68	0.01	19.06	142.36	0.46	1.49	0.68	0.02
19.11	141.99	0.45	1.49	0.68	0.01	19.20	140.73	0.44	1.50	0.67	0.02
19.24	139.95	0.43	1.51	0.67	0.01	19.33	137.57	0.41	1.54	0.67	0.02
19.38	136.14	0.40	1.55	0.67	0.01	19.46	134.77	0.39	1.57	0.67	0.02
19.51	134.17	2.00	0.00	0.67	0.00	19.60	133.58	2.00	0.00	0.67	0.00
19.64	133.44	2.00	0.00	0.67	0.00	19.72	130.53	2.00	0.00	0.67	0.00
19.78	126.14	2.00	0.00	0.66	0.00	19.82	120.04	2.00	0.00	0.66	0.00
19.91	105.41	2.00	0.00	0.66	0.00	19.95	97.04	2.00	0.00	0.66	0.00
20.06	23.11	2.00	0.00	0.66	0.00	20.12	19.40	2.00	0.00	0.66	0.00
20.17	16.78	2.00	0.00	0.66	0.00	20.22	15.24	2.00	0.00	0.66	0.00
20.32	13.14	2.00	0.00	0.66	0.00	20.39	12.69	2.00	0.00	0.65	0.00
20.44	11.69	2.00	0.00	0.65	0.00	20.50	10.87	2.00	0.00	0.65	0.00
20.57	10.06	2.00	0.00	0.65	0.00	20.61	9.51	2.00	0.00	0.65	0.00
20.71	10.04	2.00	0.00	0.65	0.00	20.79	9.58	2.00	0.00	0.65	0.00

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

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
20.84	10.03	2.00	0.00	0.65	0.00
20.96	10.36	2.00	0.00	0.64	0.00
21.10	9.99	2.00	0.00	0.64	0.00
21.23	9.89	2.00	0.00	0.64	0.00
21.37	11.11	2.00	0.00	0.64	0.00
21.46	13.32	2.00	0.00	0.64	0.00
21.59	12.58	2.00	0.00	0.63	0.00
21.73	13.00	2.00	0.00	0.63	0.00
21.85	15.54	2.00	0.00	0.63	0.00
22.01	20.97	2.00	0.00	0.63	0.00
22.12	23.05	2.00	0.00	0.63	0.00
22.30	93.34	2.00	0.00	0.62	0.00
22.38	99.59	2.00	0.00	0.62	0.00
22.59	106.93	2.00	0.00	0.62	0.00
22.70	106.26	2.00	0.00	0.62	0.00
22.80	93.33	2.00	0.00	0.61	0.00
22.94	84.52	0.19	2.32	0.61	0.02
23.09	90.88	0.20	2.15	0.61	0.02
23.18	94.06	0.21	2.08	0.61	0.02
23.32	99.69	0.22	1.95	0.60	0.01
23.44	103.14	0.23	1.88	0.60	0.02
23.58	104.46	0.23	1.84	0.60	0.02
23.72	110.40	0.25	1.73	0.60	0.02
23.84	116.01	0.27	1.64	0.60	0.02
23.98	121.80	0.29	1.55	0.59	0.02
24.11	116.21	0.27	1.62	0.59	0.02
24.25	102.76	0.23	1.84	0.59	0.02
24.38	107.89	0.24	1.74	0.59	0.02
24.53	112.55	0.26	1.66	0.58	0.02
24.69	114.08	0.26	1.63	0.58	0.02
24.79	114.54	0.26	1.62	0.58	0.01
24.91	112.73	0.26	1.64	0.58	0.01
25.00	111.80	0.25	1.65	0.58	0.01
25.13	109.57	0.24	1.68	0.57	0.01
25.29	110.28	0.25	1.66	0.57	0.01
25.44	116.56	0.27	1.56	0.57	0.01
25.54	120.16	0.28	1.50	0.57	0.01
25.67	115.84	0.26	1.56	0.56	0.01
25.81	117.20	0.27	1.53	0.56	0.01
25.95	113.65	0.26	1.58	0.56	0.01
26.07	111.61	0.25	1.60	0.56	0.01
26.18	110.41	0.25	1.61	0.56	0.01
26.32	109.80	0.24	1.62	0.55	0.01
26.51	111.36	0.25	1.58	0.55	0.01
26.64	109.57	0.24	1.60	0.55	0.02
26.77	108.55	0.24	1.61	0.55	0.01
26.87	106.46	0.23	1.64	0.54	0.01
26.98	104.77	0.23	1.66	0.54	0.01

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
20.88	9.67	2.00	0.00	0.65	0.00
21.02	10.45	2.00	0.00	0.64	0.00
21.15	9.80	2.00	0.00	0.64	0.00
21.27	9.88	2.00	0.00	0.64	0.00
21.41	12.00	2.00	0.00	0.64	0.00
21.55	13.21	2.00	0.00	0.63	0.00
21.69	12.39	2.00	0.00	0.63	0.00
21.81	14.32	2.00	0.00	0.63	0.00
21.93	18.88	2.00	0.00	0.63	0.00
22.08	22.09	2.00	0.00	0.63	0.00
22.21	27.51	2.00	0.00	0.62	0.00
22.34	96.55	2.00	0.00	0.62	0.00
22.49	103.63	2.00	0.00	0.62	0.00
22.65	105.89	2.00	0.00	0.62	0.00
22.75	106.91	2.00	0.00	0.61	0.00
22.87	82.32	2.00	0.00	0.61	0.00
23.01	87.64	0.20	2.24	0.61	0.02
23.11	91.43	0.20	2.14	0.61	0.01
23.26	97.36	0.22	2.00	0.61	0.02
23.36	101.21	0.22	1.92	0.60	0.01
23.49	104.45	0.23	1.85	0.60	0.01
23.63	106.11	0.24	1.81	0.60	0.01
23.76	111.19	0.25	1.72	0.60	0.01
23.89	118.83	0.28	1.60	0.60	0.01
24.02	122.94	0.30	1.53	0.59	0.01
24.16	109.15	0.25	1.73	0.59	0.01
24.29	104.15	0.23	1.81	0.59	0.01
24.43	109.82	0.25	1.71	0.59	0.01
24.60	114.11	0.26	1.63	0.58	0.01
24.74	113.94	0.26	1.63	0.58	0.01
24.86	113.43	0.26	1.63	0.58	0.01
24.95	112.27	0.25	1.64	0.58	0.01
25.08	110.48	0.25	1.67	0.57	0.02
25.23	113.24	0.26	1.62	0.57	0.02
25.40	113.93	0.26	1.60	0.57	0.02
25.49	118.67	0.28	1.53	0.57	0.01
25.59	117.23	0.27	1.54	0.57	0.01
25.75	117.97	0.27	1.52	0.56	0.02
25.89	115.62	0.26	1.55	0.56	0.02
26.02	112.19	0.25	1.59	0.56	0.01
26.15	110.71	0.25	1.61	0.56	0.02
26.28	109.77	0.24	1.62	0.55	0.02
26.46	111.16	0.25	1.59	0.55	0.03
26.55	110.53	0.25	1.59	0.55	0.01
26.72	109.03	0.24	1.61	0.55	0.02
26.82	107.46	0.24	1.63	0.55	0.01
26.93	105.52	0.23	1.65	0.54	0.01
27.04	104.19	0.23	1.67	0.54	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)
27.15	103.21	0.22	1.68	0.54	0.02	27.21	102.45	0.22	1.69	0.54	0.01
27.26	101.79	0.22	1.70	0.54	0.01	27.36	101.42	0.22	1.70	0.54	0.02
27.43	101.25	0.22	1.70	0.54	0.01	27.48	101.45	0.22	1.69	0.53	0.01
27.53	101.81	0.22	1.68	0.53	0.01	27.57	102.70	0.22	1.67	0.53	0.01
27.68	106.48	0.23	1.60	0.53	0.02	27.74	108.95	0.24	1.56	0.53	0.01
27.79	112.29	0.25	1.51	0.53	0.01	27.84	116.15	0.26	1.45	0.53	0.01
27.91	119.99	0.28	1.40	0.53	0.01	27.97	124.61	0.30	1.34	0.53	0.01
28.08	134.57	0.35	1.23	0.52	0.02	28.14	139.12	0.39	1.18	0.52	0.01
28.16	139.43	0.39	1.18	0.52	0.00	28.23	137.26	0.37	1.20	0.52	0.01
28.31	139.56	0.39	1.17	0.52	0.01	28.36	139.99	0.39	1.16	0.52	0.01
28.42	139.81	0.39	1.16	0.52	0.01	28.50	140.44	0.40	1.16	0.52	0.01
28.56	140.98	0.40	1.15	0.52	0.01	28.62	142.00	0.41	1.14	0.51	0.01
28.72	144.76	0.44	1.11	0.51	0.01	28.74	145.33	0.44	1.10	0.51	0.00
28.85	146.65	0.46	1.09	0.51	0.01	28.90	147.41	0.46	1.08	0.51	0.01
28.94	147.42	0.46	1.08	0.51	0.01	29.03	146.82	0.46	1.08	0.51	0.01
29.09	146.65	0.45	1.08	0.51	0.01	29.20	144.38	0.43	1.09	0.51	0.02
29.24	144.10	0.43	1.10	0.50	0.00	29.28	143.80	0.43	1.10	0.50	0.01
29.36	144.12	0.43	1.09	0.50	0.01	29.44	143.84	0.43	1.09	0.50	0.01
29.52	143.95	0.43	1.09	0.50	0.01	29.56	144.23	0.43	1.08	0.50	0.01
29.66	144.93	0.44	1.07	0.50	0.01	29.71	145.01	0.44	1.07	0.50	0.01
29.75	144.95	0.44	1.07	0.50	0.01	29.81	144.66	0.43	1.07	0.49	0.01
29.86	144.42	0.43	1.07	0.49	0.01	29.96	143.67	0.42	1.07	0.49	0.01
30.00	144.56	0.43	1.06	0.49	0.01	30.09	141.89	0.41	1.08	0.49	0.01
30.14	136.04	0.36	1.13	0.49	0.01	30.23	135.75	0.36	1.13	0.49	0.01
30.27	135.76	0.36	1.13	0.49	0.01	30.32	135.87	0.36	1.13	0.49	0.01
30.41	138.47	0.38	1.10	0.48	0.01	30.49	139.12	0.38	1.09	0.48	0.01
30.53	139.53	0.39	1.09	0.48	0.01	30.59	140.32	0.39	1.08	0.48	0.01
30.66	140.81	0.40	1.07	0.48	0.01	30.72	141.65	0.40	1.06	0.48	0.01
30.80	141.90	0.41	1.06	0.48	0.01	30.89	140.31	0.39	1.07	0.48	0.01
30.93	139.69	0.39	1.07	0.48	0.01	30.98	138.47	0.38	1.08	0.47	0.01
31.07	135.54	0.36	1.10	0.47	0.01	31.10	133.84	0.34	1.11	0.47	0.00
31.19	128.15	0.31	1.17	0.47	0.01	31.24	123.76	0.29	1.21	0.47	0.01
31.35	115.58	0.26	1.30	0.47	0.02	31.39	112.97	0.25	1.33	0.47	0.01
31.46	108.33	0.23	1.38	0.47	0.01	31.50	106.82	0.23	1.40	0.47	0.01
31.57	104.97	0.22	1.42	0.46	0.01	31.66	103.95	0.22	1.43	0.46	0.02
31.74	98.97	0.21	1.50	0.46	0.01	31.78	97.62	0.21	1.52	0.46	0.01
31.87	97.06	0.21	1.52	0.46	0.02	31.91	96.68	0.21	1.53	0.46	0.01
32.01	97.42	0.21	1.51	0.46	0.02	32.05	97.34	0.21	1.51	0.46	0.01
32.10	97.12	0.21	1.51	0.46	0.01	32.19	99.90	0.21	1.46	0.45	0.02
32.23	100.93	0.21	1.44	0.45	0.01	32.29	101.49	0.22	1.43	0.45	0.01
32.38	102.08	0.22	1.42	0.45	0.01	32.45	101.74	0.22	1.42	0.45	0.01
32.48	101.83	0.22	1.42	0.45	0.00	32.55	101.31	0.22	1.42	0.45	0.01
32.63	100.36	0.21	1.43	0.45	0.01	32.72	99.48	0.21	1.44	0.45	0.01
32.76	98.94	0.21	1.44	0.44	0.01	32.82	97.20	0.21	1.47	0.44	0.01
32.89	95.28	0.20	1.49	0.44	0.01	32.94	94.30	0.20	1.51	0.44	0.01
33.04	92.48	0.20	1.53	0.44	0.02	33.08	94.17	0.20	1.50	0.44	0.01
33.16	92.71	0.20	1.52	0.44	0.02	33.25	93.31	0.20	1.50	0.44	0.02
33.30	93.49	0.20	1.50	0.44	0.01	33.34	93.58	0.20	1.49	0.44	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)
33.42	93.81	0.20	1.49	0.43	0.02	33.47	94.25	0.20	1.48	0.43	0.01
33.56	95.22	0.20	1.46	0.43	0.01	33.64	95.50	0.20	1.45	0.43	0.02
33.68	95.46	0.20	1.45	0.43	0.01	33.73	95.86	0.20	1.44	0.43	0.01
33.80	98.32	0.21	1.40	0.43	0.01	33.89	104.23	0.22	1.31	0.43	0.01
33.94	106.06	0.23	1.28	0.42	0.01	34.01	106.00	0.23	1.28	0.42	0.01
34.10	113.62	0.25	1.19	0.42	0.01	34.15	111.30	0.24	1.21	0.42	0.01
34.25	115.30	0.25	1.16	0.42	0.01	34.29	116.88	0.26	1.14	0.42	0.01
34.32	117.48	0.26	1.14	0.42	0.00	34.42	120.56	2.00	0.00	0.42	0.00
34.52	123.18	2.00	0.00	0.41	0.00	34.56	122.52	2.00	0.00	0.41	0.00
34.60	120.00	2.00	0.00	0.41	0.00	34.68	111.46	2.00	0.00	0.41	0.00
34.76	100.43	2.00	0.00	0.41	0.00	34.79	34.00	2.00	0.00	0.41	0.00
34.87	26.55	2.00	0.00	0.41	0.00	34.91	24.24	2.00	0.00	0.41	0.00
35.00	21.92	2.00	0.00	0.41	0.00	35.06	20.62	2.00	0.00	0.41	0.00
35.11	20.07	2.00	0.00	0.40	0.00	35.18	19.22	2.00	0.00	0.40	0.00
35.26	18.83	2.00	0.00	0.40	0.00	35.31	19.34	2.00	0.00	0.40	0.00
35.39	20.62	2.00	0.00	0.40	0.00	35.44	21.51	2.00	0.00	0.40	0.00
35.53	23.69	2.00	0.00	0.40	0.00	35.57	24.74	2.00	0.00	0.40	0.00
35.67	25.62	2.00	0.00	0.40	0.00	35.71	25.68	2.00	0.00	0.39	0.00
35.80	25.19	2.00	0.00	0.39	0.00	35.85	24.81	2.00	0.00	0.39	0.00
35.91	24.56	2.00	0.00	0.39	0.00	36.02	24.61	2.00	0.00	0.39	0.00
36.06	24.90	2.00	0.00	0.39	0.00	36.10	25.65	2.00	0.00	0.39	0.00
36.20	86.87	2.00	0.00	0.39	0.00	36.25	88.53	2.00	0.00	0.39	0.00
36.32	88.57	2.00	0.00	0.38	0.00	36.37	89.66	2.00	0.00	0.38	0.00
36.48	92.20	2.00	0.00	0.38	0.00	36.59	94.47	2.00	0.00	0.38	0.00
36.64	95.30	0.20	1.28	0.38	0.01	36.65	92.31	0.20	1.32	0.38	0.00
36.69	96.54	0.20	1.26	0.38	0.01	36.76	98.28	0.21	1.23	0.38	0.01
36.82	98.94	0.21	1.22	0.38	0.01	36.90	99.57	0.21	1.21	0.37	0.01
36.95	99.86	2.00	0.00	0.37	0.00	37.03	100.80	2.00	0.00	0.37	0.00
37.07	100.94	2.00	0.00	0.37	0.00	37.18	99.92	2.00	0.00	0.37	0.00
37.21	98.98	2.00	0.00	0.37	0.00	37.30	96.24	2.00	0.00	0.37	0.00
37.42	29.50	2.00	0.00	0.37	0.00	37.46	27.45	2.00	0.00	0.37	0.00
37.49	25.49	2.00	0.00	0.36	0.00	37.55	23.82	2.00	0.00	0.36	0.00
37.64	22.15	2.00	0.00	0.36	0.00	37.72	20.94	2.00	0.00	0.36	0.00
37.77	20.19	2.00	0.00	0.36	0.00	37.82	20.18	2.00	0.00	0.36	0.00
37.86	20.18	2.00	0.00	0.36	0.00	37.97	20.05	2.00	0.00	0.36	0.00
38.01	19.93	2.00	0.00	0.36	0.00	38.06	20.07	2.00	0.00	0.35	0.00
38.19	20.70	2.00	0.00	0.35	0.00	38.23	20.18	2.00	0.00	0.35	0.00
38.27	19.42	2.00	0.00	0.35	0.00	38.34	18.46	2.00	0.00	0.35	0.00
38.40	17.64	2.00	0.00	0.35	0.00	38.46	16.74	2.00	0.00	0.35	0.00
38.56	15.19	2.00	0.00	0.35	0.00	38.63	14.67	2.00	0.00	0.35	0.00
38.67	14.59	2.00	0.00	0.34	0.00	38.72	14.15	2.00	0.00	0.34	0.00
38.81	13.33	2.00	0.00	0.34	0.00	38.85	12.09	2.00	0.00	0.34	0.00
38.94	11.00	2.00	0.00	0.34	0.00	38.99	10.21	2.00	0.00	0.34	0.00
39.07	9.19	2.00	0.00	0.34	0.00	39.12	8.83	2.00	0.00	0.34	0.00
39.20	9.14	2.00	0.00	0.34	0.00	39.25	8.89	2.00	0.00	0.33	0.00
39.33	9.10	2.00	0.00	0.33	0.00	39.38	10.03	2.00	0.00	0.33	0.00
39.46	10.52	2.00	0.00	0.33	0.00	39.52	10.30	2.00	0.00	0.33	0.00
39.57	11.15	2.00	0.00	0.33	0.00	39.65	12.59	2.00	0.00	0.33	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)
39.74	15.10	2.00	0.00	0.33	0.00	39.78	16.18	2.00	0.00	0.33	0.00
39.85	20.36	2.00	0.00	0.32	0.00	39.92	23.33	2.00	0.00	0.32	0.00
39.97	84.30	2.00	0.00	0.32	0.00	40.04	87.80	2.00	0.00	0.32	0.00
40.14	93.63	2.00	0.00	0.32	0.00	40.18	96.53	2.00	0.00	0.32	0.00
40.25	99.04	0.21	1.03	0.32	0.01	40.31	101.07	0.21	1.01	0.32	0.01
40.37	103.90	0.22	0.98	0.32	0.01	40.46	107.21	0.23	0.94	0.31	0.01
40.51	108.77	0.23	0.92	0.31	0.00	40.62	112.50	0.24	0.89	0.31	0.01
40.68	114.31	0.25	0.87	0.31	0.01	40.75	115.58	0.25	0.85	0.31	0.01
40.85	118.56	0.26	0.83	0.31	0.01	40.90	120.46	0.27	0.81	0.31	0.00
40.98	122.89	0.28	0.79	0.31	0.01	41.02	124.87	0.29	0.78	0.30	0.00
41.08	127.59	0.30	0.75	0.30	0.01	41.20	131.43	0.32	0.73	0.30	0.01
41.24	132.63	0.33	0.72	0.30	0.00	41.30	134.57	0.34	0.70	0.30	0.01
41.37	136.16	0.35	0.69	0.30	0.01	41.42	137.36	0.36	0.68	0.30	0.00
41.59	138.52	0.37	0.67	0.30	0.01	41.64	137.21	0.36	0.67	0.29	0.00
41.69	135.95	0.35	0.68	0.29	0.00	41.77	134.53	0.34	0.68	0.29	0.01
41.82	132.52	0.33	0.69	0.29	0.00	41.87	129.63	0.31	0.71	0.29	0.00
41.94	125.02	0.29	0.73	0.29	0.01	41.99	119.67	0.27	0.77	0.29	0.01
42.08	113.72	0.25	0.81	0.29	0.01	42.13	109.62	0.24	0.84	0.29	0.01
42.21	106.00	0.23	0.86	0.28	0.01	42.27	103.67	0.22	0.88	0.28	0.01
42.35	103.64	0.22	0.87	0.28	0.01	42.41	103.62	0.22	0.87	0.28	0.01
42.48	103.73	0.22	0.87	0.28	0.01	42.50	103.84	0.22	0.86	0.28	0.00
42.54	103.94	0.22	0.86	0.28	0.00	42.59	106.15	0.23	0.84	0.28	0.00
42.67	108.02	0.23	0.82	0.28	0.01	42.72	108.61	0.23	0.81	0.28	0.00
42.84	108.75	2.00	0.00	0.27	0.00	42.86	108.26	2.00	0.00	0.27	0.00
42.92	107.35	2.00	0.00	0.27	0.00	42.99	105.59	2.00	0.00	0.27	0.00
43.07	103.88	2.00	0.00	0.27	0.00	43.17	101.45	2.00	0.00	0.27	0.00
43.21	101.17	2.00	0.00	0.27	0.00	43.25	100.88	2.00	0.00	0.27	0.00
43.34	100.78	2.00	0.00	0.27	0.00	43.40	101.14	2.00	0.00	0.26	0.00
43.44	100.67	2.00	0.00	0.26	0.00	43.54	97.97	2.00	0.00	0.26	0.00
43.58	95.74	2.00	0.00	0.26	0.00	43.69	30.22	2.00	0.00	0.26	0.00
43.74	27.49	2.00	0.00	0.26	0.00	43.78	25.41	2.00	0.00	0.26	0.00
43.92	26.28	2.00	0.00	0.26	0.00	43.96	91.44	2.00	0.00	0.25	0.00
44.01	100.66	2.00	0.00	0.25	0.00	44.11	114.88	2.00	0.00	0.25	0.00
44.17	120.55	2.00	0.00	0.25	0.00	44.23	123.43	0.28	0.64	0.25	0.00
44.27	124.49	0.29	0.64	0.25	0.00	44.32	125.07	0.29	0.63	0.25	0.00
44.38	125.69	0.30	0.63	0.25	0.00	44.45	126.16	0.30	0.62	0.25	0.00
44.49	127.38	0.30	0.61	0.25	0.00	44.56	129.46	0.31	0.60	0.24	0.00
44.63	132.09	0.33	0.58	0.24	0.00	44.70	134.50	0.34	0.57	0.24	0.00
44.76	136.32	0.36	0.56	0.24	0.00	44.84	135.02	0.35	0.56	0.24	0.01
44.89	138.89	0.37	0.54	0.24	0.00	44.98	143.08	0.41	0.52	0.24	0.01
45.03	143.24	0.41	0.52	0.24	0.00	45.10	142.46	0.40	0.52	0.24	0.00
45.16	143.20	0.41	0.51	0.23	0.00	45.24	144.09	0.42	0.51	0.23	0.00
45.28	144.58	0.42	0.50	0.23	0.00	45.35	145.09	0.43	0.50	0.23	0.00
45.41	145.56	0.43	0.49	0.23	0.00	45.51	144.92	0.43	0.49	0.23	0.01
45.55	144.32	0.42	0.49	0.23	0.00	45.63	142.53	2.00	0.00	0.23	0.00
45.70	140.65	2.00	0.00	0.23	0.00	45.78	137.97	2.00	0.00	0.22	0.00
45.81	136.68	2.00	0.00	0.22	0.00	45.90	133.44	2.00	0.00	0.22	0.00
45.95	129.68	2.00	0.00	0.22	0.00	46.05	118.27	2.00	0.00	0.22	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)
46.10	112.28	2.00	0.00	0.22	0.00	46.15	107.21	2.00	0.00	0.22	0.00
46.20	102.30	2.00	0.00	0.22	0.00	46.30	32.00	2.00	0.00	0.22	0.00
46.35	29.01	2.00	0.00	0.21	0.00	46.50	21.15	2.00	0.00	0.21	0.00
46.56	19.75	2.00	0.00	0.21	0.00	46.63	17.82	2.00	0.00	0.21	0.00
46.69	15.21	2.00	0.00	0.21	0.00	46.74	13.18	2.00	0.00	0.21	0.00
46.81	11.22	2.00	0.00	0.21	0.00	46.88	10.01	2.00	0.00	0.21	0.00
46.92	8.94	2.00	0.00	0.20	0.00	46.97	9.00	2.00	0.00	0.20	0.00
47.00	9.00	2.00	0.00	0.20	0.00	47.07	9.06	2.00	0.00	0.20	0.00
47.15	8.72	2.00	0.00	0.20	0.00	47.18	8.72	2.00	0.00	0.20	0.00
47.28	8.05	2.00	0.00	0.20	0.00	47.32	7.99	2.00	0.00	0.20	0.00
47.38	7.71	2.00	0.00	0.20	0.00	47.46	7.78	2.00	0.00	0.20	0.00
47.55	8.50	2.00	0.00	0.19	0.00	47.62	9.36	2.00	0.00	0.19	0.00
47.64	9.68	2.00	0.00	0.19	0.00	47.72	9.67	2.00	0.00	0.19	0.00
47.81	9.60	2.00	0.00	0.19	0.00	47.88	9.27	2.00	0.00	0.19	0.00
47.91	9.46	2.00	0.00	0.19	0.00	47.99	8.93	2.00	0.00	0.19	0.00
48.07	8.86	2.00	0.00	0.19	0.00	48.16	8.65	2.00	0.00	0.18	0.00
48.21	8.85	2.00	0.00	0.18	0.00	48.25	8.78	2.00	0.00	0.18	0.00
48.30	8.84	2.00	0.00	0.18	0.00	48.38	9.50	2.00	0.00	0.18	0.00
48.43	10.09	2.00	0.00	0.18	0.00	48.52	10.67	2.00	0.00	0.18	0.00
48.56	11.13	2.00	0.00	0.18	0.00	48.65	11.85	2.00	0.00	0.18	0.00
48.70	12.05	2.00	0.00	0.17	0.00	48.78	11.70	2.00	0.00	0.17	0.00
48.83	11.50	2.00	0.00	0.17	0.00	48.93	10.96	2.00	0.00	0.17	0.00
48.99	10.95	2.00	0.00	0.17	0.00	49.09	10.49	2.00	0.00	0.17	0.00
49.14	10.48	2.00	0.00	0.17	0.00	49.18	10.48	2.00	0.00	0.17	0.00
49.22	10.47	2.00	0.00	0.17	0.00	49.29	10.33	2.00	0.00	0.16	0.00
49.36	10.20	2.00	0.00	0.16	0.00	49.42	10.85	2.00	0.00	0.16	0.00
49.49	11.18	2.00	0.00	0.16	0.00	49.55	11.49	2.00	0.00	0.16	0.00
49.63	11.76	2.00	0.00	0.16	0.00	49.70	12.54	2.00	0.00	0.16	0.00
49.76	13.92	2.00	0.00	0.16	0.00	49.83	17.45	2.00	0.00	0.16	0.00
49.90	22.18	2.00	0.00	0.15	0.00	49.98	24.51	2.00	0.00	0.15	0.00
50.03	24.02	2.00	0.00	0.15	0.00	50.11	23.95	2.00	0.00	0.15	0.00
50.17	27.11	2.00	0.00	0.15	0.00						

Total estimated settlement: 5.07

Abbreviations

$Q_{tn,cs}$: Equivalent dean 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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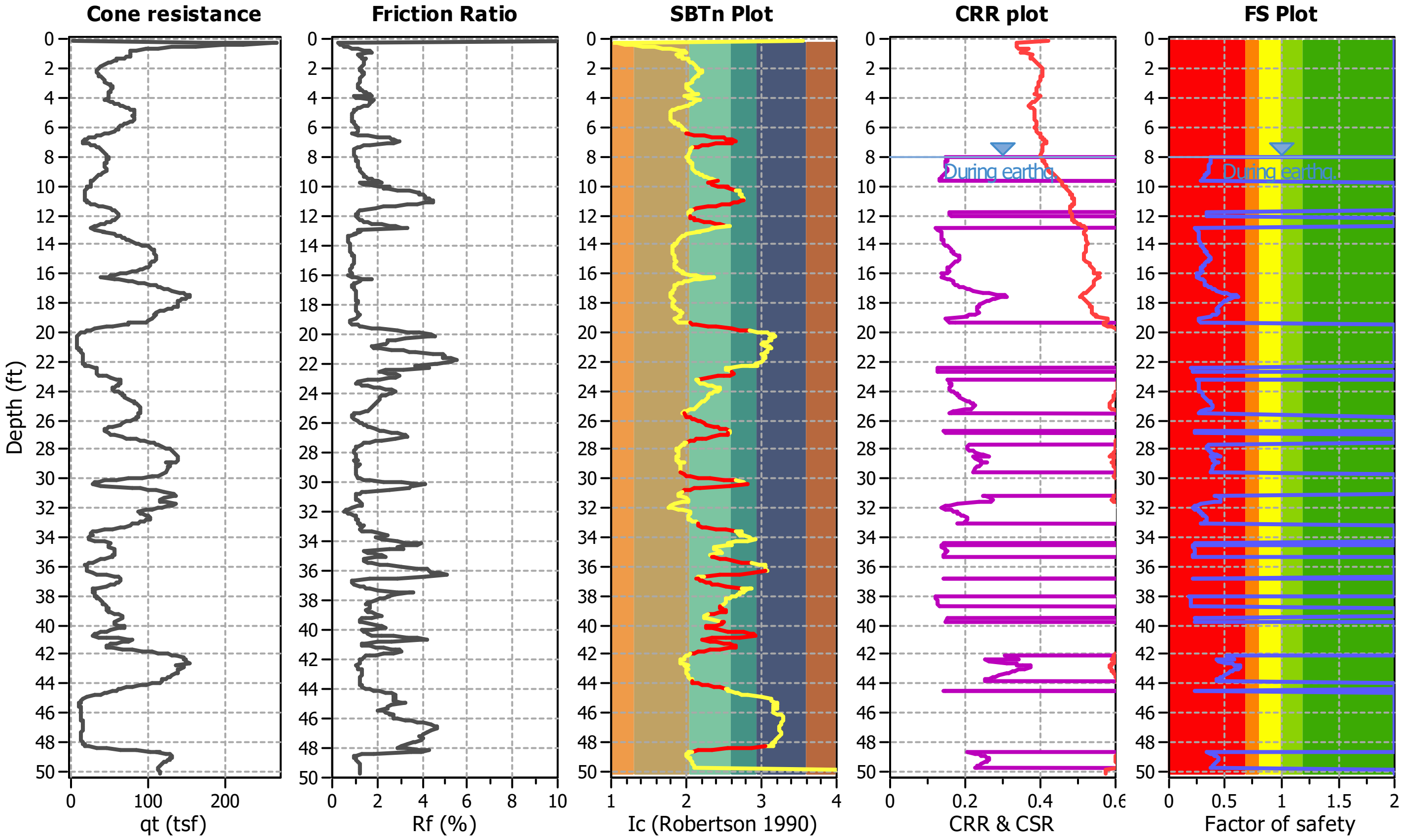
LIQUEFACTION ANALYSIS REPORT

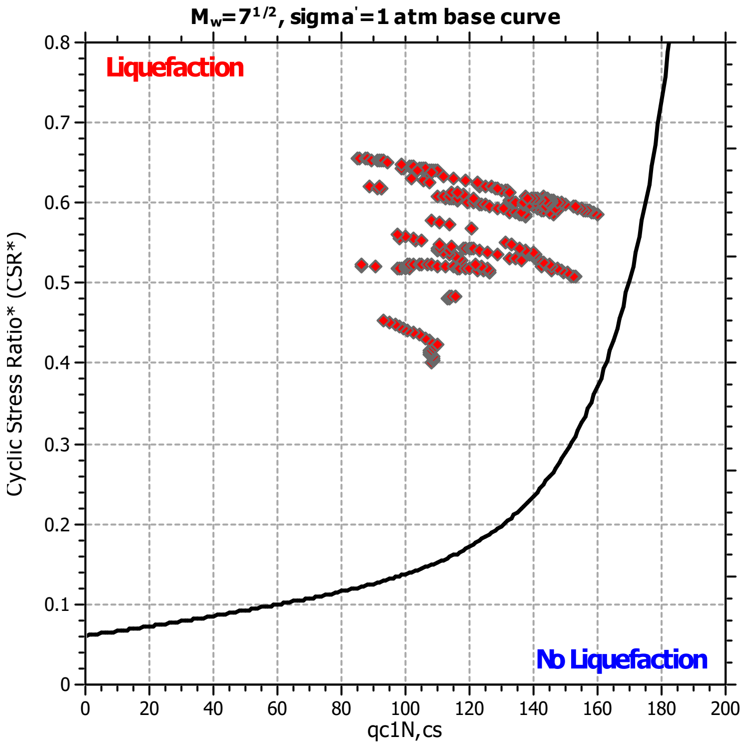
Project title : Proposed Warehouse **Location : Cerritos, CA**

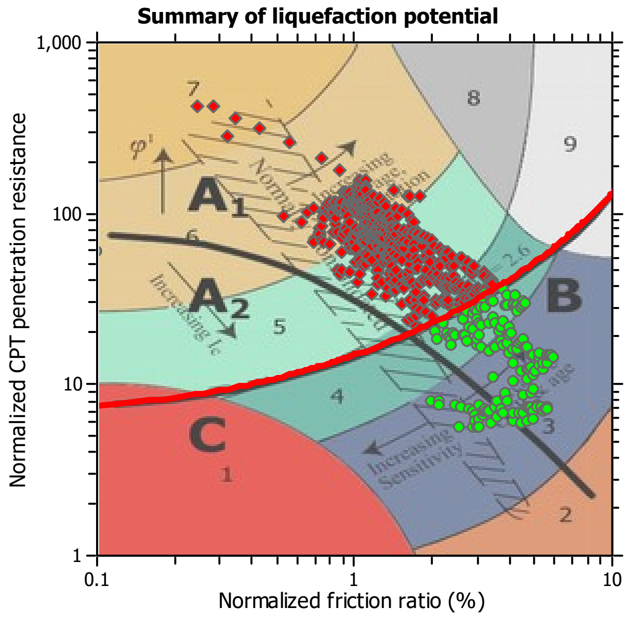
CPT file : CPT-3

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.81	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

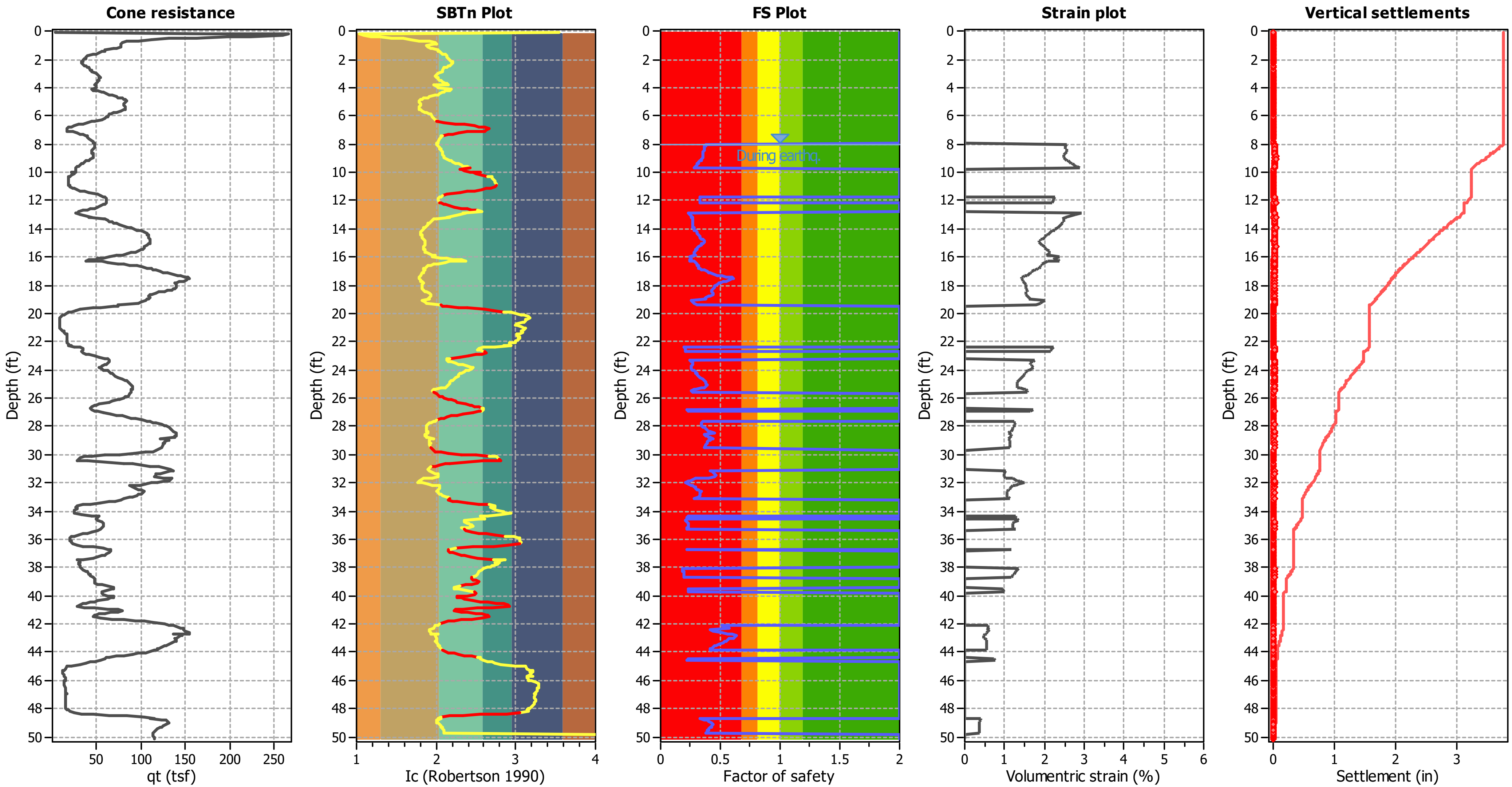






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and 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)
8.04	108.33	0.37	2.56	0.86	0.02	8.11	108.70	0.37	2.54	0.86	0.02
8.18	108.87	0.37	2.54	0.86	0.02	8.24	108.77	0.37	2.53	0.86	0.02
8.29	108.46	0.37	2.54	0.86	0.02	8.37	108.10	0.36	2.54	0.86	0.02
8.43	107.80	0.36	2.55	0.86	0.02	8.50	107.56	0.36	2.55	0.86	0.02
8.55	107.66	0.36	2.55	0.86	0.02	8.64	108.18	0.36	2.53	0.85	0.03
8.69	108.50	0.36	2.52	0.85	0.02	8.77	108.97	0.36	2.50	0.85	0.02
8.89	109.66	0.36	2.48	0.85	0.04	8.95	109.69	0.36	2.48	0.85	0.02
9.08	107.81	0.35	2.52	0.85	0.04	9.15	106.08	0.34	2.55	0.84	0.02
9.21	104.08	0.33	2.60	0.84	0.02	9.27	102.29	0.32	2.65	0.84	0.02
9.34	100.66	0.31	2.69	0.84	0.02	9.41	99.23	0.31	2.72	0.84	0.02
9.48	98.02	0.30	2.75	0.84	0.02	9.54	96.86	0.30	2.78	0.84	0.02
9.61	95.31	0.29	2.82	0.84	0.02	9.66	93.43	0.29	2.88	0.84	0.02
9.74	89.45	2.00	0.00	0.83	0.00	9.79	89.74	2.00	0.00	0.83	0.00
9.87	91.10	2.00	0.00	0.83	0.00	10.01	92.21	2.00	0.00	0.83	0.00
10.01	86.54	2.00	0.00	0.83	0.00	10.04	91.23	2.00	0.00	0.83	0.00
10.11	90.17	2.00	0.00	0.83	0.00	10.19	87.59	2.00	0.00	0.83	0.00
10.28	25.84	2.00	0.00	0.83	0.00	10.34	24.37	2.00	0.00	0.82	0.00
10.40	24.18	2.00	0.00	0.82	0.00	10.45	24.36	2.00	0.00	0.82	0.00
10.56	24.22	2.00	0.00	0.82	0.00	10.62	23.92	2.00	0.00	0.82	0.00
10.67	23.86	2.00	0.00	0.82	0.00	10.72	23.81	2.00	0.00	0.82	0.00
10.82	23.68	2.00	0.00	0.82	0.00	10.90	24.15	2.00	0.00	0.82	0.00
10.96	24.75	2.00	0.00	0.81	0.00	11.03	25.79	2.00	0.00	0.81	0.00
11.10	27.91	2.00	0.00	0.81	0.00	11.16	30.59	2.00	0.00	0.81	0.00
11.23	93.80	2.00	0.00	0.81	0.00	11.29	98.04	2.00	0.00	0.81	0.00
11.37	102.52	2.00	0.00	0.81	0.00	11.43	106.68	2.00	0.00	0.81	0.00
11.56	111.98	2.00	0.00	0.80	0.00	11.64	112.31	2.00	0.00	0.80	0.00
11.70	113.00	2.00	0.00	0.80	0.00	11.78	113.38	0.33	2.26	0.80	0.02
11.84	113.60	0.33	2.25	0.80	0.02	11.91	113.73	0.33	2.24	0.80	0.02
11.97	114.44	0.33	2.23	0.80	0.02	12.11	115.50	0.33	2.20	0.79	0.04
12.18	115.84	2.00	0.00	0.79	0.00	12.26	116.60	2.00	0.00	0.79	0.00
12.31	116.55	2.00	0.00	0.79	0.00	12.40	116.90	2.00	0.00	0.79	0.00
12.45	115.34	2.00	0.00	0.79	0.00	12.53	113.57	2.00	0.00	0.79	0.00
12.59	109.81	2.00	0.00	0.79	0.00	12.66	105.09	2.00	0.00	0.79	0.00
12.72	99.95	2.00	0.00	0.78	0.00	12.80	95.20	2.00	0.00	0.78	0.00
12.86	86.00	0.23	2.92	0.78	0.02	12.93	86.22	0.23	2.91	0.78	0.02
13.00	90.57	0.24	2.77	0.78	0.02	13.17	98.95	0.26	2.52	0.78	0.05
13.18	97.75	0.26	2.55	0.78	0.00	13.19	98.43	0.26	2.54	0.78	0.01
13.28	101.11	0.27	2.46	0.77	0.03	13.35	99.93	0.26	2.49	0.77	0.02
13.41	99.88	0.26	2.49	0.77	0.02	13.46	99.89	0.26	2.48	0.77	0.02
13.54	100.52	0.26	2.46	0.77	0.02	13.59	100.75	0.26	2.45	0.77	0.02
13.66	101.06	0.27	2.44	0.77	0.02	13.72	102.29	0.27	2.41	0.77	0.02
13.90	104.50	0.27	2.35	0.76	0.05	13.95	105.12	0.28	2.33	0.76	0.01
14.03	107.14	0.28	2.28	0.76	0.02	14.08	108.18	0.29	2.26	0.76	0.01
14.15	110.06	0.29	2.21	0.76	0.02	14.22	112.22	0.30	2.16	0.76	0.02
14.34	114.94	0.31	2.10	0.76	0.03	14.41	116.26	0.31	2.08	0.76	0.02
14.48	117.17	0.32	2.06	0.75	0.02	14.55	118.66	0.32	2.02	0.75	0.02
14.61	119.78	0.33	2.00	0.75	0.02	14.70	122.68	0.34	1.95	0.75	0.02
14.75	124.34	0.35	1.92	0.75	0.01	14.83	126.03	0.36	1.89	0.75	0.02

:: 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)
14.88	126.20	0.36	1.88	0.75	0.01	14.97	126.19	0.36	1.88	0.75	0.02
15.02	125.40	0.36	1.89	0.75	0.01	15.10	123.90	0.35	1.91	0.74	0.02
15.15	121.89	0.34	1.94	0.74	0.01	15.30	117.67	0.31	2.01	0.74	0.04
15.36	116.68	0.31	2.02	0.74	0.01	15.44	115.35	0.30	2.04	0.74	0.02
15.50	114.03	0.30	2.07	0.74	0.01	15.58	111.75	0.29	2.11	0.74	0.02
15.64	110.27	0.28	2.13	0.73	0.02	15.71	109.85	0.28	2.14	0.73	0.02
15.79	110.62	0.28	2.12	0.73	0.02	15.85	113.72	0.29	2.06	0.73	0.01
15.92	110.91	0.28	2.11	0.73	0.02	15.98	97.92	0.24	2.39	0.73	0.02
16.06	103.22	0.26	2.26	0.73	0.02	16.12	102.49	0.25	2.28	0.73	0.02
16.20	104.74	0.26	2.22	0.73	0.02	16.22	99.82	0.25	2.33	0.73	0.01
16.24	97.59	0.24	2.39	0.72	0.00	16.31	110.71	0.28	2.09	0.72	0.02
16.37	114.40	0.29	2.02	0.72	0.01	16.49	117.95	0.31	1.95	0.72	0.03
16.54	118.80	0.31	1.93	0.72	0.01	16.59	118.67	0.31	1.93	0.72	0.01
16.65	119.72	0.31	1.91	0.72	0.01	16.71	120.32	0.32	1.90	0.72	0.01
16.76	121.21	0.32	1.88	0.72	0.01	16.81	123.34	0.33	1.84	0.72	0.01
16.87	125.65	0.34	1.80	0.71	0.01	16.94	128.58	0.36	1.76	0.71	0.01
17.04	132.77	0.39	1.69	0.71	0.02	17.11	134.32	0.40	1.67	0.71	0.01
17.16	136.28	0.42	1.64	0.71	0.01	17.28	142.30	0.47	1.56	0.71	0.02
17.33	145.76	0.51	1.51	0.71	0.01	17.38	149.09	0.55	1.47	0.71	0.01
17.46	152.06	0.60	1.44	0.70	0.01	17.52	153.10	0.61	1.43	0.70	0.01
17.56	152.49	0.60	1.43	0.70	0.01	17.64	150.66	0.57	1.45	0.70	0.01
17.69	148.76	0.54	1.47	0.70	0.01	17.74	147.33	0.53	1.48	0.70	0.01
17.82	145.81	0.51	1.50	0.70	0.01	17.87	144.79	0.49	1.51	0.70	0.01
17.92	143.11	0.47	1.52	0.70	0.01	18.09	140.78	0.45	1.55	0.69	0.03
18.14	140.97	0.45	1.54	0.69	0.01	18.21	140.53	0.45	1.54	0.69	0.01
18.26	139.89	0.44	1.55	0.69	0.01	18.31	139.19	0.43	1.56	0.69	0.01
18.39	138.96	0.43	1.56	0.69	0.01	18.44	138.97	0.43	1.55	0.69	0.01
18.50	139.09	0.43	1.55	0.69	0.01	18.56	139.86	0.44	1.54	0.69	0.01
18.61	140.37	0.44	1.53	0.68	0.01	18.67	139.98	0.44	1.53	0.68	0.01
18.75	137.79	0.42	1.56	0.68	0.02	18.79	135.88	0.40	1.58	0.68	0.01
18.86	133.32	0.38	1.61	0.68	0.01	18.93	131.07	0.36	1.64	0.68	0.01
18.99	132.97	0.38	1.61	0.68	0.01	19.06	108.43	0.26	2.00	0.68	0.02
19.19	110.67	0.27	1.95	0.67	0.03	19.28	113.72	0.28	1.89	0.67	0.02
19.35	120.50	0.30	1.78	0.67	0.02	19.37	120.54	0.30	1.77	0.67	0.00
19.45	121.22	2.00	0.00	0.67	0.00	19.49	119.28	2.00	0.00	0.67	0.00
19.59	110.84	2.00	0.00	0.67	0.00	19.63	103.87	2.00	0.00	0.67	0.00
19.74	31.25	2.00	0.00	0.67	0.00	19.81	25.39	2.00	0.00	0.66	0.00
19.86	20.79	2.00	0.00	0.66	0.00	19.94	17.36	2.00	0.00	0.66	0.00
20.07	12.85	2.00	0.00	0.66	0.00	20.12	11.58	2.00	0.00	0.66	0.00
20.17	9.97	2.00	0.00	0.66	0.00	20.31	8.15	2.00	0.00	0.66	0.00
20.39	8.15	2.00	0.00	0.65	0.00	20.44	8.14	2.00	0.00	0.65	0.00
20.52	8.13	2.00	0.00	0.65	0.00	20.60	8.21	2.00	0.00	0.65	0.00
20.65	8.47	2.00	0.00	0.65	0.00	20.74	8.73	2.00	0.00	0.65	0.00
20.79	9.00	2.00	0.00	0.65	0.00	20.86	8.99	2.00	0.00	0.65	0.00
20.92	8.63	2.00	0.00	0.65	0.00	21.00	8.62	2.00	0.00	0.64	0.00
21.07	9.06	2.00	0.00	0.64	0.00	21.13	10.12	2.00	0.00	0.64	0.00
21.20	11.52	2.00	0.00	0.64	0.00	21.27	12.31	2.00	0.00	0.64	0.00
21.33	13.36	2.00	0.00	0.64	0.00	21.40	14.49	2.00	0.00	0.64	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)
21.48	15.72	2.00	0.00	0.64	0.00	21.54	16.32	2.00	0.00	0.63	0.00
21.62	16.04	2.00	0.00	0.63	0.00	21.68	15.76	2.00	0.00	0.63	0.00
21.76	15.83	2.00	0.00	0.63	0.00	21.81	16.08	2.00	0.00	0.63	0.00
21.89	16.15	2.00	0.00	0.63	0.00	21.97	16.14	2.00	0.00	0.63	0.00
22.02	16.12	2.00	0.00	0.63	0.00	22.11	16.37	2.00	0.00	0.63	0.00
22.16	17.15	2.00	0.00	0.62	0.00	22.24	18.78	2.00	0.00	0.62	0.00
22.29	20.51	2.00	0.00	0.62	0.00	22.31	17.90	2.00	0.00	0.62	0.00
22.33	23.83	2.00	0.00	0.62	0.00	22.41	88.97	0.20	2.24	0.62	0.02
22.46	91.18	0.21	2.18	0.62	0.01	22.54	92.42	0.21	2.15	0.62	0.02
22.60	92.41	0.21	2.15	0.62	0.02	22.67	91.91	0.21	2.15	0.62	0.02
22.72	31.44	2.00	0.00	0.61	0.00	22.84	32.35	2.00	0.00	0.61	0.00
22.90	95.73	2.00	0.00	0.61	0.00	22.95	99.78	2.00	0.00	0.61	0.00
23.01	104.08	2.00	0.00	0.61	0.00	23.07	106.53	2.00	0.00	0.61	0.00
23.17	108.31	2.00	0.00	0.61	0.00	23.24	109.31	2.00	0.00	0.61	0.00
23.30	109.85	0.25	1.76	0.61	0.01	23.38	112.54	0.26	1.72	0.60	0.02
23.44	114.19	0.26	1.69	0.60	0.01	23.52	116.09	0.27	1.65	0.60	0.02
23.58	115.95	0.27	1.65	0.60	0.01	23.65	114.85	0.27	1.67	0.60	0.01
23.70	113.46	0.26	1.69	0.60	0.01	23.78	111.86	0.26	1.71	0.60	0.02
23.84	112.03	0.26	1.70	0.60	0.01	23.91	113.73	0.26	1.67	0.59	0.01
24.00	116.55	0.27	1.63	0.59	0.02	24.04	119.61	0.28	1.58	0.59	0.01
24.13	121.51	0.29	1.55	0.59	0.02	24.19	123.04	0.30	1.52	0.59	0.01
24.27	124.04	0.30	1.51	0.59	0.01	24.33	124.87	0.31	1.49	0.59	0.01
24.40	125.99	0.31	1.48	0.59	0.01	24.53	128.91	0.33	1.44	0.58	0.02
24.67	132.76	0.35	1.38	0.58	0.02	24.75	134.33	0.36	1.36	0.58	0.01
24.80	135.64	0.37	1.35	0.58	0.01	24.88	136.54	0.38	1.33	0.58	0.01
24.94	137.39	0.38	1.32	0.58	0.01	25.02	137.78	0.39	1.32	0.58	0.01
25.15	137.38	0.38	1.31	0.57	0.02	25.22	136.06	0.37	1.33	0.57	0.01
25.29	130.90	0.34	1.38	0.57	0.01	25.37	121.50	0.29	1.49	0.57	0.01
25.42	118.27	0.28	1.53	0.57	0.01	25.50	114.22	0.26	1.59	0.57	0.01
25.56	116.16	0.27	1.56	0.57	0.01	25.72	119.42	2.00	0.00	0.56	0.00
25.86	123.09	2.00	0.00	0.56	0.00	25.90	124.32	2.00	0.00	0.56	0.00
25.94	124.85	2.00	0.00	0.56	0.00	25.99	125.14	2.00	0.00	0.56	0.00
26.08	124.22	2.00	0.00	0.56	0.00	26.12	122.58	2.00	0.00	0.56	0.00
26.21	118.17	2.00	0.00	0.56	0.00	26.26	116.31	2.00	0.00	0.55	0.00
26.45	107.57	2.00	0.00	0.55	0.00	26.49	105.32	2.00	0.00	0.55	0.00
26.54	102.82	2.00	0.00	0.55	0.00	26.58	100.84	2.00	0.00	0.55	0.00
26.65	100.51	2.00	0.00	0.55	0.00	26.69	100.31	2.00	0.00	0.55	0.00
26.74	100.68	2.00	0.00	0.55	0.00	26.78	101.85	0.22	1.72	0.55	0.01
26.88	105.82	0.23	1.65	0.54	0.02	26.92	107.31	0.24	1.62	0.54	0.01
26.97	108.67	2.00	0.00	0.54	0.00	27.06	114.35	2.00	0.00	0.54	0.00
27.17	121.17	2.00	0.00	0.54	0.00	27.22	124.83	2.00	0.00	0.54	0.00
27.27	127.90	2.00	0.00	0.54	0.00	27.32	130.19	2.00	0.00	0.54	0.00
27.39	132.20	2.00	0.00	0.54	0.00	27.45	133.69	2.00	0.00	0.53	0.00
27.55	134.84	2.00	0.00	0.53	0.00	27.60	134.37	2.00	0.00	0.53	0.00
27.67	134.12	0.35	1.25	0.53	0.01	27.71	133.40	0.35	1.25	0.53	0.01
27.77	132.19	0.34	1.27	0.53	0.01	27.89	132.49	0.34	1.26	0.53	0.02
27.93	132.56	0.34	1.25	0.53	0.01	27.98	133.07	0.34	1.25	0.53	0.01
28.04	134.08	0.35	1.24	0.52	0.01	28.11	136.58	0.37	1.21	0.52	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)
28.16	137.45	0.37	1.20	0.52	0.01	28.32	141.14	0.40	1.16	0.52	0.02
28.38	142.25	0.41	1.14	0.52	0.01	28.47	146.19	0.45	1.11	0.52	0.01
28.54	137.01	0.37	1.19	0.52	0.01	28.65	138.50	0.38	1.17	0.51	0.02
28.70	139.94	0.39	1.15	0.51	0.01	28.76	141.23	0.40	1.14	0.51	0.01
28.82	141.94	0.41	1.13	0.51	0.01	28.84	145.07	0.44	1.10	0.51	0.00
28.89	143.39	0.42	1.11	0.51	0.01	28.95	141.78	0.41	1.13	0.51	0.01
29.03	141.12	0.40	1.13	0.51	0.01	29.12	140.12	0.39	1.14	0.51	0.01
29.16	139.65	0.39	1.14	0.51	0.01	29.21	139.46	0.39	1.14	0.50	0.01
29.33	138.64	0.38	1.14	0.50	0.02	29.52	136.98	0.37	1.15	0.50	0.03
29.73	139.66	2.00	0.00	0.50	0.00	29.78	140.62	2.00	0.00	0.50	0.00
29.86	142.29	2.00	0.00	0.49	0.00	29.91	137.81	2.00	0.00	0.49	0.00
29.99	127.75	2.00	0.00	0.49	0.00	30.04	114.37	2.00	0.00	0.49	0.00
30.13	37.71	2.00	0.00	0.49	0.00	30.18	30.92	2.00	0.00	0.49	0.00
30.26	27.52	2.00	0.00	0.49	0.00	30.33	25.26	2.00	0.00	0.49	0.00
30.40	23.73	2.00	0.00	0.48	0.00	30.48	94.98	2.00	0.00	0.48	0.00
30.61	125.29	2.00	0.00	0.48	0.00	30.67	130.72	2.00	0.00	0.48	0.00
30.75	133.38	2.00	0.00	0.48	0.00	30.81	134.59	2.00	0.00	0.48	0.00
30.88	135.94	2.00	0.00	0.48	0.00	31.01	138.34	2.00	0.00	0.47	0.00
31.14	142.46	0.41	1.04	0.47	0.02	31.19	143.76	0.42	1.03	0.47	0.01
31.35	147.00	0.46	1.00	0.47	0.02	31.41	147.07	0.46	0.99	0.47	0.01
31.50	147.78	0.46	0.98	0.47	0.01	31.55	146.96	0.46	0.99	0.47	0.01
31.62	140.85	0.40	1.03	0.46	0.01	31.68	123.35	0.29	1.19	0.46	0.01
31.83	112.08	0.25	1.31	0.46	0.02	31.89	106.82	0.23	1.38	0.46	0.01
31.97	99.46	0.21	1.48	0.46	0.01	32.03	98.61	0.21	1.49	0.46	0.01
32.11	107.94	0.23	1.35	0.46	0.01	32.16	114.88	0.25	1.27	0.45	0.01
32.21	118.91	0.27	1.22	0.45	0.01	32.30	122.54	0.28	1.17	0.45	0.01
32.34	123.30	0.29	1.17	0.45	0.01	32.42	124.86	0.29	1.15	0.45	0.01
32.47	126.58	0.30	1.13	0.45	0.01	32.51	128.43	0.31	1.11	0.45	0.00
32.60	131.61	0.33	1.07	0.45	0.01	32.65	132.53	0.34	1.06	0.45	0.01
32.74	132.24	0.33	1.06	0.45	0.01	32.79	132.17	0.33	1.06	0.44	0.01
32.83	131.86	0.33	1.06	0.44	0.01	32.88	131.44	0.33	1.06	0.44	0.01
32.97	130.01	0.32	1.07	0.44	0.01	33.01	128.48	0.31	1.09	0.44	0.01
33.11	122.75	0.28	1.14	0.44	0.01	33.19	118.85	2.00	0.00	0.44	0.00
33.29	114.65	2.00	0.00	0.44	0.00	33.35	110.09	2.00	0.00	0.43	0.00
33.41	104.05	2.00	0.00	0.43	0.00	33.49	95.65	2.00	0.00	0.43	0.00
33.55	28.55	2.00	0.00	0.43	0.00	33.63	23.66	2.00	0.00	0.43	0.00
33.70	22.03	2.00	0.00	0.43	0.00	33.76	22.39	2.00	0.00	0.43	0.00
33.85	22.68	2.00	0.00	0.43	0.00	33.90	20.76	2.00	0.00	0.43	0.00
34.04	20.29	2.00	0.00	0.42	0.00	34.12	20.27	2.00	0.00	0.42	0.00
34.18	19.81	2.00	0.00	0.42	0.00	34.25	26.60	2.00	0.00	0.42	0.00
34.33	36.16	2.00	0.00	0.42	0.00	34.38	105.83	0.23	1.26	0.42	0.01
34.45	104.69	0.22	1.28	0.42	0.01	34.52	40.05	2.00	0.00	0.41	0.00
34.59	101.33	0.22	1.31	0.41	0.01	34.65	98.64	0.21	1.34	0.41	0.01
34.78	105.27	0.23	1.25	0.41	0.02	34.87	107.67	0.23	1.22	0.41	0.01
34.92	108.18	0.23	1.21	0.41	0.01	34.94	108.43	0.23	1.21	0.41	0.00
34.98	108.77	0.23	1.20	0.41	0.01	35.08	108.31	0.23	1.20	0.41	0.02
35.14	106.18	0.23	1.22	0.40	0.01	35.19	103.25	0.22	1.26	0.40	0.01
35.27	102.75	0.22	1.26	0.40	0.01	35.32	101.64	0.22	1.27	0.40	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)
35.40	98.92	2.00	0.00	0.40	0.00	35.45	94.48	2.00	0.00	0.40	0.00
35.54	90.61	2.00	0.00	0.40	0.00	35.63	27.69	2.00	0.00	0.40	0.00
35.68	23.68	2.00	0.00	0.40	0.00	35.76	19.99	2.00	0.00	0.39	0.00
35.82	17.29	2.00	0.00	0.39	0.00	35.89	16.08	2.00	0.00	0.39	0.00
35.95	15.92	2.00	0.00	0.39	0.00	36.09	16.19	2.00	0.00	0.39	0.00
36.16	16.78	2.00	0.00	0.39	0.00	36.23	17.06	2.00	0.00	0.39	0.00
36.29	18.02	2.00	0.00	0.38	0.00	36.42	22.85	2.00	0.00	0.38	0.00
36.49	27.93	2.00	0.00	0.38	0.00	36.55	95.61	2.00	0.00	0.38	0.00
36.63	99.82	2.00	0.00	0.38	0.00	36.70	101.29	2.00	0.00	0.38	0.00
36.77	102.60	0.22	1.18	0.38	0.01	36.84	103.47	2.00	0.00	0.38	0.00
36.91	104.01	2.00	0.00	0.37	0.00	36.97	104.40	2.00	0.00	0.37	0.00
37.04	105.01	2.00	0.00	0.37	0.00	37.10	105.10	2.00	0.00	0.37	0.00
37.18	102.82	2.00	0.00	0.37	0.00	37.25	100.06	2.00	0.00	0.37	0.00
37.31	97.02	2.00	0.00	0.37	0.00	37.44	27.89	2.00	0.00	0.37	0.00
37.45	22.31	2.00	0.00	0.37	0.00	37.48	22.86	2.00	0.00	0.36	0.00
37.57	24.27	2.00	0.00	0.36	0.00	37.62	23.37	2.00	0.00	0.36	0.00
37.71	24.35	2.00	0.00	0.36	0.00	37.75	22.89	2.00	0.00	0.36	0.00
37.82	24.42	2.00	0.00	0.36	0.00	37.86	24.79	2.00	0.00	0.36	0.00
37.94	24.99	2.00	0.00	0.36	0.00	38.02	25.11	2.00	0.00	0.36	0.00
38.10	84.80	0.18	1.34	0.35	0.01	38.15	85.68	0.18	1.33	0.35	0.01
38.23	87.40	0.19	1.29	0.35	0.01	38.28	88.30	0.19	1.28	0.35	0.01
38.33	89.22	0.19	1.26	0.35	0.01	38.43	91.17	0.19	1.23	0.35	0.02
38.50	91.99	0.20	1.21	0.35	0.01	38.54	92.50	0.20	1.21	0.35	0.01
38.59	92.87	0.20	1.20	0.35	0.01	38.71	94.33	0.20	1.17	0.34	0.02
38.81	96.46	2.00	0.00	0.34	0.00	38.88	97.62	2.00	0.00	0.34	0.00
38.95	97.88	2.00	0.00	0.34	0.00	39.02	97.85	2.00	0.00	0.34	0.00
39.08	98.28	2.00	0.00	0.34	0.00	39.16	99.77	2.00	0.00	0.34	0.00
39.21	102.72	2.00	0.00	0.34	0.00	39.27	106.31	2.00	0.00	0.33	0.00
39.39	109.70	2.00	0.00	0.33	0.00	39.48	109.62	0.24	0.97	0.33	0.01
39.54	109.86	0.24	0.96	0.33	0.01	39.74	105.73	0.23	0.99	0.33	0.02
39.76	105.89	0.23	0.99	0.33	0.00	39.77	106.01	0.23	0.99	0.33	0.00
39.84	106.43	2.00	0.00	0.32	0.00	39.90	107.21	2.00	0.00	0.32	0.00
39.99	109.35	2.00	0.00	0.32	0.00	40.07	112.06	2.00	0.00	0.32	0.00
40.13	112.11	2.00	0.00	0.32	0.00	40.17	110.16	2.00	0.00	0.32	0.00
40.23	107.64	2.00	0.00	0.32	0.00	40.37	98.99	2.00	0.00	0.32	0.00
40.41	97.06	2.00	0.00	0.32	0.00	40.47	33.19	2.00	0.00	0.31	0.00
40.53	29.65	2.00	0.00	0.31	0.00	40.57	26.29	2.00	0.00	0.31	0.00
40.63	24.01	2.00	0.00	0.31	0.00	40.72	21.96	2.00	0.00	0.31	0.00
40.81	27.80	2.00	0.00	0.31	0.00	40.86	97.16	2.00	0.00	0.31	0.00
40.92	108.18	2.00	0.00	0.31	0.00	40.99	115.80	2.00	0.00	0.31	0.00
41.03	119.35	2.00	0.00	0.30	0.00	41.12	117.64	2.00	0.00	0.30	0.00
41.17	112.73	2.00	0.00	0.30	0.00	41.25	107.29	2.00	0.00	0.30	0.00
41.30	102.27	2.00	0.00	0.30	0.00	41.37	37.41	2.00	0.00	0.30	0.00
41.41	36.14	2.00	0.00	0.30	0.00	41.47	35.75	2.00	0.00	0.30	0.00
41.59	104.79	2.00	0.00	0.30	0.00	41.65	114.57	2.00	0.00	0.29	0.00
41.70	125.29	2.00	0.00	0.29	0.00	41.75	133.20	2.00	0.00	0.29	0.00
41.83	140.04	2.00	0.00	0.29	0.00	41.87	144.81	2.00	0.00	0.29	0.00
41.99	150.55	2.00	0.00	0.29	0.00	42.05	151.58	2.00	0.00	0.29	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)
42.14	152.54	0.51	0.58	0.29	0.01	42.27	154.90	0.55	0.57	0.28	0.01
42.32	156.64	0.57	0.55	0.28	0.00	42.40	143.42	0.41	0.61	0.28	0.01
42.47	146.05	0.44	0.60	0.28	0.00	42.61	152.77	0.52	0.56	0.28	0.01
42.67	156.04	0.57	0.55	0.28	0.00	42.69	156.36	0.57	0.54	0.28	0.00
42.72	158.31	0.60	0.50	0.28	0.00	42.81	160.16	0.64	0.47	0.27	0.01
42.86	160.15	0.64	0.47	0.27	0.00	42.99	158.56	0.61	0.49	0.27	0.01
43.03	157.57	0.59	0.51	0.27	0.00	43.07	156.70	0.58	0.52	0.27	0.00
43.11	155.88	0.56	0.53	0.27	0.00	43.19	154.80	0.55	0.54	0.27	0.00
43.25	154.00	0.53	0.54	0.27	0.00	43.35	152.85	0.52	0.54	0.27	0.01
43.46	150.11	0.48	0.55	0.26	0.01	43.52	148.67	0.47	0.55	0.26	0.00
43.57	147.29	0.45	0.55	0.26	0.00	43.63	145.65	0.43	0.56	0.26	0.00
43.70	144.58	0.42	0.56	0.26	0.00	43.75	144.02	0.42	0.56	0.26	0.00
43.83	143.70	0.42	0.56	0.26	0.01	43.89	143.06	2.00	0.00	0.26	0.00
44.01	139.00	2.00	0.00	0.25	0.00	44.08	135.66	2.00	0.00	0.25	0.00
44.15	131.33	2.00	0.00	0.25	0.00	44.22	125.11	2.00	0.00	0.25	0.00
44.28	118.13	2.00	0.00	0.25	0.00	44.36	112.02	2.00	0.00	0.25	0.00
44.42	109.49	2.00	0.00	0.25	0.00	44.50	108.16	0.23	0.73	0.25	0.01
44.55	103.47	0.22	0.76	0.24	0.01	44.63	35.59	2.00	0.00	0.24	0.00
44.68	31.92	2.00	0.00	0.24	0.00	44.76	27.99	2.00	0.00	0.24	0.00
44.83	23.48	2.00	0.00	0.24	0.00	44.96	15.72	2.00	0.00	0.24	0.00
45.03	12.37	2.00	0.00	0.24	0.00	45.17	10.60	2.00	0.00	0.23	0.00
45.25	9.59	2.00	0.00	0.23	0.00	45.30	8.92	2.00	0.00	0.23	0.00
45.38	8.91	2.00	0.00	0.23	0.00	45.44	8.91	2.00	0.00	0.23	0.00
45.52	8.90	2.00	0.00	0.23	0.00	45.60	9.56	2.00	0.00	0.23	0.00
45.66	10.09	2.00	0.00	0.23	0.00	45.70	9.29	2.00	0.00	0.23	0.00
45.74	10.22	2.00	0.00	0.22	0.00	45.80	10.61	2.00	0.00	0.22	0.00
45.88	10.54	2.00	0.00	0.22	0.00	45.98	10.53	2.00	0.00	0.22	0.00
46.06	9.99	2.00	0.00	0.22	0.00	46.15	9.52	2.00	0.00	0.22	0.00
46.19	9.25	2.00	0.00	0.22	0.00	46.28	9.57	2.00	0.00	0.22	0.00
46.37	9.70	2.00	0.00	0.21	0.00	46.50	10.55	2.00	0.00	0.21	0.00
46.57	10.82	2.00	0.00	0.21	0.00	46.63	11.14	2.00	0.00	0.21	0.00
46.71	11.07	2.00	0.00	0.21	0.00	46.76	11.00	2.00	0.00	0.21	0.00
46.85	11.19	2.00	0.00	0.21	0.00	46.90	11.39	2.00	0.00	0.21	0.00
46.98	11.18	2.00	0.00	0.20	0.00	47.11	11.17	2.00	0.00	0.20	0.00
47.18	11.09	2.00	0.00	0.20	0.00	47.25	11.08	2.00	0.00	0.20	0.00
47.32	10.68	2.00	0.00	0.20	0.00	47.38	10.64	2.00	0.00	0.20	0.00
47.45	10.64	2.00	0.00	0.20	0.00	47.60	10.59	2.00	0.00	0.19	0.00
47.65	10.59	2.00	0.00	0.19	0.00	47.74	10.58	2.00	0.00	0.19	0.00
47.87	10.57	2.00	0.00	0.19	0.00	47.93	10.70	2.00	0.00	0.19	0.00
48.00	11.21	2.00	0.00	0.19	0.00	48.07	12.13	2.00	0.00	0.19	0.00
48.13	12.79	2.00	0.00	0.18	0.00	48.21	13.91	2.00	0.00	0.18	0.00
48.28	17.12	2.00	0.00	0.18	0.00	48.36	23.68	2.00	0.00	0.18	0.00
48.41	93.80	2.00	0.00	0.18	0.00	48.49	104.71	2.00	0.00	0.18	0.00
48.55	116.64	2.00	0.00	0.18	0.00	48.62	125.19	2.00	0.00	0.18	0.00
48.68	130.12	2.00	0.00	0.17	0.00	48.72	132.27	0.33	0.42	0.17	0.00
48.78	137.47	0.37	0.40	0.17	0.00	48.85	140.17	0.39	0.39	0.17	0.00
48.89	141.50	0.40	0.38	0.17	0.00	48.98	144.26	0.42	0.37	0.17	0.00
49.03	145.11	0.43	0.36	0.17	0.00	49.13	145.90	0.44	0.36	0.17	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)
49.16	145.89	0.44	0.36	0.17	0.00	49.25	145.17	0.43	0.36	0.17	0.00
49.29	144.64	0.43	0.36	0.16	0.00	49.35	143.71	0.42	0.36	0.16	0.00
49.42	142.18	0.41	0.36	0.16	0.00	49.51	141.22	0.40	0.36	0.16	0.00
49.55	140.60	0.39	0.36	0.16	0.00	49.63	139.68	0.39	0.36	0.16	0.00
49.70	139.17	0.38	0.36	0.16	0.00	49.78	138.33	0.37	0.36	0.16	0.00
49.82	86.32	2.00	0.00	0.16	0.00	49.90	86.46	2.00	0.00	0.15	0.00
49.95	87.00	2.00	0.00	0.15	0.00	50.01	88.08	2.00	0.00	0.15	0.00
50.07	88.54	2.00	0.00	0.15	0.00	50.15	87.74	2.00	0.00	0.15	0.00

Total estimated settlement: 3.76

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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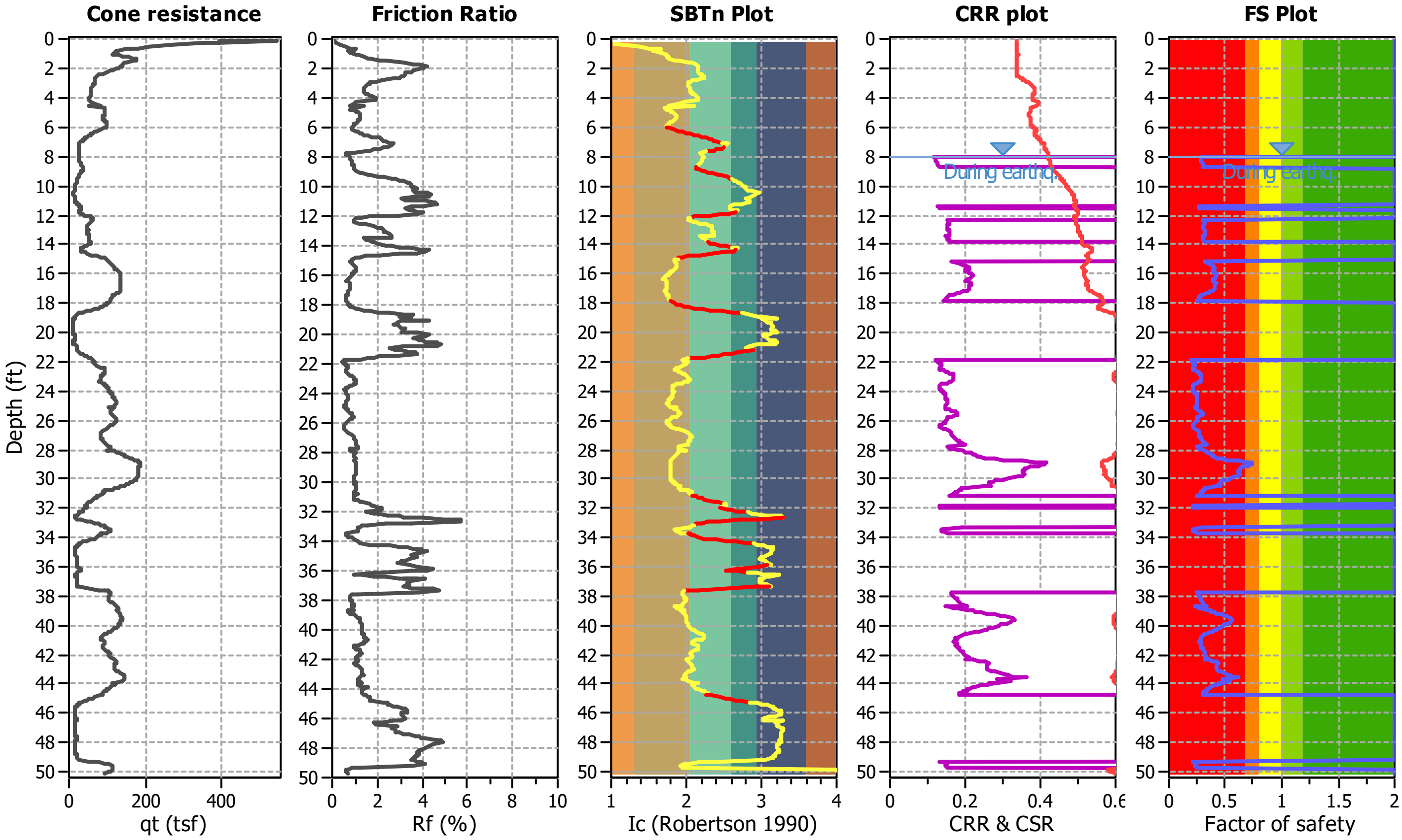
LIQUEFACTION ANALYSIS REPORT

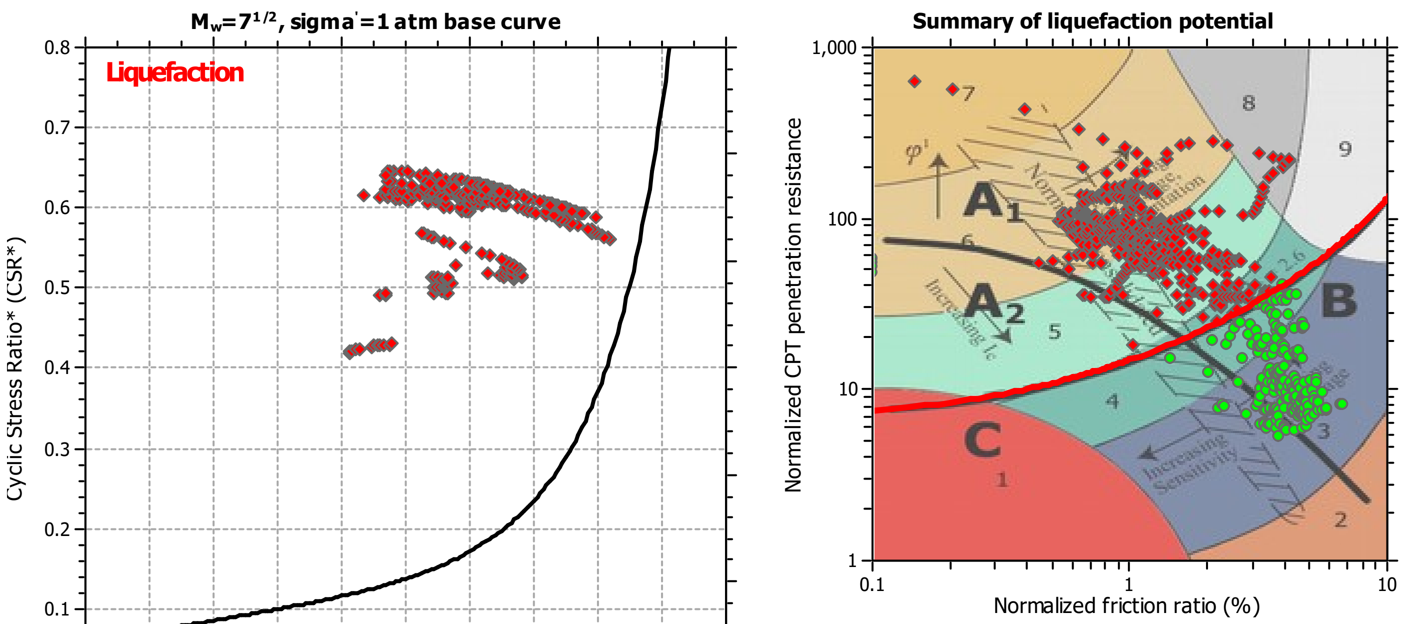
Project title : Proposed Warehouse **Location : Cerritos, CA**

CPT file : CPT-4

Input parameters and analysis data

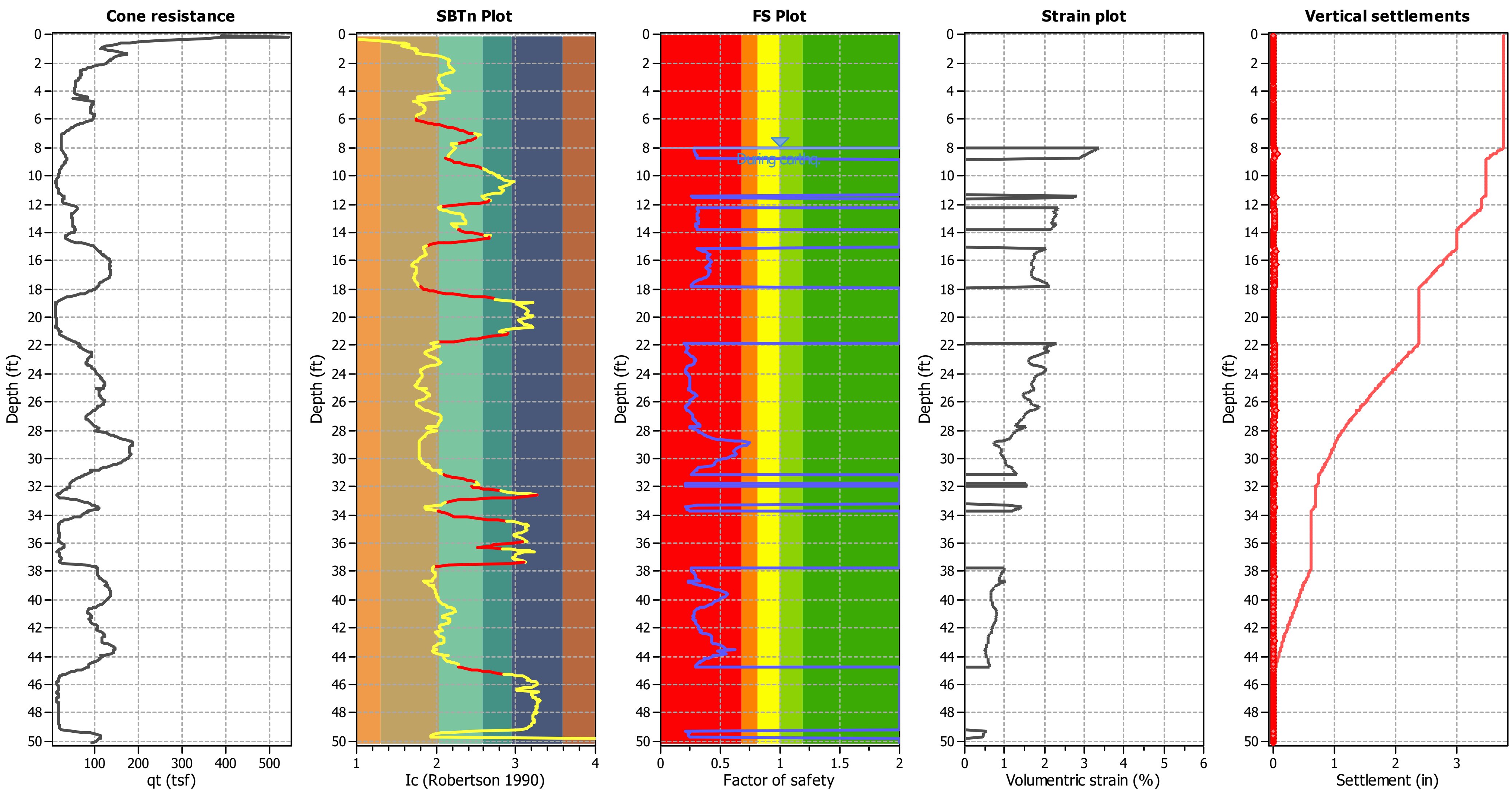
Analysis method:	B&I (2014)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.81	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.74	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)
8.00	82.40	0.28	3.37	0.86	0.01	8.04	82.52	0.28	3.36	0.86	0.01
8.07	83.12	0.28	3.33	0.86	0.02	8.14	84.20	0.28	3.29	0.86	0.03
8.23	85.34	0.29	3.24	0.86	0.04	8.27	85.92	0.29	3.22	0.86	0.01
8.44	89.14	0.29	3.09	0.86	0.06	8.48	90.06	0.29	3.06	0.86	0.02
8.52	91.12	0.30	3.02	0.86	0.01	8.57	92.16	0.30	2.98	0.85	0.02
8.62	93.30	0.30	2.94	0.85	0.02	8.70	95.18	0.31	2.88	0.85	0.03
8.74	95.85	0.31	2.86	0.85	0.01	8.83	96.43	2.00	0.00	0.85	0.00
8.88	96.33	2.00	0.00	0.85	0.00	8.96	95.91	2.00	0.00	0.85	0.00
9.01	95.80	2.00	0.00	0.85	0.00	9.10	94.99	2.00	0.00	0.85	0.00
9.14	94.13	2.00	0.00	0.85	0.00	9.23	91.83	2.00	0.00	0.84	0.00
9.27	90.38	2.00	0.00	0.84	0.00	9.37	87.63	2.00	0.00	0.84	0.00
9.45	86.87	2.00	0.00	0.84	0.00	9.49	85.98	2.00	0.00	0.84	0.00
9.54	26.62	2.00	0.00	0.84	0.00	9.58	26.32	2.00	0.00	0.84	0.00
9.69	25.12	2.00	0.00	0.84	0.00	9.74	23.99	2.00	0.00	0.83	0.00
9.79	22.77	2.00	0.00	0.83	0.00	9.89	20.79	2.00	0.00	0.83	0.00
9.98	19.64	2.00	0.00	0.83	0.00	10.02	18.76	2.00	0.00	0.83	0.00
10.07	18.96	2.00	0.00	0.83	0.00	10.12	18.45	2.00	0.00	0.83	0.00
10.24	16.59	2.00	0.00	0.83	0.00	10.29	15.50	2.00	0.00	0.83	0.00
10.37	13.55	2.00	0.00	0.82	0.00	10.42	12.94	2.00	0.00	0.82	0.00
10.50	13.35	2.00	0.00	0.82	0.00	10.56	14.01	2.00	0.00	0.82	0.00
10.63	14.42	2.00	0.00	0.82	0.00	10.69	15.31	2.00	0.00	0.82	0.00
10.77	15.26	2.00	0.00	0.82	0.00	10.82	16.93	2.00	0.00	0.82	0.00
10.89	18.23	2.00	0.00	0.82	0.00	10.95	19.29	2.00	0.00	0.81	0.00
11.01	19.23	2.00	0.00	0.81	0.00	11.04	19.71	2.00	0.00	0.81	0.00
11.13	20.13	2.00	0.00	0.81	0.00	11.17	20.87	2.00	0.00	0.81	0.00
11.24	23.78	2.00	0.00	0.81	0.00	11.31	28.51	2.00	0.00	0.81	0.00
11.39	92.13	0.26	2.82	0.81	0.03	11.44	93.48	0.26	2.77	0.81	0.02
11.55	93.76	0.26	2.76	0.80	0.04	11.59	32.20	2.00	0.00	0.80	0.00
11.64	30.86	2.00	0.00	0.80	0.00	11.70	28.96	2.00	0.00	0.80	0.00
11.78	28.87	2.00	0.00	0.80	0.00	11.82	28.82	2.00	0.00	0.80	0.00
11.91	92.54	2.00	0.00	0.80	0.00	11.96	96.35	2.00	0.00	0.80	0.00
12.02	104.35	2.00	0.00	0.80	0.00	12.11	110.66	2.00	0.00	0.79	0.00
12.15	110.08	2.00	0.00	0.79	0.00	12.25	109.01	2.00	0.00	0.79	0.00
12.29	108.98	0.31	2.33	0.79	0.01	12.35	109.01	0.30	2.32	0.79	0.02
12.45	112.19	0.32	2.25	0.79	0.03	12.50	113.07	0.32	2.23	0.79	0.01
12.54	113.07	0.32	2.23	0.79	0.01	12.65	111.20	0.31	2.26	0.79	0.03
12.70	109.57	0.30	2.29	0.78	0.01	12.76	109.00	0.30	2.30	0.78	0.02
12.81	109.19	0.30	2.30	0.78	0.01	12.90	110.70	0.31	2.26	0.78	0.02
12.96	111.79	0.31	2.23	0.78	0.02	13.01	112.48	0.31	2.22	0.78	0.01
13.07	112.37	0.31	2.22	0.78	0.02	13.18	112.89	0.31	2.20	0.78	0.03
13.25	112.94	0.31	2.20	0.78	0.02	13.30	113.38	0.31	2.18	0.77	0.01
13.38	114.15	0.32	2.17	0.77	0.02	13.43	107.91	0.29	2.29	0.77	0.01
13.51	108.95	0.29	2.27	0.77	0.02	13.57	109.96	0.30	2.24	0.77	0.02
13.64	111.53	0.30	2.21	0.77	0.02	13.69	112.70	0.31	2.18	0.77	0.01
13.76	113.33	0.31	2.16	0.77	0.02	13.79	113.38	0.31	2.16	0.77	0.01
13.85	116.59	2.00	0.00	0.77	0.00	13.94	117.80	2.00	0.00	0.76	0.00
13.99	115.43	2.00	0.00	0.76	0.00	14.05	111.09	2.00	0.00	0.76	0.00
14.12	105.08	2.00	0.00	0.76	0.00	14.20	38.08	2.00	0.00	0.76	0.00

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

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
14.25	33.45	2.00	0.00	0.76	0.00
14.38	33.59	2.00	0.00	0.76	0.00
14.50	100.26	2.00	0.00	0.75	0.00
14.68	113.49	2.00	0.00	0.75	0.00
14.77	114.66	2.00	0.00	0.75	0.00
14.92	116.42	2.00	0.00	0.75	0.00
15.03	111.67	2.00	0.00	0.75	0.00
15.31	125.43	0.36	1.87	0.74	0.04
15.44	130.74	0.39	1.79	0.74	0.01
15.57	133.77	0.41	1.74	0.74	0.01
15.71	131.29	0.39	1.77	0.73	0.02
15.84	131.78	0.39	1.75	0.73	0.01
15.97	134.69	0.41	1.71	0.73	0.01
16.10	136.37	0.43	1.68	0.73	0.02
16.37	134.53	0.41	1.69	0.72	0.02
16.50	131.31	0.38	1.73	0.72	0.02
16.67	135.35	0.41	1.67	0.72	0.03
16.75	135.28	0.41	1.67	0.72	0.01
16.91	134.37	0.40	1.67	0.71	0.02
17.03	133.97	0.40	1.68	0.71	0.02
17.16	131.04	0.38	1.71	0.71	0.01
17.29	125.96	0.34	1.78	0.71	0.02
17.40	118.66	0.31	1.89	0.71	0.02
17.52	111.80	0.28	2.01	0.70	0.01
17.65	108.37	0.27	2.07	0.70	0.01
17.80	105.67	0.26	2.12	0.70	0.01
17.95	105.83	2.00	0.00	0.70	0.00
18.05	111.14	2.00	0.00	0.69	0.00
18.18	121.77	2.00	0.00	0.69	0.00
18.31	124.43	2.00	0.00	0.69	0.00
18.45	112.52	2.00	0.00	0.69	0.00
18.58	34.23	2.00	0.00	0.69	0.00
18.71	22.76	2.00	0.00	0.68	0.00
18.98	12.00	2.00	0.00	0.68	0.00
19.06	11.26	2.00	0.00	0.68	0.00
19.17	10.07	2.00	0.00	0.68	0.00
19.30	9.05	2.00	0.00	0.67	0.00
19.46	8.58	2.00	0.00	0.67	0.00
19.59	8.57	2.00	0.00	0.67	0.00
19.77	8.91	2.00	0.00	0.66	0.00
19.88	8.90	2.00	0.00	0.66	0.00
19.98	9.43	2.00	0.00	0.66	0.00
20.08	10.84	2.00	0.00	0.66	0.00
20.22	12.43	2.00	0.00	0.66	0.00
20.37	11.69	2.00	0.00	0.65	0.00
20.52	10.78	2.00	0.00	0.65	0.00
20.62	9.78	2.00	0.00	0.65	0.00
20.74	9.86	2.00	0.00	0.65	0.00

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
14.34	33.64	2.00	0.00	0.76	0.00
14.45	33.80	2.00	0.00	0.76	0.00
14.59	108.36	2.00	0.00	0.75	0.00
14.72	115.78	2.00	0.00	0.75	0.00
14.83	114.66	2.00	0.00	0.75	0.00
14.96	116.47	2.00	0.00	0.75	0.00
15.13	115.81	0.31	2.05	0.74	0.03
15.40	129.16	0.38	1.81	0.74	0.02
15.53	133.49	0.41	1.74	0.74	0.02
15.63	133.20	0.40	1.74	0.74	0.01
15.78	130.61	0.38	1.77	0.73	0.02
15.91	133.37	0.40	1.73	0.73	0.02
16.02	136.18	0.43	1.69	0.73	0.01
16.30	134.30	0.41	1.70	0.72	0.04
16.42	132.76	0.39	1.72	0.72	0.01
16.55	132.86	0.39	1.71	0.72	0.01
16.72	135.26	0.41	1.67	0.72	0.01
16.82	134.62	0.41	1.67	0.71	0.01
16.94	134.49	0.40	1.67	0.71	0.01
17.12	132.20	0.39	1.70	0.71	0.02
17.20	129.71	0.37	1.73	0.71	0.01
17.33	123.54	0.33	1.82	0.71	0.01
17.48	114.02	0.29	1.97	0.70	0.02
17.61	108.98	0.27	2.06	0.70	0.02
17.74	106.92	0.26	2.10	0.70	0.02
17.86	105.04	0.25	2.13	0.70	0.02
18.00	107.69	2.00	0.00	0.69	0.00
18.12	117.02	2.00	0.00	0.69	0.00
18.25	124.68	2.00	0.00	0.69	0.00
18.39	120.27	2.00	0.00	0.69	0.00
18.52	103.31	2.00	0.00	0.69	0.00
18.64	27.76	2.00	0.00	0.68	0.00
18.90	14.19	2.00	0.00	0.68	0.00
18.99	8.90	2.00	0.00	0.68	0.00
19.10	11.16	2.00	0.00	0.68	0.00
19.26	9.33	2.00	0.00	0.67	0.00
19.37	8.59	2.00	0.00	0.67	0.00
19.50	8.58	2.00	0.00	0.67	0.00
19.68	8.74	2.00	0.00	0.67	0.00
19.82	8.91	2.00	0.00	0.66	0.00
19.93	8.89	2.00	0.00	0.66	0.00
20.03	9.96	2.00	0.00	0.66	0.00
20.17	12.26	2.00	0.00	0.66	0.00
20.31	12.15	2.00	0.00	0.66	0.00
20.47	11.14	2.00	0.00	0.65	0.00
20.57	10.24	2.00	0.00	0.65	0.00
20.68	9.51	2.00	0.00	0.65	0.00
20.83	13.66	2.00	0.00	0.65	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)
20.89	16.31	2.00	0.00	0.65	0.00	21.01	19.55	2.00	0.00	0.64	0.00
21.05	19.37	2.00	0.00	0.64	0.00	21.12	18.38	2.00	0.00	0.64	0.00
21.18	17.65	2.00	0.00	0.64	0.00	21.24	18.00	2.00	0.00	0.64	0.00
21.37	24.92	2.00	0.00	0.64	0.00	21.45	88.31	2.00	0.00	0.64	0.00
21.50	92.08	2.00	0.00	0.64	0.00	21.57	96.16	2.00	0.00	0.63	0.00
21.64	100.27	2.00	0.00	0.63	0.00	21.71	102.32	2.00	0.00	0.63	0.00
21.76	90.69	2.00	0.00	0.63	0.00	21.85	82.52	2.00	0.00	0.63	0.00
21.89	87.18	0.20	2.32	0.63	0.01	21.97	92.16	0.21	2.19	0.63	0.02
22.02	94.67	0.21	2.13	0.63	0.01	22.08	98.02	0.22	2.05	0.63	0.01
22.12	96.98	0.22	2.07	0.63	0.01	22.20	100.66	0.23	1.99	0.62	0.02
22.26	99.89	0.23	2.00	0.62	0.02	22.35	96.12	0.22	2.08	0.62	0.02
22.43	93.56	0.21	2.13	0.62	0.02	22.47	94.88	0.21	2.10	0.62	0.01
22.52	97.26	0.22	2.04	0.62	0.01	22.59	101.62	0.23	1.95	0.62	0.02
22.66	106.75	0.24	1.85	0.62	0.02	22.74	111.59	0.26	1.76	0.61	0.02
22.79	114.05	0.27	1.72	0.61	0.01	22.87	117.39	0.28	1.66	0.61	0.02
22.92	118.45	0.28	1.65	0.61	0.01	22.99	119.55	0.29	1.63	0.61	0.02
23.04	119.61	0.29	1.62	0.61	0.01	23.12	119.74	0.29	1.62	0.61	0.02
23.18	119.52	0.29	1.62	0.61	0.01	23.27	118.44	0.28	1.63	0.61	0.02
23.32	117.04	0.28	1.65	0.60	0.01	23.40	112.37	0.26	1.72	0.60	0.02
23.45	109.09	0.25	1.77	0.60	0.01	23.53	100.44	0.22	1.92	0.60	0.02
23.58	96.10	0.21	2.01	0.60	0.01	23.68	94.00	0.21	2.05	0.60	0.03
23.72	94.44	0.21	2.04	0.60	0.01	23.76	93.67	0.21	2.05	0.60	0.01
23.87	96.01	0.21	1.99	0.60	0.03	23.93	97.53	0.22	1.96	0.59	0.01
23.98	98.83	0.22	1.93	0.59	0.01	24.07	102.76	0.23	1.85	0.59	0.02
24.12	104.35	0.23	1.82	0.59	0.01	24.20	107.05	0.24	1.77	0.59	0.02
24.24	107.02	0.24	1.77	0.59	0.01	24.32	106.97	0.24	1.76	0.59	0.02
24.38	107.06	0.24	1.76	0.59	0.01	24.44	107.39	0.24	1.75	0.59	0.01
24.51	108.07	0.24	1.73	0.58	0.02	24.55	109.37	0.25	1.71	0.58	0.01
24.64	110.33	0.25	1.69	0.58	0.02	24.71	110.64	0.25	1.68	0.58	0.01
24.77	110.30	0.25	1.68	0.58	0.01	24.82	109.79	0.25	1.69	0.58	0.01
24.89	109.44	0.25	1.69	0.58	0.01	24.95	108.50	0.24	1.70	0.58	0.01
25.08	106.80	0.24	1.73	0.57	0.03	25.09	108.07	0.24	1.70	0.57	0.00
25.14	105.91	0.24	1.74	0.57	0.01	25.20	108.03	0.24	1.70	0.57	0.01
25.26	112.31	0.25	1.63	0.57	0.01	25.34	117.04	0.27	1.56	0.57	0.01
25.39	120.07	0.28	1.51	0.57	0.01	25.49	122.18	0.29	1.48	0.57	0.02
25.53	123.81	0.30	1.46	0.57	0.01	25.62	122.68	0.29	1.47	0.57	0.02
25.68	120.98	0.29	1.49	0.56	0.01	25.75	116.29	0.27	1.55	0.56	0.01
25.84	110.41	0.25	1.63	0.56	0.02	25.89	109.47	0.24	1.64	0.56	0.01
26.02	108.37	0.24	1.65	0.56	0.03	26.08	107.87	0.24	1.66	0.56	0.01
26.15	106.16	0.23	1.68	0.56	0.01	26.20	102.76	0.23	1.74	0.56	0.01
26.28	98.50	0.22	1.81	0.55	0.02	26.33	95.69	0.21	1.86	0.55	0.01
26.41	94.35	0.21	1.88	0.55	0.02	26.59	102.08	0.22	1.73	0.55	0.04
26.65	105.87	0.23	1.66	0.55	0.01	26.78	114.63	0.26	1.52	0.55	0.02
26.91	116.56	0.27	1.49	0.54	0.02	26.99	116.39	0.27	1.49	0.54	0.01
27.11	117.46	0.27	1.47	0.54	0.02	27.17	118.72	0.27	1.45	0.54	0.01
27.24	120.16	0.28	1.43	0.54	0.01	27.30	121.52	0.29	1.41	0.54	0.01
27.36	122.86	0.29	1.39	0.54	0.01	27.43	124.28	0.30	1.37	0.54	0.01
27.49	125.43	0.30	1.35	0.53	0.01	27.56	127.78	0.31	1.32	0.53	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)
27.62	130.22	0.33	1.29	0.53	0.01	27.70	130.58	0.33	1.29	0.53	0.01
27.75	109.64	0.24	1.55	0.53	0.01	27.83	111.53	0.25	1.52	0.53	0.02
27.88	113.96	0.26	1.48	0.53	0.01	28.03	128.52	0.32	1.29	0.52	0.02
28.04	129.12	0.32	1.29	0.52	0.00	28.11	129.46	0.32	1.28	0.52	0.01
28.16	131.53	0.34	1.26	0.52	0.01	28.24	135.01	0.36	1.22	0.52	0.01
28.31	136.71	0.37	1.20	0.52	0.01	28.37	138.37	0.38	1.18	0.52	0.01
28.41	139.32	0.39	1.17	0.52	0.01	28.48	140.60	0.40	1.15	0.52	0.01
28.55	143.48	0.43	1.13	0.52	0.01	28.64	148.21	0.48	1.08	0.51	0.01
28.68	151.52	0.52	1.05	0.51	0.01	28.76	158.14	0.62	0.94	0.51	0.01
28.82	161.02	0.68	0.85	0.51	0.01	28.92	163.76	0.74	0.73	0.51	0.01
28.98	163.65	0.74	0.74	0.51	0.01	29.03	163.06	0.72	0.77	0.51	0.00
29.08	161.98	0.70	0.82	0.51	0.00	29.20	159.95	0.65	0.87	0.51	0.01
29.26	159.18	0.64	0.89	0.50	0.01	29.30	158.75	0.63	0.91	0.50	0.01
29.36	158.48	0.63	0.91	0.50	0.01	29.42	158.18	0.62	0.92	0.50	0.01
29.48	157.81	0.61	0.93	0.50	0.01	29.58	158.23	0.62	0.91	0.50	0.01
29.65	158.77	0.63	0.90	0.50	0.01	29.71	158.60	0.63	0.90	0.50	0.01
29.78	158.16	0.62	0.91	0.50	0.01	29.83	157.01	0.60	0.94	0.49	0.01
29.96	153.63	0.54	0.99	0.49	0.02	30.02	151.98	0.52	1.00	0.49	0.01
30.09	149.85	0.49	1.02	0.49	0.01	30.14	148.59	0.48	1.03	0.49	0.01
30.20	147.36	0.46	1.03	0.49	0.01	30.27	146.12	0.45	1.04	0.49	0.01
30.32	146.01	0.45	1.04	0.49	0.01	30.40	145.93	0.45	1.04	0.48	0.01
30.45	146.35	0.45	1.03	0.48	0.01	30.54	143.56	0.42	1.05	0.48	0.01
30.59	131.33	0.33	1.16	0.48	0.01	30.67	128.04	0.31	1.19	0.48	0.01
30.72	127.16	0.31	1.20	0.48	0.01	30.80	125.03	0.30	1.21	0.48	0.01
30.82	125.45	0.30	1.21	0.48	0.00	30.85	123.92	0.29	1.22	0.48	0.00
30.93	121.93	0.28	1.24	0.48	0.01	31.00	119.40	0.27	1.27	0.47	0.01
31.12	114.34	0.25	1.32	0.47	0.02	31.16	112.75	2.00	0.00	0.47	0.00
31.20	111.71	2.00	0.00	0.47	0.00	31.25	110.98	2.00	0.00	0.47	0.00
31.33	109.80	2.00	0.00	0.47	0.00	31.37	109.22	2.00	0.00	0.47	0.00
31.51	104.08	2.00	0.00	0.47	0.00	31.53	102.19	2.00	0.00	0.47	0.00
31.57	100.54	2.00	0.00	0.46	0.00	31.65	97.22	2.00	0.00	0.46	0.00
31.72	96.16	2.00	0.00	0.46	0.00	31.80	96.23	0.21	1.54	0.46	0.02
31.84	95.23	0.20	1.55	0.46	0.01	31.94	94.54	0.20	1.56	0.46	0.02
31.99	93.17	2.00	0.00	0.46	0.00	32.03	91.99	2.00	0.00	0.46	0.00
32.12	89.40	2.00	0.00	0.46	0.00	32.17	88.30	2.00	0.00	0.45	0.00
32.22	27.00	2.00	0.00	0.45	0.00	32.30	21.84	2.00	0.00	0.45	0.00
32.36	19.04	2.00	0.00	0.45	0.00	32.45	14.40	2.00	0.00	0.45	0.00
32.52	12.32	2.00	0.00	0.45	0.00	32.57	10.86	2.00	0.00	0.45	0.00
32.61	10.77	2.00	0.00	0.45	0.00	32.75	16.19	2.00	0.00	0.44	0.00
32.82	24.50	2.00	0.00	0.44	0.00	32.88	95.10	2.00	0.00	0.44	0.00
32.94	104.14	2.00	0.00	0.44	0.00	33.01	110.80	2.00	0.00	0.44	0.00
33.06	116.15	2.00	0.00	0.44	0.00	33.14	121.72	2.00	0.00	0.44	0.00
33.19	124.52	2.00	0.00	0.44	0.00	33.26	126.55	0.30	1.09	0.44	0.01
33.33	127.29	0.31	1.08	0.44	0.01	33.41	101.37	0.22	1.37	0.43	0.01
33.45	98.70	0.21	1.41	0.43	0.01	33.53	100.50	0.21	1.38	0.43	0.01
33.59	104.49	0.22	1.32	0.43	0.01	33.66	109.55	0.24	1.26	0.43	0.01
33.68	112.78	0.25	1.22	0.43	0.00	33.75	113.50	2.00	0.00	0.43	0.00
33.83	114.32	2.00	0.00	0.43	0.00	33.87	114.15	2.00	0.00	0.43	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)
33.96	112.94	2.00	0.00	0.42	0.00	34.09	109.40	2.00	0.00	0.42	0.00
34.19	102.45	2.00	0.00	0.42	0.00	34.27	96.54	2.00	0.00	0.42	0.00
34.31	29.71	2.00	0.00	0.42	0.00	34.36	24.10	2.00	0.00	0.42	0.00
34.44	20.05	2.00	0.00	0.42	0.00	34.49	17.47	2.00	0.00	0.42	0.00
34.54	15.34	2.00	0.00	0.41	0.00	34.61	13.75	2.00	0.00	0.41	0.00
34.67	12.24	2.00	0.00	0.41	0.00	34.73	11.94	2.00	0.00	0.41	0.00
34.80	11.48	2.00	0.00	0.41	0.00	34.91	11.76	2.00	0.00	0.41	0.00
34.98	11.76	2.00	0.00	0.41	0.00	35.10	11.74	2.00	0.00	0.41	0.00
35.24	12.91	2.00	0.00	0.40	0.00	35.29	13.58	2.00	0.00	0.40	0.00
35.36	14.47	2.00	0.00	0.40	0.00	35.42	14.91	2.00	0.00	0.40	0.00
35.48	14.82	2.00	0.00	0.40	0.00	35.62	13.61	2.00	0.00	0.40	0.00
35.68	13.30	2.00	0.00	0.40	0.00	35.86	13.13	2.00	0.00	0.39	0.00
35.93	14.38	2.00	0.00	0.39	0.00	35.99	16.61	2.00	0.00	0.39	0.00
36.12	22.85	2.00	0.00	0.39	0.00	36.18	23.73	2.00	0.00	0.39	0.00
36.26	78.56	2.00	0.00	0.39	0.00	36.30	19.06	2.00	0.00	0.38	0.00
36.39	16.30	2.00	0.00	0.38	0.00	36.45	14.39	2.00	0.00	0.38	0.00
36.53	13.19	2.00	0.00	0.38	0.00	36.55	10.76	2.00	0.00	0.38	0.00
36.56	12.97	2.00	0.00	0.38	0.00	36.57	10.10	2.00	0.00	0.38	0.00
36.66	12.73	2.00	0.00	0.38	0.00	36.71	13.76	2.00	0.00	0.38	0.00
36.75	14.57	2.00	0.00	0.38	0.00	36.85	15.88	2.00	0.00	0.38	0.00
36.89	15.88	2.00	0.00	0.37	0.00	36.96	15.87	2.00	0.00	0.37	0.00
37.01	16.15	2.00	0.00	0.37	0.00	37.10	16.29	2.00	0.00	0.37	0.00
37.15	16.28	2.00	0.00	0.37	0.00	37.26	14.86	2.00	0.00	0.37	0.00
37.30	14.41	2.00	0.00	0.37	0.00	37.36	15.28	2.00	0.00	0.37	0.00
37.41	20.01	2.00	0.00	0.37	0.00	37.56	116.45	2.00	0.00	0.36	0.00
37.61	118.45	2.00	0.00	0.36	0.00	37.67	116.60	2.00	0.00	0.36	0.00
37.72	115.44	2.00	0.00	0.36	0.00	37.77	115.20	0.25	1.00	0.36	0.01
37.82	115.49	0.25	0.99	0.36	0.01	37.88	115.77	0.26	0.99	0.36	0.01
37.94	116.83	0.26	0.98	0.36	0.01	38.04	119.26	0.27	0.95	0.36	0.01
38.11	121.47	0.28	0.93	0.35	0.01	38.16	122.58	0.28	0.92	0.35	0.01
38.21	123.35	0.28	0.91	0.35	0.01	38.26	124.01	0.29	0.90	0.35	0.01
38.38	124.99	0.29	0.89	0.35	0.01	38.43	125.21	0.29	0.88	0.35	0.00
38.48	125.73	0.30	0.88	0.35	0.01	38.54	126.77	0.30	0.87	0.35	0.01
38.65	131.76	0.33	0.83	0.34	0.01	38.69	106.40	0.23	1.04	0.34	0.00
38.76	110.00	0.24	1.00	0.34	0.01	38.82	117.57	0.26	0.93	0.34	0.01
38.87	122.24	0.28	0.89	0.34	0.01	38.98	132.59	0.33	0.81	0.34	0.01
39.01	137.05	0.36	0.78	0.34	0.00	39.05	138.13	0.37	0.77	0.34	0.00
39.11	142.02	0.40	0.74	0.34	0.01	39.18	145.19	0.43	0.72	0.34	0.01
39.25	148.79	0.47	0.70	0.33	0.01	39.31	150.86	0.49	0.69	0.33	0.00
39.39	152.71	0.52	0.68	0.33	0.01	39.53	154.89	0.55	0.66	0.33	0.01
39.61	155.46	0.56	0.65	0.33	0.01	39.65	155.56	0.56	0.65	0.33	0.00
39.70	155.30	0.55	0.65	0.33	0.00	39.78	154.48	0.54	0.65	0.33	0.01
39.86	152.89	0.52	0.66	0.32	0.01	39.94	151.01	0.49	0.67	0.32	0.01
39.98	150.01	0.48	0.67	0.32	0.00	40.11	147.70	0.46	0.68	0.32	0.01
40.15	147.09	0.45	0.68	0.32	0.00	40.20	146.64	0.44	0.68	0.32	0.00
40.23	146.12	0.44	0.68	0.32	0.00	40.32	144.68	0.42	0.68	0.32	0.01
40.37	143.30	0.41	0.69	0.32	0.00	40.45	140.17	0.39	0.70	0.31	0.01
40.50	137.84	0.37	0.72	0.31	0.00	40.59	132.86	0.33	0.74	0.31	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)
40.63	130.19	0.32	0.76	0.31	0.00	40.68	127.46	0.30	0.77	0.31	0.00
40.80	122.55	0.28	0.80	0.31	0.01	40.85	121.45	0.28	0.81	0.31	0.00
40.92	121.43	0.28	0.80	0.31	0.01	40.98	122.68	0.28	0.79	0.31	0.01
41.04	121.20	0.28	0.80	0.30	0.01	41.11	119.36	0.27	0.81	0.30	0.01
41.17	120.19	0.27	0.80	0.30	0.01	41.24	121.87	0.28	0.79	0.30	0.01
41.30	123.20	0.28	0.77	0.30	0.01	41.35	120.45	0.27	0.79	0.30	0.00
41.42	122.80	0.28	0.77	0.30	0.01	41.50	123.92	0.29	0.76	0.30	0.01
41.55	124.37	0.29	0.76	0.30	0.00	41.64	125.46	0.29	0.74	0.29	0.01
41.68	126.23	0.30	0.74	0.29	0.00	41.77	127.93	0.31	0.72	0.29	0.01
41.81	128.60	0.31	0.72	0.29	0.00	41.87	129.14	0.31	0.71	0.29	0.00
41.94	129.28	0.31	0.71	0.29	0.01	42.06	130.55	0.32	0.70	0.29	0.01
42.20	133.36	0.34	0.67	0.28	0.01	42.31	131.11	0.32	0.68	0.28	0.01
42.39	136.43	0.36	0.65	0.28	0.01	42.44	139.98	0.38	0.63	0.28	0.00
42.52	143.86	0.42	0.61	0.28	0.01	42.58	145.46	0.43	0.60	0.28	0.00
42.66	145.43	0.43	0.60	0.28	0.01	42.71	144.30	0.42	0.60	0.28	0.00
42.89	144.44	0.42	0.59	0.27	0.01	42.97	144.42	0.42	0.59	0.27	0.01
43.03	144.36	0.42	0.59	0.27	0.00	43.09	146.28	0.44	0.58	0.27	0.00
43.16	148.46	0.46	0.56	0.27	0.00	43.23	151.17	0.50	0.55	0.27	0.00
43.36	153.63	0.53	0.54	0.27	0.01	43.41	154.09	0.54	0.53	0.26	0.00
43.49	155.28	0.55	0.52	0.26	0.01	43.54	159.11	0.62	0.47	0.26	0.00
43.62	149.13	0.47	0.54	0.26	0.00	43.67	149.25	0.47	0.54	0.26	0.00
43.74	152.06	0.51	0.53	0.26	0.00	43.87	154.70	0.55	0.51	0.26	0.01
43.90	149.03	0.47	0.54	0.26	0.00	43.91	145.53	0.43	0.55	0.26	0.00
43.98	148.82	0.47	0.53	0.25	0.00	44.06	144.85	0.43	0.55	0.25	0.01
44.11	142.10	0.40	0.56	0.25	0.00	44.20	138.11	0.37	0.57	0.25	0.01
44.25	136.79	0.36	0.58	0.25	0.00	44.34	134.35	0.34	0.58	0.25	0.01
44.41	132.71	0.33	0.59	0.25	0.01	44.46	131.66	0.33	0.59	0.25	0.00
44.51	129.65	0.32	0.60	0.25	0.00	44.60	126.37	0.30	0.61	0.24	0.01
44.65	125.40	0.30	0.62	0.24	0.00	44.73	125.16	0.30	0.61	0.24	0.01
44.77	124.50	2.00	0.00	0.24	0.00	44.82	123.48	2.00	0.00	0.24	0.00
44.94	113.68	2.00	0.00	0.24	0.00	44.99	107.16	2.00	0.00	0.24	0.00
45.05	100.56	2.00	0.00	0.24	0.00	45.12	32.70	2.00	0.00	0.24	0.00
45.18	28.09	2.00	0.00	0.23	0.00	45.25	22.69	2.00	0.00	0.23	0.00
45.32	18.47	2.00	0.00	0.23	0.00	45.39	15.85	2.00	0.00	0.23	0.00
45.44	13.80	2.00	0.00	0.23	0.00	45.52	12.31	2.00	0.00	0.23	0.00
45.57	11.56	2.00	0.00	0.23	0.00	45.65	11.29	2.00	0.00	0.23	0.00
45.70	10.61	2.00	0.00	0.23	0.00	45.78	10.14	2.00	0.00	0.22	0.00
45.84	9.40	2.00	0.00	0.22	0.00	45.92	9.20	2.00	0.00	0.22	0.00
45.96	9.06	2.00	0.00	0.22	0.00	46.05	9.12	2.00	0.00	0.22	0.00
46.10	9.78	2.00	0.00	0.22	0.00	46.18	9.90	2.00	0.00	0.22	0.00
46.23	10.77	2.00	0.00	0.22	0.00	46.32	11.56	2.00	0.00	0.21	0.00
46.37	12.03	2.00	0.00	0.21	0.00	46.45	12.02	2.00	0.00	0.21	0.00
46.56	8.49	2.00	0.00	0.21	0.00	46.59	9.35	2.00	0.00	0.21	0.00
46.67	9.67	2.00	0.00	0.21	0.00	46.72	9.66	2.00	0.00	0.21	0.00
46.83	9.69	2.00	0.00	0.21	0.00	46.90	9.72	2.00	0.00	0.21	0.00
46.95	9.25	2.00	0.00	0.20	0.00	47.03	9.38	2.00	0.00	0.20	0.00
47.08	9.11	2.00	0.00	0.20	0.00	47.15	9.31	2.00	0.00	0.20	0.00
47.21	9.56	2.00	0.00	0.20	0.00	47.29	10.21	2.00	0.00	0.20	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)
47.34	10.54	2.00	0.00	0.20	0.00	47.48	11.32	2.00	0.00	0.20	0.00
47.54	11.45	2.00	0.00	0.19	0.00	47.61	11.71	2.00	0.00	0.19	0.00
47.66	11.70	2.00	0.00	0.19	0.00	47.79	11.55	2.00	0.00	0.19	0.00
47.87	11.09	2.00	0.00	0.19	0.00	47.93	10.88	2.00	0.00	0.19	0.00
48.00	11.07	2.00	0.00	0.19	0.00	48.05	10.87	2.00	0.00	0.19	0.00
48.12	11.06	2.00	0.00	0.18	0.00	48.19	11.26	2.00	0.00	0.18	0.00
48.24	11.31	2.00	0.00	0.18	0.00	48.32	11.44	2.00	0.00	0.18	0.00
48.39	11.43	2.00	0.00	0.18	0.00	48.45	11.17	2.00	0.00	0.18	0.00
48.50	10.97	2.00	0.00	0.18	0.00	48.58	10.89	2.00	0.00	0.18	0.00
48.66	10.88	2.00	0.00	0.18	0.00	48.71	11.21	2.00	0.00	0.17	0.00
48.78	11.40	2.00	0.00	0.17	0.00	48.85	11.65	2.00	0.00	0.17	0.00
48.91	11.98	2.00	0.00	0.17	0.00	48.98	12.76	2.00	0.00	0.17	0.00
49.04	13.35	2.00	0.00	0.17	0.00	49.11	13.93	2.00	0.00	0.17	0.00
49.18	15.38	2.00	0.00	0.17	0.00	49.29	93.55	0.20	0.57	0.16	0.01
49.38	105.95	0.23	0.49	0.16	0.00	49.42	110.34	0.24	0.47	0.16	0.00
49.50	106.12	0.23	0.49	0.16	0.00	49.63	107.61	0.23	0.47	0.16	0.01
49.69	110.99	0.24	0.46	0.16	0.00	49.75	114.90	0.26	0.44	0.16	0.00
49.83	85.47	2.00	0.00	0.16	0.00	49.87	84.04	2.00	0.00	0.15	0.00
49.95	80.30	2.00	0.00	0.15	0.00	50.00	76.18	2.00	0.00	0.15	0.00
50.14	68.78	2.00	0.00	0.15	0.00						

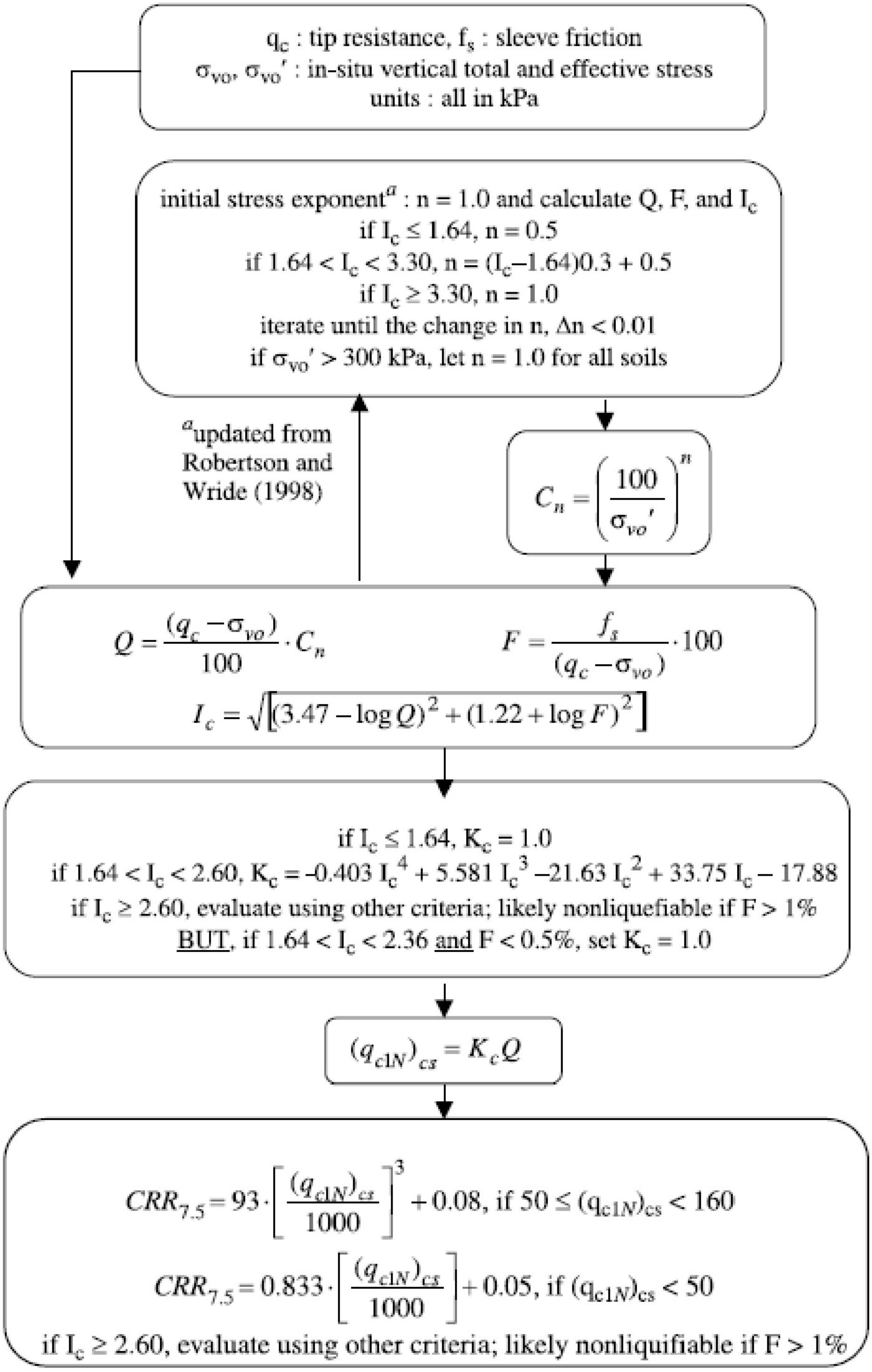
Total estimated settlement: 3.77

Abbreviations

$Q_{tn,cs}$: Equivalent dean 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

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

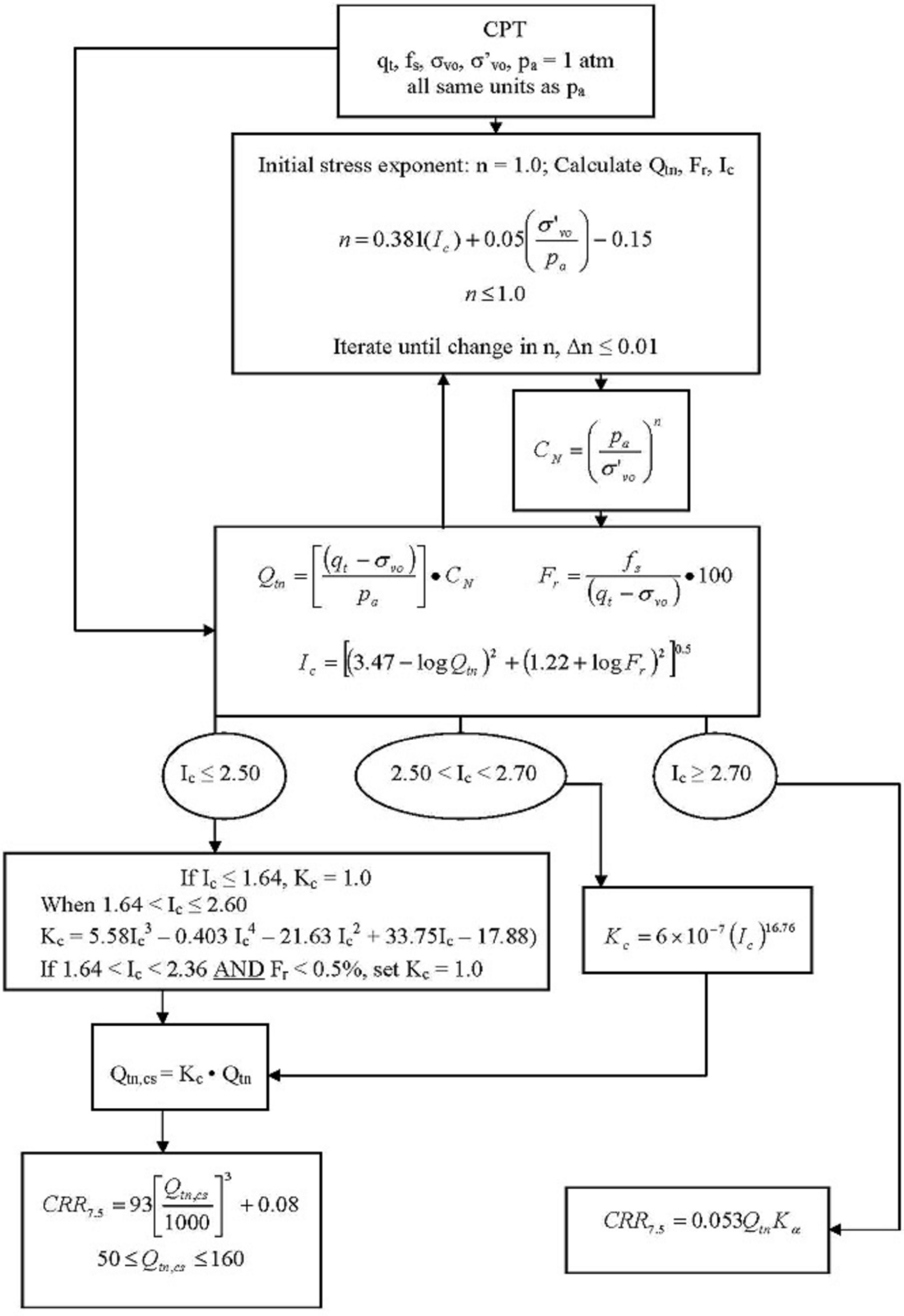
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of 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

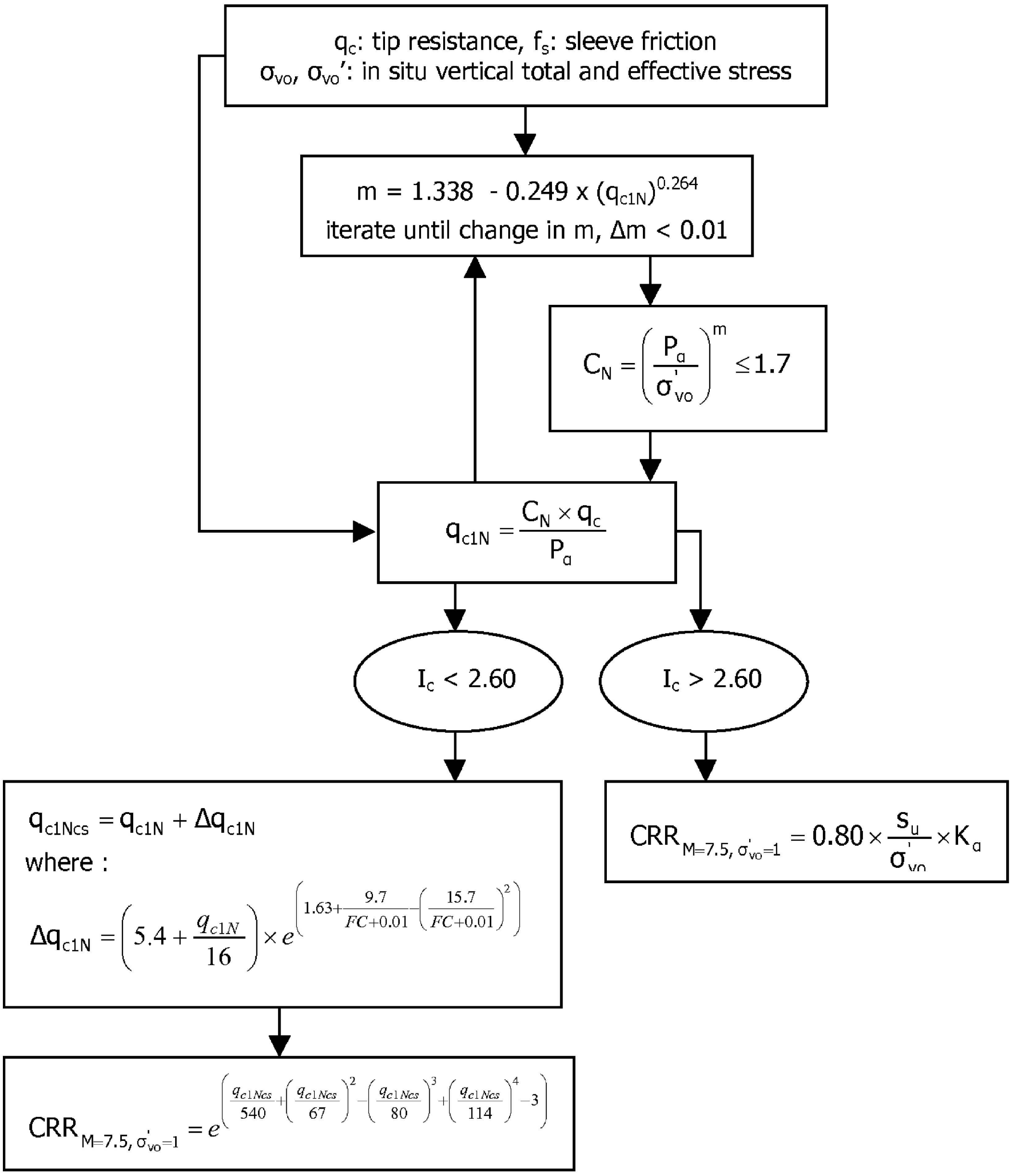
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

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]:

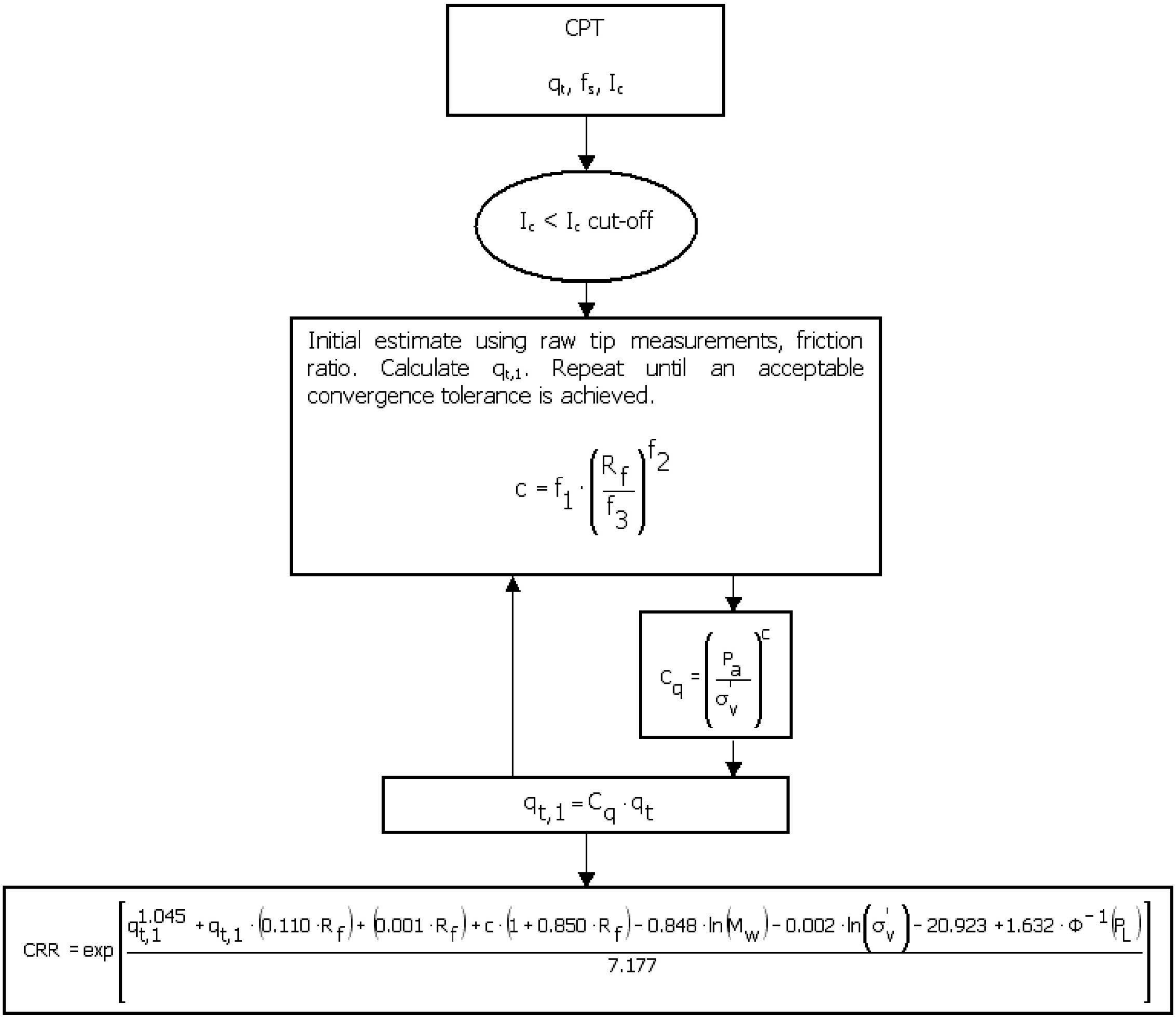


[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

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

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

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

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

$$q_{c1N} = C_{N\sigma}\frac{q_c}{P_a}$$

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

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

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

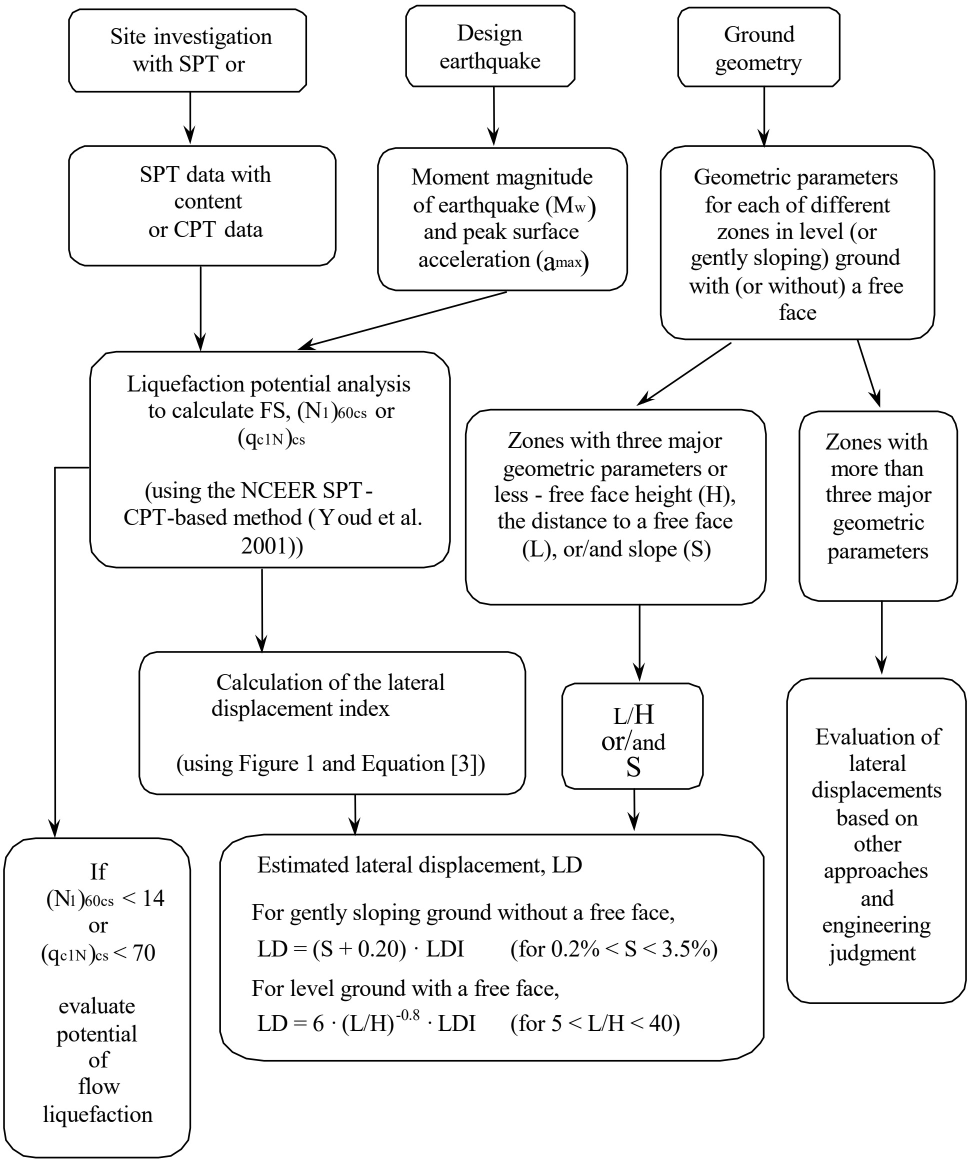
$$I_c = \left[\left(3.47-\log(Q)\right)^2+\left(1.22+\log(F)\right)^2\right]^{0.5}$$

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

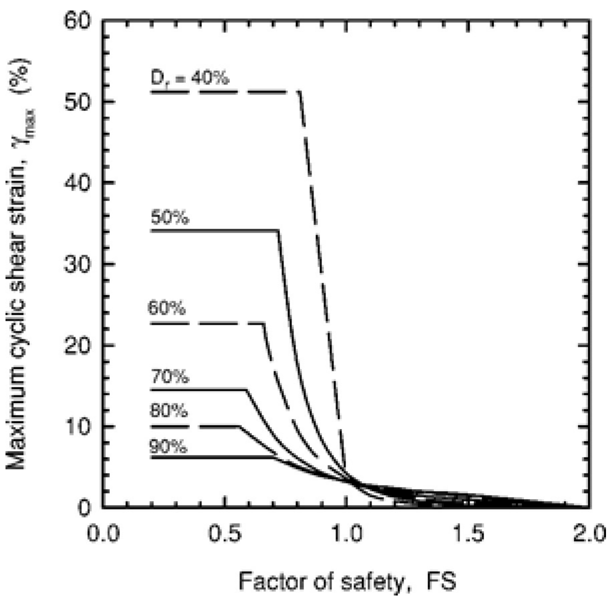
$$F = \left(\frac{f_s}{q_c-\sigma_{vc}}\right)\cdot 100\%$$

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

Average shear stress, τ_{av}

$$\tau_{av} = CSR^{*} \cdot \sigma_{v0}' = 0.65 \cdot \frac{\alpha_{max}}{g} \cdot \sigma_{v0} \cdot \tau_{a}$$

↓

Estimate small shear strain modulus, G_0

$$G_0 = 0.0188 \cdot \left[10^{(0.55 I_c - 1.68)}\right] \cdot (q_t - \sigma_v)$$

↓

Estimate shear strain amplitude, γ

(based on Pradel (1998))

$$\gamma = \left[\frac{1 + \alpha \cdot e^{bR}}{1 + \alpha}\right] \cdot R \cdot 100\ (\%)$$

$$R = \frac{\tau_{av}}{G_0}\ \text{(Note } \tau_{av} \text{ and } G_0 \text{ same units)}$$

$$\alpha = 0.0389 \cdot \left(\frac{\sigma_v}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{\sigma_v}{p_a}\right)$$

↓

Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60cs}}{20}\right]^{-1.20}$$

$$(N_1)_{60,cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]}$$

↓

Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

↓

Seismic settlement, s

$$s = 2 \cdot \int_0^{GWT} \Sigma_{vol} \cdot dz$$

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.

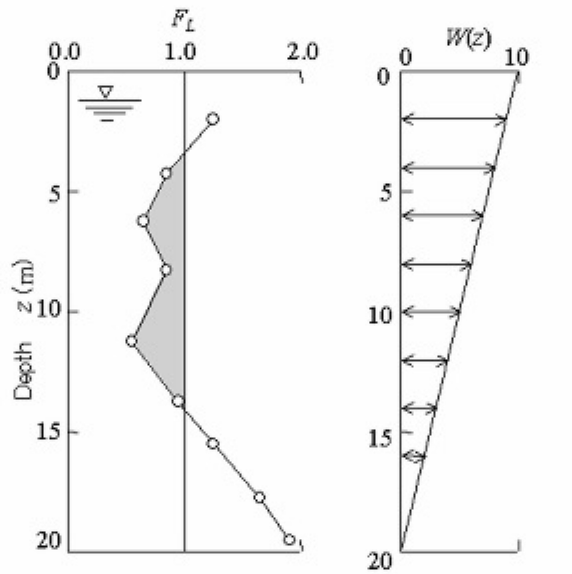
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_0^{20} (10 - 0{,}5_z) \times F_L \times d_z$$

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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