APPENDIX E

GEOTECHNICAL INVESTIGATION

GEOTECHNICAL INVESTIGATION
PROPOSED SELF STORAGE FACILITY

4301 Temple City Boulevard
El Monte, California
for
Magellan Value Partners, LLC

SoCalGeo
SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

November 23, 2021

Magellan Value Partners, LLC
1900 Avenue of the Stars, Suite 2470
Los Angeles, California 90067

Attention: Mr. Somy Mukherjee
Managing Director

Project No.: **21G226-1**

Subject: **Geotechnical Investigation**
Proposed Self-Storage Facility
4301 Temple City Boulevard
El Monte, California

Gentlemen:

In accordance with your request, we have performed 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.

REGISTERED PROFESSIONAL ENGINEER ROBERT TRAZO No. 2655 GEOTECHNICAL STATE OF CALIFORNIA

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Distribution: (1) Addressee

TABLE OF CONTENTS

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 results of the liquefaction evaluation indicate that some of the on-site soils are susceptible to liquefaction during a major seismic event. Based on the liquefaction evaluation, total dynamic settlements ranging from 0 to 1.07± inches could occur at the site during the design seismic event concurrent with historically high groundwater levels.
- Based on the predicted total settlements, the dynamic differential settlements are expected to be on the order of ½± inch.
- Undocumented fill soils were encountered at most of the boring locations extending to depths of 3 to 8½± feet below the existing site grades. These fill soils are underlain by native alluvial soils of variable composition and density.
- Remedial grading will be required to remove the undocumented fill soils and a portion of the underlying compressible native alluvial soils, and replace these materials as compacted structural fill.
- The near-surface soils possess a very low expansion potential.

Site Preparation

- Site stripping should include removal of any significant topsoil, vegetation and organic material. This should include the shrubs, native grass and weed growth which was present at the time of the subsurface exploration. These materials should be disposed of off-site.
- Demolition of the existing pavements and any associated improvements will be required in order to facilitate construction of the new building. Demolition procedures should include all foundations, floor slabs, utilities, the existing pavements, any septic systems, and any other subsurface improvements that will not remain in place for use with the new development. Debris resultant from demolition activities should be disposed of off-site. Alternatively, concrete and asphalt debris may be crushed to a maximum 2-inch particle size and mixed with sandy soils and reused within site fills.
- If a shallow foundation system with a moderate soil bearing pressure is to be used to support the proposed structure, remedial grading will be necessary.
- Remedial grading is recommended to be performed within the proposed building pad area. The excavation should extend to a depth of at least 8 feet below existing grade, and to a depth of at least 6 feet below proposed building pad subgrade elevation. Within the foundation influence zones, the overexcavation should extend to a depth of at least 6 feet below proposed foundation bearing grade. The overexcavation should also extend to a depth sufficient to remove all of the existing artificial fill soils (and any soils disturbed during demolition) from the proposed building pad area.
- After the overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated. The resulting subgrade should then be scarified to a depth of 12 inches, and

moisture conditioned or air dried to a moisture content of 0 to 4 percent above optimum. The previously excavated soils may then be replaced as compacted structural fill.

- All fill soils placed within the proposed building area should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density.

Foundation Design Parameters

- Conventional shallow foundations, supported in newly placed compacted fill.
- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2 based on an assumed bearing depth of at least 12 inches into suitable structural fill soils and 18 inches below adjacent exterior grade. An increase of 250 lbs/ft^2 per additional foot of depth below 18 inches may be used for the design. The maximum soil bearing pressure should not exceed 3,500 lbs/ft^2.
- Maximum, net allowable soil bearing pressure for wall footings constructed along the edges of the overexcavation where previously described lateral extents are not met: 1,800 lbs/ft^2 based on an assumed bearing depth of at least 12 inches into suitable structural fill soils and 18 inches below adjacent exterior grade. An increase of 180 lbs/ft^2 per additional foot of depth below 18 inches may be used for the design. The maximum soil bearing pressure for footings where the described lateral extents are not met should not exceed 2,500 lbs/ft^2.
- Reinforcement consisting of at least six (6) No. 5 rebars (3 top and 3 bottom) in strip footings due to the presence of potentially liquefiable soils. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade, 6 inches thick.
- Modulus of Subgrade Reaction: k = 150 psi/in
- Reinforcement consisting of No. 3 rebars at 16 inches on center in both directions due to the presence of liquefiable soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed slab loading.

Pavements

ASPHALT PAVEMENTS (R = 35)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	5	7	9	10	11
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 35)				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 21P412, dated September 14, 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 slab, 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 subject site is located at 4301 Temple City Boulevard in El Monte, California. The site is bounded to the north by Eaton Wash, to the west by multi-family residential buildings, to the south by a railroad easement, and to the east by Temple City Boulevard. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The site consists of an irregular-shaped parcel, 2.45± acres in size. The site is presently vacant of any structures. Ground surface cover generally consists of asphaltic concrete pavements and scattered debris and trash. The pavements are in poor condition with severe cracking throughout. The site possesses a few isolated areas of the site exhibit 2 to 4± inches of settlement. The remaining ground surface, along the northern property line, consists of exposed soil and a few palm trees. A descending slope that terminates at the Eaton Wash retaining wall is located along the northern property line.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth and visual observations made at the time of the subsurface investigation, the site slopes to the east at a gradient of 1.5± percent. The northern property line possesses a descending slope with a minor inclination of 5h:1v. The retaining wall along the northern property line retains approximately 10± feet of soil.

3.2 Proposed Development

A preliminary site plan, prepared by Jordan Architects, was provided to our office by the client. Based on this plan, the site will be developed with a self-storage building, with a footprint of 31,000± ft^2, located in the south-central area of the site. As of the writing of this report, the client has not decided what the height of the proposed building will be. It is our understanding that the building may be between one and four stories in height. **Based on the preliminary nature of the proposed building, additional subsurface investigations may be required at a later date after more details about the proposed development are known in order to verify the recommendations contained in this report.** The building is expected to be surrounded by asphaltic concrete pavements in the parking and drive areas, and limited landscape areas.

Detailed structural information has not been provided. Based on the relatively high structural loads anticipated for a one- to four-story storage building, conventional shallow foundations may not be suitable for the support of this structure. It is our understanding that no subterranean levels, such as basements or crawlspaces, are planned for the proposed development. It is assumed that maximum column and wall loads will be on the order of 100 to 400 kips and 2 to 5 kips per linear foot, respectively.

Preliminary grading plans were not available at the time of this report. Based on the existing topography, and assuming a relatively balanced site, cuts and fills of up to 4 to 5± feet are expected to be necessary to achieve the proposed site grades within the proposed building area.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration for this project consisted of six (6) borings advanced to depths of 10 to 65± feet below the existing site grades. In addition to the borings, five (5) Cone Penetration Test (CPT) soundings were advanced to depths of 26 to 31± feet at the site as a part of the liquefaction analysis. All of the CPTs encountered refusal at depths shallower than the planned minimum depth of 50 feet due to refusal on very dense soils. All of the borings were logged during drilling by a member of our staff.

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. Both of the CPT soundings were advanced to depths of 100± 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 October 13, 2021, enclosed in Appendix F.

The data generated by the cone penetrometer equipment has been reduced using CPeT-IT, V2.0, published by Geologismiki Geotechnical Software. The CPeT-IT program output as well as more details regarding the interpretation procedure are presented a report prepared by KTE, which is provided in Appendix F of this report.

General

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

4.2 Geotechnical Conditions

Pavements

Asphaltic concrete pavements were encountered at the ground surface of all boring locations. The pavement sections at the boring locations consist of 1 to 3± inches of asphaltic concrete (AC), underlain by 0 to 6± inches of aggregate base (AB).

Artificial Fill

Artificial fill soils were encountered beneath the pavements at all of the boring locations, extending to depths of 3 to 8½± feet below ground surface. The fill soils generally consist of loose to medium dense silty fine sands, fine sands, and intermixed fine sands and silty fine sands. Occasional layers of fine sandy silt and fine to coarse sand were encountered. The fill soils possess a disturbed and mottled appearance, with some samples possessing glass fragments, resulting in their classification as artificial fill.

Alluvium

Native alluvium was encountered beneath the fill soils at all of the boring locations, extending to at least the maximum depth explored of 65± feet below ground surface. The alluvial soils within the upper 12 to 19½± feet generally consist of very loose to medium dense silty fine sands, silty fine to medium sands and fine sandy silts. At greater depths, the alluvium generally consists of loose to dense fine sands, fine to medium sands, fine to coarse sands, silty fine to medium sands, and fine sandy silts. Occasional layers of medium dense to very dense gravelly fine to coarse sands, interbedded fine to coarse sands and silty fine sands, and medium stiff to hard fine to medium sandy clays were encountered. The alluvial soils occasionally possess iron oxide staining and porosity in the upper 15± feet.

Groundwater

Free water was not encountered during the drilling of any of the borings. Based on the moisture contents of the recovered soil samples and the lack of free water in the borings, the static groundwater table is at a greater depth than 65± feet below existing site grades.

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) Seismic Hazard Zone

Report 024, for the El Monte 7.5-Minute Quadrangle, which indicates that the historic high groundwater level for the site was about 20 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/. One monitoring wells on record is located 1352± feet south of the site. Water level readings within this monitoring wells indicate a high groundwater level of 95± feet below the ground surface in May 2019.

5.0 LABORATORY TESTING

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

Classification

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

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-8 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

One representative bulk sample was tested for its maximum dry density and optimum moisture content. The results were obtained using the Modified Proctor procedure, per ASTM D-1557 and is presented on Plate C-9 in Appendix C of this report. This test is generally used for comparison with the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Grain Size Analysis

Limited grain size analyses have been performed on several selected samples, in accordance with ASTM D-1140. These samples were washed over a #200 sieve to determine the percentage of fine-grained material in each sample, which is defined as the material which passes the #200 sieve. The weight of the portion of the sample retained on each screen is recorded and the percentage finer or coarser of the total weight is calculated. The results of these laboratory tests are shown on the attached boring logs.

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with the California Building Code (CBC). The testing apparatus is designed to accept a 4-inch diameter, 1-inch 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-1 @ 0 to 5 feet	0	Non-expansive
B-2 @ 0 to 5 feet	2	Very Low

Soluble Sulfates

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

Sample Identification	**Soluble Sulfates (%)**	**ACI Classification**
B-1 @ 0 to 5 feet	0.002	Not Applicable (S0)
B-2 @ 0 to 5 feet	0.010	Not Applicable (S0)

Direct Shear

Direct shear tests were performed on selected soil samples to determine their shear strength parameters. The tests were performed in accordance with ASTM D-3080. The testing apparatus is designed to accept either natural or remolded samples in a one-inch-high ring, approximately 2.416 inches in diameter. Three samples of the same soil are prepared by remolding them to 90± percent compaction and near optimum moisture. Each of the three samples are then loaded with different normal loads and the resulting shear strength is determined for that particular normal load. The shearing of the samples is performed at a rate slow enough to permit the dissipation of excess pore water pressure. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The results of the direct shear tests are presented on Plates C-10 and C-11.

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-1 @ 0 to 5 feet	2,400	7.0	6.0	265
B-2 @ 0 to 5 feet	1,760	6.8	3.2	344

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

6.1 Seismic Design Considerations

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

Faulting and Seismicity

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

Seismic Design Parameters

The 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 building proposed for this site. However, the structural engineer should verify that this exception is applicable to the proposed building.** 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 Spectral Acceleration at 0.2 sec Period	S_S	1.937
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.699
Site Class	---	D*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.937
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.188
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.291
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.792

*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 building has a fundamental period greater than 0.5 seconds, a site-specific seismic hazard 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 building at this site.

Ground Motion Parameters

For the purposes of the liquefaction analysis performed for this study, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2019 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-16. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-16. The web-based software application SEAOC/OSHPD Seismic Design Maps Tool (described in the previous section) was used to determine PGA_M, which is 0.921g. A portion of the program output is included as Plate E-1 of this report. An associated earthquake magnitude was obtained from the USGS Unified Hazard Tool, Interactive Deaggregation application available on the USGS website. The deaggregated mean magnitude is 6.92, based on the peak ground acceleration and soil classification D.

Liquefaction

Research of the Earthquake Zones of Required Investigation Map for the El Monte Quadrangle, published by the California Geological Survey (CGS) indicates that the site subject site is located within a liquefaction hazard zone. Based on this mapping, and the subsurface conditions encountered at the borings, 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 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.

As part of the liquefaction evaluation, Boring Nos. B-1 through B-4 and CPT-1 through CPT-4, inclusive, were planned to be extended to depths of at least 50 feet. Borings were also drilled in close proximity to the CPT locations to provide physical samples for further analysis and correlation with the CPT data. However, all of the CPTs were terminated at depths shallower than planned due to refusal on very dense soils. The soils below the refusal depths for the CPT soundings are considered non-liquefiable. The liquefaction potential for the on-site soils was evaluated the computer program CLiq V3.0.3.2, which was developed by Geologismiki, copyright 2007. The analysis method is based on Boulanger and Idriss, 2014. The liquefaction potential of the site was analyzed utilizing a PGA_M of 0.92g for a magnitude 6.92 seismic event. The potential settlements that could occur as a result of liquefaction for each of the potentially liquefiable layers was calculated. A copy of the program output is presented in Appendix G of this report. The historic high groundwater depth used in the liquefaction analysis was obtained from CGS Open File Report 98-15, the Seismic Hazard Evaluation of the El Monte Quadrangle, which indicates a historic high groundwater depth was conservatively estimated to be 20± feet below the ground surface.

Conclusions and Recommendations

The results of the liquefaction analysis have identified potentially liquefiable soils at three (3) of the boring locations and one (1) 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 encountered between depths of 20 and 31± feet are considered to be non-liquefiable due to their cohesive characteristics, as interpreted by the CPT. Settlement analyses were conducted for each of the potentially liquefiable strata. The total dynamic settlement for each CPT location, based on the results of the dynamic settlement analyses (presented in Appendix G) are presented below:

- Boring No. B-1: 0.57± inch
- Boring No. B-2: 0.70± inch
- Boring No. B-3: 1.07± inches
- Boring No. B-4: 0.19± inch
- CPT-1: 0.45± inch
- CPT-2: 0.00± inch
- CPT-3: 0.00± inch
- CPT-4: 0.00± inch

Based on these total settlements, differential settlements of up to ½± inch 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 0.001± inch per inch.

Based on our understanding of the proposed development, it is considered feasible to support the proposed structure on shallow foundations. Such a foundation system can be designed to resist the effects of the anticipated differential settlements, to the extent that the structure would not catastrophically fail. Designing the proposed structure to remain completely undamaged during a major seismic event is not considered to be economically feasible. Based on this understanding, the use of a shallow foundation system is considered to be the most economical means of supporting the proposed structure.

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

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

6.2 Geotechnical Design Considerations

General

As discussed previously, detailed structural information is not available at this time. Estimates for the column and wall loads for the project have not been provided. Therefore, we are providing preliminary geotechnical design parameters, and various alternatives for foundation construction. Once a final building configuration has been determined and a foundation system has been selected, and the structural details have been determined, the geotechnical engineer should be contacted for final design recommendations.

Artificial fill soils were encountered at the boring locations within the proposed building area, extending to depths of 3 to 8½± feet below the existing site grades. The fill soils generally consist of sands and silty sands. Based on their variable composition and strength, and the lack of any documentation regarding the placement or compaction of the fill soils, the existing fill materials are considered to represent undocumented fill soils.

The fill soils are underlain by native alluvium of varying composition. The results of consolidation testing on samples of on-site soils from the upper 6 to 8± feet indicate that these soils are subject to moderate consolidation settlement when loaded. Based on the presence of undocumented fill soils and compressible native alluvium within the foundation influence zones of the proposed building, these existing near-surface soils, in their present state are not considered suitable to support the foundations and floor slab of the proposed building. Remedial grading is considered warranted in order to remove the artificial fill soils in their entirety and a portion of the near surface alluvial soils in order to replace these materials as compacted structural fill.

As discussed in the previous section of this report, potentially liquefiable soils were identified at this site. The presence of the recommended layer of newly placed compacted structural fill above these liquefiable soils will help to reduce any surface manifestations that could occur as a result of liquefaction. The foundation design recommendations presented in the subsequent sections of this report also contain recommendations to provide additional rigidity in order to reduce the potential effects of differential settlement that could occur as a result of liquefaction.

A significant design concern for this project is the potential difficulty associated with making excavations near the adjacent property line and other adjacent improvements. Provisions should be made to incorporate shoring in order to help facilitate construction of the proposed structure. More detailed recommendations for shoring are presented in subsequent sections of this report.

Settlement

The recommended remedial grading will remove the existing undocumented fill soils and a portion of the compressible near surface alluvium and replace these materials as compacted structural fill. The native soils that will remain in place below the recommended depth of overexcavation will not be subject to significant load increases from the foundations of the new building. Provided that the recommended remedial grading is completed, the post-construction static settlement of the proposed building is expected to be within tolerable limits.

Soluble Sulfates

The results of the soluble sulfate testing indicate that the selected samples of the on-site soils contain concentrations of soluble sulfates that correspond to Class S0 with respect to the American Concrete Institute (ACI) Publication 318-14 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building area.

Expansion

The near-surface soils at this site generally consist of sands and silty sands. The results of laboratory testing indicate that the near-surface soils face possess very low to non-expansive potentials (EI = 0 and 2). The foundation and floor slab design recommendations contained within this report are made in consideration of the expansion index test results. It is recommended that additional expansion index testing be conducted at the completion of rough grading to verify the expansion potential of the as-graded building pad.

Corrosion Potential

The results of the electrical resistivity and pH testing indicate that samples of the on-site soils have resistivity values ranging from 1,760 to 2,400 ohm-cm, and pH values ranging from 6.8 to 7.0. 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. Redox potential, relative soil moisture content and sulfides are also included. Although sulfide testing was 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 the on-site soils are considered to be corrosive to ductile iron pipe. Therefore, polyethylene protection is expected to be required for cast iron or ductile iron pipes.** It should be noted that SCG does not practice in the field of corrosion engineering, and therefore, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.

Relatively low concentrations of chlorides, 3.2 to 6.0 mg/kg, were detected in the samples submitted for corrosivity testing. In general, soils possessing chloride concentrations in excess of 500 parts per million (ppm) are considered to be corrosive with respect to steel reinforcement within reinforced concrete. Based on the relatively low chloride concentration in the tested sample, the site is considered to have a C1 chloride exposure in accordance with the American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary. Therefore, a specialized concrete mix design for protection against chloride exposure is not considered warranted.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested samples possess nitrate concentrations ranging from 265 to 344 mg/kg. **Based on these test results, the on-site soils are considered to be corrosive to copper pipe and protection may be required.**

Since SCG does not practice in the area of corrosion engineering, the client may wish to contact a corrosion engineer to provide a more thorough evaluation**.**

Shrinkage/Subsidence

Removal and recompaction of the near surface fill soils and alluvium is estimated to result in an average shrinkage of 7 to 17 percent. 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 0.10± feet. This estimate may be used for grading in areas that are underlain by native alluvial soils.

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

Grading and Foundation Plan Review

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

6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring locations and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

Site Stripping and Demolition

It is anticipated that the existing pavements and associated improvements will be demolished. Any existing subsurface improvements that will not remain in place for use with the new development should be removed in their entirety. This should include all foundations, slabs, septic systems, utilities, and any other subsurface improvements associated with any previously existing structures and associated improvements. Debris resultant from demolition should be disposed of off-site in accordance with any applicable regulations. Alternatively, concrete and asphalt debris may be crushed to a maximum 2-inch particle size, well mixed with the sandy soils, and incorporated into new structural fills or it may be processed into CMB, if desired.

Initial site stripping should also include removal of any surficial vegetation from the site. This should include any weeds, grasses, shrubs, and trees. Root masses associated with the trees should be removed in their entirety, and the resultant excavations should be backfilled with compacted structural fill soils. 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.

Treatment of Existing Soils: Building Supported on Conventional Shallow Foundations

Remedial grading should be performed within the proposed building area to remove all of the existing undocumented fill materials, which extend to depths of at least 3 to 8½± feet at the boring locations. In order to provide more uniform support characteristics for the proposed structure, the overexcavation should extend to a depth of at least 8 feet below the existing site grades and to a depth of at least 6 feet below the proposed building pad grades. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 6 feet below proposed foundation bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building perimeters and foundations, 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 overexcavation should also encompass these areas.

Based on the configuration of the proposed structure with respect to the apparent property lines, it appears that the horizontal limits of overexcavation will not be achievable along portions of all of the property lines. Therefore, temporary or permanent shoring will be necessary. Geotechnical recommendations for shoring are included in a later section of this report. After reviewing the grading and foundation plans, the geotechnical engineer may need to provide supplemental geotechnical shoring design recommendations. Additional considerations related to excavations adjacent to existing improvements are presented in Section 6.4 of this report.

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

Based on the conditions encountered at the exploratory boring locations, very moist 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, but it is expected that an extended period of drying may be required. 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, will likely 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 RS580I, 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 may be used as a subgrade stabilization material. Other options, including lime or cement treatment are also available. Typically, an unstable subgrade may be stabilized by treating the upper 12 inches of subgrade material with cement to a concentration of 5 percent (by dry weight of soil).

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, thoroughly moisture conditioned, and recompacted. Overexcavation bottoms should be thoroughly moisture conditioned to achieve a moisture content of 0 to 4 percent above the optimum moisture content. The previously excavated soils may then be replaced as compacted structural fill. **Any fill soils placed within the building area should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density.**

<u>Treatment of Existing Soils: Retaining Walls and Site Walls</u>

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 pads. Any undocumented fill soils within the foundation areas should be removed in their entirety. The overexcavation areas should extend horizontally beyond the foundation perimeters to a distance equal to the depth of fill below the new foundations. These overexcavation recommendations also apply to any erection pads for tilt-up concrete walls, since these pads are part of the foundation system. 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 lateral and/or vertical extents of overexcavation are not achievable for the retaining walls or site walls, as may occur along the property lines, then additional recommendations including, but not limited to reduced design bearing pressures may be required. Additionally, specialized grading techniques such as slot cutting or shoring may be required in order to facilitate construction.

Treatment of Existing Soils: Flatwork, Parking and Drive Areas

Based on economic considerations, overexcavation of the existing undocumented fill soils and the lower strength, compressible, native alluvium in the new parking and flatwork 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 flatwork areas should initially consist of removal of all soils disturbed during stripping and demolition operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 0 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 alluvial 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 flatwork, parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within the proposed parking and flatwork areas. The grading recommendations presented above do not mitigate the extent of potentially compressible soils and undocumented fill soils in the parking, drive and flatwork areas. As such, some movement 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 overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 0 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer. Drying of some of the on-site soils may be necessary to achieve a moisture content suitable for recompaction.

- 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 El Monte.
- All new structural fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. **All fill soils within the new building pad area should be compacted to at least 95 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 soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). It is recommended that materials in excess of 3 inches in size not be used for utility trench backfill. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by City of El Monte. 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 generally consist of sands and silty sands. 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, the inclination of temporary slopes should not exceed 2h:1v. 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.

Remedial grading for the proposed structure will require excavation adjacent to the south property line. The contractor should take all necessary provisions to protect any improvements on the adjacent property. **The use of shoring or slot cutting techniques is expected to be necessary during remedial grading for the proposed structure pad in order to maintain lateral support for any improvements located on the adjacent property to the south of the subject site.** Typically, A-B-C slot cuts on 6 to 8-foot centers are suitable to maintain excavation stability. The geotechnical engineer should observe the conditions and determine the appropriate slot cutting procedures at the time of site grading. Based on the anticipated depth of required overexcavation, it is anticipated that shoring will be required.

Moisture Sensitive Subgrade Soils

Most of the near surface soils possess appreciable silt content and may become unstable if exposed to significant moisture infiltration or disturbance by construction traffic. In addition, based on their granular content, some of the on-site soils will also be susceptible to erosion. The site should, therefore, be graded to prevent ponding of surface water and to prevent water from running into excavations.

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.

Elevator Equipment Shafts

Depending on the final height of the proposed building, the proposed building may incorporate elevators. Typically, these elevators require installation of relatively large-diameter steel pipes as part of the elevator counterweights. It is expected that the pipes will be installed within slightly oversized borings. Where these pipes are installed, the annulus between the borehole wall and the elevator pipe should be backfilled with a lean concrete slurry or grout. Placement of loose backfill soils around these pipes could result in localized settlement of the structural soils and/or foundation elements.

Groundwater

Ground water was not encountered at any of the boring locations, which extended to a depth of up to 65± feet below the existing site grades. Groundwater is therefore not expected to impact the grading or foundation construction activities.

6.5 Shallow Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by structural fill soils used to replace the undocumented fill soils and near-surface alluvial soils. These new structural fill soils are expected to extend to depths of at least 6 feet below proposed foundation bearing grades, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, and based on the design considerations presented in Section 6.1 of this report, the proposed structure may be supported on conventional shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2 based on an assumed bearing depth of at least 12 inches into suitable structural fill soils and 18 inches below adjacent exterior grade. An increase of 250 lbs/ft^2 per additional foot of depth below 18 inches may be used for the design. The maximum soil bearing pressure should not exceed 3,500 lbs/ft^2.

- Maximum, net allowable soil bearing pressure for wall footings constructed along the edges of the overexcavation where previously described lateral extents are not met: 1,800 lbs/ft^2 based on an assumed bearing depth of at least 12 inches into suitable structural fill soils and 18 inches below adjacent exterior grade. An increase of 180 lbs/ft^2 per additional foot of depth below 18 inches may be used for the design. The maximum soil bearing pressure for footings where the described lateral extents are not met should not exceed 2,500 lbs/ft^2.

- Minimum wall/column footing width: 14 inches/24 inches.

- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom) in strip footings, due to the presence of potentially liquefiable 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.

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

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 95 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 0 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. 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. It should be noted that the projected potential dynamic settlement is related to a major seismic event and a conservative historic high groundwater level.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slab 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: 300 lbs/ft^3
- Friction Coefficient: 0.30

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 2500 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support the new floor slab should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, and based on the design considerations presented in Section 6.1 of this report, the floor of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, extending to a depth of at least 6 feet below proposed finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches, due to the liquefaction potential of the on-site soils.

- Modulus of Subgrade Reaction: k = 150 psi/in.

- Minimum slab reinforcement: Minimum slab reinforcement: No. 3 bars at 16 inches on-center, in both directions, due to the presence of potentially liquefiable soils at the site. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed loading, and the 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 0 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

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

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement. 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 sands and silty sands. Based on laboratory testing, sand and silty sand materials are expected to possess a friction angle of at least 31 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 Sands and Silty Sands
Internal Friction Angle (ϕ)		31°
Unit Weight		130 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	40 lbs/ft^3
	Active Condition (2h:1v backfill)	63 lbs/ft^3
	At-Rest Condition (level backfill)	61 lbs/ft^3

The walls should be designed using a soil-footing coefficient of friction of 0.30 and an equivalent passive pressure of 300 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

On-site sands, silty sands and sandy silts may be used to backfill the retaining walls. All backfill material placed within 3 feet of the back-wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

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

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

Subsurface Drainage

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

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

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

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

6.8 Temporary Shoring Recommendations

Shoring will be required during grading and/or foundation construction activities. The following recommendations assume that the retained soil heights will not exceed 10 to 12± feet. If surcharge loads are located within 10± feet of the shoring, the effect of these loads upon the shoring system must be considered by the shoring engineer. The shoring should be designed to withstand the effects of nearby surcharge loads, including traffic from the adjacent streets, and any construction loads or traffic.

Lateral Earth Pressures

It is assumed that the soil behind the shoring system will be relatively level. It is assumed that the shoring will consist of either sheet piles or soldier piles and lagging. Plate 3 included in this report illustrates the at-rest and active lateral earth pressure distributions. As shown on Plate 3, the at-rest and active pressures to be used in the shoring design should be 67H and 42H, respectively. These distributions are based on static conditions. As previously discussed, if surcharge loads are imposed upon the shoring, they must be considered by the shoring engineer. In accordance with the Caltrans Trenching and Shoring Manual, a construction surcharge of 83 lbs/ft^2, per foot of depth, should also be applied to the back of the shoring system, extending to a depth of 10 feet below the top of the shoring system or to the excavation line, whichever is less. These loads assume normal construction traffic, consisting of lightly loaded vehicles and storage of small amounts of materials. If large stockpiles of soil, concentrated pallet loads, or crane loads are expected, SCG should be contacted for additional surcharge load recommendations. The passive resistance value of the soil below the level of excavation may be assumed to be 300 lbs/ft^2, per foot of depth, to a maximum of 3,000 lbs/ft^2. The passive resistance was calculated in accordance with Section 6 of the Caltrans Trenching and Shoring Manual using the equations $s_P = \gamma k_P$ where $k_P = \tan^2(45+\phi/2)$ and γ is the unit weight of the soil.

Shoring Construction

If soldier piles are utilized, they should be spaced no closer than 3 times the nominal soldier pile diameter. The contractor should take all necessary provisions to assure firm contact between the retained soils and the shoring system. A 2-sack cement slurry may be used to fill voids where inadequate contact between the shoring system and the retained soils are observed.

Since the shoring system will be designed as a cantilever wall, some deflection will occur. In order to develop the full active pressure, a deflection of ½ to 1 inch is expected to occur at the top of the shoring system. The design of the shoring system as well as the protection of adjacent improvements should take this deflection into consideration.

6.9 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface soils consist of various soil materials including sands and silty sands. These soils are considered to possess poor to fair pavement support characteristics with estimated R-values ranging from 35 to 45. The subsequent pavement design is therefore based upon an assumed R-value of 35. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering-controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

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

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

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

ASPHALT PAVEMENTS (R = 35)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	5	7	9	10	11
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>.

<u>Portland Cement Concrete</u>

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

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 35)				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

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

7.0 GENERAL COMMENTS

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

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

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

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

8.0 REFERENCES

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.

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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.

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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

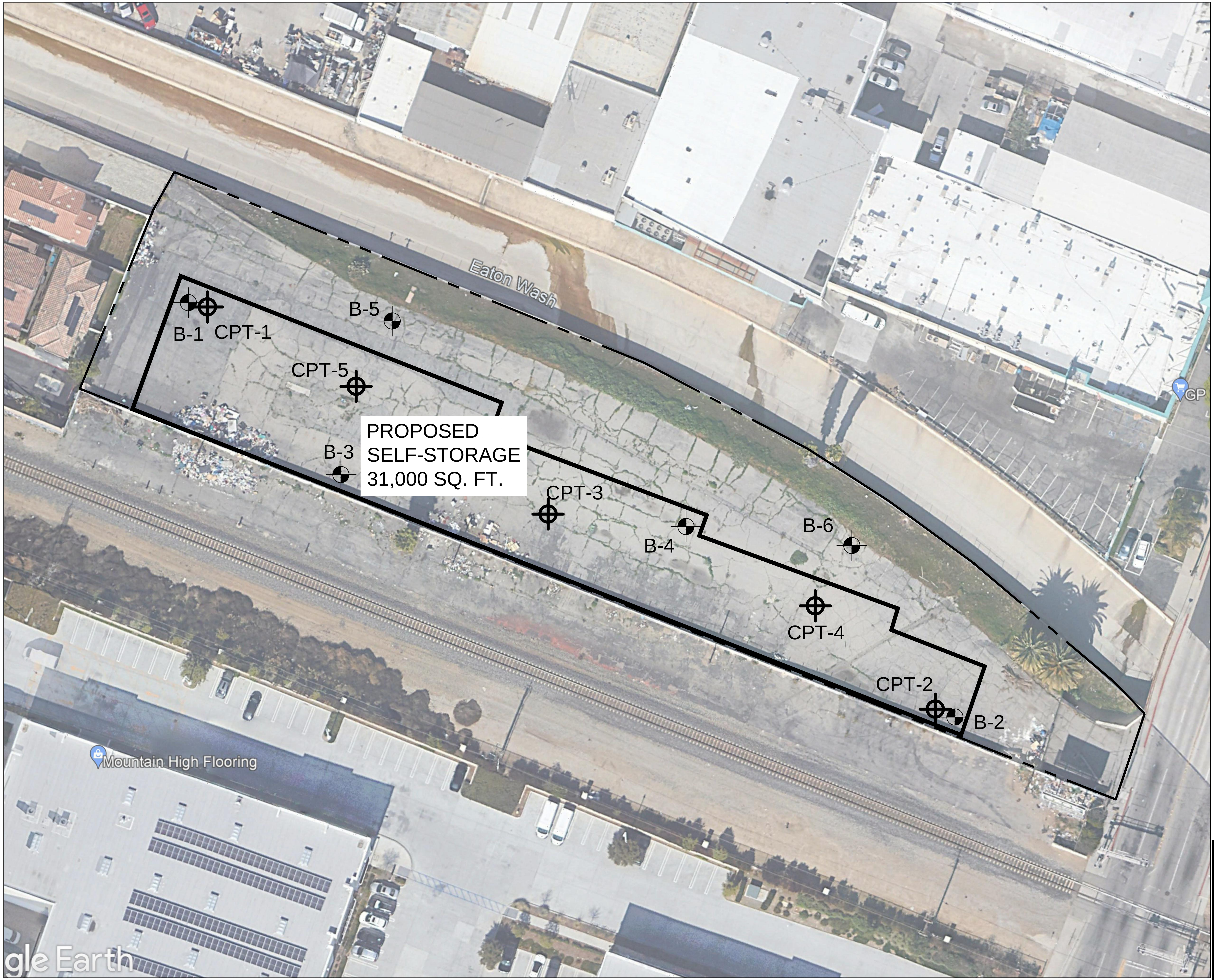
SOURCE: USGS TOPOGRAPHIC MAP OF THE EL MONTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA, 2018.

SITE LOCATION MAP

PROPOSED SELF-STORAGE FACILITY

EL MONTE, CALIFORNIA

SCALE: 1" = 2000'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAH CHKD: RGT	
SCG PROJECT 21G226-1	
PLATE 1	



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

APPROXIMATE CPT LOCATION

NOTE: SITE PLAN PROVIDED BY JORDAN ARCHITECTS.

BORING AND CPT LOCATION PLAN

PROPOSED SELF-STORAGE FACILITY

EL MONTE, CALIFORNIA

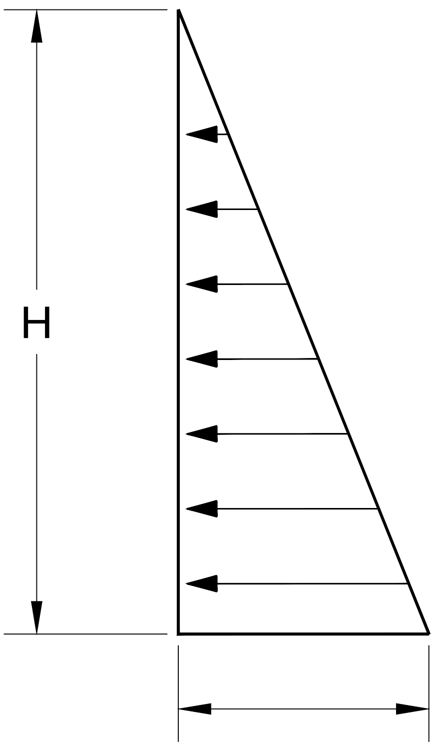
SCALE: 1" = 50'

DRAWN: JAH
CHKD: RGT

SCG PROJECT
21G226-1

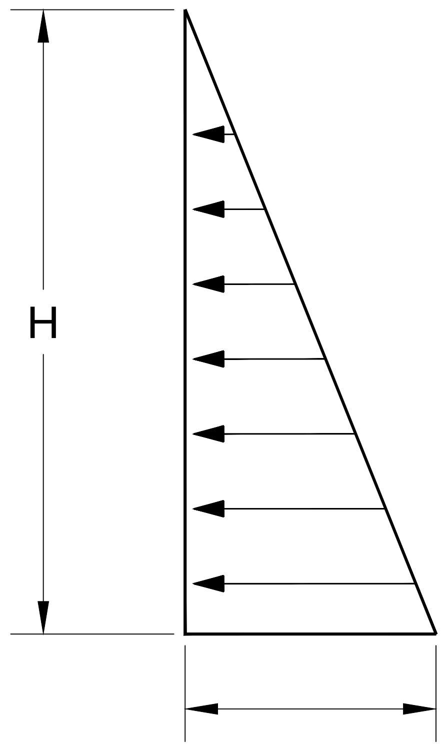
PLATE 2

SoCalGeo
SOUTHERN
CALIFORNIA
GEOTECHNICAL



67H (static)

AT-REST PRESSURE



42H (static)

ACTIVE PRESSURE

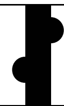
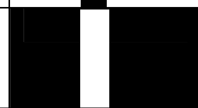
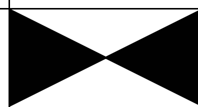
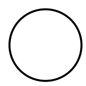
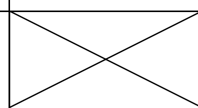
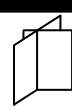
NOTES: THE LATERAL EARTH PRESSURES DEPICTED IN THIS DIAGRAM DO NOT INCLUDE ANY SURCHARGE LOADS.

PRESSURE DIAGRAM FOR SHORING

NOT TO SCALE	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: PM CHKD: RGT	
SCG PROJECT 21G226-1	
PLATE 3	

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.: 21G226-1
PROJECT: Proposed Self-Storage Facility
LOCATION: El Monte, California

DRILLING DATE: 10/5/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 58 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS						LABORATORY RESULTS						
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
					3± inches Asphaltic concrete; 3± inches Aggregate Base							
	X	16			FILL: Brown Silty fine Sand, trace to little medium to coarse Sand, trace fine Gravel, medium dense-damp		7					EI = 0 @ 0-5'
	X	6			FILL: Brown Interbedded fine Sand and Silty fine Sand, loose-very moist		15					
5												
	X	4			ALLUVIUM: Dark Brown fine Sandy Silt, loose to medium dense-very moist		17					
	X	11					19					
10												
					Brown Silty fine Sand, trace medium Sand, medium dense-very moist							
	X	10					13					
15												
	X	17					13					
20					Light Brown fine Sand, trace medium to coarse Sand, medium dense-damp		5			10		
					Light Gray Brown fine to medium Sand, little coarse Sand, medium dense-damp							
	X	28					3					
25												
					Light Gray Brown fine to coarse Sand, little fine to coarse Gravel, dense-damp							
	X	46					2					
30												
					Light Brown fine to coarse Sand, little Silt, little fine to coarse Gravel, very dense-damp							
	X	50					5					

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 21G226-1
PROJECT: Proposed Self-Storage Facility
LOCATION: El Monte, California

DRILLING DATE: 10/5/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 58 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Light Brown fine to coarse Sand, little Silt, little fine to coarse Gravel, very dense-damp							
	X	58					3					
40												
	X	50/5"					2					
45												
					Brown Silty fine to medium Sand, trace Clay, trace coarse Sand, little to some Iron oxide staining, medium dense-very moist							
	X	22					14			30		
50												
					Light Brown fine to coarse Sand, little fine Gravel, dense-damp							
	X	43					3					
55												
					Brown Silty fine Sand to fine Sandy Silt, trace medium to coarse Sand, medium dense-very moist							
	X	24					17					
60												
	X	28					14					
65												
					Boring Terminated at 65'							

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 21G226-1
PROJECT: Proposed Self-Storage Facility
LOCATION: El Monte, California

DRILLING DATE: 10/5/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 49 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
					2± inches Asphaltic concrete; 3± inches Aggregate Base							
	X	12			FILL: Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, medium dense-damp		6					EI = 2 @ 0-5'
	X	5			ALLUVIUM: Dark Brown Silty fine Sand to fine Sandy Silt, loose to medium dense-very moist		17					
5												
	X	4			@ 6 feet, trace fine root fibers		16					
	X	10			@ 8½ feet, trace Clay, trace Calcareous veining		15					
10												
					Brown Clayey fine to medium Sand to fine to medium Sandy Silt, little Iron oxide staining, medium dense-moist							
	X	10					12					
15												
					Brown fine to coarse Sand, trace to little Silt, trace fine Gravel, medium dense-damp							
	X	10					5			10		
20												
					Light Gray Brown fine to coarse Sand, little fine to coarse Gravel, dense-dry to damp							
	X	43					2					
25												
	X	37					2					
30												
					Light Gray Gravelly fine to coarse Sand, dense to very dense-dry to damp							
	X	60					2					

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 21G226-1
PROJECT: Proposed Self-Storage Facility
LOCATION: El Monte, California

DRILLING DATE: 10/5/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 49 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Light Gray Gravelly fine to coarse Sand, very dense-dry to damp							
	X	48					2					
40												
					Gray fine Sandy Clay, trace medium Sand, some Iron oxide staining, hard-moist							
	X	34	4.5				19					
45												
					Brown Silty fine Sand, trace to little medium to coarse Sand, dense-damp to moist							
	X	35					8					
50												
					Gray Brown fine to coarse Sand, trace fine Gravel, dense-damp							
	X	35					3					
55												
					Brown fine to coarse Sand, little Silt, trace fine Gravel, dense to very dense-moist to very moist							
	X	33					16					
60												
	X	62					7					
65												
					Boring Terminated at 65'							

TBL 21G226-1.GPJ SOCALGEO.GDT 11/24/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 21G226-1
PROJECT: Proposed Self-Storage Facility
LOCATION: El Monte, California

DRILLING DATE: 10/5/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 46 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; 6± inches Aggregate Base							
		19			FILL: Gray Brown Silty fine Sand, little medium to coarse Sand, little fine to coarse Gravel, loose to medium dense-dry to damp	103	2					
		12			@ 3 feet, trace glass fragments		3					Disturbed Sample
5		12				111	3					
					FILL: Dark Brown fine Sandy Silt, little Clay, sightly mottled, loose-damp to moist							
		11				102	14					
					ALLUVIUM: Brown fine Sandy Silt, little Clay, slightly mottled, medium dense-moist							
		15				114	14					
10												
					Brown Silty fine Sand to fine Sandy Silt, loose-very moist							
		10				104	18					
15												
					Light Gray Brown fine to medium Sand, little Silt, little coarse Sand, medium dense-damp							
		23				105	6			20		
20												
					Light Gray Brown fine to coarse Sand, little fine Gravel, very dense-dry to damp							
		55					3					
25												
		59					3					
30												
		80					2					

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 21G226-1
PROJECT: Proposed Self-Storage Facility
LOCATION: El Monte, California

DRILLING DATE: 10/5/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 46 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 (%)	
					Light Gray Brown fine to coarse Sand, little fine Gravel, very dense-dry to damp							
	X	75/9"			@ 38½ feet, little fine to coarse Gravel		3					
40												
					Light Gray Brown fine to medium Sand, little coarse Sand, trace fine Gravel, dense-damp							
	X	35					4					
45												
					Brown Silty fine Sand to fine Sandy Silt, some Iron oxide staining, medium dense-very moist							
	X	22					23					
50												
					Brown Silty fine to medium Sand, some Iron oxide staining, dense-moist							
	X	41					14					
55												
					Boring Terminated at 55'							

TBL 21G226-1.GPJ SOCALGEO.GDT 11/24/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 21G226-1
PROJECT: Proposed Self-Storage Facility
LOCATION: El Monte, California

DRILLING DATE: 10/5/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 48 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS						LABORATORY RESULTS						
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
					1± inch Asphaltic concrete; No Discernible Aggregate Base							
	X	22			FILL: Light Gray Brown fine Sand, trace medium to coarse Sand, trace fine Gravel, medium dense-damp	104	3					
	X	15				105	4					
5	X	6			ALLUVIUM: Brown Silty fine Sand, slightly mottled, loose-very moist	95	16					
	X	13				105	14					
	X	13			Brown Silty fine to medium Sand, little coarse Sand, porous, trace Calcareous nodules, loose-moist	117	11					
10												
					Brown fine Sand, little medium to coarse Sand, trace fine Gravel, loose-damp							
	X	14				101	6					
15												
					Light Gray fine to medium Sand, little fine Gravel, medium dense-damp							
	X	32				106	3					
20												
					Light Gray Brown fine to coarse Sand, little fine Gravel, dense to very dense-dry to damp							
	X	45					2					
25												
	X	32			@ 28½ feet, little coarse Gravel		2					
30												
	X	66					2					

TBL 21G226-1.GPJ SOCALGEO.GDT 11/24/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 21G226-1
PROJECT: Proposed Self-Storage Facility
LOCATION: El Monte, California

DRILLING DATE: 10/5/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 48 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Light Gray Brown fine to coarse Sand, little fine Gravel, dense to very dense-dry to damp							
					Brown Clayey fine to medium Sand, little Silt, little to some Iron oxide staining, dense-very moist							
	X	32					19					
40												
					Gray Brown Interbedded fine to coarse Sand and Silty fine Sand, little Iron oxide staining, dense-moist							
	X	30					12					
45												
					Brown Silty fine Sand, medium dense to dense-moist to very moist							
	X	26					17					
50												
	X	36			@ 53½ feet, 3-inch fine Sandy Silt lense with little Iron oxide staining		13					
55												
					Boring Terminated at 55'							

TBL 21G226-1.GPJ SOCALGEO.GDT 11/24/21

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 21G226-1
PROJECT: Proposed Self-Storage Facility
LOCATION: El Monte, California

DRILLING DATE: 10/5/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 9 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
					2± inches Asphaltic concrete; 4± inches Aggregate Base							
	X	12			FILL: Gray Brown Intermixed Silty fine Sand and fine Sand, trace to little medium to coarse Sand, medium dense-damp		6					
	X	8			FILL: Brown Silty fine Sand, little Iron oxide staining, loose-moist		10					
5					FILL: Light Gray Brown fine Sand, trace medium to coarse Sand, trace fine Gravel, loose-dry to damp		2					
	X	4			ALLUVIUM: Brown Silty fine Sand, porous, loose-very moist		15					
	X	8			@ 8½ feet, trace Calcareous veining		15					
10												

Boring Terminated at 10'

TBL 21G226-1.GPJ SOCALGEO.GDT 11/24/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 21G226-1
PROJECT: Proposed Self-Storage Facility
LOCATION: El Monte, California

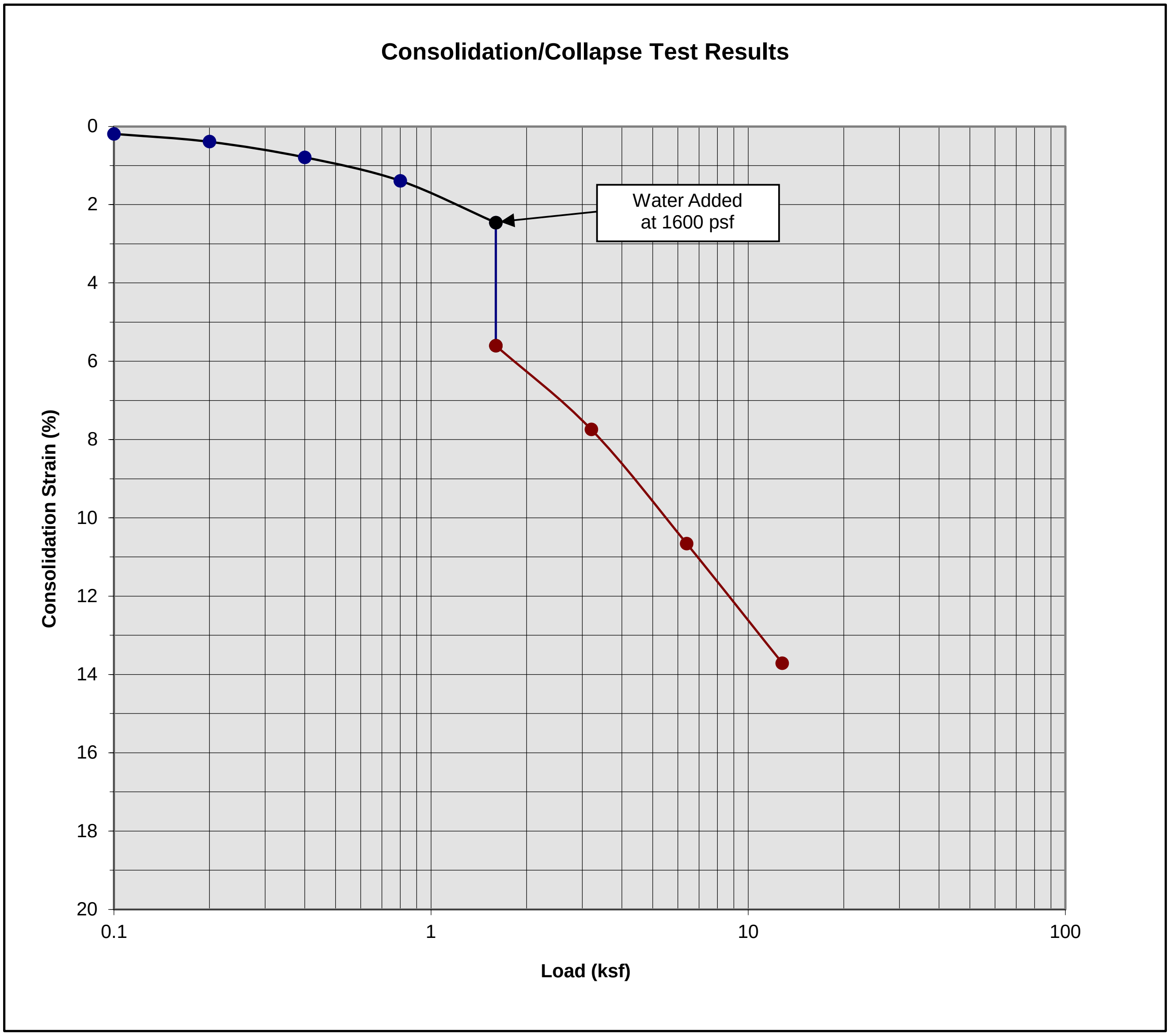
DRILLING DATE: 10/5/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 8 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					2± inches Asphaltic concrete; 4± inches Aggregate Base							
	X	8			FILL: Gray Brown fine to coarse Sand, trace to little fine Gravel, loose-dry to damp		2					
	X	8					2					
5												
	X	6			ALLUVIUM: Brown Silty fine Sand, trace medium to coarse Sand, slightly porous, loose-very moist		27					
	X	9			Brown Silty fine to medium Sand, porous, trace Calcareous veining, loose-very moist		28					
10												
					Boring Terminated at 10'							

TBL 21G226-1.GPJ SOCALGEO.GDT 11/24/21

APPENDIX C

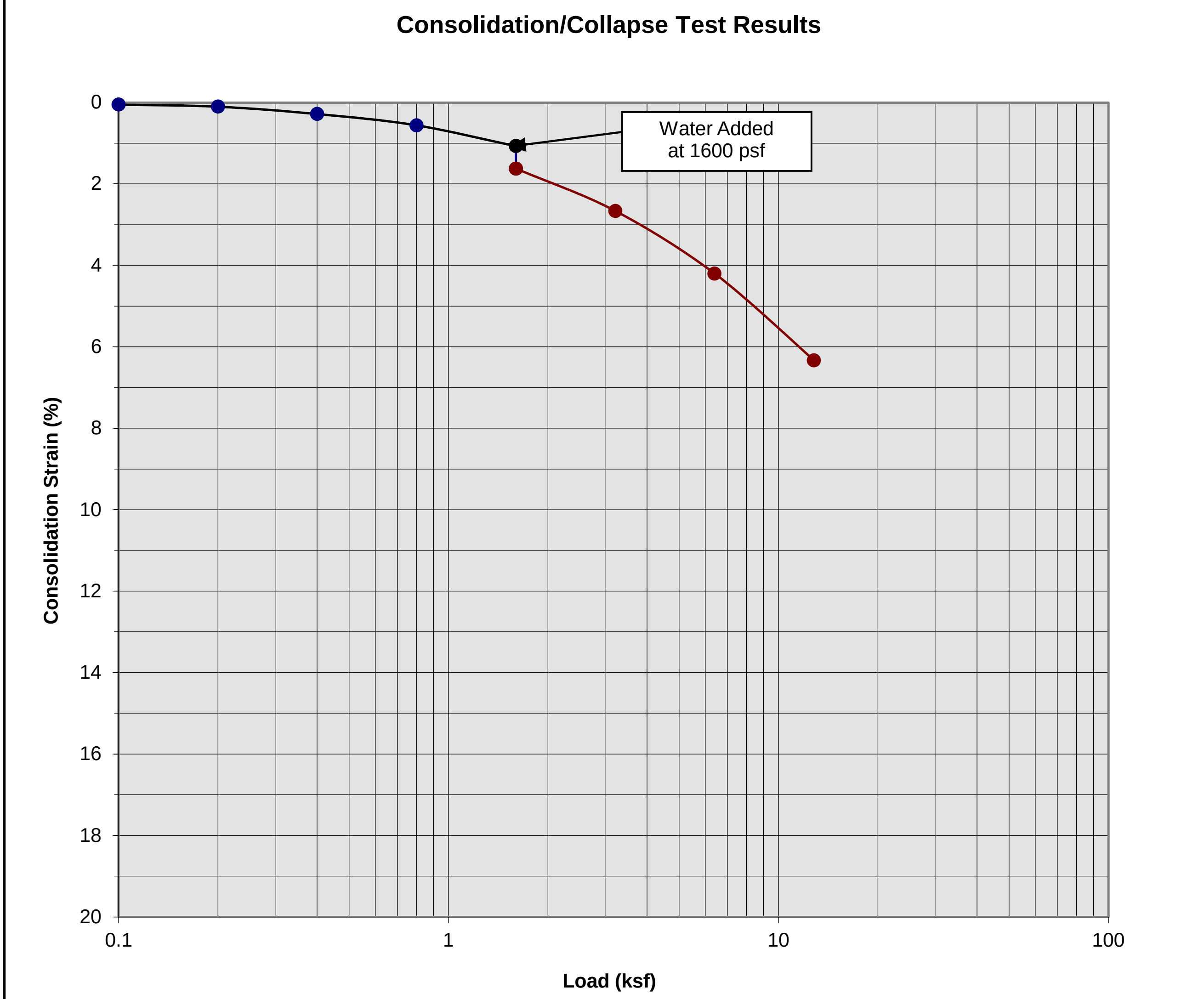


Classification: FILL: Dark Brown fine Sandy Silt, little Clay

Boring Number:	B-3	Initial Moisture Content (%)	14
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	7 to 8	Initial Dry Density (pcf)	101.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	117.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	3.14

Proposed Self-Storage Facility
El Monte, CA
Project No. 21G226-1
PLATE C- 1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



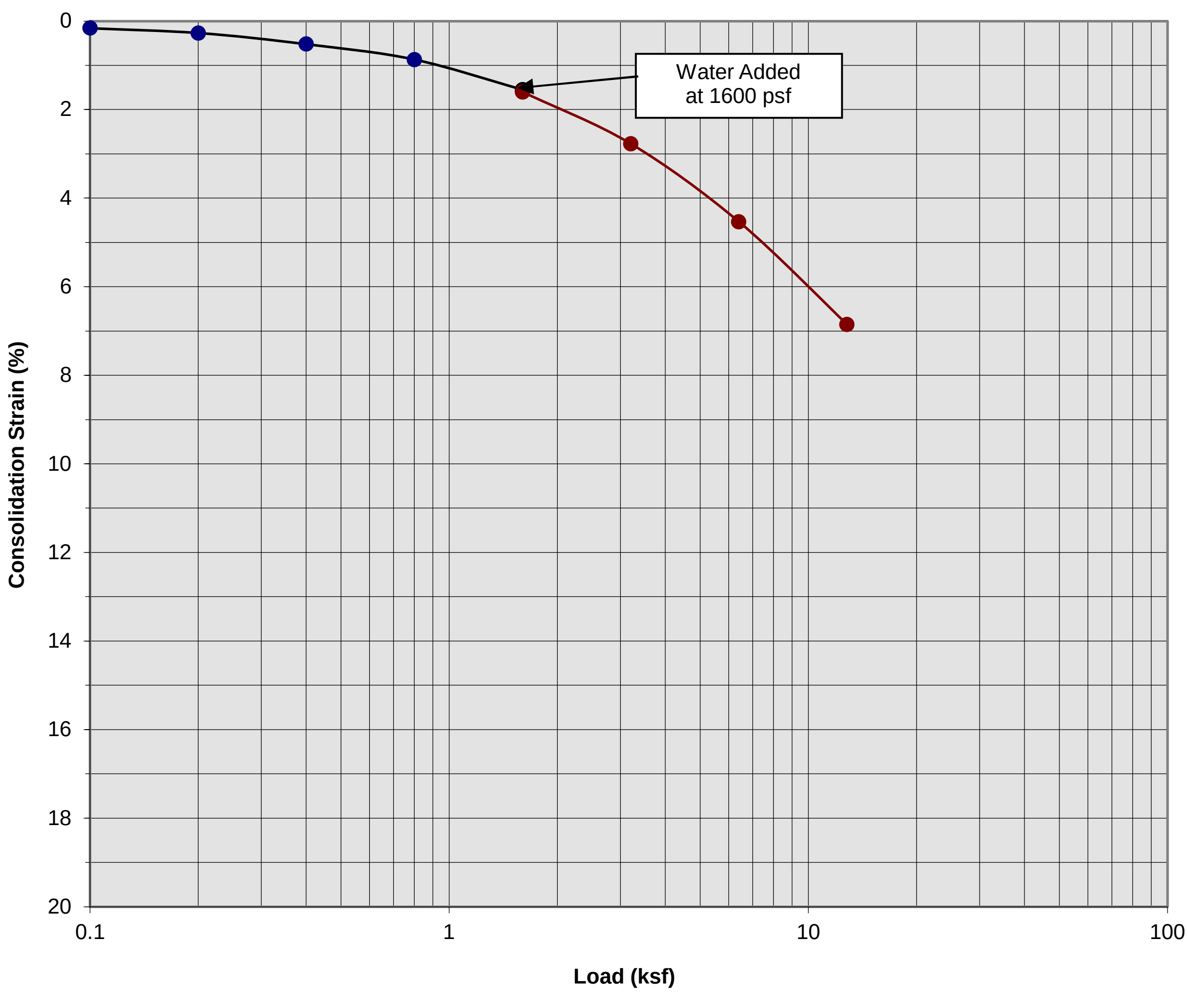
Classification: Brown fine Sandy Silt, little Clay

Boring Number:	B-3	Initial Moisture Content (%)	14
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	9 to 10	Initial Dry Density (pcf)	114.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	121.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.55

Proposed Self-Storage Facility
El Monte, CA
Project No. 21G226-1
PLATE C- 2

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

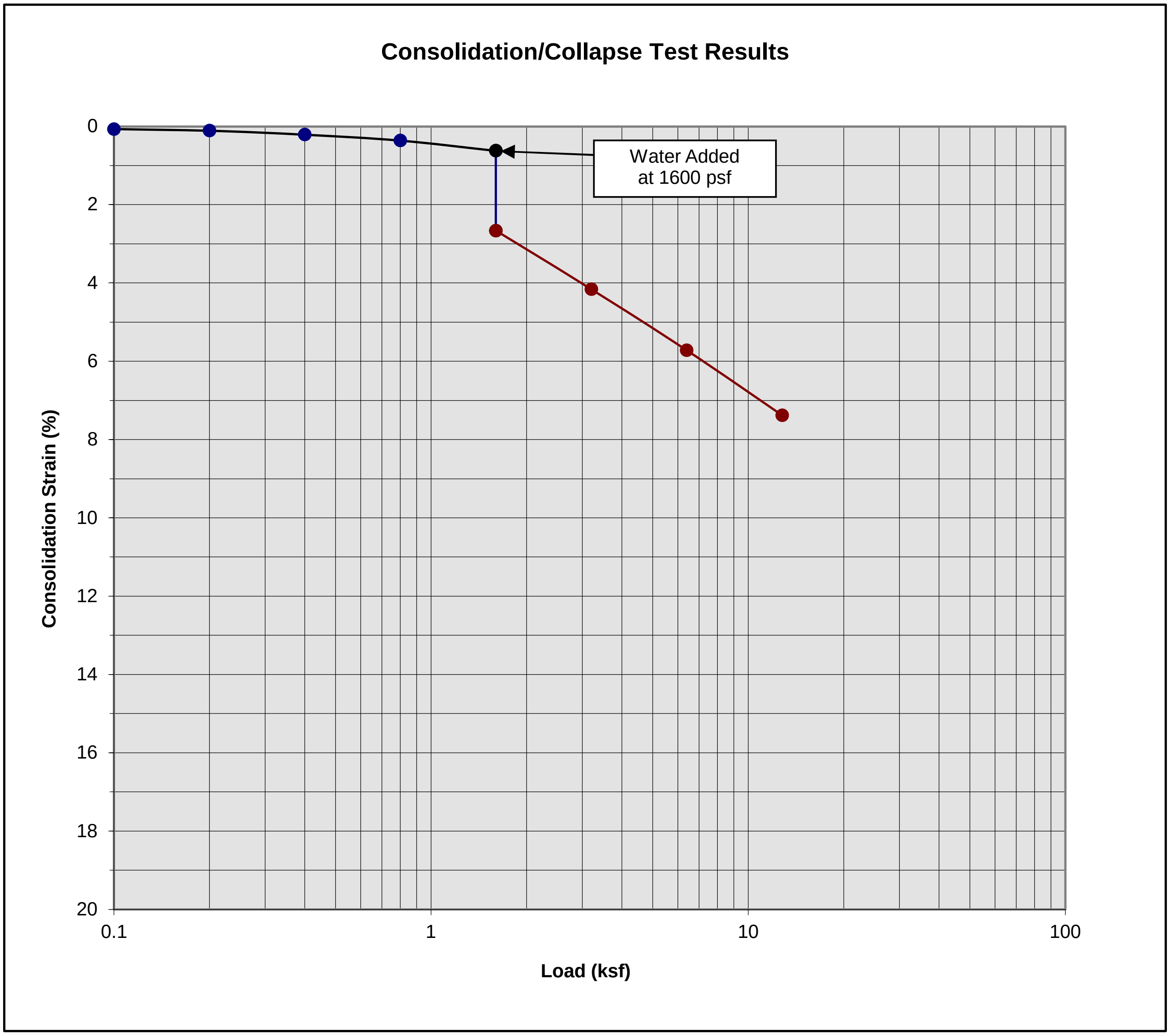


Classification: Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-3	Initial Moisture Content (%)	18
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	14 to 15	Initial Dry Density (pcf)	104.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.05

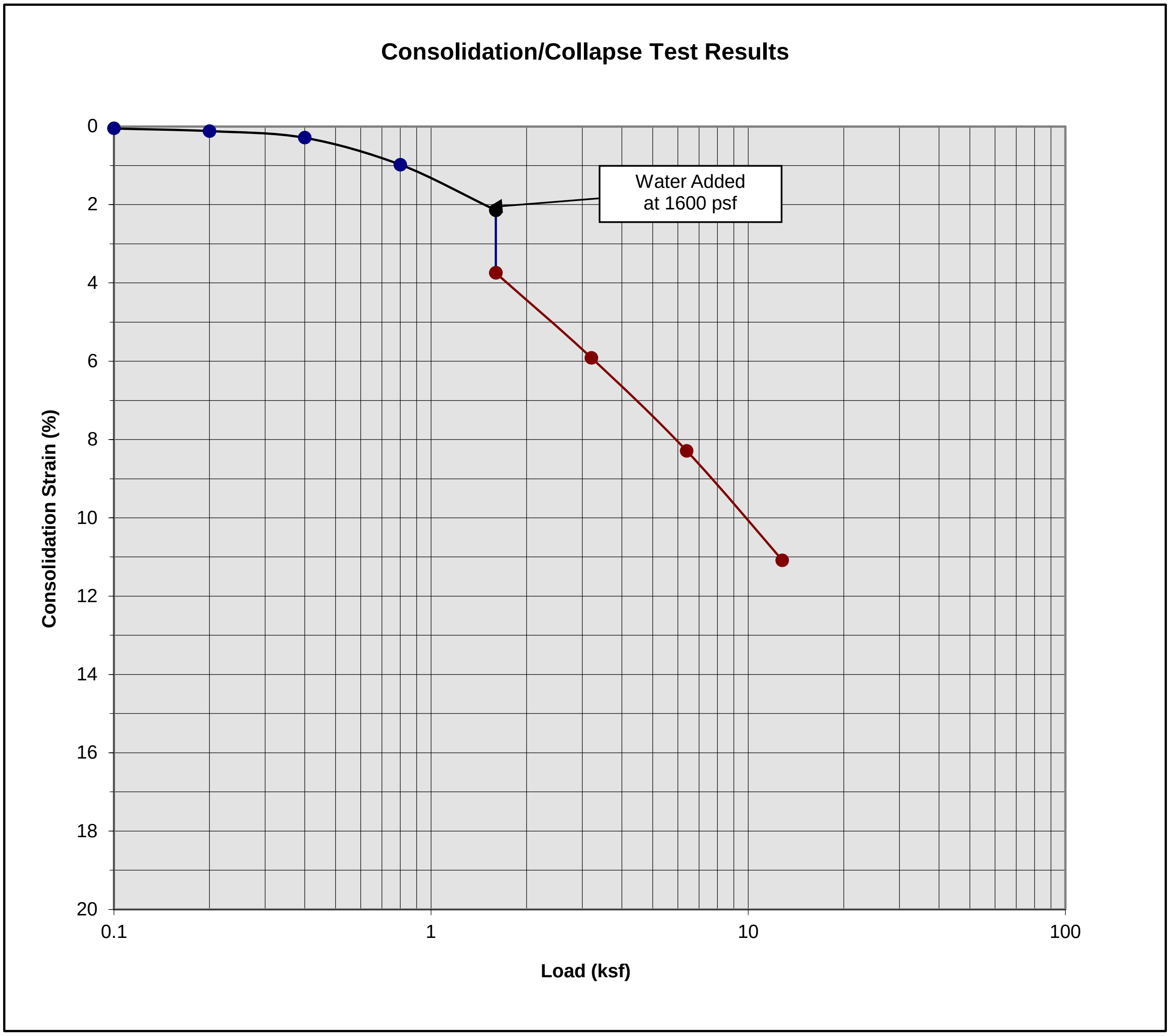
Proposed Self-Storage Facility
El Monte, CA
Project No. 21G226-1
PLATE C- 3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: Light Gray Brown fine to medium Sand, little Silt, little course Sand

Boring Number:	B-3	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	19 to 20	Initial Dry Density (pcf)	104.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	113.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.04

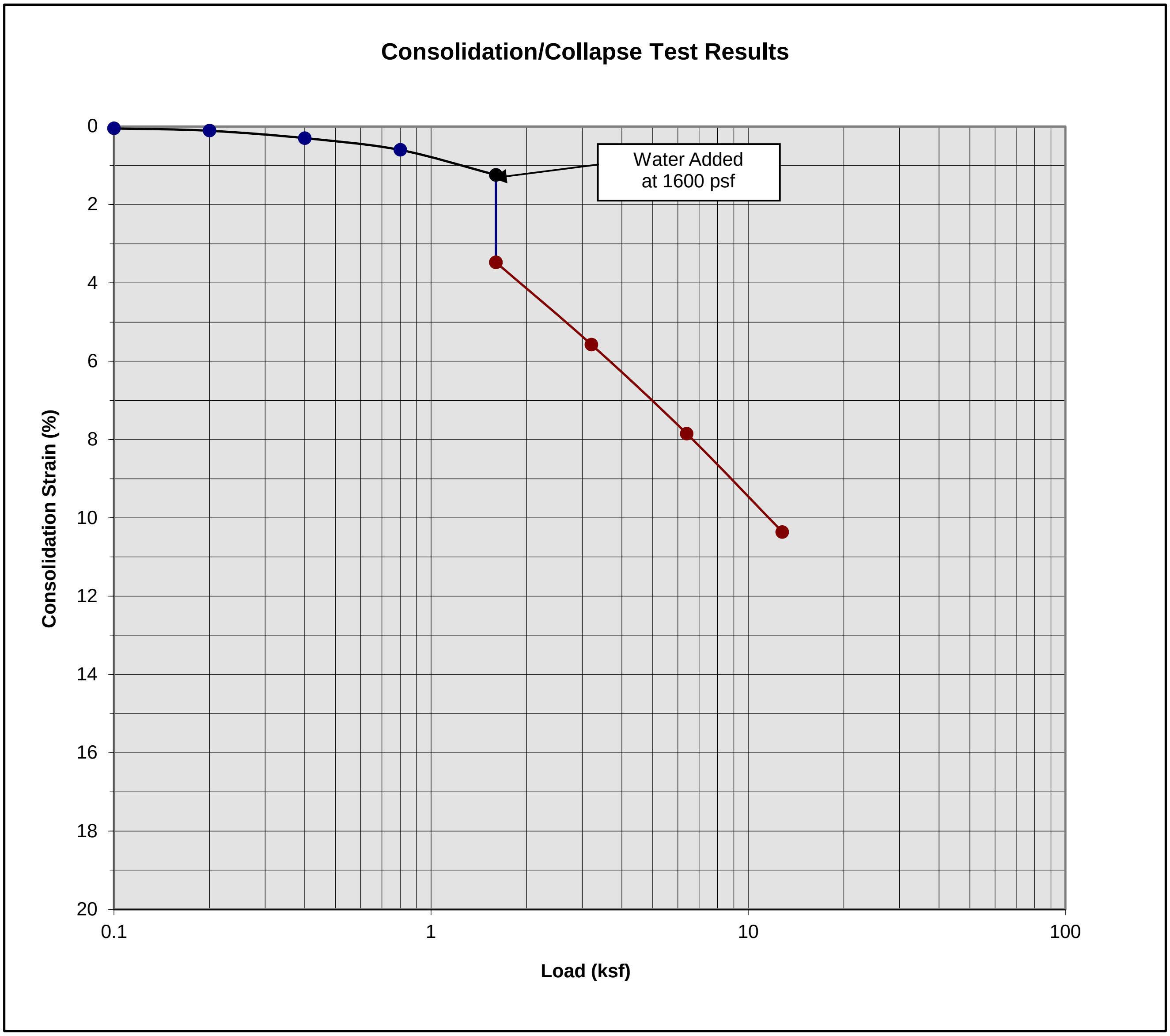


Classification: Brown Silty fine Sand

Boring Number:	B-4	Initial Moisture Content (%)	13
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	7 to 8	Initial Dry Density (pcf)	104.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	116.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.60

Proposed Self-Storage Facility
El Monte, CA
Project No. 21G226-1
PLATE C- 5

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

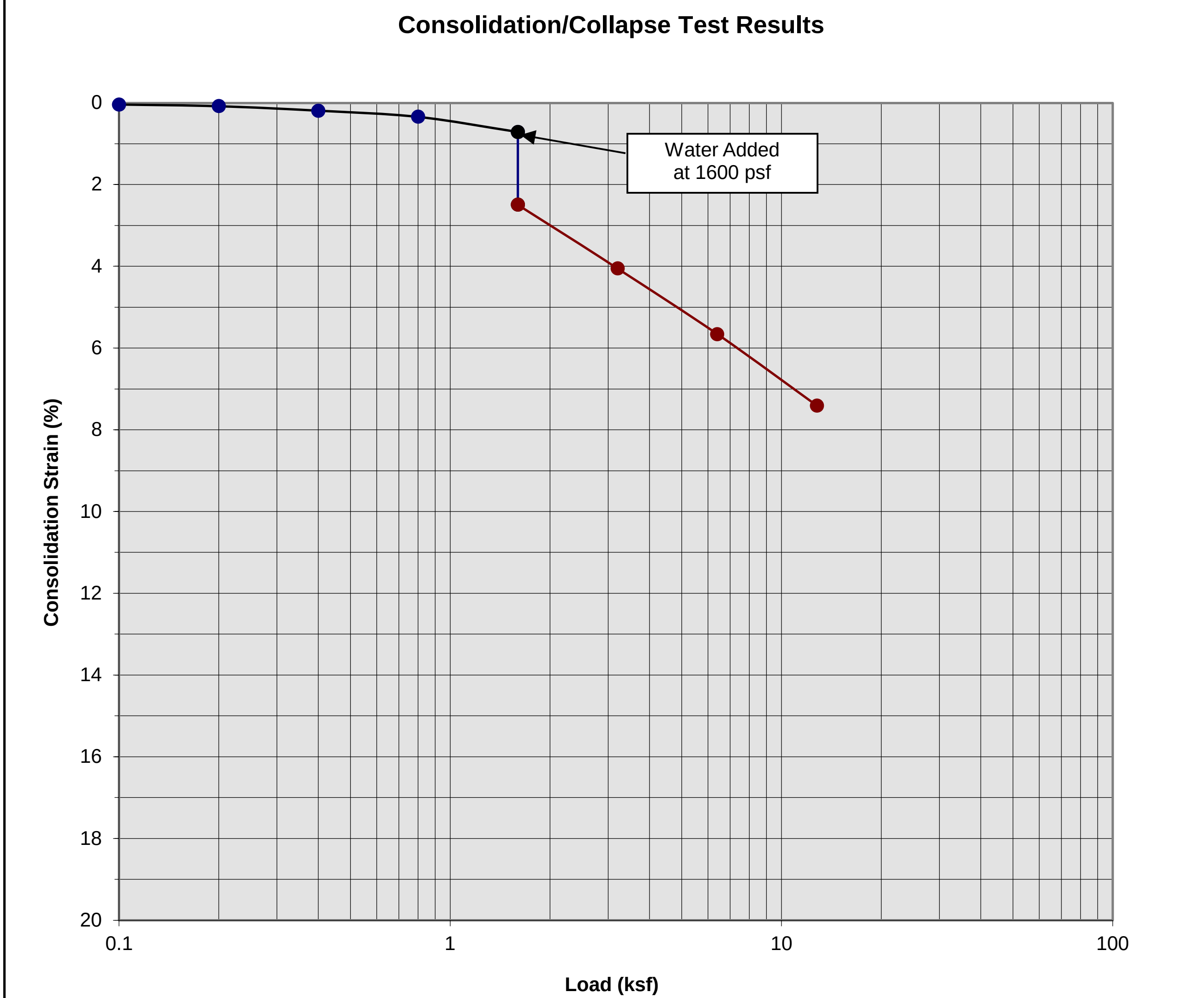


Classification: Brown Silty fine to medium Sand, little coarse Sand

Boring Number:	B-4	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	9 to 10	Initial Dry Density (pcf)	116.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	129.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.23

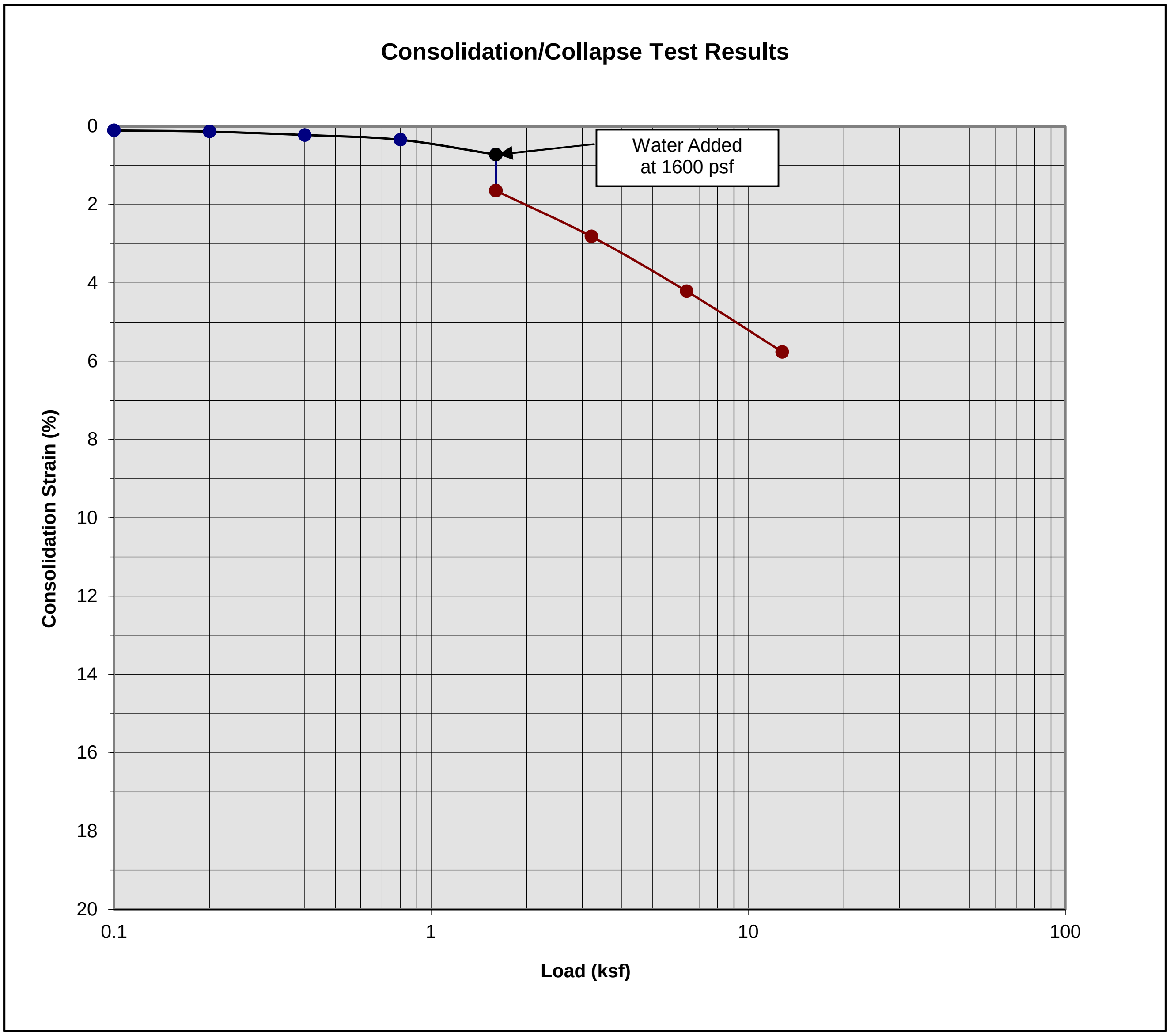
Proposed Self-Storage Facility
El Monte, CA
Project No. 21G226-1
PLATE C- 6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: Brown fine Sand, little medium to coarse Sand

Boring Number:	B-4	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	14 to 15	Initial Dry Density (pcf)	101.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.78



Classification: Light Gray fine to medium Sand, little fine Gravel

Boring Number:	B-4	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	19 to 20	Initial Dry Density (pcf)	105.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	112.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.92

Proposed Self-Storage Facility
El Monte, CA
Project No. 21G226-1
PLATE C- 8

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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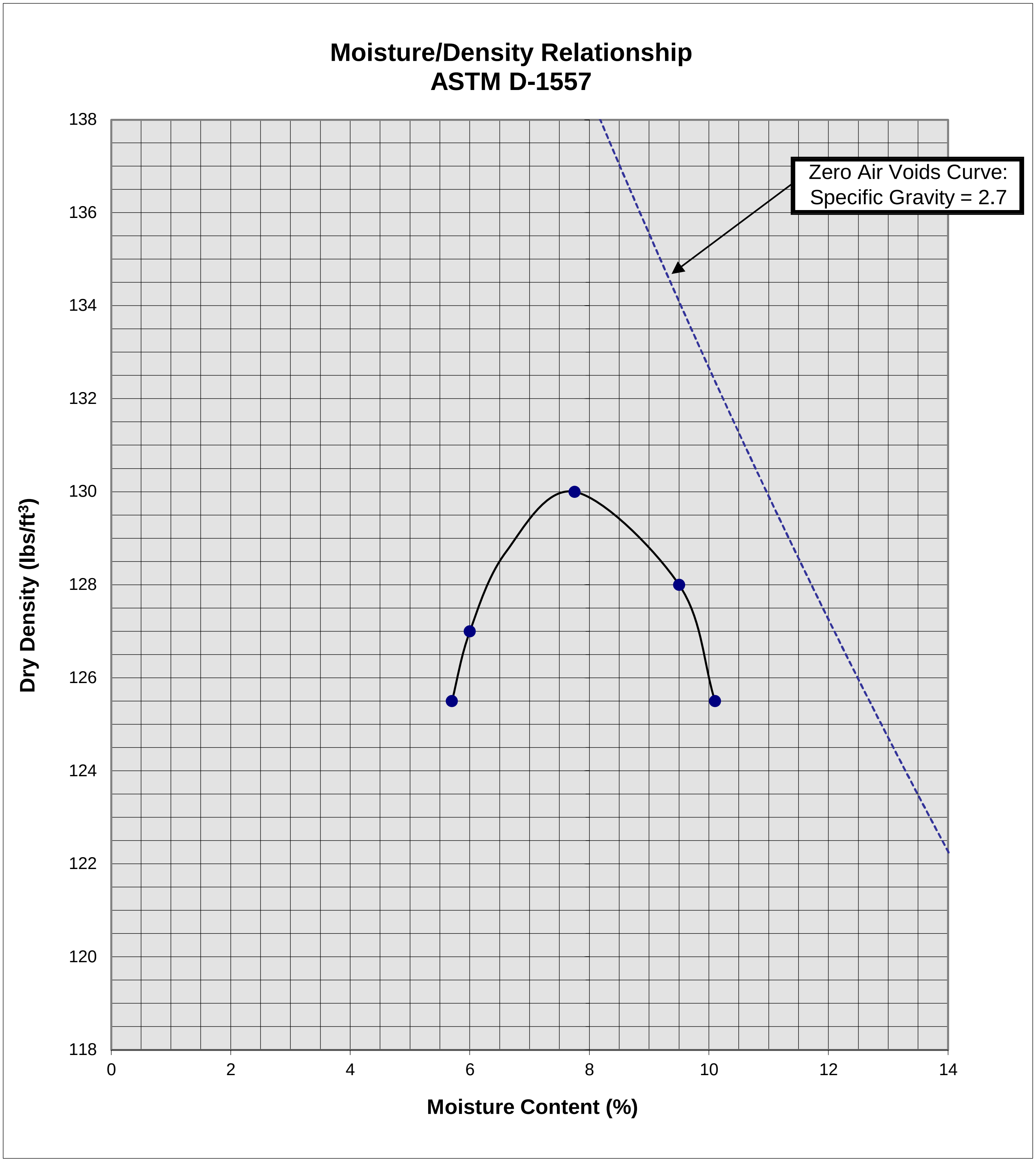

Soil ID Number	B-3 @ 0 to 5'
Optimum Moisture (%)	7.5
Maximum Dry Density (pcf)	130
Soil Classification	Gray Brown Silty fine Sand, little medium to coarse Sand

Proposed Self-Storage Facility
El Monte, California
Project No. 21G226-1
PLATE C-9

SOUTHERN CALIFORNIA GEOTECHNICAL
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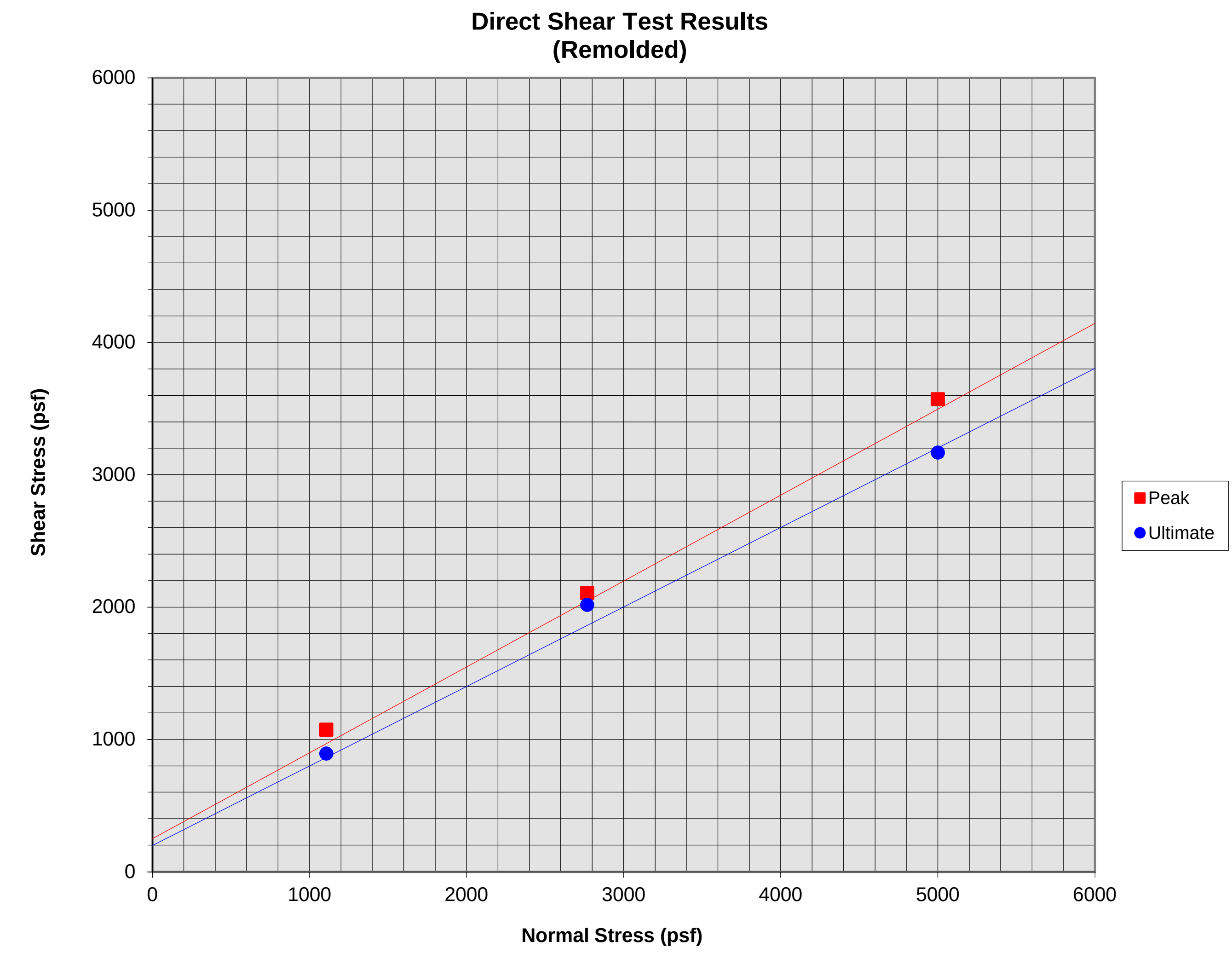

Sample Description: B-3 @ 0 to 5'

Classification: Gray Brown Silty fine Sand, little medium to coarse Sand

Sample Data		Test Results		
Remolded Moisture Content	7.5			
Final Moisture Content	15.0		Peak	Ultimate
Remolded Dry Density	117.0	ϕ (°)	33.0	31.0
Percent Compaction	90.0	C (psf)	250	200
Final Dry Density	----			
Specimen Diameter (in)	2.4			
Specimen Thickness (in)	1.0			

Proposed Self Storage Facility
El Monte, California
Project No. 21G226-1

PLATE C- 10

SOUTHERN CALIFORNIA GEOTECHNICAL
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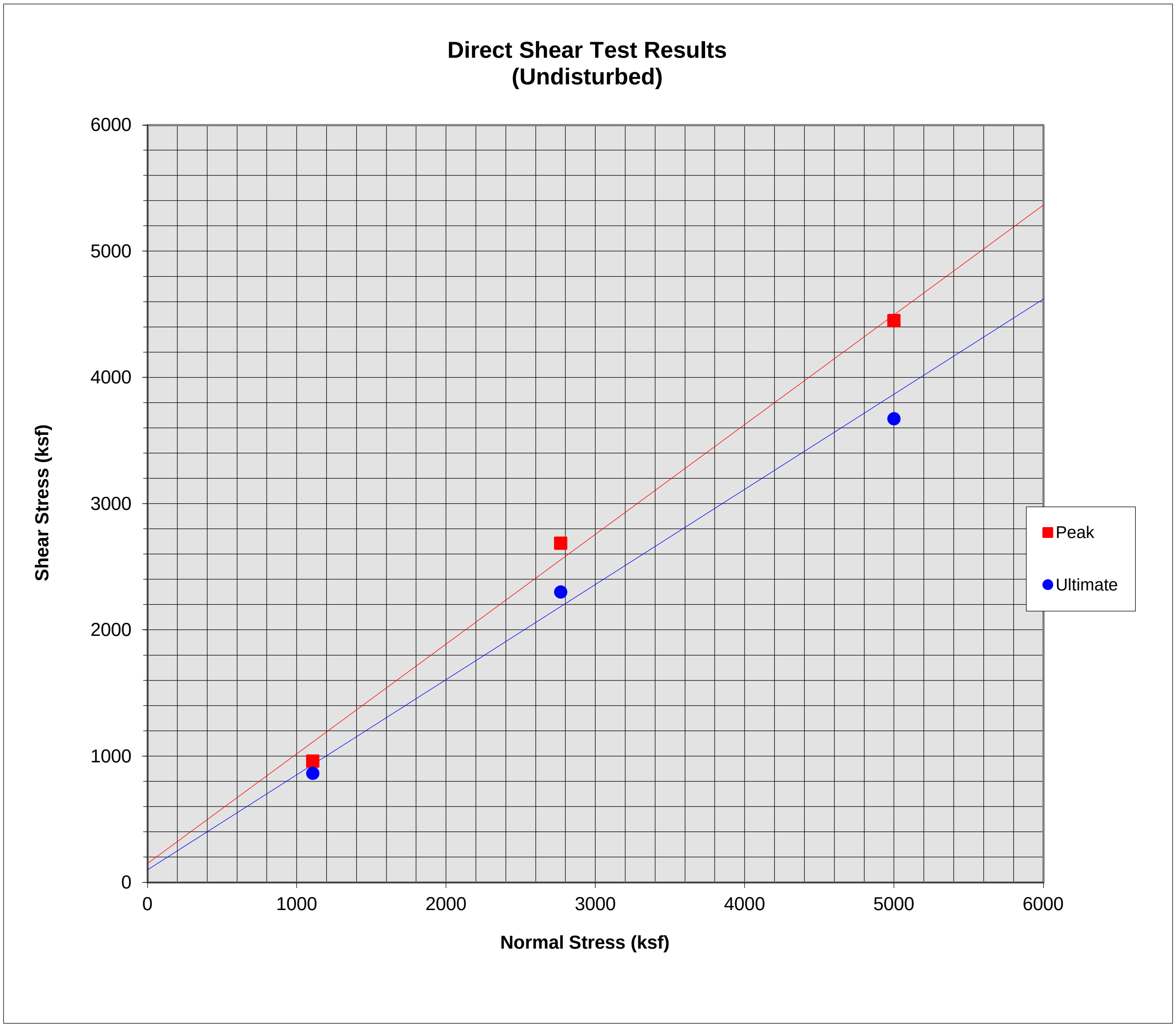


Sample Description: B-4 @ 9 to 10'

Classification: Brown Silty fine to medium Sand, little coarse Sand

Sample Data	
Initial Moisture Content	11.0
Final Moisture Content	15.0
Initial Dry Density	117.0
Final Dry Density	--
Specimen Diameter (in)	2.4
Specimen Thickness (in)	1.0

Test Results	Peak	Ultimate
ϕ (°)	41.0	37.0
C (psf)	150	100

Proposed Self Storage Facility
El Monte, California
Project No. 21G226-1
PLATE C- 11

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

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

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

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

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

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

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

Site Preparation

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

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

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

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

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

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

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

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

Compacted Fills

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated," and shall be very low to non-expansive with a maximum expansion index (EI) of 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.

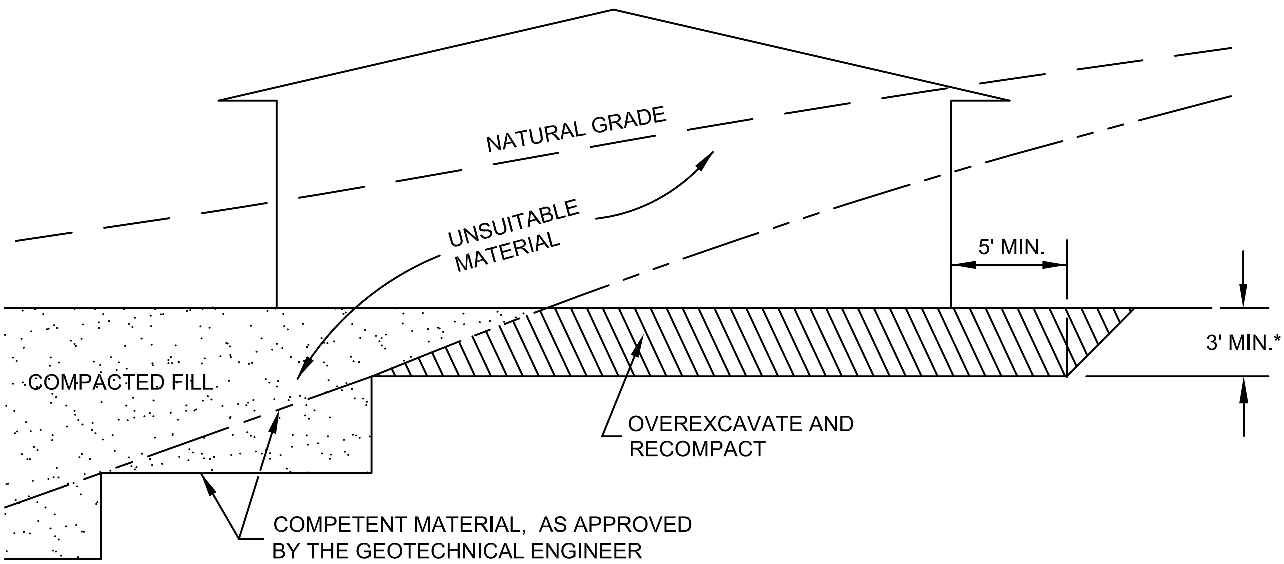
Subdrains

- 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.

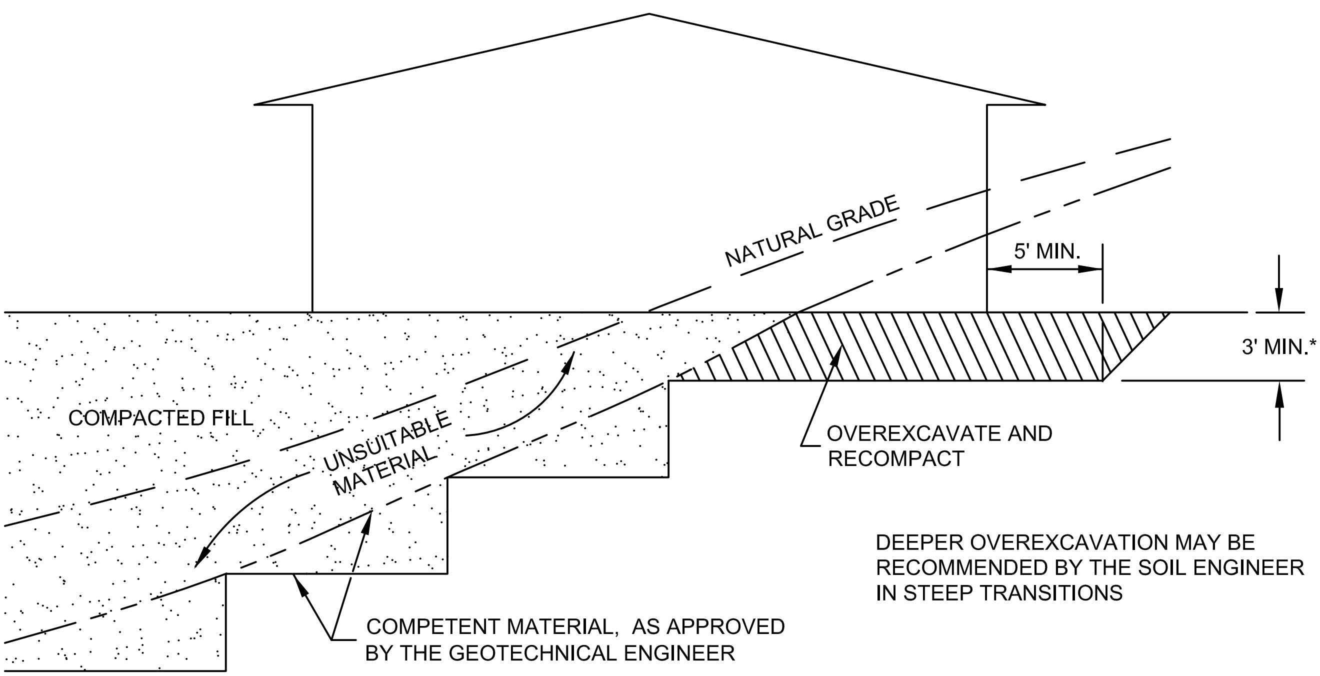
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- 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)



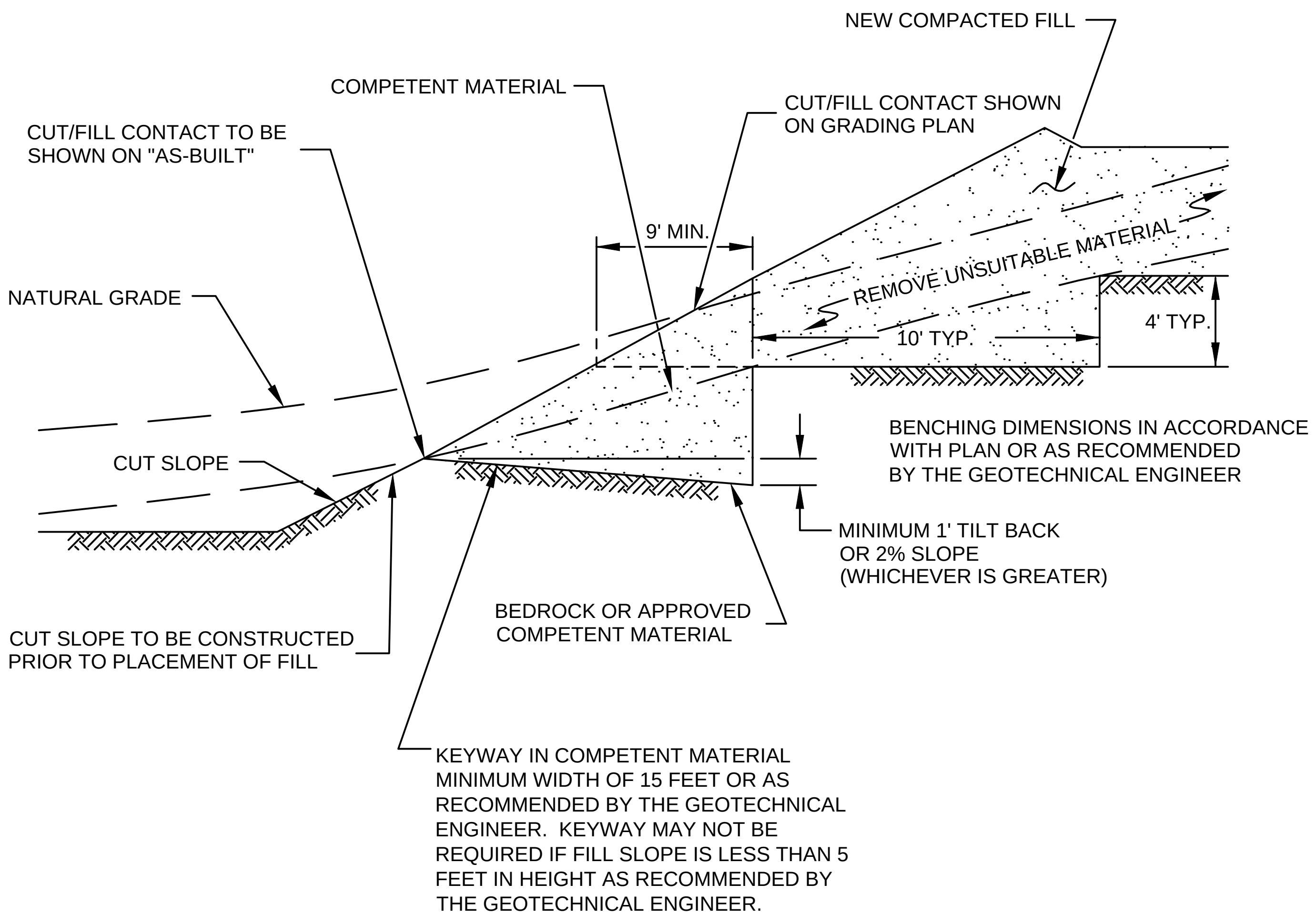
DEEPER OVEREXCAVATION MAY BE RECOMMENDED BY THE SOIL ENGINEER IN STEEP TRANSITIONS

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

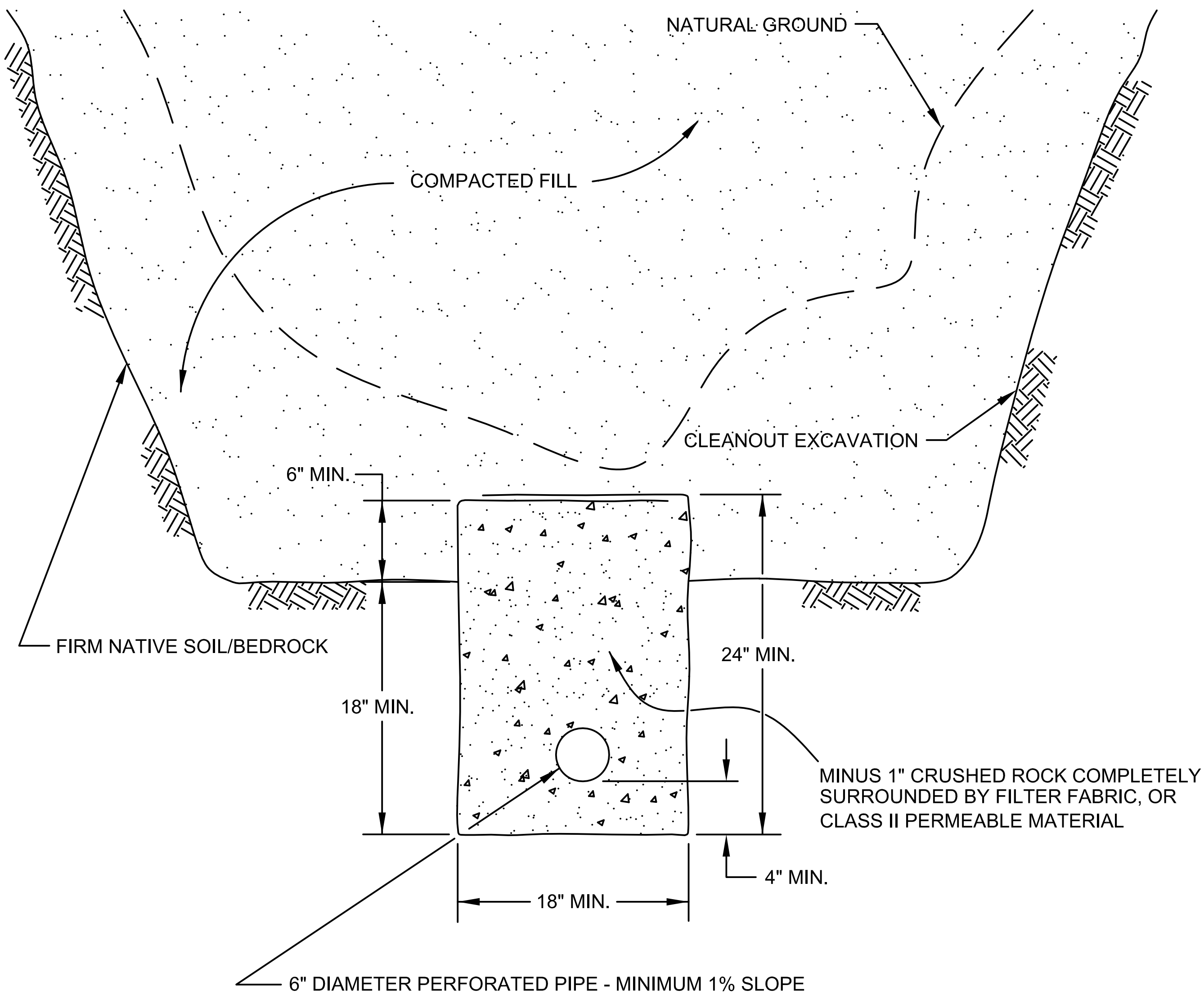
TRANSITION LOT DETAIL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-1	



FILL ABOVE CUT SLOPE DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-2	



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

GRADING GUIDE SPECIFICATIONS

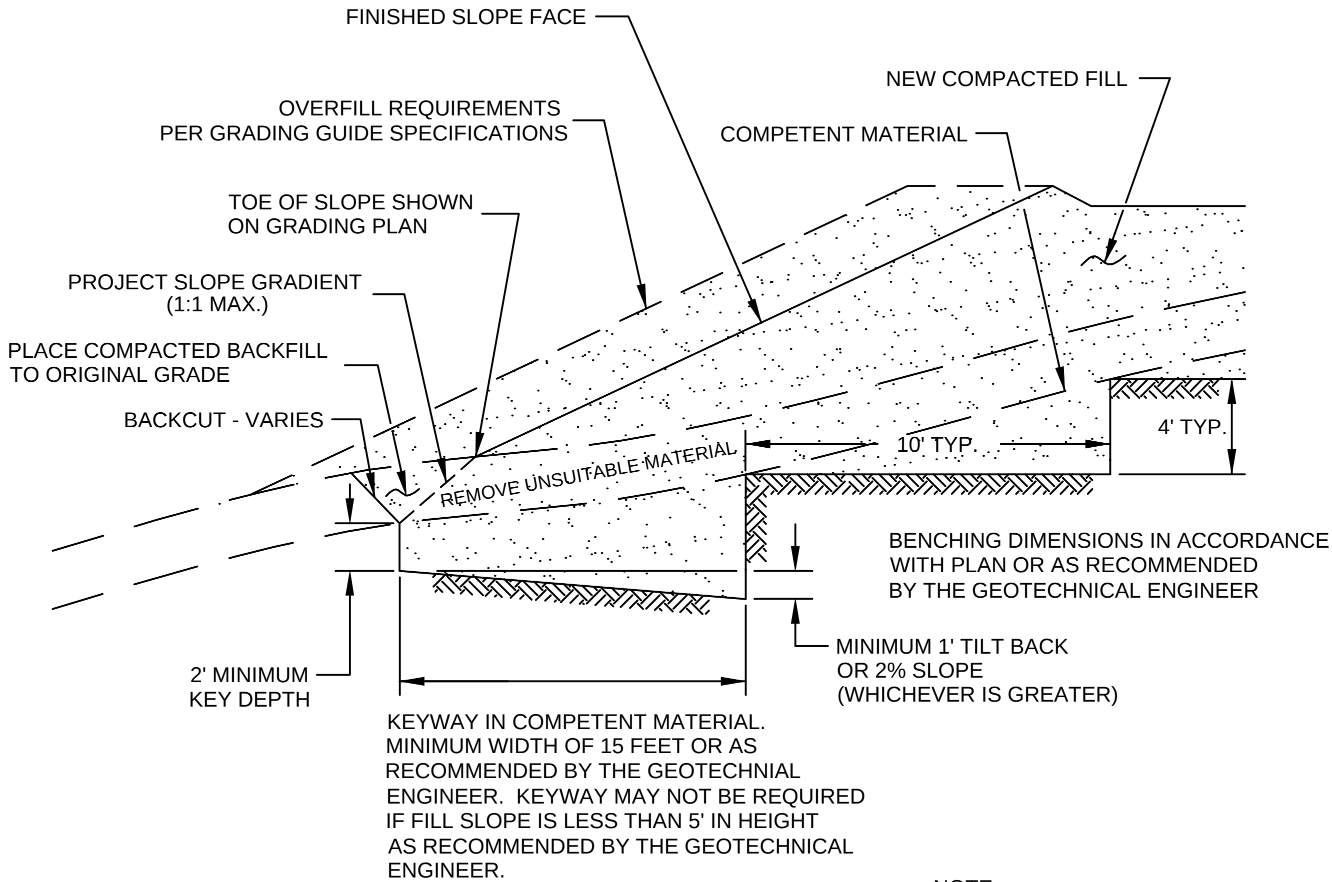
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



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

FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

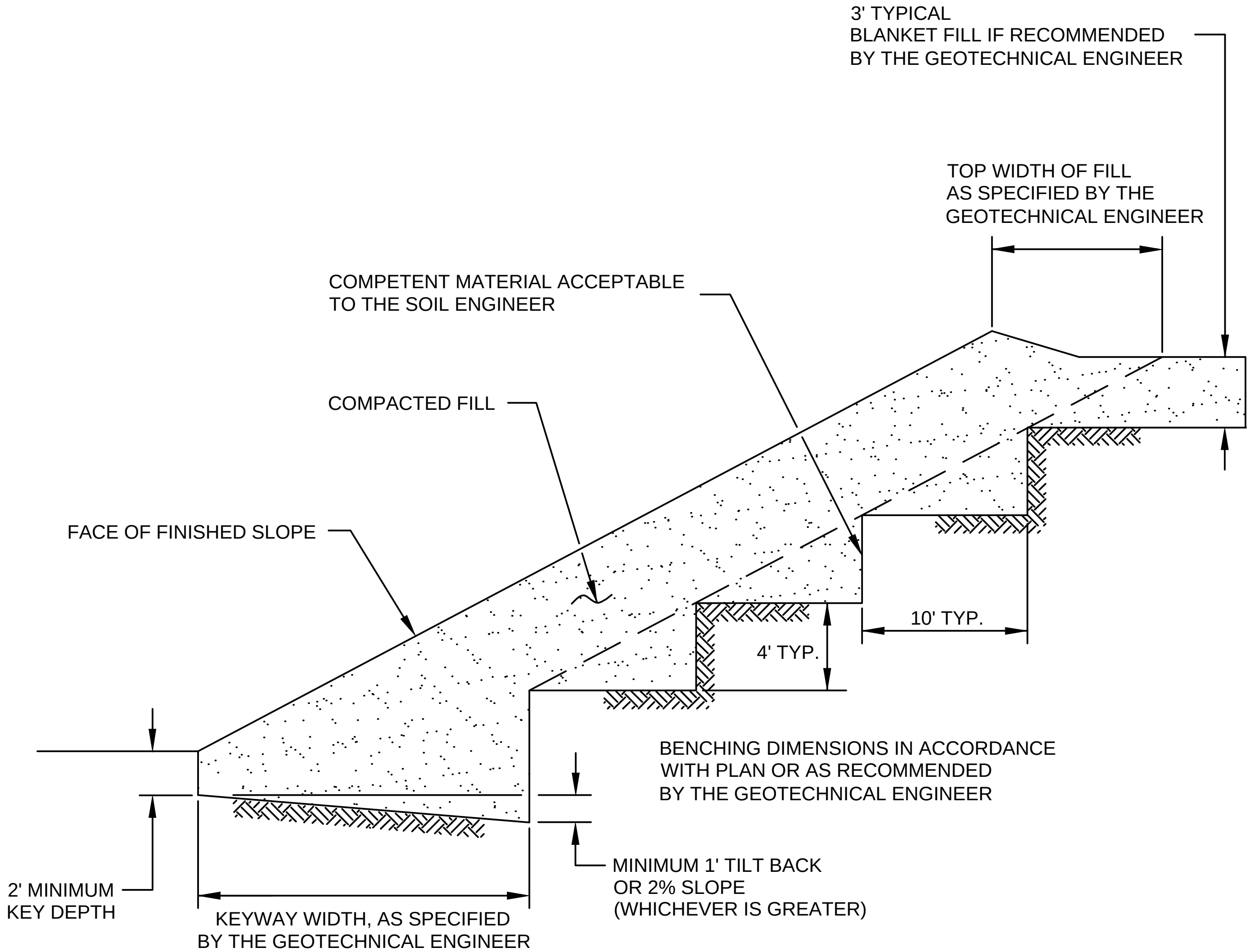
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

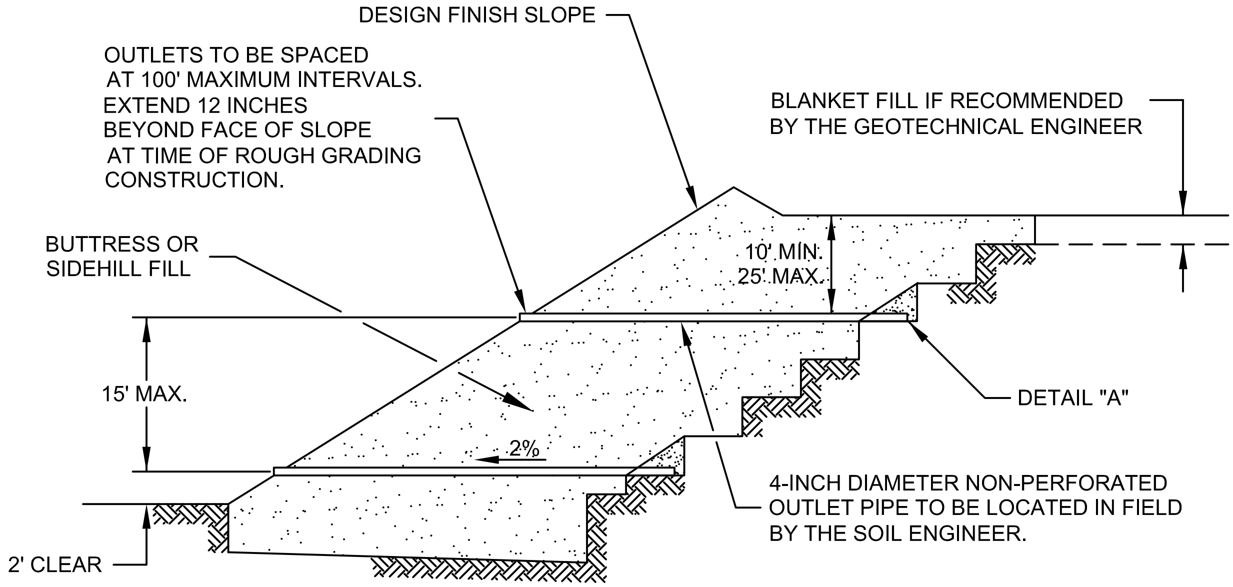
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



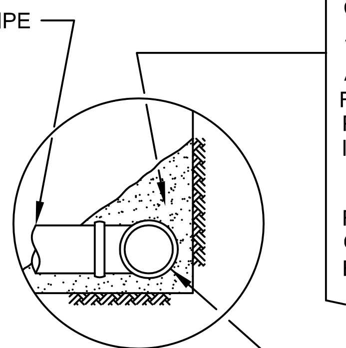
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50



DETAIL "A"

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

SLOPE FILL SUBDRAINS

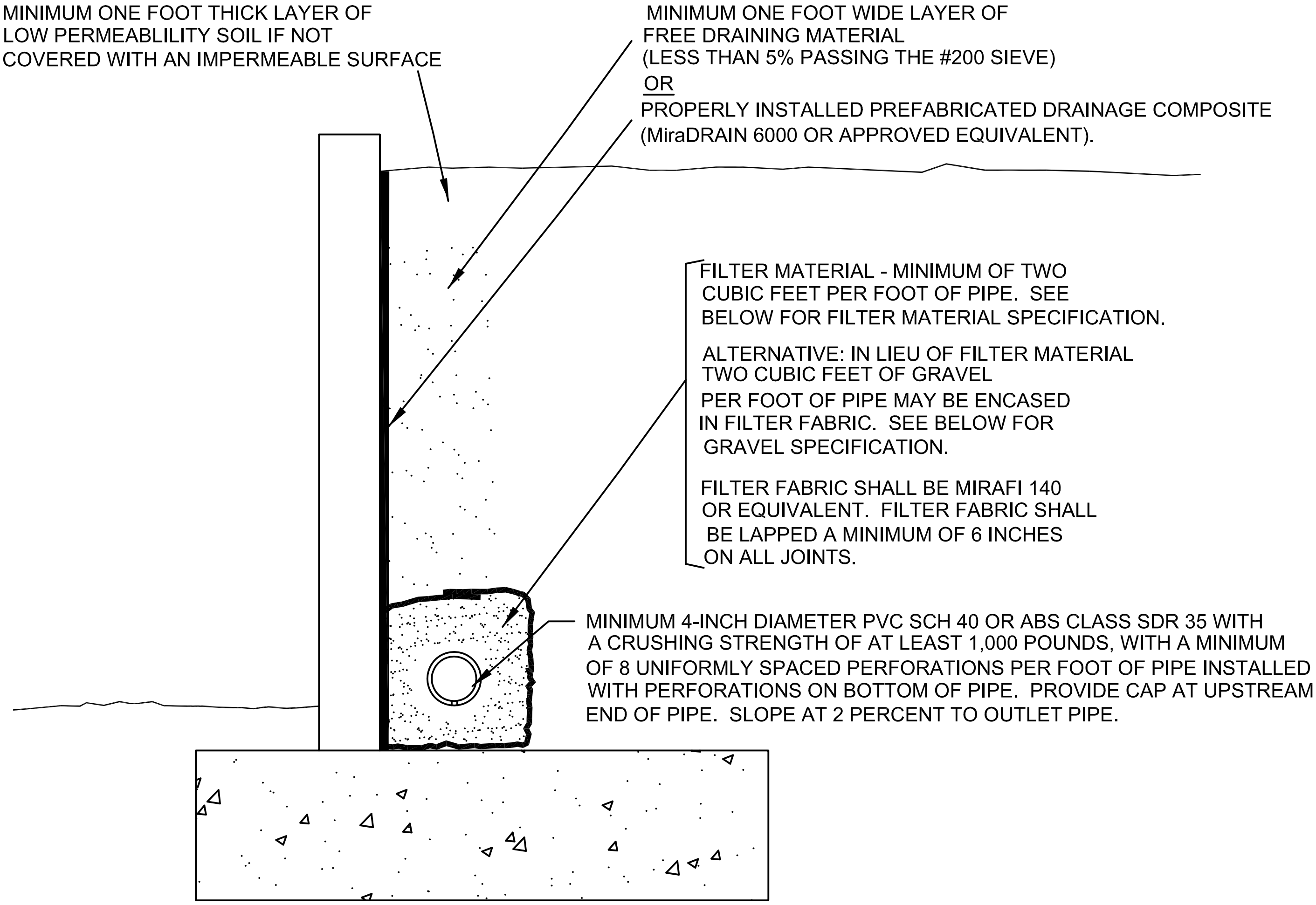
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

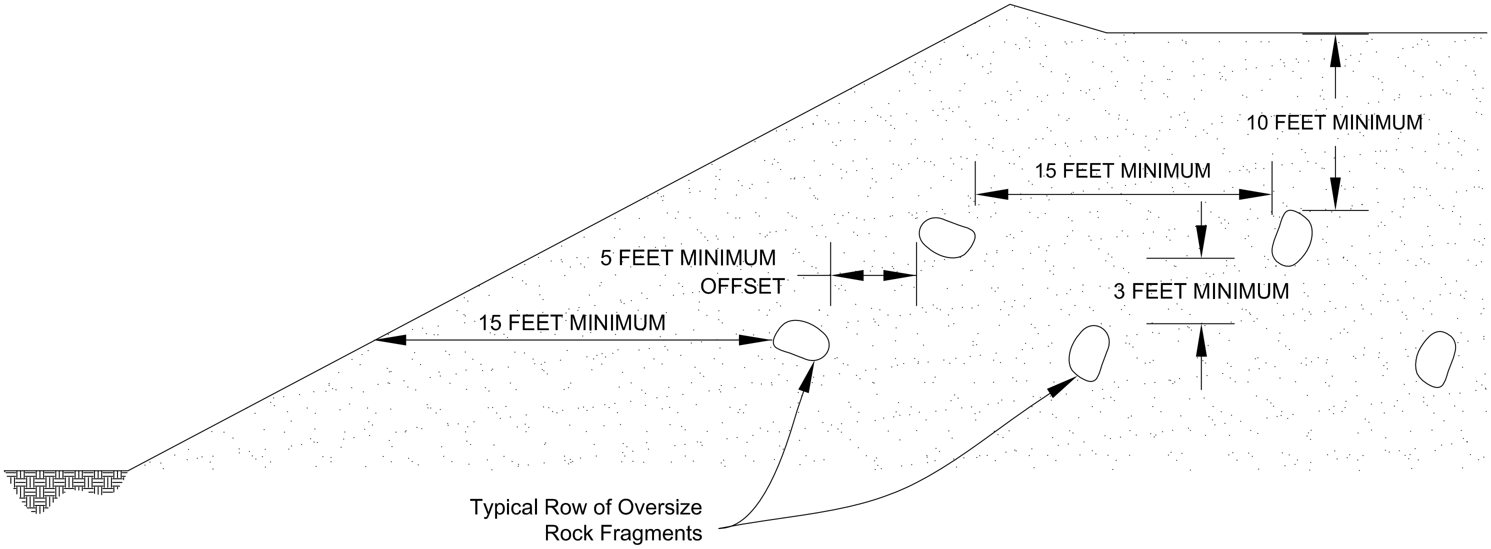
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

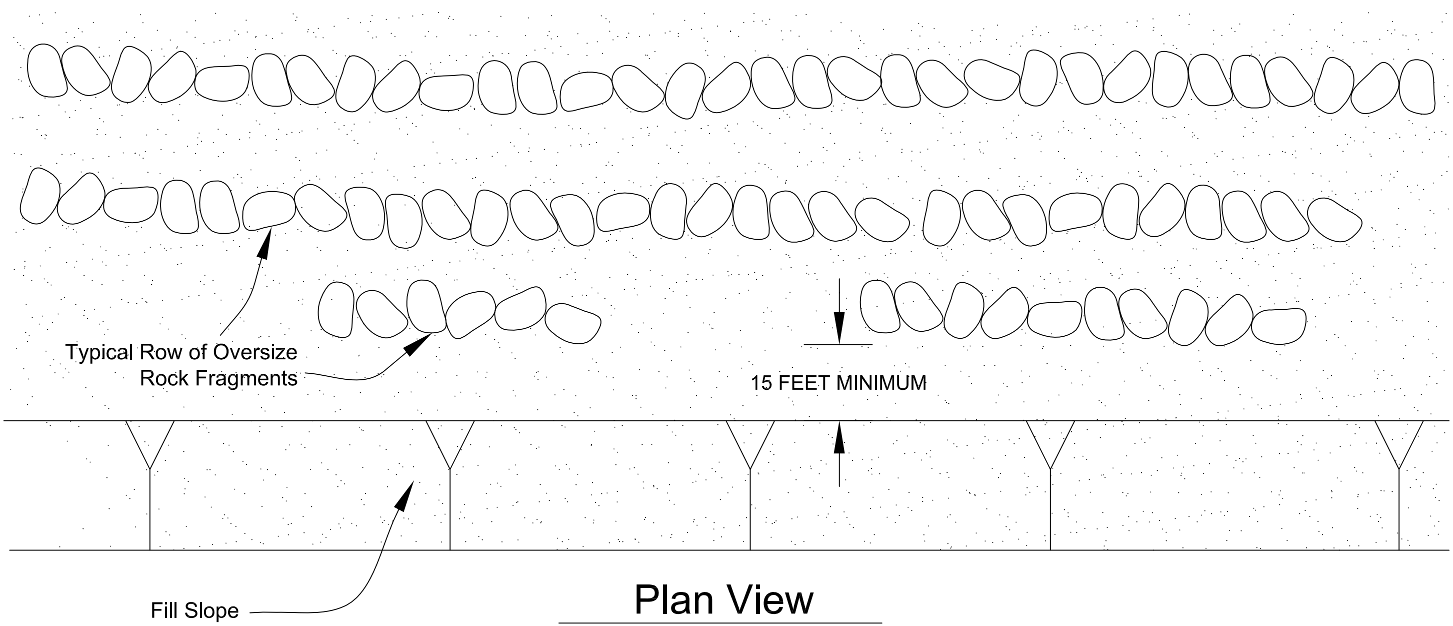
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: PM CHKD: GKM	
PLATE D-8	

APPENDIX E

OSHPD

Latitude, Longitude: 34.08565470, -118.05771546

Date	10/20/2021, 3:20:45 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	**Value**	**Description**
S_S	1.937	MCE_R ground motion. (for 0.2 second period)
S_1	0.699	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.937	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.291	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	0.837	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.921	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.937	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.172	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.283	Factored deterministic acceleration value. (0.2 second)
S1RT	0.703	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.787	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.699	Factored deterministic acceleration value. (1.0 second)
PGAd	0.913	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.892	Mapped value of the risk coefficient at short periods
C_{R1}	0.893	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 SELF STORAGE

EL MONTE, CALIFORNIA

DRAWN: MD
CHKD: RGT

SCG PROJECT
21G226-1

PLATE E-1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

TABLE OF CONTENTS

Southern California Geotechnical, Inc.
info@socalgeo.com

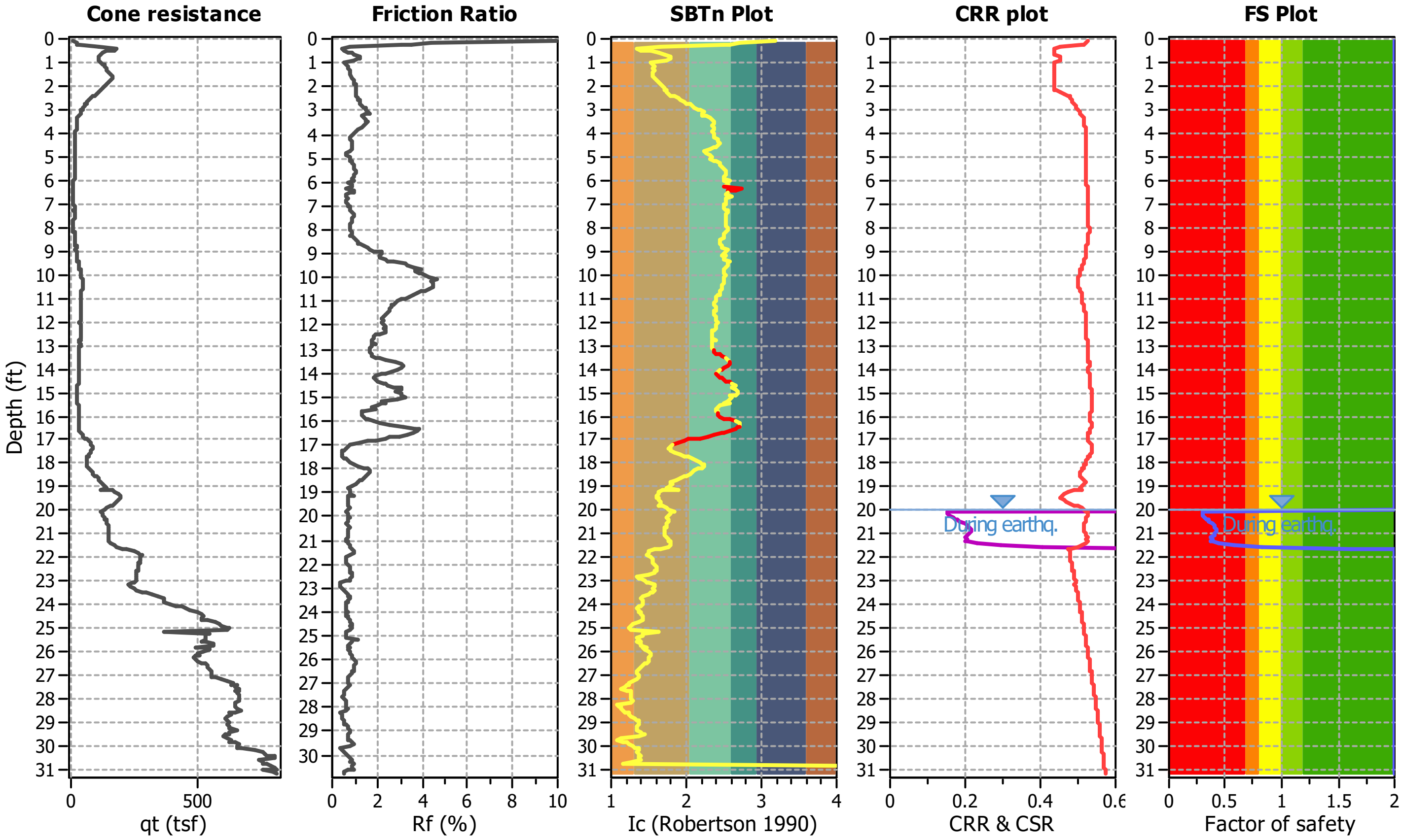
LIQUEFACTION ANALYSIS REPORT

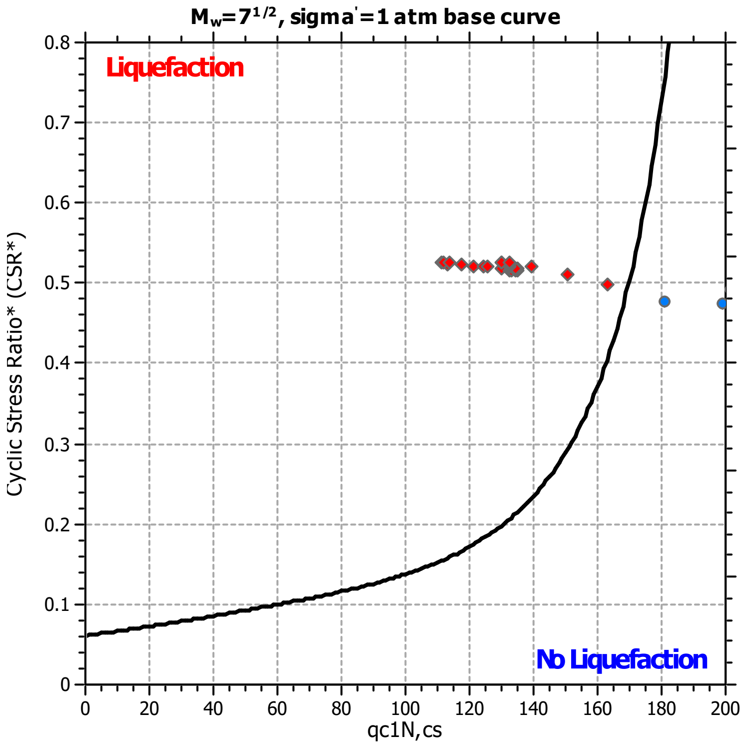
Project title : Proposed Self Storage Facility **Location : El Monte, CA**

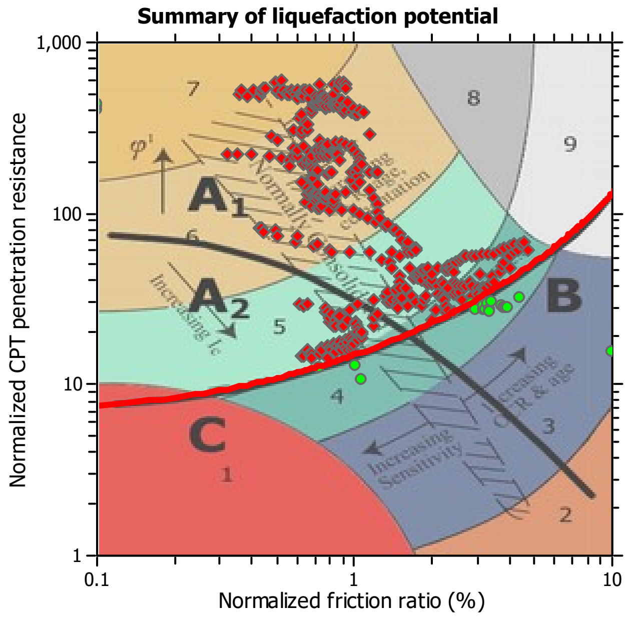
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	60.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.92	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.92	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

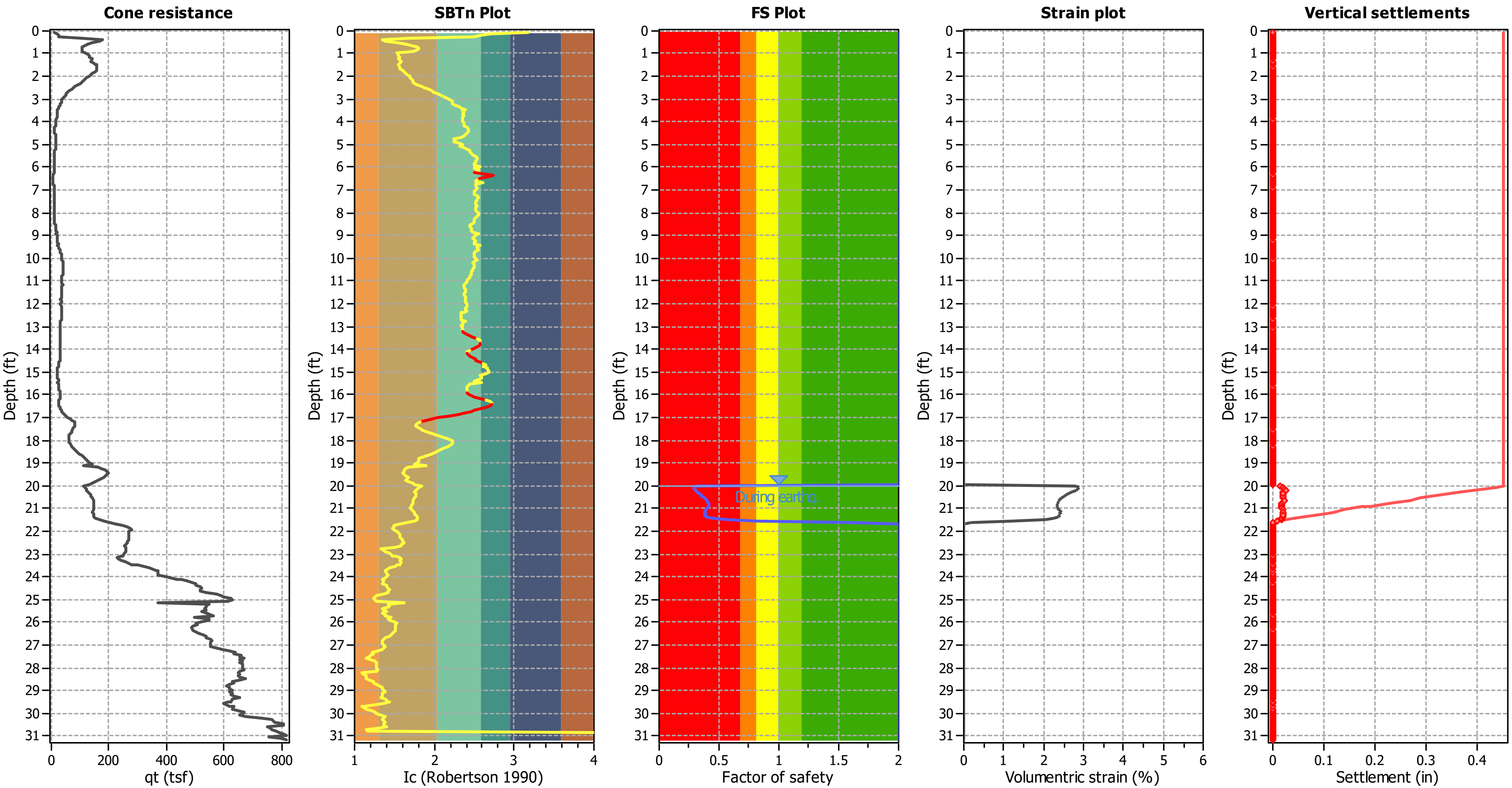






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

Estimation of post-earthquake settlements



Abbreviations

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

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

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
20.02	113.24	0.30	2.82	1.00	0.02	20.08	111.01	0.29	2.88	1.00	0.02
20.15	111.81	0.30	2.86	1.00	0.02	20.23	113.59	0.30	2.81	1.00	0.02
20.29	117.61	0.32	2.71	1.00	0.02	20.35	121.04	0.33	2.63	1.00	0.02
20.41	124.06	0.35	2.56	1.00	0.02	20.48	125.35	0.35	2.53	1.00	0.02
20.55	130.06	0.38	2.43	1.00	0.02	20.63	132.48	0.40	2.38	1.00	0.02
20.70	133.14	0.40	2.37	1.00	0.02	20.76	134.49	0.41	2.35	1.00	0.02
20.82	135.15	0.42	2.33	1.00	0.02	20.88	135.14	0.42	2.33	1.00	0.02
20.94	134.46	0.41	2.35	1.00	0.02	21.00	134.34	0.41	2.35	1.00	0.02
21.07	132.68	0.40	2.38	1.00	0.02	21.14	130.00	0.38	2.43	1.00	0.02
21.20	132.37	0.39	2.39	1.00	0.02	21.27	130.10	0.38	2.43	1.00	0.02
21.34	132.38	0.39	2.39	1.00	0.02	21.40	139.08	0.44	2.26	1.00	0.02
21.47	150.55	0.57	2.07	1.00	0.02	21.54	163.24	0.82	1.21	1.00	0.01
21.60	181.31	1.61	0.19	1.00	0.00	21.67	199.38	2.00	0.00	1.00	0.00
21.74	223.82	2.00	0.00	1.00	0.00	21.81	240.95	2.00	0.00	1.00	0.00
21.87	249.43	2.00	0.00	1.00	0.00	21.93	254.39	2.00	0.00	1.00	0.00
22.00	244.24	2.00	0.00	1.00	0.00	22.06	243.22	2.00	0.00	1.00	0.00
22.12	243.90	2.00	0.00	1.00	0.00	22.20	241.92	2.00	0.00	1.00	0.00
22.25	243.59	2.00	0.00	1.00	0.00	22.33	239.25	2.00	0.00	1.00	0.00
22.39	239.21	2.00	0.00	1.00	0.00	22.46	238.26	2.00	0.00	1.00	0.00
22.52	234.07	2.00	0.00	1.00	0.00	22.59	231.83	2.00	0.00	1.00	0.00
22.66	227.47	2.00	0.00	1.00	0.00	22.72	228.56	2.00	0.00	1.00	0.00
22.79	228.95	2.00	0.00	1.00	0.00	22.86	229.59	2.00	0.00	1.00	0.00
22.93	228.93	2.00	0.00	1.00	0.00	23.00	225.98	2.00	0.00	1.00	0.00
23.06	222.00	2.00	0.00	1.00	0.00	23.10	213.88	2.00	0.00	1.00	0.00
23.18	201.91	2.00	0.00	1.00	0.00	23.24	207.45	2.00	0.00	1.00	0.00
23.31	216.70	2.00	0.00	1.00	0.00	23.38	228.27	2.00	0.00	1.00	0.00
23.45	248.44	2.00	0.00	1.00	0.00	23.50	265.18	2.00	0.00	1.00	0.00
23.56	283.11	2.00	0.00	1.00	0.00	23.64	302.70	2.00	0.00	1.00	0.00
23.71	322.60	2.00	0.00	1.00	0.00	23.76	326.00	2.00	0.00	1.00	0.00
23.83	326.10	2.00	0.00	1.00	0.00	23.90	326.24	2.00	0.00	1.00	0.00
23.97	347.67	2.00	0.00	1.00	0.00	24.04	364.52	2.00	0.00	1.00	0.00
24.09	383.29	2.00	0.00	1.00	0.00	24.15	396.79	2.00	0.00	1.00	0.00
24.21	411.96	2.00	0.00	1.00	0.00	24.28	438.47	2.00	0.00	1.00	0.00
24.36	440.57	2.00	0.00	1.00	0.00	24.42	450.88	2.00	0.00	1.00	0.00
24.49	456.00	2.00	0.00	1.00	0.00	24.55	449.88	2.00	0.00	1.00	0.00
24.62	450.96	2.00	0.00	1.00	0.00	24.69	469.44	2.00	0.00	1.00	0.00
24.75	499.88	2.00	0.00	1.00	0.00	24.82	511.29	2.00	0.00	1.00	0.00
24.89	520.53	2.00	0.00	1.00	0.00	24.95	540.65	2.00	0.00	1.00	0.00
25.01	543.85	2.00	0.00	1.00	0.00	25.07	534.16	2.00	0.00	1.00	0.00
25.14	319.68	2.00	0.00	1.00	0.00	25.20	475.93	2.00	0.00	1.00	0.00
25.27	463.71	2.00	0.00	1.00	0.00	25.34	462.79	2.00	0.00	1.00	0.00
25.42	458.58	2.00	0.00	1.00	0.00	25.47	462.31	2.00	0.00	1.00	0.00
25.53	448.88	2.00	0.00	1.00	0.00	25.59	461.93	2.00	0.00	1.00	0.00
25.69	482.84	2.00	0.00	1.00	0.00	25.74	484.37	2.00	0.00	1.00	0.00
25.79	425.83	2.00	0.00	1.00	0.00	25.85	469.44	2.00	0.00	1.00	0.00
25.93	469.97	2.00	0.00	1.00	0.00	25.99	448.34	2.00	0.00	1.00	0.00
26.06	428.55	2.00	0.00	1.00	0.00	26.13	433.91	2.00	0.00	1.00	0.00
26.20	419.67	2.00	0.00	1.00	0.00	26.25	414.38	2.00	0.00	1.00	0.00

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

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
26.34	421.27	2.00	0.00	1.00	0.00	26.40	427.08	2.00	0.00	1.00	0.00
26.47	438.47	2.00	0.00	1.00	0.00	26.53	451.24	2.00	0.00	1.00	0.00
26.59	458.09	2.00	0.00	1.00	0.00	26.66	459.16	2.00	0.00	1.00	0.00
26.73	469.47	2.00	0.00	1.00	0.00	26.80	474.25	2.00	0.00	1.00	0.00
26.86	470.50	2.00	0.00	1.00	0.00	26.92	470.57	2.00	0.00	1.00	0.00
26.98	470.30	2.00	0.00	1.00	0.00	27.05	470.32	2.00	0.00	1.00	0.00
27.12	484.59	2.00	0.00	1.00	0.00	27.17	502.04	2.00	0.00	1.00	0.00
27.24	522.65	2.00	0.00	1.00	0.00	27.30	540.96	2.00	0.00	1.00	0.00
27.37	533.41	2.00	0.00	1.00	0.00	27.44	556.71	2.00	0.00	1.00	0.00
27.50	551.34	2.00	0.00	1.00	0.00	27.56	562.53	2.00	0.00	1.00	0.00
27.63	548.43	2.00	0.00	1.00	0.00	27.70	560.64	2.00	0.00	1.00	0.00
27.77	551.66	2.00	0.00	1.00	0.00	27.83	558.76	2.00	0.00	1.00	0.00
27.90	558.80	2.00	0.00	1.00	0.00	27.96	558.00	2.00	0.00	1.00	0.00
28.02	557.61	2.00	0.00	1.00	0.00	28.10	559.58	2.00	0.00	1.00	0.00
28.16	546.61	2.00	0.00	1.00	0.00	28.23	541.61	2.00	0.00	1.00	0.00
28.29	544.97	2.00	0.00	1.00	0.00	28.36	543.75	2.00	0.00	1.00	0.00
28.43	546.62	2.00	0.00	1.00	0.00	28.50	560.62	2.00	0.00	1.00	0.00
28.55	543.03	2.00	0.00	1.00	0.00	28.62	525.33	2.00	0.00	1.00	0.00
28.68	528.29	2.00	0.00	1.00	0.00	28.75	513.79	2.00	0.00	1.00	0.00
28.81	506.74	2.00	0.00	1.00	0.00	28.88	514.17	2.00	0.00	1.00	0.00
28.94	514.92	2.00	0.00	1.00	0.00	29.01	520.83	2.00	0.00	1.00	0.00
29.07	511.74	2.00	0.00	1.00	0.00	29.14	519.01	2.00	0.00	1.00	0.00
29.21	517.48	2.00	0.00	1.00	0.00	29.27	522.34	2.00	0.00	1.00	0.00
29.34	542.11	2.00	0.00	1.00	0.00	29.40	518.04	2.00	0.00	1.00	0.00
29.47	515.52	2.00	0.00	1.00	0.00	29.53	504.93	2.00	0.00	1.00	0.00
29.60	495.24	2.00	0.00	1.00	0.00	29.67	509.49	2.00	0.00	1.00	0.00
29.73	521.94	2.00	0.00	1.00	0.00	29.79	517.92	2.00	0.00	1.00	0.00
29.86	534.94	2.00	0.00	1.00	0.00	29.93	549.14	2.00	0.00	1.00	0.00
29.99	543.48	2.00	0.00	1.00	0.00	30.05	537.85	2.00	0.00	1.00	0.00
30.12	553.41	2.00	0.00	1.00	0.00	30.19	595.01	2.00	0.00	1.00	0.00
30.26	625.14	2.00	0.00	1.00	0.00	30.32	628.56	2.00	0.00	1.00	0.00
30.39	635.33	2.00	0.00	1.00	0.00	30.45	657.89	2.00	0.00	1.00	0.00
30.52	657.15	2.00	0.00	1.00	0.00	30.59	610.51	2.00	0.00	1.00	0.00
30.65	619.69	2.00	0.00	1.00	0.00	30.71	623.16	2.00	0.00	1.00	0.00
30.78	634.35	2.00	0.00	1.00	0.00	30.84	645.08	2.00	0.00	1.00	0.00
30.91	656.92	2.00	0.00	1.00	0.00	30.97	662.78	2.00	0.00	1.00	0.00
31.04	616.16	2.00	0.00	1.00	0.00	31.10	649.55	2.00	0.00	1.00	0.00
31.17	663.11	2.00	0.00	1.00	0.00						

Total estimated settlement: 0.45

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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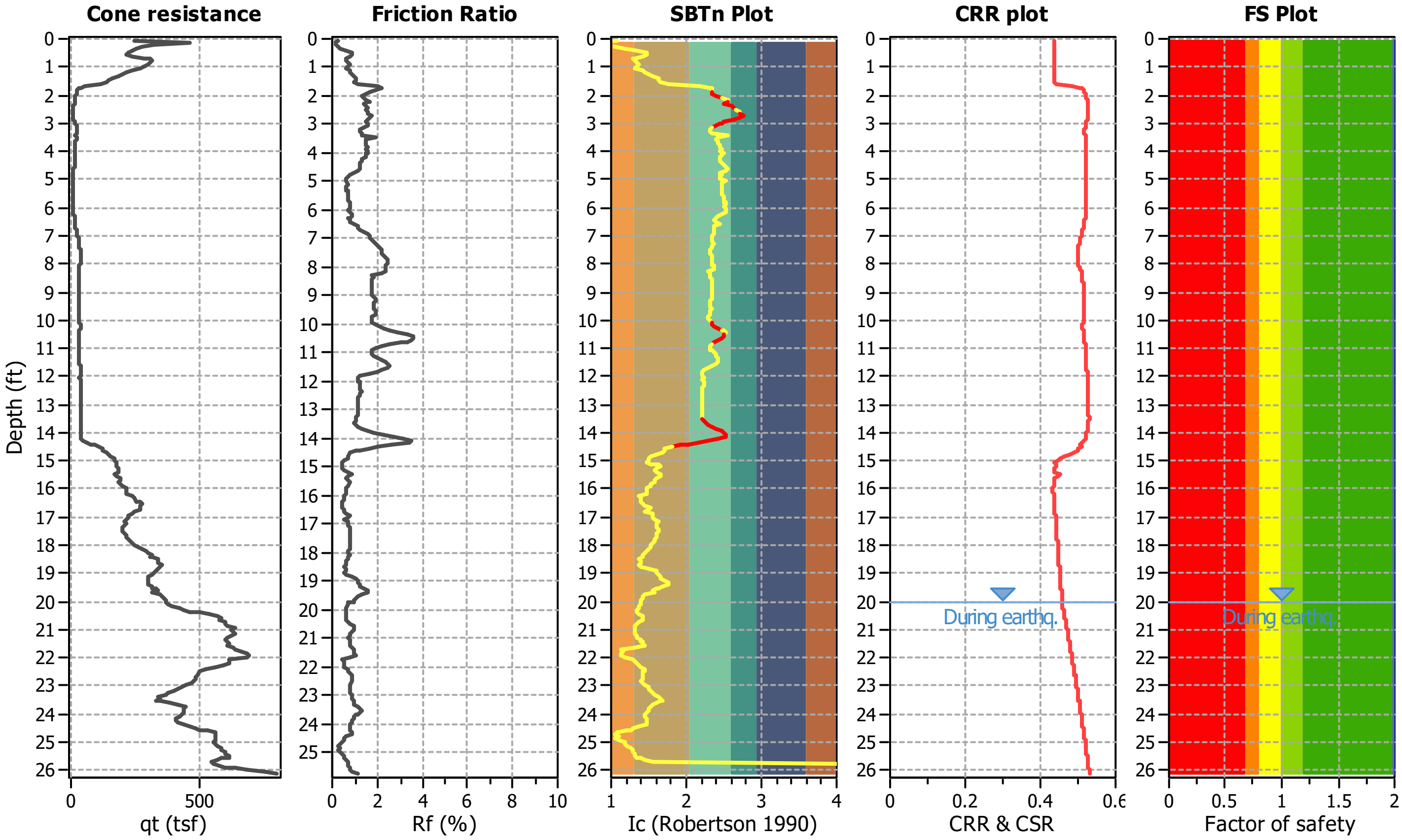
LIQUEFACTION ANALYSIS REPORT

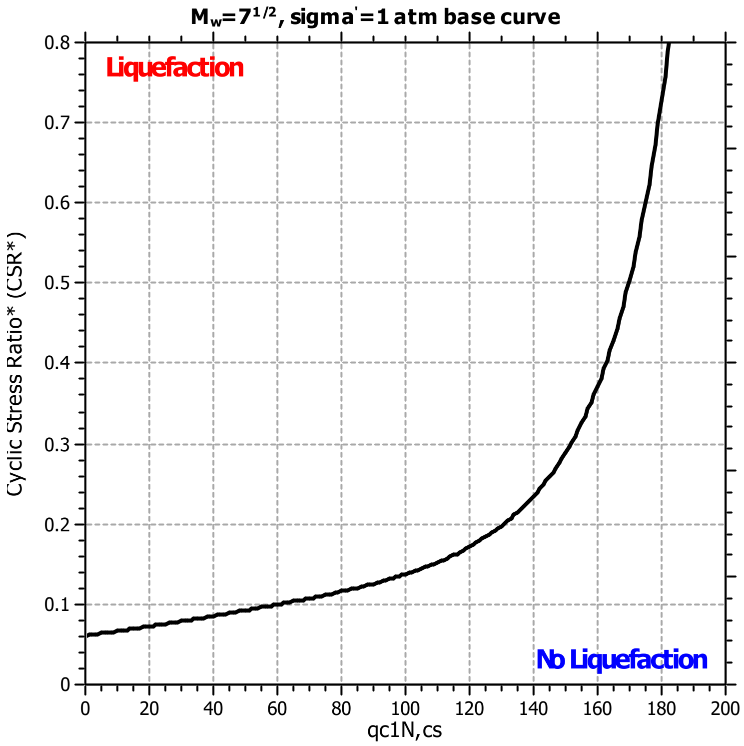
Project title : Proposed Self Storage Facility **Location : El Monte, CA**

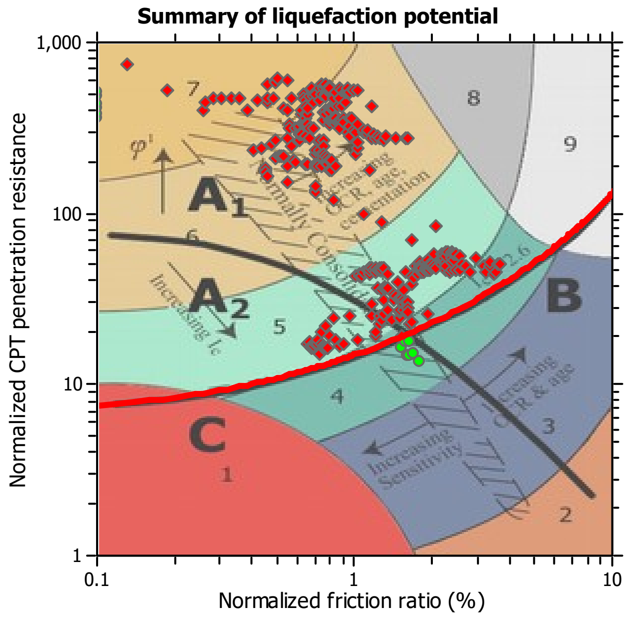
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	60.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.92	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.92	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

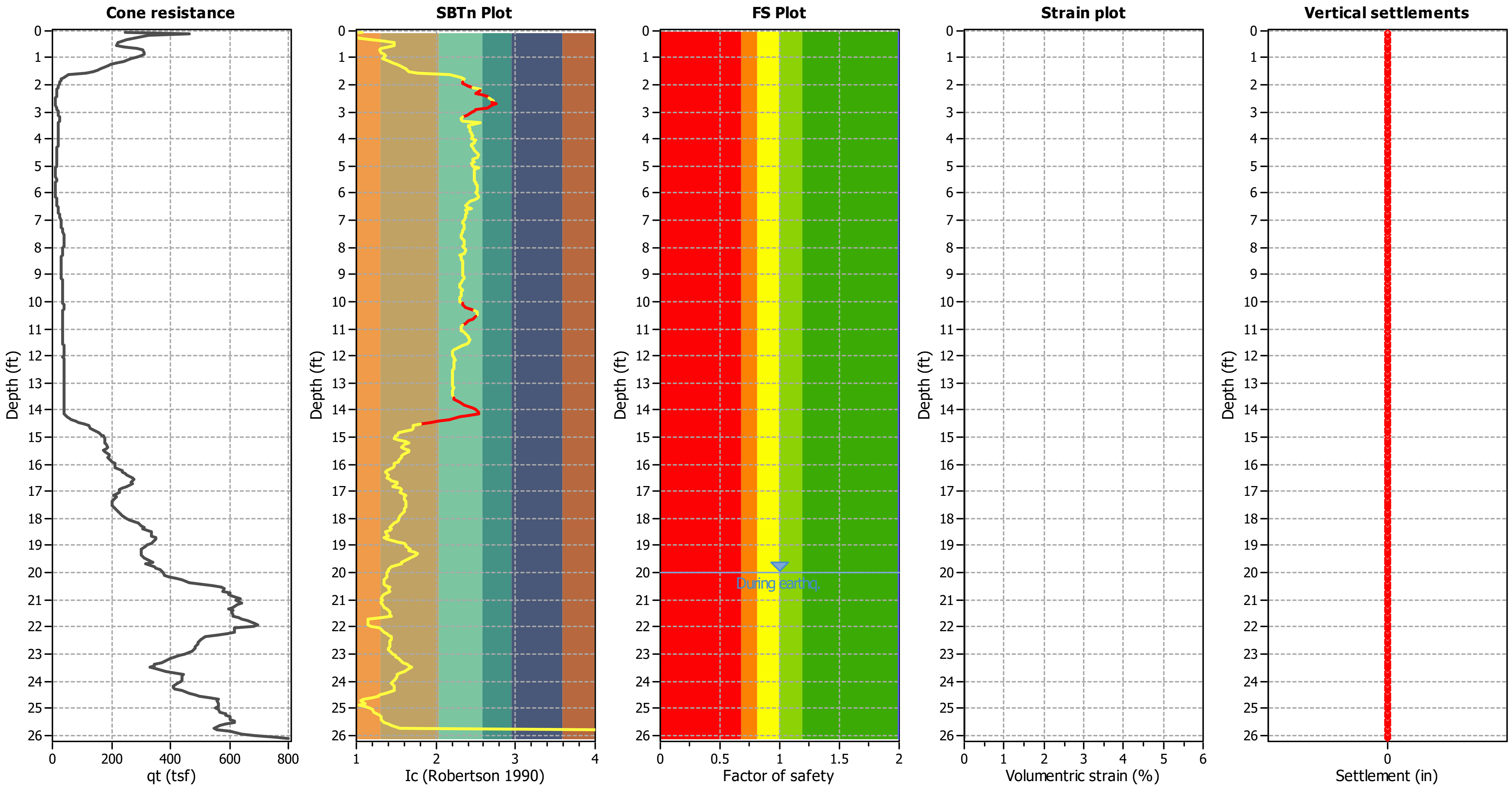






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

Estimation of post-earthquake settlements



Abbreviations

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

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

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
20.02	342.86	2.00	0.00	1.00	0.00	20.11	349.61	2.00	0.00	1.00	0.00
20.15	364.07	2.00	0.00	1.00	0.00	20.22	385.88	2.00	0.00	1.00	0.00
20.29	403.87	2.00	0.00	1.00	0.00	20.35	425.44	2.00	0.00	1.00	0.00
20.41	458.72	2.00	0.00	1.00	0.00	20.47	494.08	2.00	0.00	1.00	0.00
20.54	522.53	2.00	0.00	1.00	0.00	20.61	530.93	2.00	0.00	1.00	0.00
20.67	527.61	2.00	0.00	1.00	0.00	20.74	546.08	2.00	0.00	1.00	0.00
20.81	544.08	2.00	0.00	1.00	0.00	20.88	550.49	2.00	0.00	1.00	0.00
20.94	576.23	2.00	0.00	1.00	0.00	21.01	561.86	2.00	0.00	1.00	0.00
21.08	572.76	2.00	0.00	1.00	0.00	21.14	579.55	2.00	0.00	1.00	0.00
21.20	564.75	2.00	0.00	1.00	0.00	21.26	552.03	2.00	0.00	1.00	0.00
21.33	538.15	2.00	0.00	1.00	0.00	21.40	551.35	2.00	0.00	1.00	0.00
21.46	546.70	2.00	0.00	1.00	0.00	21.53	550.69	2.00	0.00	1.00	0.00
21.59	549.42	2.00	0.00	1.00	0.00	21.66	569.17	2.00	0.00	1.00	0.00
21.72	574.14	2.00	0.00	1.00	0.00	21.79	594.98	2.00	0.00	1.00	0.00
21.85	616.46	2.00	0.00	1.00	0.00	21.92	622.79	2.00	0.00	1.00	0.00
21.98	609.27	2.00	0.00	1.00	0.00	22.05	551.38	2.00	0.00	1.00	0.00
22.12	550.95	2.00	0.00	1.00	0.00	22.19	550.72	2.00	0.00	1.00	0.00
22.25	528.90	2.00	0.00	1.00	0.00	22.31	498.12	2.00	0.00	1.00	0.00
22.38	462.68	2.00	0.00	1.00	0.00	22.45	447.19	2.00	0.00	1.00	0.00
22.51	443.39	2.00	0.00	1.00	0.00	22.60	437.00	2.00	0.00	1.00	0.00
22.65	435.69	2.00	0.00	1.00	0.00	22.72	429.03	2.00	0.00	1.00	0.00
22.77	428.34	2.00	0.00	1.00	0.00	22.85	423.85	2.00	0.00	1.00	0.00
22.90	415.93	2.00	0.00	1.00	0.00	22.99	395.84	2.00	0.00	1.00	0.00
23.06	376.89	2.00	0.00	1.00	0.00	23.12	365.02	2.00	0.00	1.00	0.00
23.19	349.98	2.00	0.00	1.00	0.00	23.26	331.52	2.00	0.00	1.00	0.00
23.32	322.91	2.00	0.00	1.00	0.00	23.37	300.57	2.00	0.00	1.00	0.00
23.43	302.78	2.00	0.00	1.00	0.00	23.50	291.13	2.00	0.00	1.00	0.00
23.56	304.65	2.00	0.00	1.00	0.00	23.63	334.33	2.00	0.00	1.00	0.00
23.69	361.34	2.00	0.00	1.00	0.00	23.76	386.80	2.00	0.00	1.00	0.00
23.83	384.99	2.00	0.00	1.00	0.00	23.89	383.63	2.00	0.00	1.00	0.00
23.96	380.94	2.00	0.00	1.00	0.00	24.02	378.16	2.00	0.00	1.00	0.00
24.10	370.12	2.00	0.00	1.00	0.00	24.16	353.97	2.00	0.00	1.00	0.00
24.24	354.86	2.00	0.00	1.00	0.00	24.30	360.32	2.00	0.00	1.00	0.00
24.37	378.66	2.00	0.00	1.00	0.00	24.43	399.96	2.00	0.00	1.00	0.00
24.50	420.44	2.00	0.00	1.00	0.00	24.55	432.56	2.00	0.00	1.00	0.00
24.61	460.62	2.00	0.00	1.00	0.00	24.68	485.91	2.00	0.00	1.00	0.00
24.75	483.37	2.00	0.00	1.00	0.00	24.81	483.97	2.00	0.00	1.00	0.00
24.87	483.62	2.00	0.00	1.00	0.00	24.94	484.20	2.00	0.00	1.00	0.00
25.01	475.01	2.00	0.00	1.00	0.00	25.07	485.36	2.00	0.00	1.00	0.00
25.13	486.87	2.00	0.00	1.00	0.00	25.20	502.12	2.00	0.00	1.00	0.00
25.27	502.33	2.00	0.00	1.00	0.00	25.33	516.92	2.00	0.00	1.00	0.00
25.40	514.10	2.00	0.00	1.00	0.00	25.46	527.36	2.00	0.00	1.00	0.00
25.53	527.16	2.00	0.00	1.00	0.00	25.59	500.95	2.00	0.00	1.00	0.00
25.66	486.57	2.00	0.00	1.00	0.00	25.73	469.12	2.00	0.00	1.00	0.00
25.79	475.33	2.00	0.00	1.00	0.00	25.88	513.09	2.00	0.00	1.00	0.00
25.94	545.75	2.00	0.00	1.00	0.00	25.99	582.45	2.00	0.00	1.00	0.00
26.06	639.49	2.00	0.00	1.00	0.00	26.12	680.59	2.00	0.00	1.00	0.00

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

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

Total estimated settlement: 0.00

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

Southern California Geotechnical, Inc.
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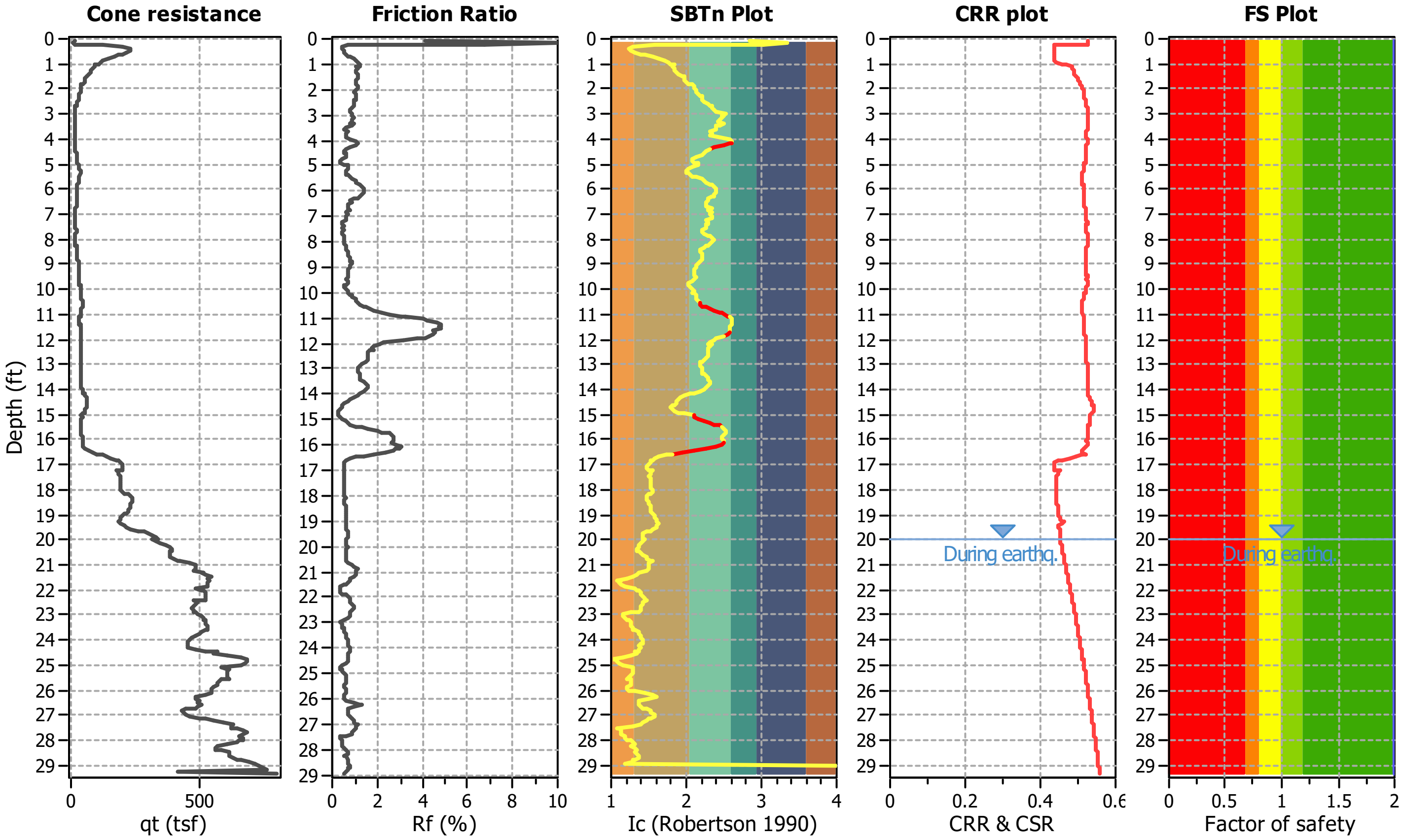
LIQUEFACTION ANALYSIS REPORT

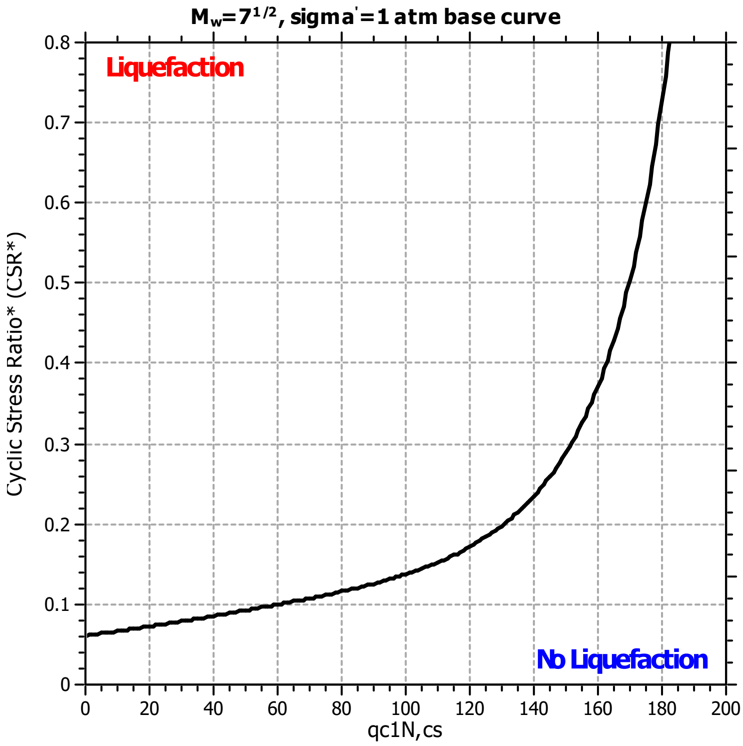
Project title : Proposed Self Storage Facility **Location : El Monte, CA**

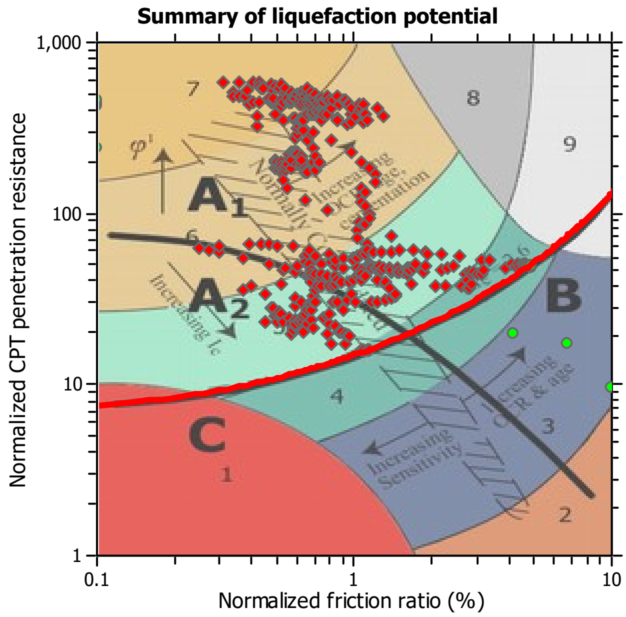
CPT file : CPT-3

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	60.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.92	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.92	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

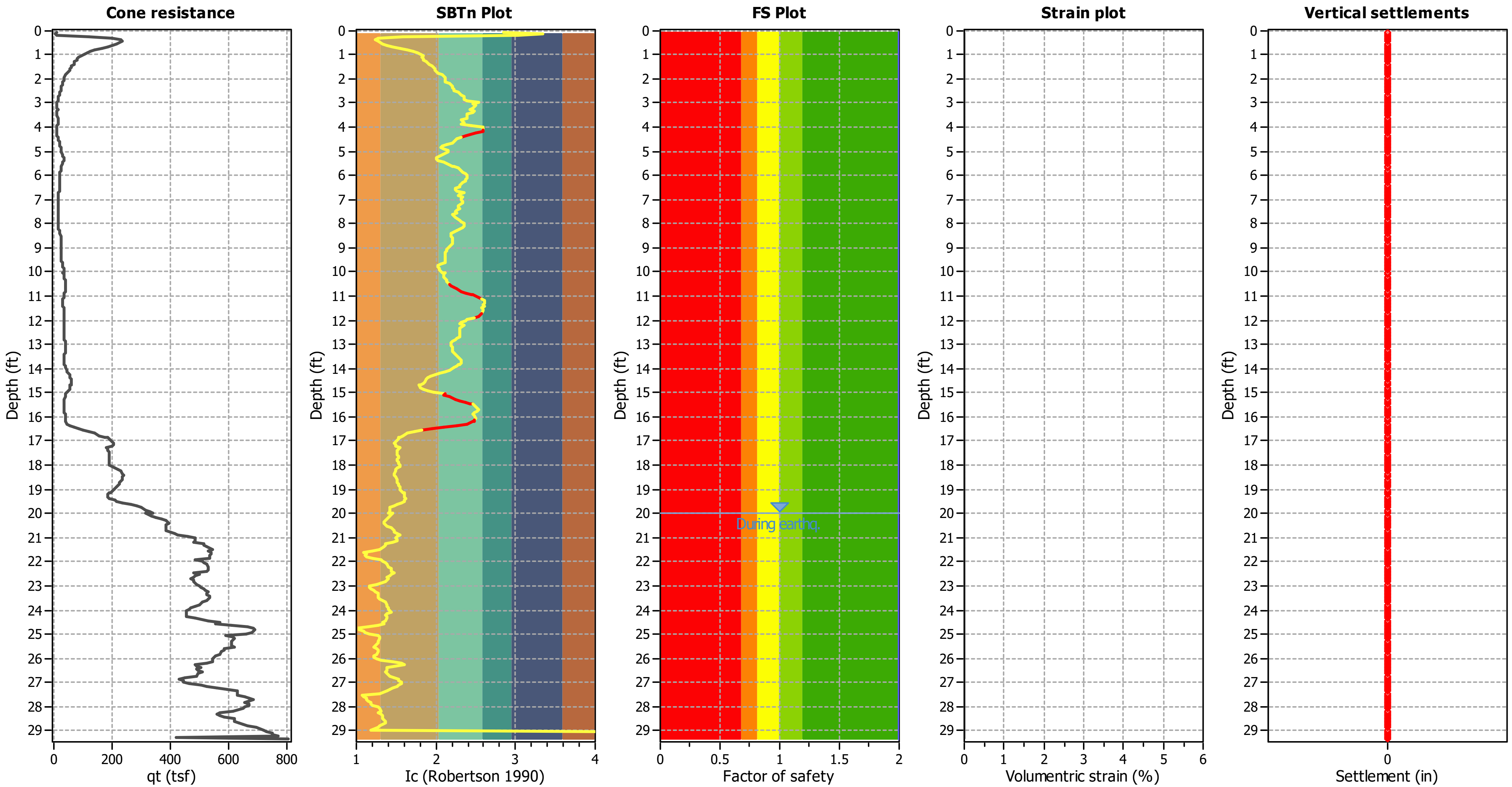






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

Estimation of post-earthquake settlements



Abbreviations

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

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

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
20.02	292.04	2.00	0.00	1.00	0.00	20.09	311.52	2.00	0.00	1.00	0.00
20.16	326.18	2.00	0.00	1.00	0.00	20.22	345.08	2.00	0.00	1.00	0.00
20.29	355.32	2.00	0.00	1.00	0.00	20.35	363.74	2.00	0.00	1.00	0.00
20.42	365.85	2.00	0.00	1.00	0.00	20.49	354.42	2.00	0.00	1.00	0.00
20.54	354.62	2.00	0.00	1.00	0.00	20.61	354.25	2.00	0.00	1.00	0.00
20.69	354.30	2.00	0.00	1.00	0.00	20.76	367.91	2.00	0.00	1.00	0.00
20.83	380.13	2.00	0.00	1.00	0.00	20.87	390.77	2.00	0.00	1.00	0.00
20.95	424.65	2.00	0.00	1.00	0.00	21.01	447.20	2.00	0.00	1.00	0.00
21.07	446.04	2.00	0.00	1.00	0.00	21.13	444.82	2.00	0.00	1.00	0.00
21.21	440.74	2.00	0.00	1.00	0.00	21.28	471.72	2.00	0.00	1.00	0.00
21.33	468.86	2.00	0.00	1.00	0.00	21.40	477.95	2.00	0.00	1.00	0.00
21.47	498.32	2.00	0.00	1.00	0.00	21.53	484.29	2.00	0.00	1.00	0.00
21.59	490.10	2.00	0.00	1.00	0.00	21.66	492.26	2.00	0.00	1.00	0.00
21.73	485.39	2.00	0.00	1.00	0.00	21.79	483.91	2.00	0.00	1.00	0.00
21.86	484.97	2.00	0.00	1.00	0.00	21.92	441.45	2.00	0.00	1.00	0.00
21.99	462.41	2.00	0.00	1.00	0.00	22.05	465.95	2.00	0.00	1.00	0.00
22.12	474.43	2.00	0.00	1.00	0.00	22.19	477.07	2.00	0.00	1.00	0.00
22.26	476.28	2.00	0.00	1.00	0.00	22.33	475.47	2.00	0.00	1.00	0.00
22.38	473.44	2.00	0.00	1.00	0.00	22.44	433.35	2.00	0.00	1.00	0.00
22.53	447.16	2.00	0.00	1.00	0.00	22.59	434.31	2.00	0.00	1.00	0.00
22.65	431.55	2.00	0.00	1.00	0.00	22.72	422.84	2.00	0.00	1.00	0.00
22.78	427.40	2.00	0.00	1.00	0.00	22.85	428.64	2.00	0.00	1.00	0.00
22.91	435.63	2.00	0.00	1.00	0.00	22.98	444.52	2.00	0.00	1.00	0.00
23.04	451.57	2.00	0.00	1.00	0.00	23.12	458.90	2.00	0.00	1.00	0.00
23.17	461.19	2.00	0.00	1.00	0.00	23.23	469.72	2.00	0.00	1.00	0.00
23.30	467.35	2.00	0.00	1.00	0.00	23.37	464.96	2.00	0.00	1.00	0.00
23.44	474.64	2.00	0.00	1.00	0.00	23.50	475.48	2.00	0.00	1.00	0.00
23.57	469.08	2.00	0.00	1.00	0.00	23.63	463.77	2.00	0.00	1.00	0.00
23.69	452.02	2.00	0.00	1.00	0.00	23.77	441.36	2.00	0.00	1.00	0.00
23.84	429.47	2.00	0.00	1.00	0.00	23.91	413.22	2.00	0.00	1.00	0.00
23.97	407.68	2.00	0.00	1.00	0.00	24.04	399.17	2.00	0.00	1.00	0.00
24.11	398.34	2.00	0.00	1.00	0.00	24.18	398.64	2.00	0.00	1.00	0.00
24.22	398.21	2.00	0.00	1.00	0.00	24.29	398.27	2.00	0.00	1.00	0.00
24.36	424.70	2.00	0.00	1.00	0.00	24.43	462.34	2.00	0.00	1.00	0.00
24.49	497.44	2.00	0.00	1.00	0.00	24.54	483.36	2.00	0.00	1.00	0.00
24.62	534.71	2.00	0.00	1.00	0.00	24.68	578.82	2.00	0.00	1.00	0.00
24.74	598.51	2.00	0.00	1.00	0.00	24.81	600.62	2.00	0.00	1.00	0.00
24.87	596.49	2.00	0.00	1.00	0.00	24.94	592.15	2.00	0.00	1.00	0.00
25.00	575.10	2.00	0.00	1.00	0.00	25.07	512.91	2.00	0.00	1.00	0.00
25.14	532.12	2.00	0.00	1.00	0.00	25.20	539.17	2.00	0.00	1.00	0.00
25.27	529.59	2.00	0.00	1.00	0.00	25.33	529.13	2.00	0.00	1.00	0.00
25.41	528.60	2.00	0.00	1.00	0.00	25.46	528.59	2.00	0.00	1.00	0.00
25.53	537.69	2.00	0.00	1.00	0.00	25.60	507.21	2.00	0.00	1.00	0.00
25.67	502.84	2.00	0.00	1.00	0.00	25.74	496.16	2.00	0.00	1.00	0.00
25.82	489.46	2.00	0.00	1.00	0.00	25.88	477.40	2.00	0.00	1.00	0.00
25.93	472.51	2.00	0.00	1.00	0.00	26.00	470.08	2.00	0.00	1.00	0.00
26.07	469.69	2.00	0.00	1.00	0.00	26.14	469.30	2.00	0.00	1.00	0.00
26.20	450.63	2.00	0.00	1.00	0.00	26.25	416.95	2.00	0.00	1.00	0.00

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

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
26.33	419.67	2.00	0.00	1.00	0.00	26.39	431.40	2.00	0.00	1.00	0.00
26.46	420.45	2.00	0.00	1.00	0.00	26.52	422.24	2.00	0.00	1.00	0.00
26.58	435.93	2.00	0.00	1.00	0.00	26.65	421.38	2.00	0.00	1.00	0.00
26.72	415.83	2.00	0.00	1.00	0.00	26.78	381.57	2.00	0.00	1.00	0.00
26.85	366.71	2.00	0.00	1.00	0.00	26.93	377.37	2.00	0.00	1.00	0.00
26.99	377.59	2.00	0.00	1.00	0.00	27.04	394.37	2.00	0.00	1.00	0.00
27.11	426.58	2.00	0.00	1.00	0.00	27.17	444.49	2.00	0.00	1.00	0.00
27.24	474.36	2.00	0.00	1.00	0.00	27.30	503.64	2.00	0.00	1.00	0.00
27.37	533.18	2.00	0.00	1.00	0.00	27.44	532.01	2.00	0.00	1.00	0.00
27.51	530.82	2.00	0.00	1.00	0.00	27.57	550.89	2.00	0.00	1.00	0.00
27.63	565.72	2.00	0.00	1.00	0.00	27.70	577.82	2.00	0.00	1.00	0.00
27.77	564.77	2.00	0.00	1.00	0.00	27.83	550.66	2.00	0.00	1.00	0.00
27.90	563.78	2.00	0.00	1.00	0.00	27.96	564.28	2.00	0.00	1.00	0.00
28.02	549.90	2.00	0.00	1.00	0.00	28.09	547.68	2.00	0.00	1.00	0.00
28.16	513.88	2.00	0.00	1.00	0.00	28.24	477.55	2.00	0.00	1.00	0.00
28.29	469.94	2.00	0.00	1.00	0.00	28.37	475.17	2.00	0.00	1.00	0.00
28.43	491.15	2.00	0.00	1.00	0.00	28.50	518.49	2.00	0.00	1.00	0.00
28.55	517.57	2.00	0.00	1.00	0.00	28.61	516.62	2.00	0.00	1.00	0.00
28.68	528.69	2.00	0.00	1.00	0.00	28.75	542.82	2.00	0.00	1.00	0.00
28.82	554.07	2.00	0.00	1.00	0.00	28.88	577.46	2.00	0.00	1.00	0.00
28.95	598.76	2.00	0.00	1.00	0.00	29.01	608.20	2.00	0.00	1.00	0.00
29.07	622.88	2.00	0.00	1.00	0.00	29.14	622.84	2.00	0.00	1.00	0.00
29.21	638.99	2.00	0.00	1.00	0.00	29.27	347.85	2.00	0.00	1.00	0.00
29.34	667.73	2.00	0.00	1.00	0.00						

Total estimated settlement: 0.00

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

Southern California Geotechnical, Inc.
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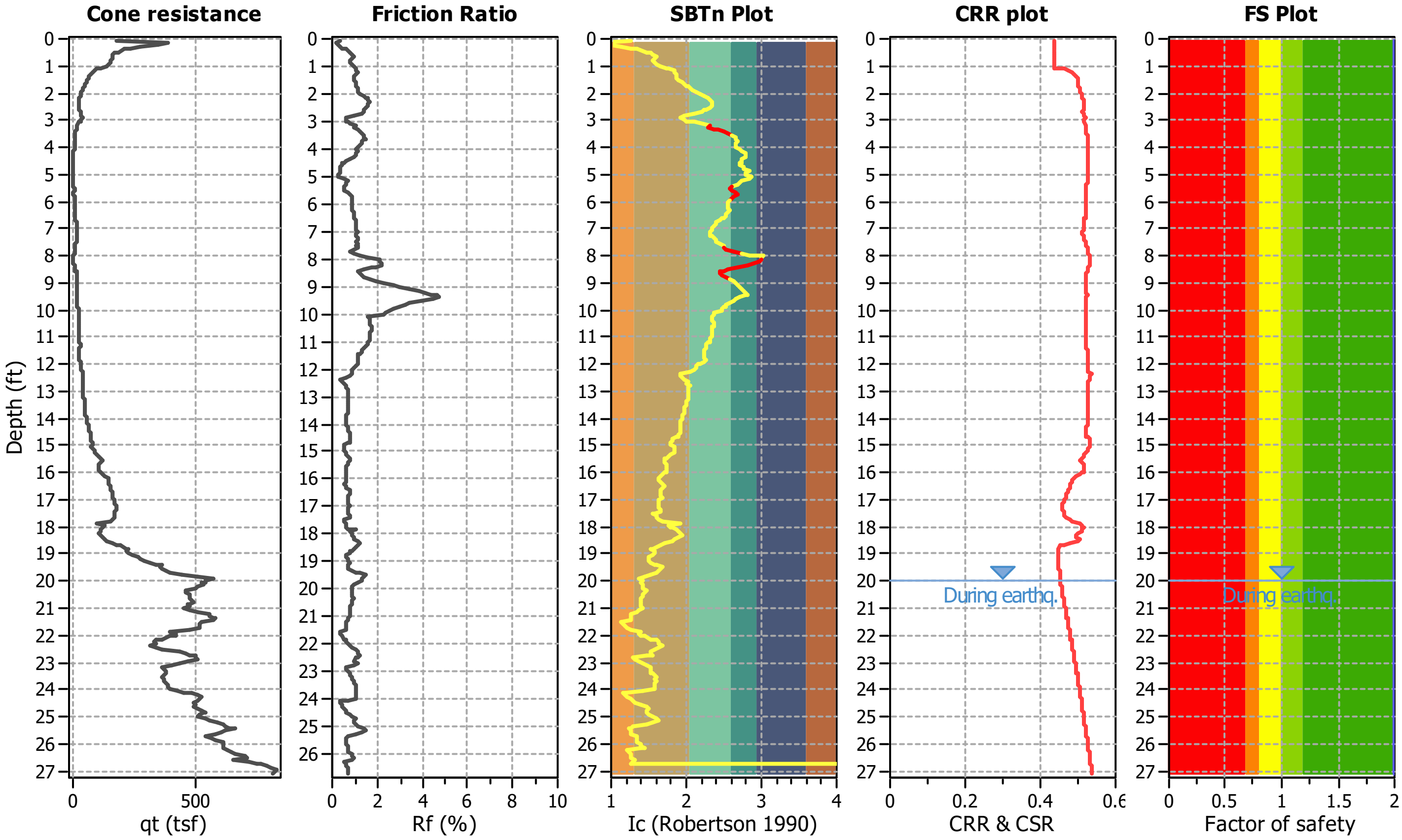
LIQUEFACTION ANALYSIS REPORT

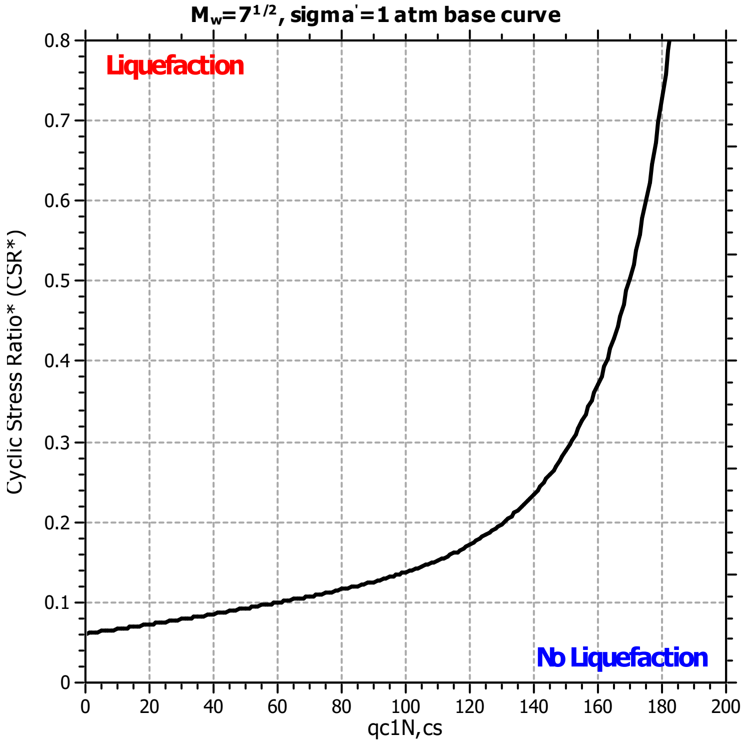
Project title : Proposed Self Storage Facility **Location : El Monte, CA**

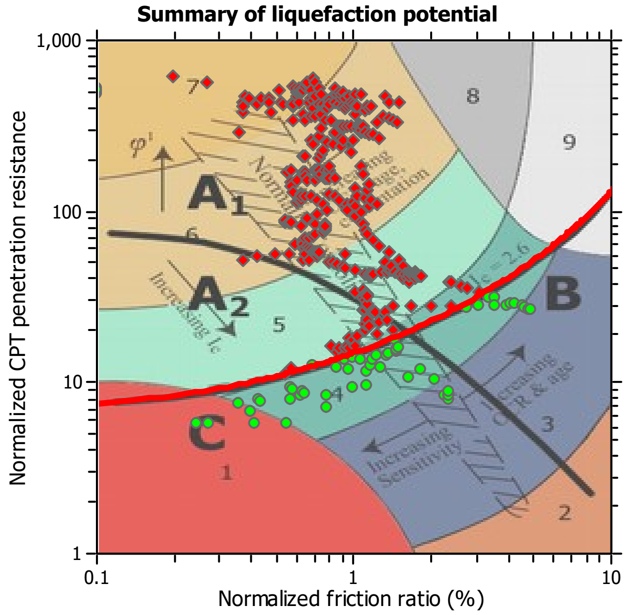
CPT file : CPT-4

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	60.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.92	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.92	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

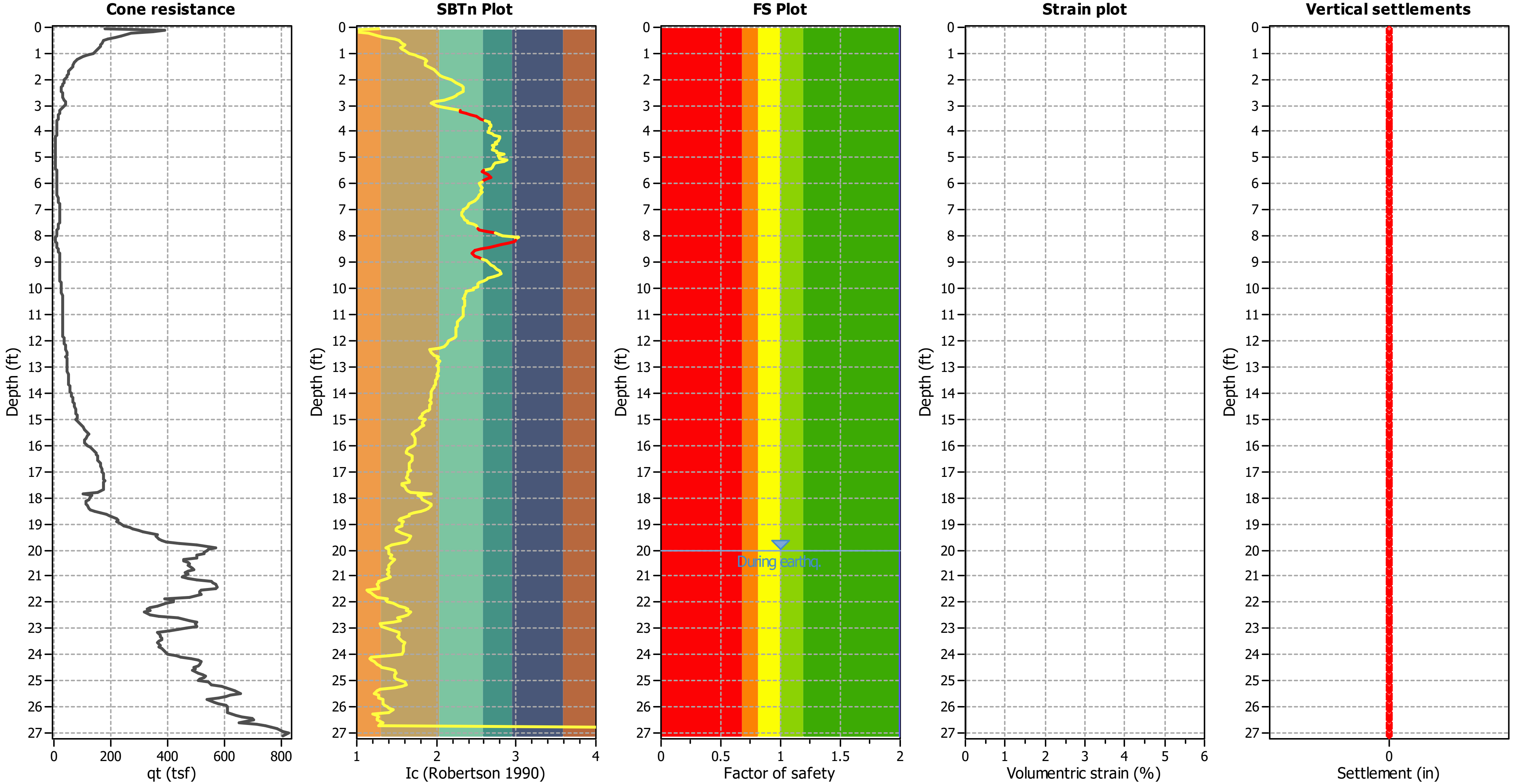






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

Estimation of post-earthquake settlements



Abbreviations

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

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

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
20.02	501.91	2.00	0.00	1.00	0.00	20.09	488.08	2.00	0.00	1.00	0.00
20.15	489.09	2.00	0.00	1.00	0.00	20.22	466.11	2.00	0.00	1.00	0.00
20.29	465.04	2.00	0.00	1.00	0.00	20.35	421.66	2.00	0.00	1.00	0.00
20.41	421.68	2.00	0.00	1.00	0.00	20.48	435.52	2.00	0.00	1.00	0.00
20.55	440.84	2.00	0.00	1.00	0.00	20.61	436.46	2.00	0.00	1.00	0.00
20.69	446.53	2.00	0.00	1.00	0.00	20.75	451.12	2.00	0.00	1.00	0.00
20.82	434.46	2.00	0.00	1.00	0.00	20.87	424.99	2.00	0.00	1.00	0.00
20.94	432.96	2.00	0.00	1.00	0.00	21.00	412.91	2.00	0.00	1.00	0.00
21.08	431.24	2.00	0.00	1.00	0.00	21.14	460.67	2.00	0.00	1.00	0.00
21.20	504.92	2.00	0.00	1.00	0.00	21.27	508.76	2.00	0.00	1.00	0.00
21.33	519.61	2.00	0.00	1.00	0.00	21.39	524.88	2.00	0.00	1.00	0.00
21.47	515.46	2.00	0.00	1.00	0.00	21.54	472.50	2.00	0.00	1.00	0.00
21.59	466.98	2.00	0.00	1.00	0.00	21.67	465.92	2.00	0.00	1.00	0.00
21.72	468.33	2.00	0.00	1.00	0.00	21.81	433.14	2.00	0.00	1.00	0.00
21.86	354.07	2.00	0.00	1.00	0.00	21.93	377.61	2.00	0.00	1.00	0.00
22.00	376.85	2.00	0.00	1.00	0.00	22.05	353.63	2.00	0.00	1.00	0.00
22.13	327.33	2.00	0.00	1.00	0.00	22.20	305.72	2.00	0.00	1.00	0.00
22.25	295.16	2.00	0.00	1.00	0.00	22.31	303.78	2.00	0.00	1.00	0.00
22.38	284.20	2.00	0.00	1.00	0.00	22.46	301.72	2.00	0.00	1.00	0.00
22.51	328.86	2.00	0.00	1.00	0.00	22.61	388.45	2.00	0.00	1.00	0.00
22.67	409.45	2.00	0.00	1.00	0.00	22.71	425.61	2.00	0.00	1.00	0.00
22.77	446.38	2.00	0.00	1.00	0.00	22.84	441.76	2.00	0.00	1.00	0.00
22.91	448.89	2.00	0.00	1.00	0.00	22.98	415.84	2.00	0.00	1.00	0.00
23.04	385.34	2.00	0.00	1.00	0.00	23.11	357.63	2.00	0.00	1.00	0.00
23.17	324.87	2.00	0.00	1.00	0.00	23.24	329.52	2.00	0.00	1.00	0.00
23.30	332.95	2.00	0.00	1.00	0.00	23.37	336.24	2.00	0.00	1.00	0.00
23.43	337.21	2.00	0.00	1.00	0.00	23.49	326.78	2.00	0.00	1.00	0.00
23.56	321.90	2.00	0.00	1.00	0.00	23.64	328.51	2.00	0.00	1.00	0.00
23.69	324.14	2.00	0.00	1.00	0.00	23.77	333.60	2.00	0.00	1.00	0.00
23.84	339.30	2.00	0.00	1.00	0.00	23.89	342.41	2.00	0.00	1.00	0.00
23.96	349.28	2.00	0.00	1.00	0.00	24.03	378.23	2.00	0.00	1.00	0.00
24.09	392.28	2.00	0.00	1.00	0.00	24.15	421.05	2.00	0.00	1.00	0.00
24.22	442.82	2.00	0.00	1.00	0.00	24.28	455.46	2.00	0.00	1.00	0.00
24.35	448.72	2.00	0.00	1.00	0.00	24.42	442.02	2.00	0.00	1.00	0.00
24.48	429.06	2.00	0.00	1.00	0.00	24.55	432.79	2.00	0.00	1.00	0.00
24.61	425.18	2.00	0.00	1.00	0.00	24.68	442.75	2.00	0.00	1.00	0.00
24.76	453.49	2.00	0.00	1.00	0.00	24.82	465.23	2.00	0.00	1.00	0.00
24.88	460.13	2.00	0.00	1.00	0.00	24.94	444.14	2.00	0.00	1.00	0.00
25.00	442.27	2.00	0.00	1.00	0.00	25.07	469.50	2.00	0.00	1.00	0.00
25.14	479.77	2.00	0.00	1.00	0.00	25.21	509.25	2.00	0.00	1.00	0.00
25.26	525.05	2.00	0.00	1.00	0.00	25.34	537.90	2.00	0.00	1.00	0.00
25.40	551.19	2.00	0.00	1.00	0.00	25.47	566.99	2.00	0.00	1.00	0.00
25.53	536.38	2.00	0.00	1.00	0.00	25.60	520.61	2.00	0.00	1.00	0.00
25.66	500.32	2.00	0.00	1.00	0.00	25.72	464.69	2.00	0.00	1.00	0.00
25.80	485.03	2.00	0.00	1.00	0.00	25.85	499.77	2.00	0.00	1.00	0.00
25.92	520.70	2.00	0.00	1.00	0.00	25.98	524.05	2.00	0.00	1.00	0.00
26.05	524.03	2.00	0.00	1.00	0.00	26.13	524.60	2.00	0.00	1.00	0.00
26.19	523.59	2.00	0.00	1.00	0.00	26.26	535.08	2.00	0.00	1.00	0.00

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

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
26.33	548.97	2.00	0.00	1.00	0.00	26.38	564.31	2.00	0.00	1.00	0.00
26.45	593.22	2.00	0.00	1.00	0.00	26.52	599.10	2.00	0.00	1.00	0.00
26.58	555.60	2.00	0.00	1.00	0.00	26.64	608.50	2.00	0.00	1.00	0.00
26.71	633.14	2.00	0.00	1.00	0.00	26.78	649.68	2.00	0.00	1.00	0.00
26.84	665.08	2.00	0.00	1.00	0.00	26.91	682.77	2.00	0.00	1.00	0.00
26.97	701.14	2.00	0.00	1.00	0.00	27.04	692.39	2.00	0.00	1.00	0.00
27.11	684.57	2.00	0.00	1.00	0.00						

Total estimated settlement: 0.00

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

Southern California Geotechnical, Inc.
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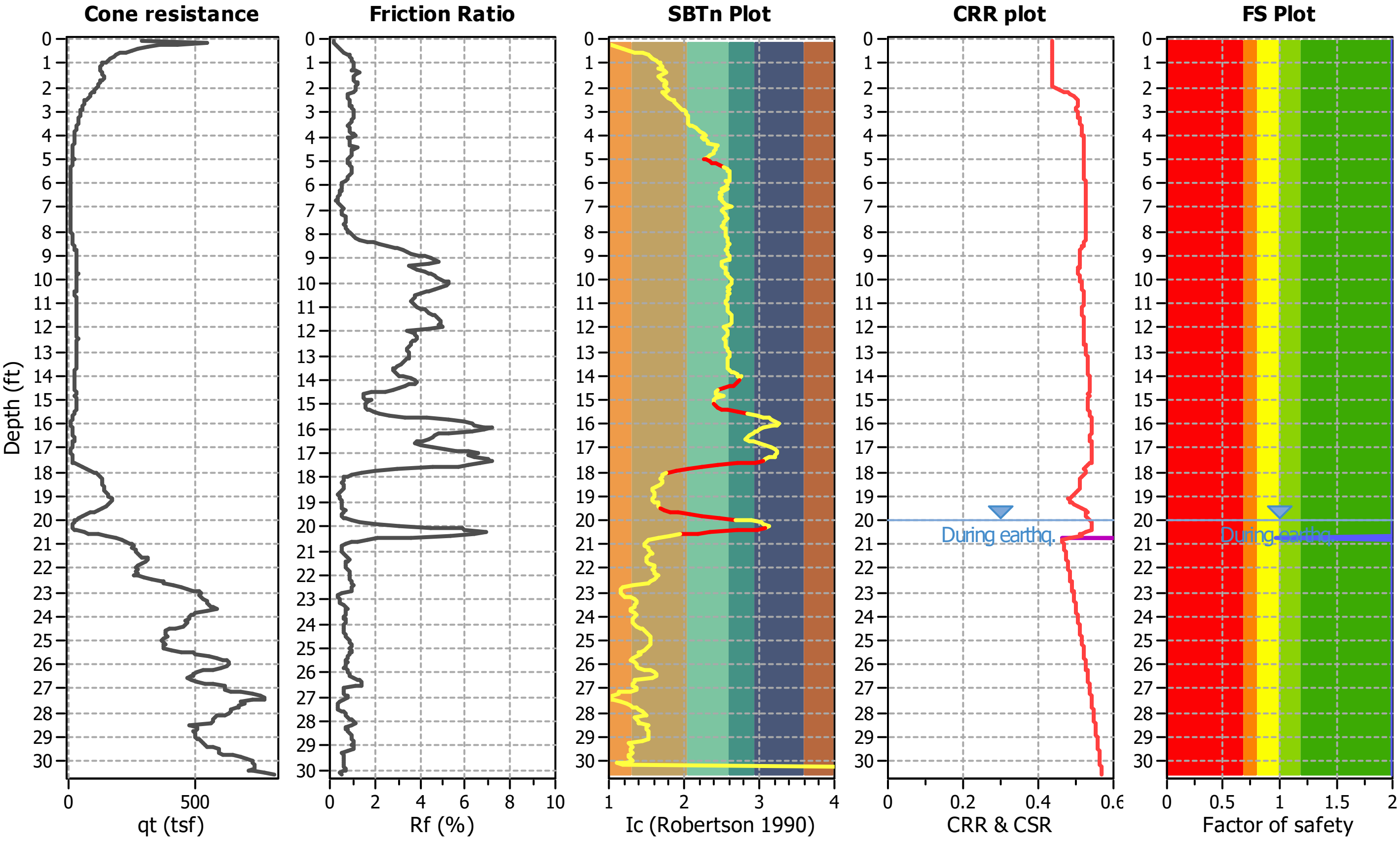
LIQUEFACTION ANALYSIS REPORT

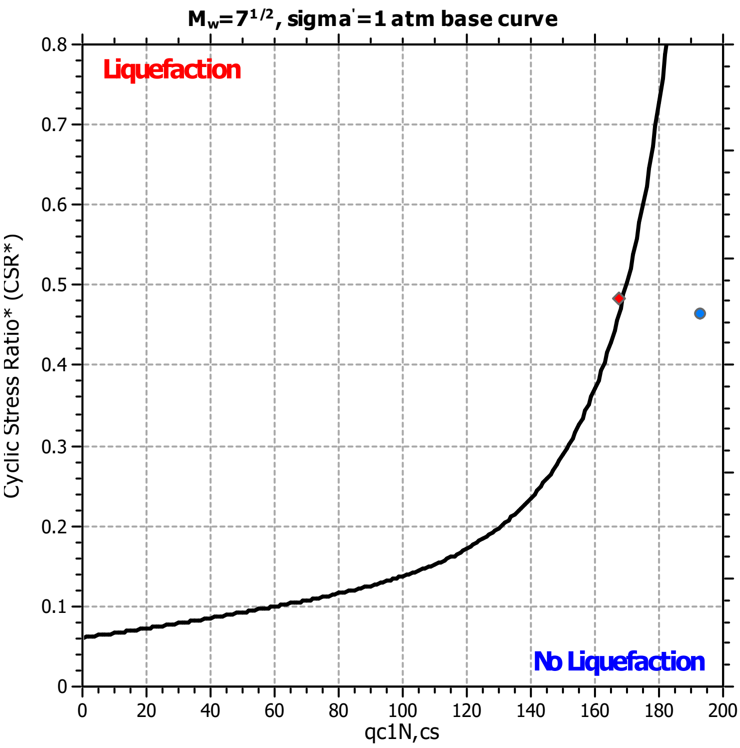
Project title : Proposed Self Storage Facility **Location : El Monte, CA**

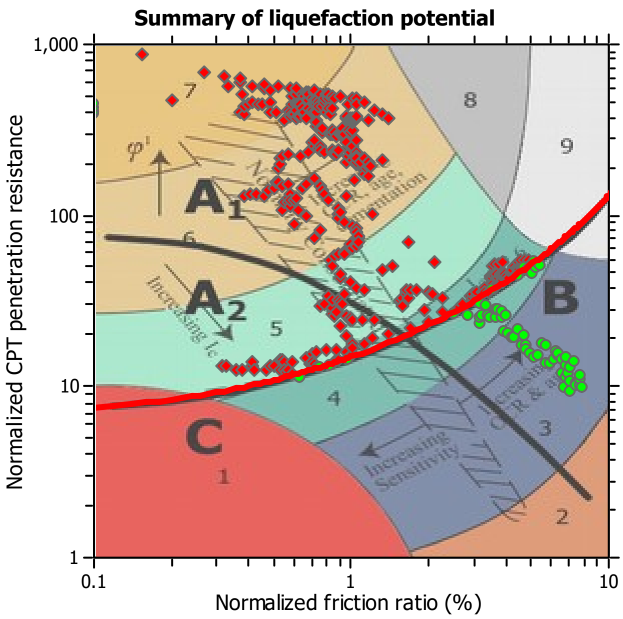
CPT file : CPT-5

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	60.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.92	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.92	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

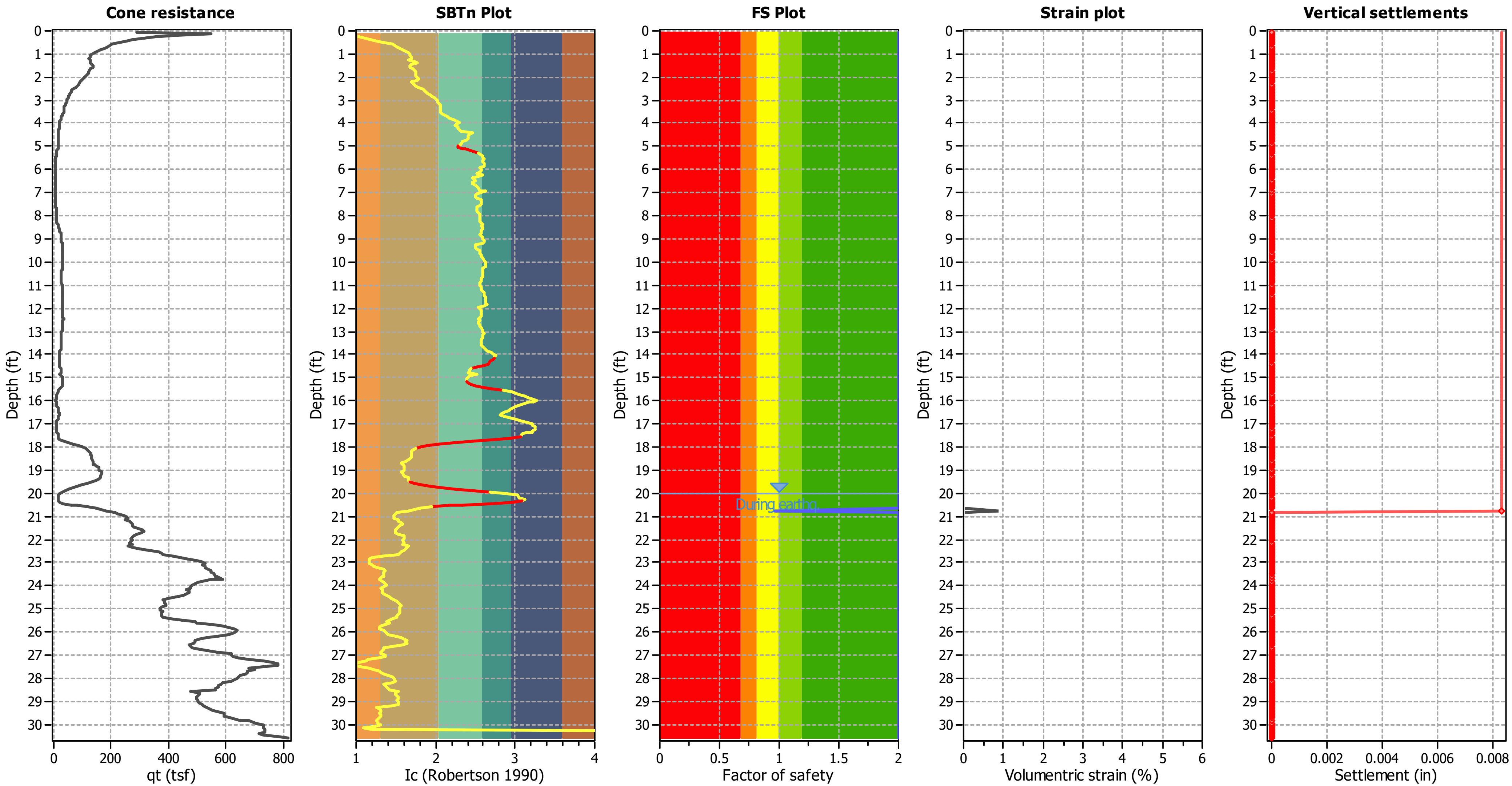






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

Estimation of post-earthquake settlements



Abbreviations

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

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

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
20.03	21.57	2.00	0.00	1.00	0.00	20.10	17.31	2.00	0.00	1.00	0.00
20.17	16.93	2.00	0.00	1.00	0.00	20.24	15.47	2.00	0.00	1.00	0.00
20.30	16.45	2.00	0.00	1.00	0.00	20.37	19.27	2.00	0.00	1.00	0.00
20.44	30.14	2.00	0.00	1.00	0.00	20.50	116.48	2.00	0.00	1.00	0.00
20.54	130.12	2.00	0.00	1.00	0.00	20.61	141.55	2.00	0.00	1.00	0.00
20.67	135.68	2.00	0.00	1.00	0.00	20.75	167.47	0.96	0.87	1.00	0.01
20.83	193.16	2.00	0.00	1.00	0.00	20.87	205.97	2.00	0.00	1.00	0.00
20.96	221.29	2.00	0.00	1.00	0.00	21.02	230.69	2.00	0.00	1.00	0.00
21.09	236.76	2.00	0.00	1.00	0.00	21.16	226.97	2.00	0.00	1.00	0.00
21.22	239.78	2.00	0.00	1.00	0.00	21.29	244.80	2.00	0.00	1.00	0.00
21.36	248.95	2.00	0.00	1.00	0.00	21.43	251.95	2.00	0.00	1.00	0.00
21.46	253.86	2.00	0.00	1.00	0.00	21.53	266.28	2.00	0.00	1.00	0.00
21.59	281.27	2.00	0.00	1.00	0.00	21.66	283.78	2.00	0.00	1.00	0.00
21.73	276.46	2.00	0.00	1.00	0.00	21.79	264.54	2.00	0.00	1.00	0.00
21.86	250.73	2.00	0.00	1.00	0.00	21.92	249.51	2.00	0.00	1.00	0.00
21.99	240.74	2.00	0.00	1.00	0.00	22.06	247.01	2.00	0.00	1.00	0.00
22.13	236.56	2.00	0.00	1.00	0.00	22.19	246.45	2.00	0.00	1.00	0.00
22.26	232.74	2.00	0.00	1.00	0.00	22.32	245.88	2.00	0.00	1.00	0.00
22.38	267.68	2.00	0.00	1.00	0.00	22.45	294.06	2.00	0.00	1.00	0.00
22.52	327.18	2.00	0.00	1.00	0.00	22.58	335.79	2.00	0.00	1.00	0.00
22.65	339.06	2.00	0.00	1.00	0.00	22.73	369.79	2.00	0.00	1.00	0.00
22.78	396.30	2.00	0.00	1.00	0.00	22.84	415.92	2.00	0.00	1.00	0.00
22.92	442.58	2.00	0.00	1.00	0.00	22.97	460.46	2.00	0.00	1.00	0.00
23.04	468.58	2.00	0.00	1.00	0.00	23.10	458.18	2.00	0.00	1.00	0.00
23.17	467.10	2.00	0.00	1.00	0.00	23.24	466.00	2.00	0.00	1.00	0.00
23.30	475.56	2.00	0.00	1.00	0.00	23.36	484.61	2.00	0.00	1.00	0.00
23.43	483.41	2.00	0.00	1.00	0.00	23.50	493.93	2.00	0.00	1.00	0.00
23.57	495.27	2.00	0.00	1.00	0.00	23.62	503.29	2.00	0.00	1.00	0.00
23.70	516.64	2.00	0.00	1.00	0.00	23.75	481.91	2.00	0.00	1.00	0.00
23.82	443.82	2.00	0.00	1.00	0.00	23.91	432.32	2.00	0.00	1.00	0.00
23.95	425.68	2.00	0.00	1.00	0.00	24.03	420.12	2.00	0.00	1.00	0.00
24.09	417.39	2.00	0.00	1.00	0.00	24.15	403.38	2.00	0.00	1.00	0.00
24.22	414.27	2.00	0.00	1.00	0.00	24.29	412.22	2.00	0.00	1.00	0.00
24.36	402.45	2.00	0.00	1.00	0.00	24.42	394.85	2.00	0.00	1.00	0.00
24.49	370.45	2.00	0.00	1.00	0.00	24.55	351.11	2.00	0.00	1.00	0.00
24.63	332.42	2.00	0.00	1.00	0.00	24.69	335.10	2.00	0.00	1.00	0.00
24.75	335.07	2.00	0.00	1.00	0.00	24.81	340.71	2.00	0.00	1.00	0.00
24.88	339.40	2.00	0.00	1.00	0.00	24.94	324.94	2.00	0.00	1.00	0.00
25.00	322.70	2.00	0.00	1.00	0.00	25.08	323.13	2.00	0.00	1.00	0.00
25.14	332.07	2.00	0.00	1.00	0.00	25.20	326.19	2.00	0.00	1.00	0.00
25.27	323.76	2.00	0.00	1.00	0.00	25.33	327.52	2.00	0.00	1.00	0.00
25.40	352.18	2.00	0.00	1.00	0.00	25.47	385.22	2.00	0.00	1.00	0.00
25.54	425.73	2.00	0.00	1.00	0.00	25.60	429.08	2.00	0.00	1.00	0.00
25.66	476.06	2.00	0.00	1.00	0.00	25.73	505.94	2.00	0.00	1.00	0.00
25.79	523.30	2.00	0.00	1.00	0.00	25.86	539.63	2.00	0.00	1.00	0.00
25.92	545.96	2.00	0.00	1.00	0.00	25.99	545.20	2.00	0.00	1.00	0.00
26.05	535.66	2.00	0.00	1.00	0.00	26.12	519.46	2.00	0.00	1.00	0.00
26.20	490.85	2.00	0.00	1.00	0.00	26.27	454.27	2.00	0.00	1.00	0.00

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

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
26.33	433.41	2.00	0.00	1.00	0.00	26.39	418.77	2.00	0.00	1.00	0.00
26.45	418.94	2.00	0.00	1.00	0.00	26.51	416.82	2.00	0.00	1.00	0.00
26.58	399.74	2.00	0.00	1.00	0.00	26.66	412.04	2.00	0.00	1.00	0.00
26.72	431.30	2.00	0.00	1.00	0.00	26.78	456.26	2.00	0.00	1.00	0.00
26.84	483.22	2.00	0.00	1.00	0.00	26.91	524.57	2.00	0.00	1.00	0.00
26.97	525.70	2.00	0.00	1.00	0.00	27.04	526.82	2.00	0.00	1.00	0.00
27.10	546.47	2.00	0.00	1.00	0.00	27.17	574.19	2.00	0.00	1.00	0.00
27.23	614.73	2.00	0.00	1.00	0.00	27.30	642.19	2.00	0.00	1.00	0.00
27.37	656.46	2.00	0.00	1.00	0.00	27.43	657.73	2.00	0.00	1.00	0.00
27.50	614.19	2.00	0.00	1.00	0.00	27.56	572.44	2.00	0.00	1.00	0.00
27.63	587.24	2.00	0.00	1.00	0.00	27.69	566.83	2.00	0.00	1.00	0.00
27.76	564.37	2.00	0.00	1.00	0.00	27.83	561.38	2.00	0.00	1.00	0.00
27.90	542.66	2.00	0.00	1.00	0.00	27.95	538.52	2.00	0.00	1.00	0.00
28.02	535.01	2.00	0.00	1.00	0.00	28.10	518.71	2.00	0.00	1.00	0.00
28.16	491.48	2.00	0.00	1.00	0.00	28.24	486.11	2.00	0.00	1.00	0.00
28.30	476.95	2.00	0.00	1.00	0.00	28.36	477.26	2.00	0.00	1.00	0.00
28.44	471.53	2.00	0.00	1.00	0.00	28.49	467.55	2.00	0.00	1.00	0.00
28.55	399.32	2.00	0.00	1.00	0.00	28.61	422.10	2.00	0.00	1.00	0.00
28.69	421.92	2.00	0.00	1.00	0.00	28.76	415.97	2.00	0.00	1.00	0.00
28.81	412.63	2.00	0.00	1.00	0.00	28.87	413.69	2.00	0.00	1.00	0.00
28.96	417.72	2.00	0.00	1.00	0.00	29.01	416.18	2.00	0.00	1.00	0.00
29.08	419.74	2.00	0.00	1.00	0.00	29.14	429.60	2.00	0.00	1.00	0.00
29.21	433.22	2.00	0.00	1.00	0.00	29.27	439.88	2.00	0.00	1.00	0.00
29.34	447.24	2.00	0.00	1.00	0.00	29.40	456.18	2.00	0.00	1.00	0.00
29.46	473.73	2.00	0.00	1.00	0.00	29.54	489.51	2.00	0.00	1.00	0.00
29.60	489.53	2.00	0.00	1.00	0.00	29.66	489.53	2.00	0.00	1.00	0.00
29.73	505.48	2.00	0.00	1.00	0.00	29.80	533.76	2.00	0.00	1.00	0.00
29.86	557.87	2.00	0.00	1.00	0.00	29.93	579.06	2.00	0.00	1.00	0.00
29.99	597.59	2.00	0.00	1.00	0.00	30.05	599.89	2.00	0.00	1.00	0.00
30.13	596.40	2.00	0.00	1.00	0.00	30.19	603.12	2.00	0.00	1.00	0.00
30.25	602.72	2.00	0.00	1.00	0.00	30.32	601.40	2.00	0.00	1.00	0.00
30.38	582.84	2.00	0.00	1.00	0.00	30.46	595.79	2.00	0.00	1.00	0.00
30.52	637.01	2.00	0.00	1.00	0.00	30.58	666.58	2.00	0.00	1.00	0.00

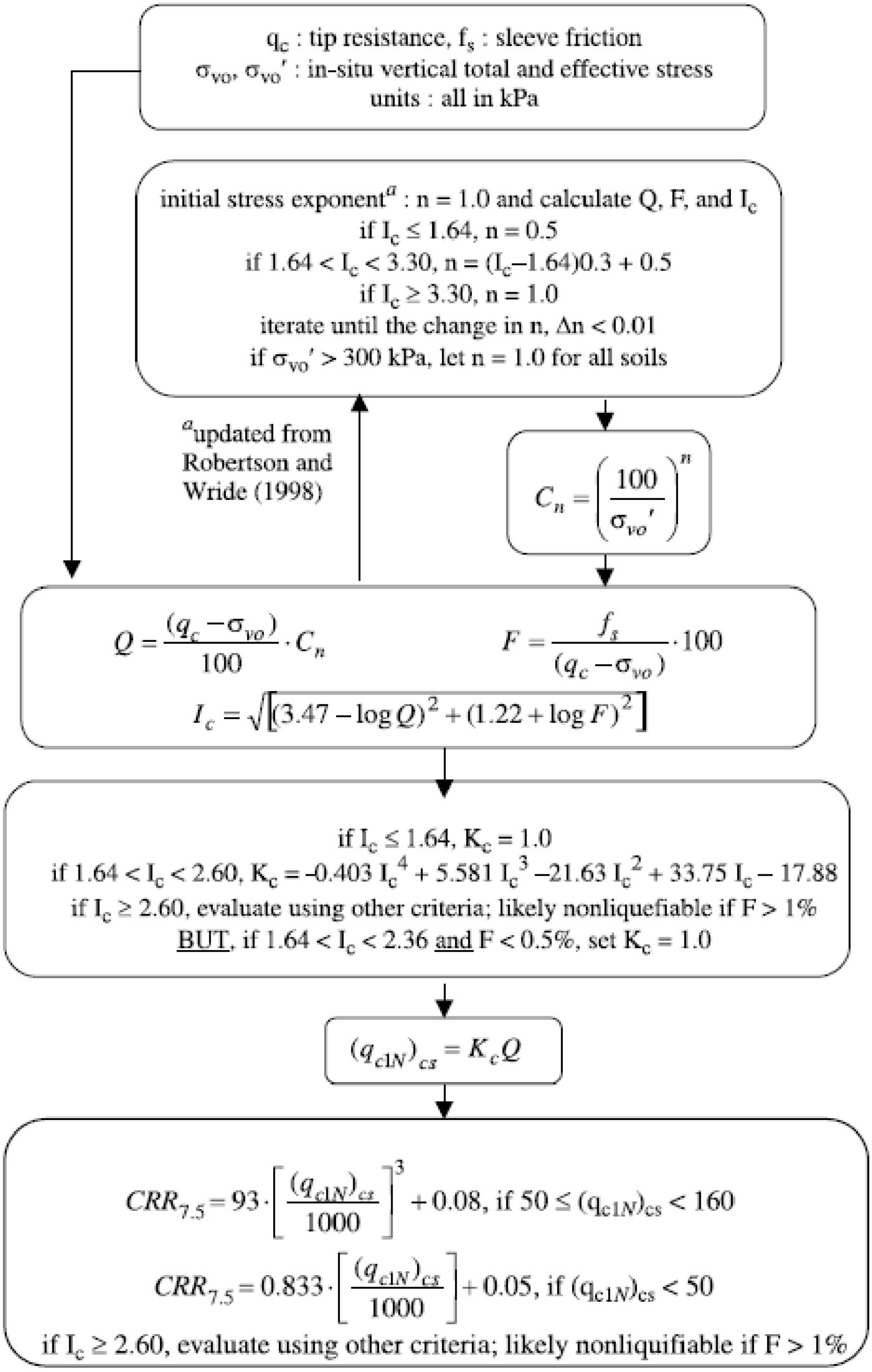
Total estimated settlement: 0.01

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)

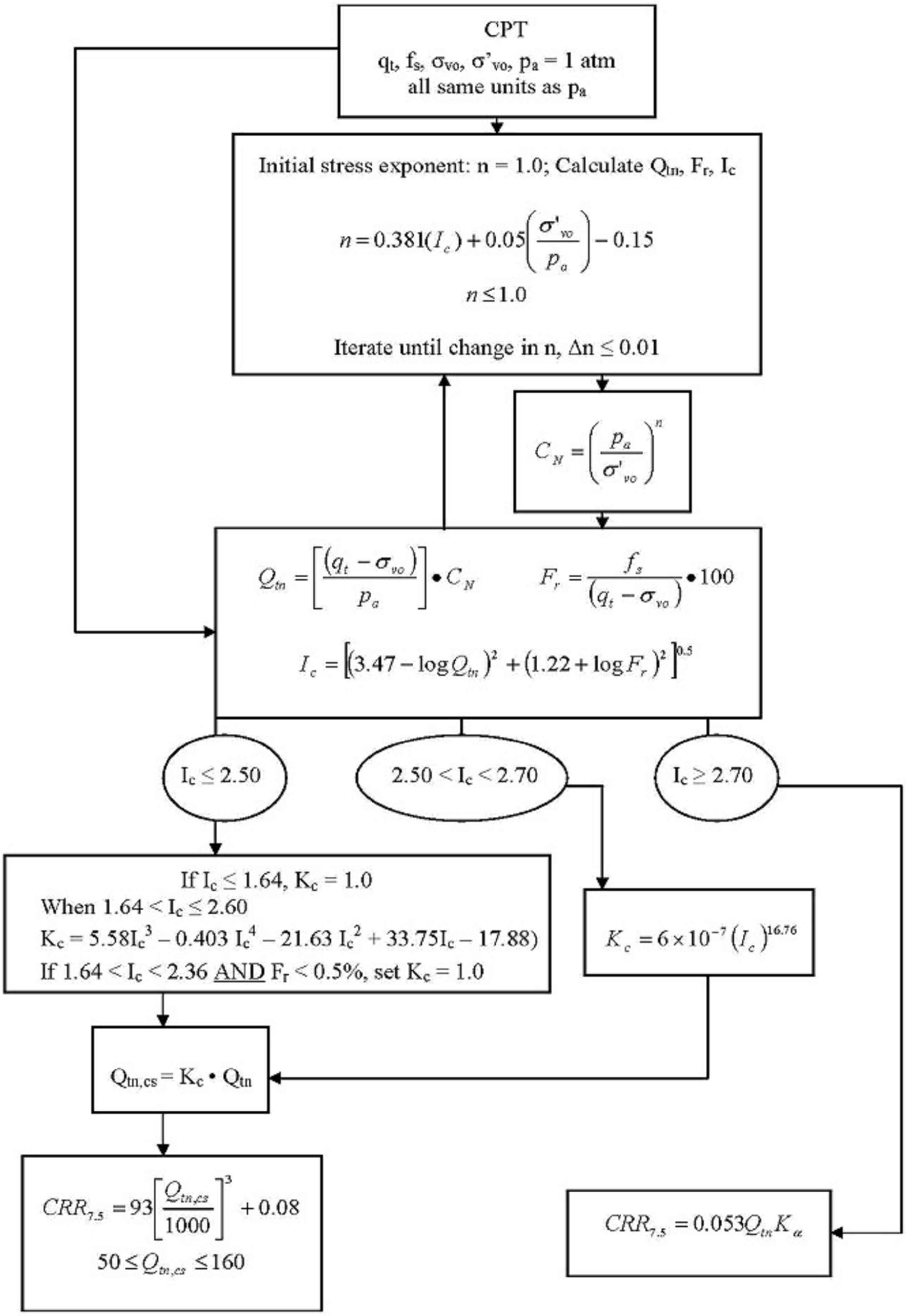
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of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

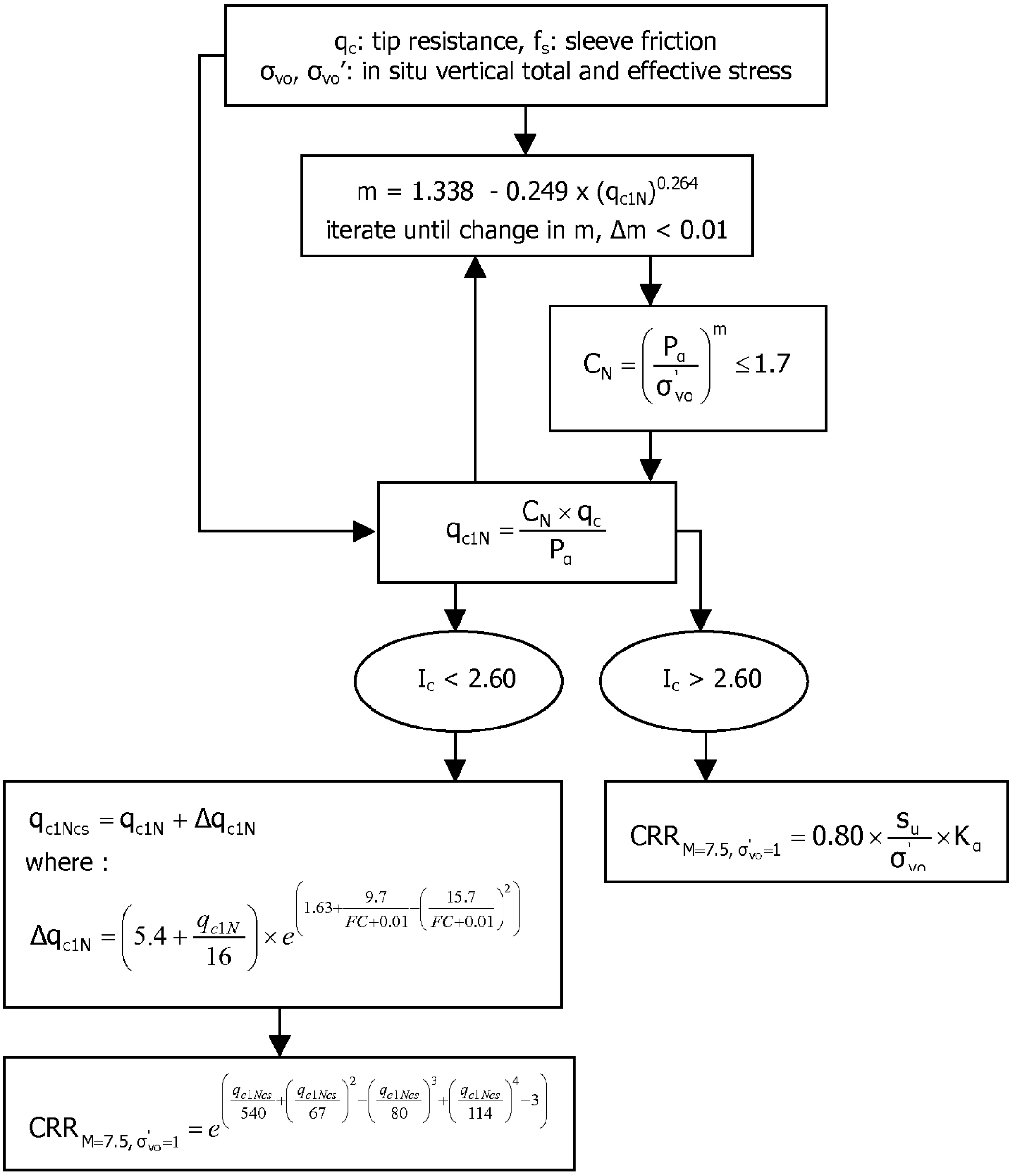
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

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]:

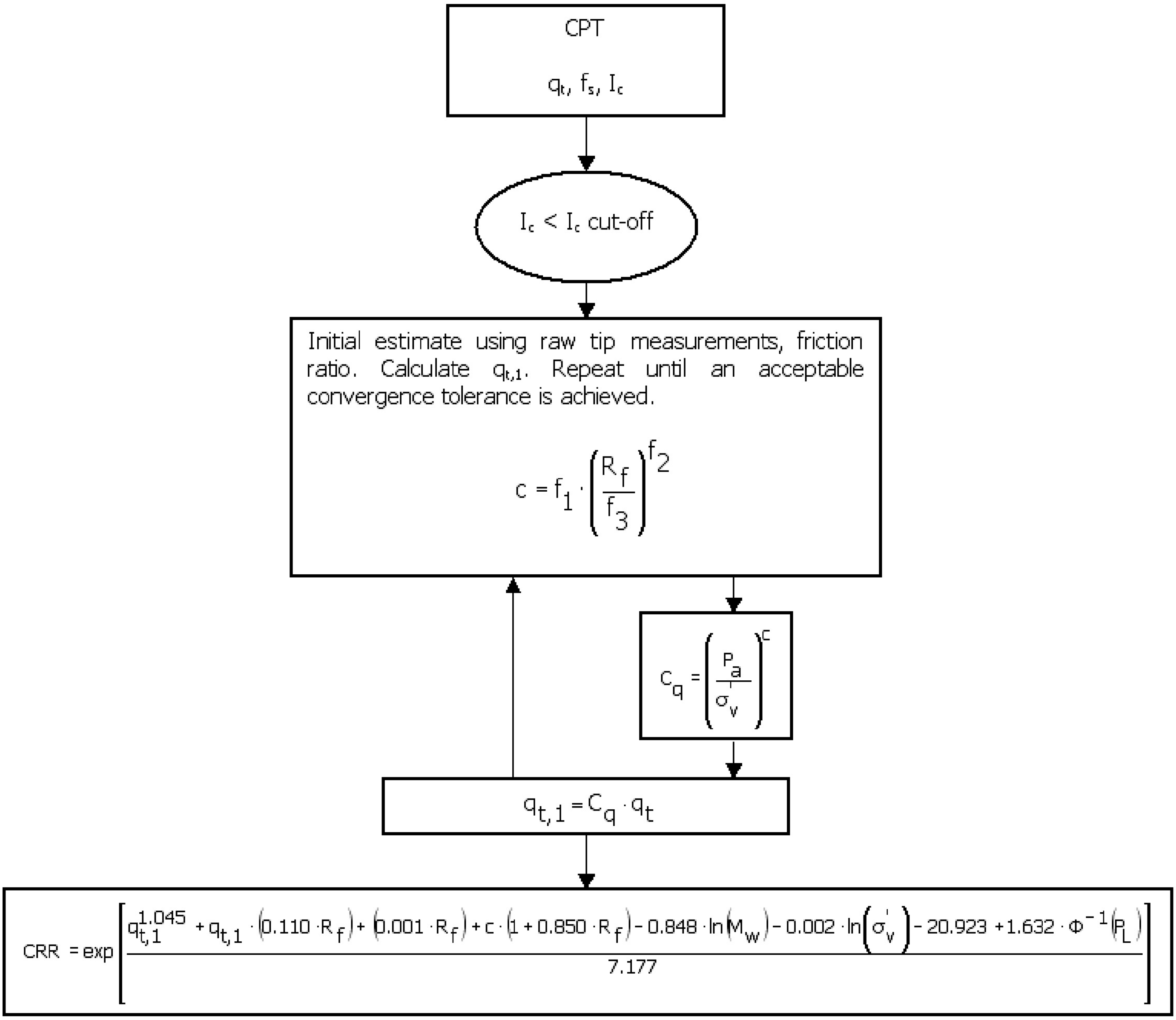


[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

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*

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

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

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

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

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

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

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

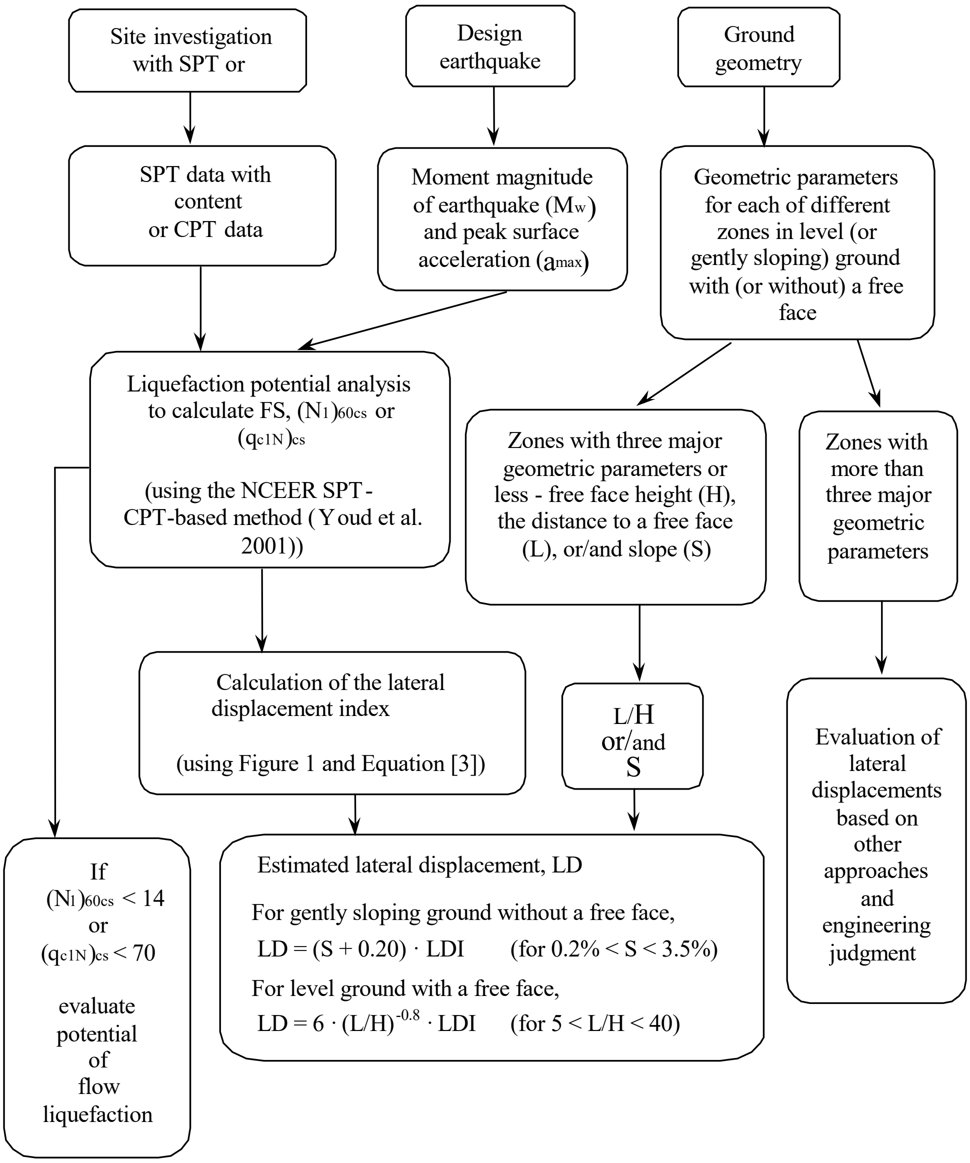
$$I_c = \left[\left(3.47 - \log(Q)\right)^2 + \left(1.22 + \log(F)\right)^2\right]^{0.5}$$

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

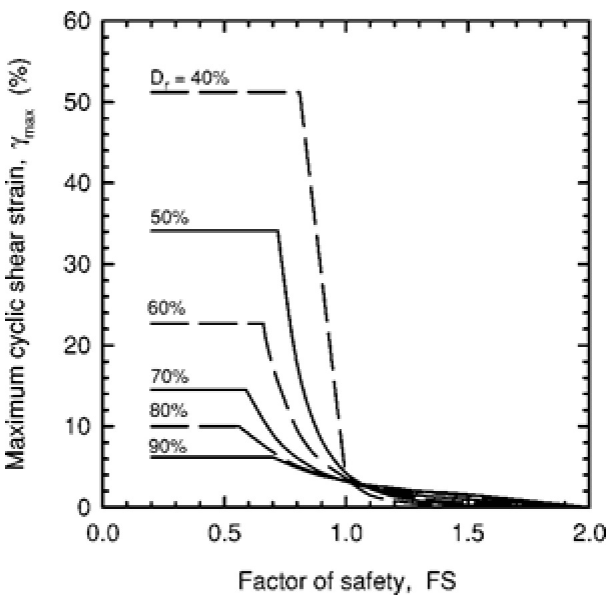
$$F = \left(\frac{f_s}{q_c - \sigma_{vo}}\right) \cdot 100\%$$

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}

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

↓

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

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

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

↓

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.

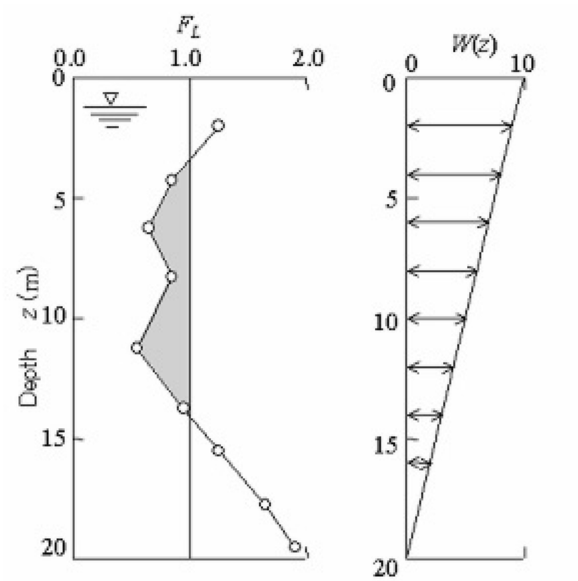
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

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

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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SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

Self-Storage Facility
El Monte, CA
October 13, 2021

Prepared for:

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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 Self-Storage Facility project located in El Monte, California. The work was performed by Kehoe Testing & Engineering (KTE) on October 13, 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 five locations to determine the soil lithology. A summary is provided in **TABLE 2.1**.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	31	Refusal
CPT-2	26	Refusal
CPT-3	29	Refusal
CPT-4	27	Refusal
CPT-5	30	Refusal

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

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

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for 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

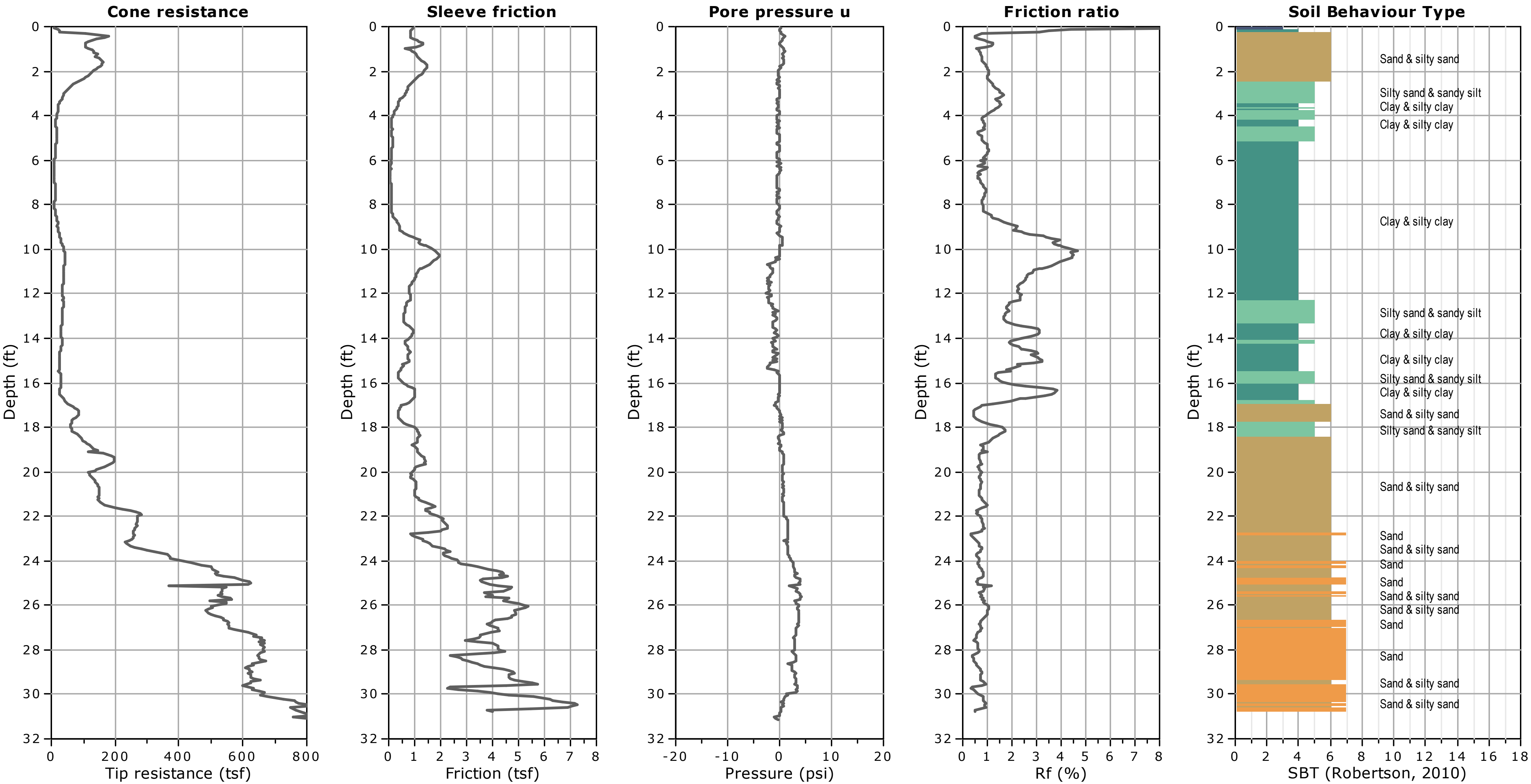
Steven P. Kehoe
President

10/21/21-wt-3401

APPENDIX

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

Project: Southern California Geotechnical / Self Storage Facility **CPT-1**
Location: El Monte, CA Total depth: 31.17 ft, Date: 10/13/2021



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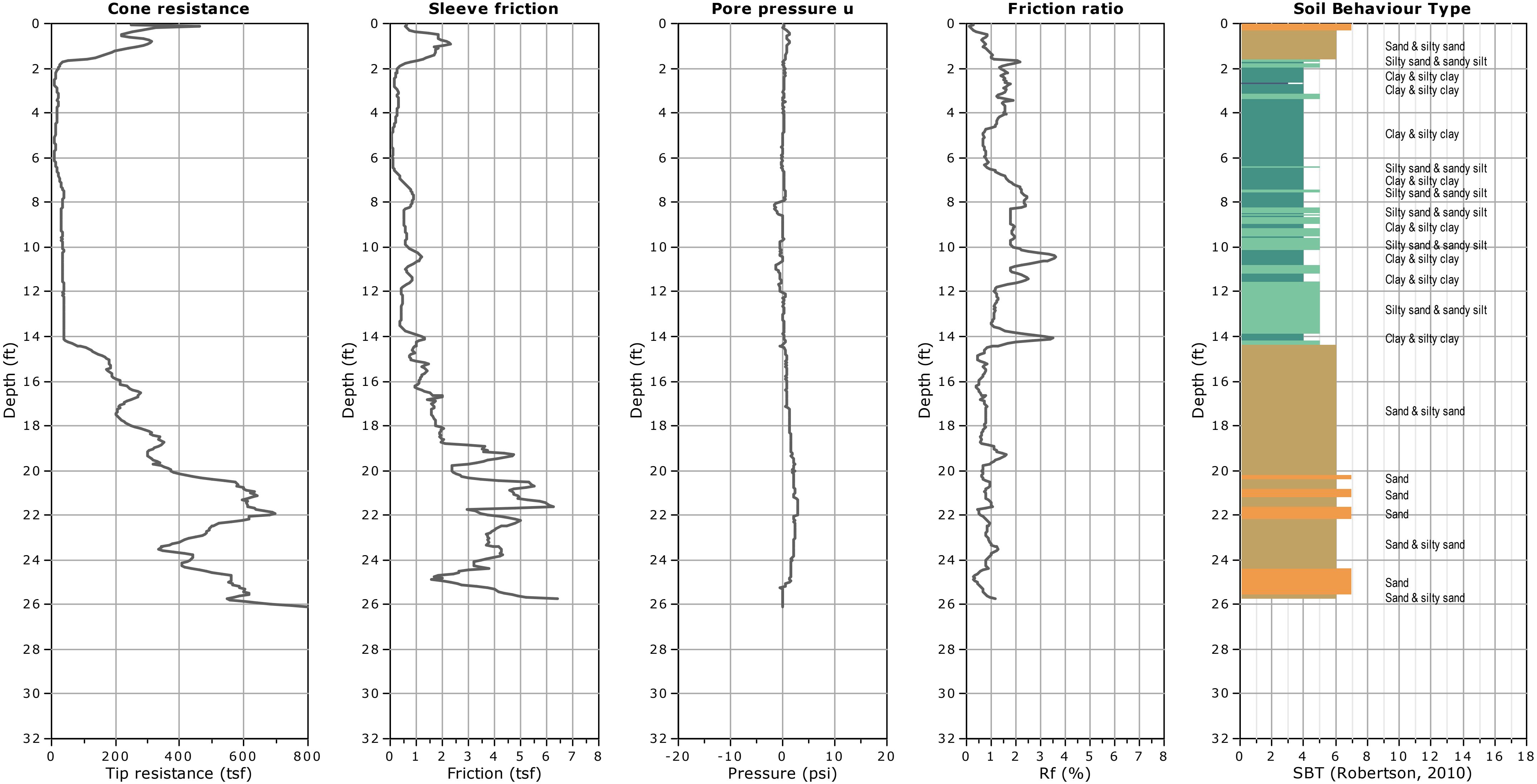
Project: Southern California Geotechnical / Self Storage Facility
Location: El Monte, CA

CPT-2
Total depth: 26.12 ft, Date: 10/13/2021



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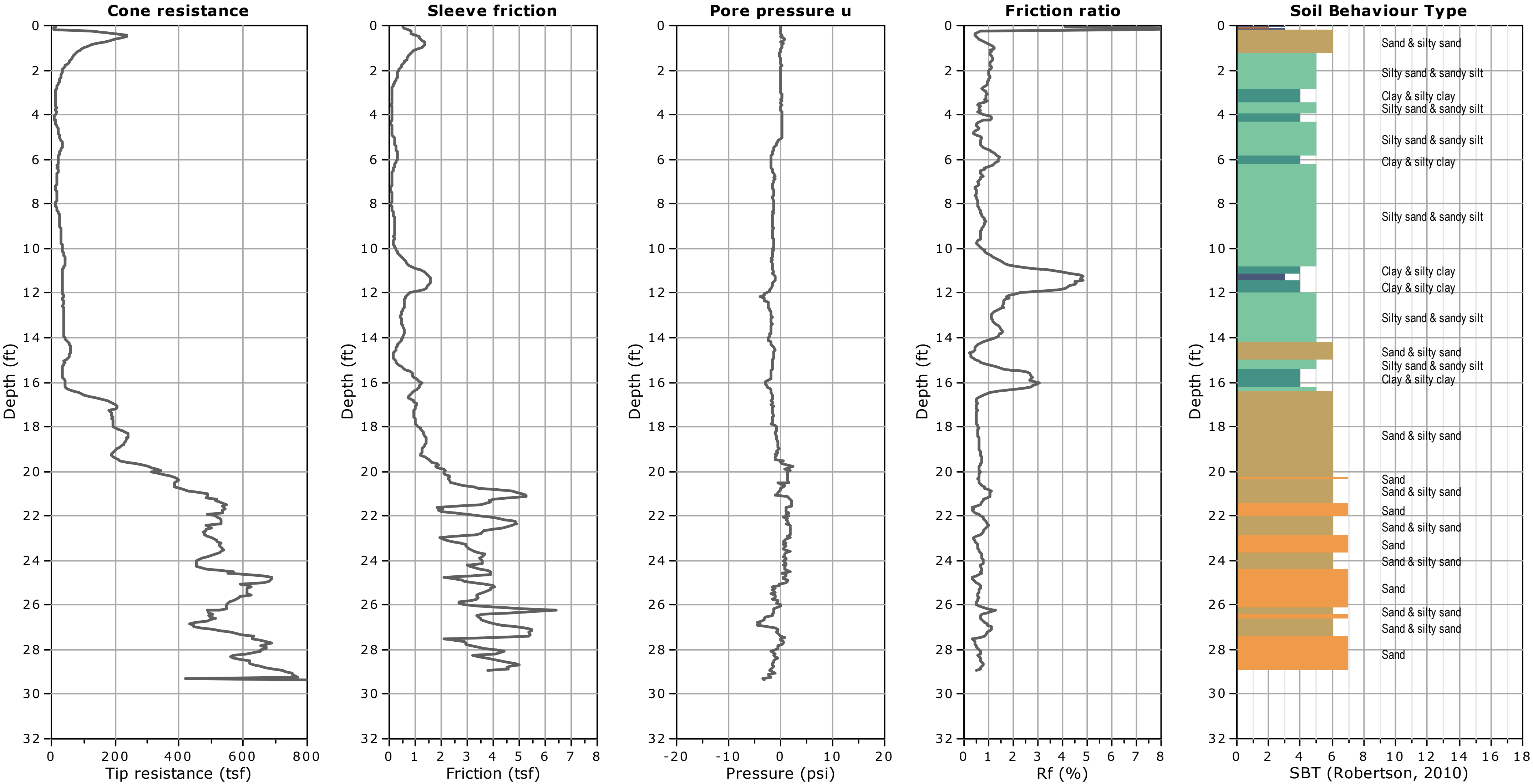
Project: Southern California Geotechnical / Self Storage Facility
Location: El Monte, CA

CPT-3
Total depth: 29.34 ft, Date: 10/13/2021



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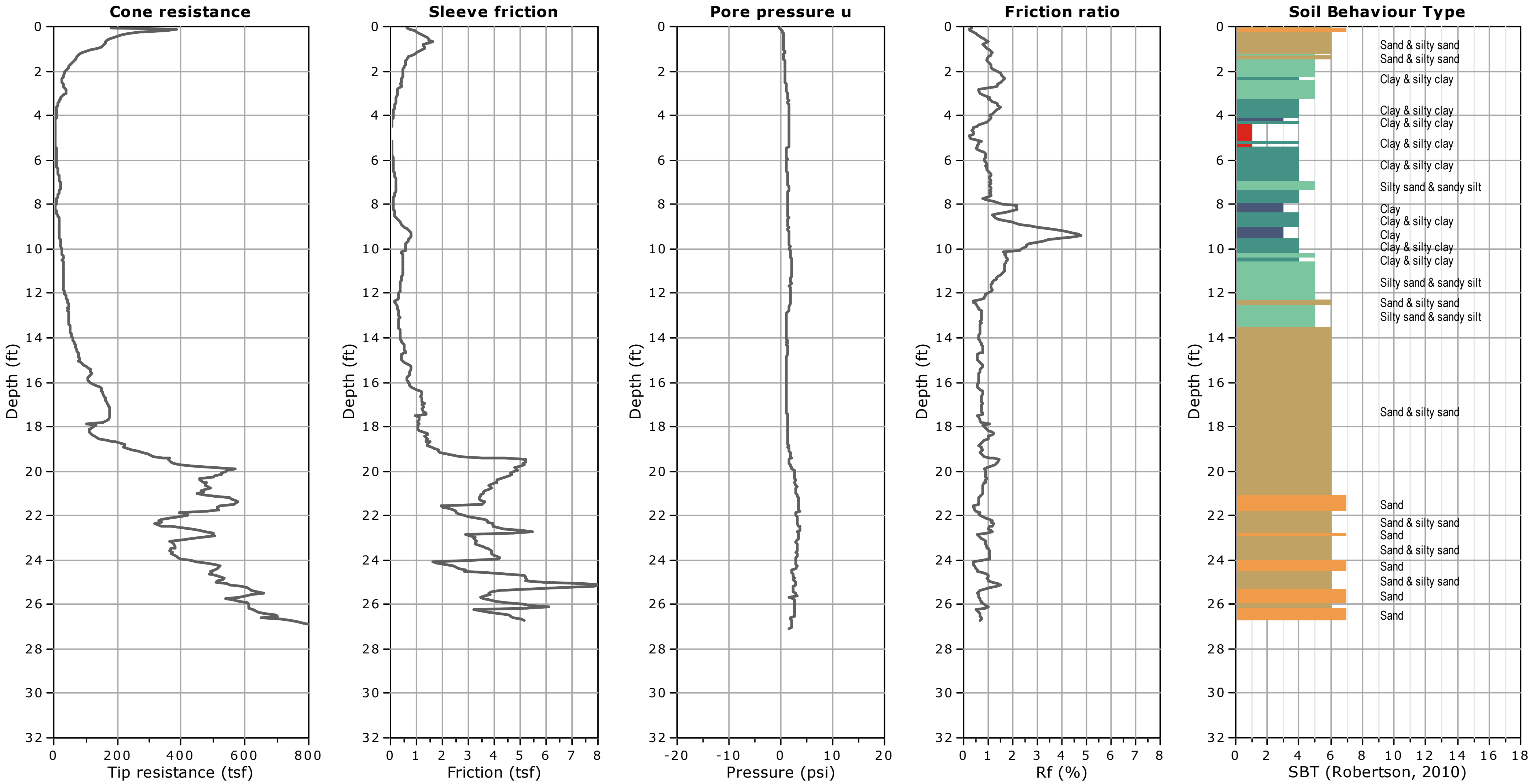
Project: Southern California Geotechnical / Self Storage Facility

Location: El Monte, CA

CPT-4

Total depth: 27.11 ft, Date: 10/13/2021



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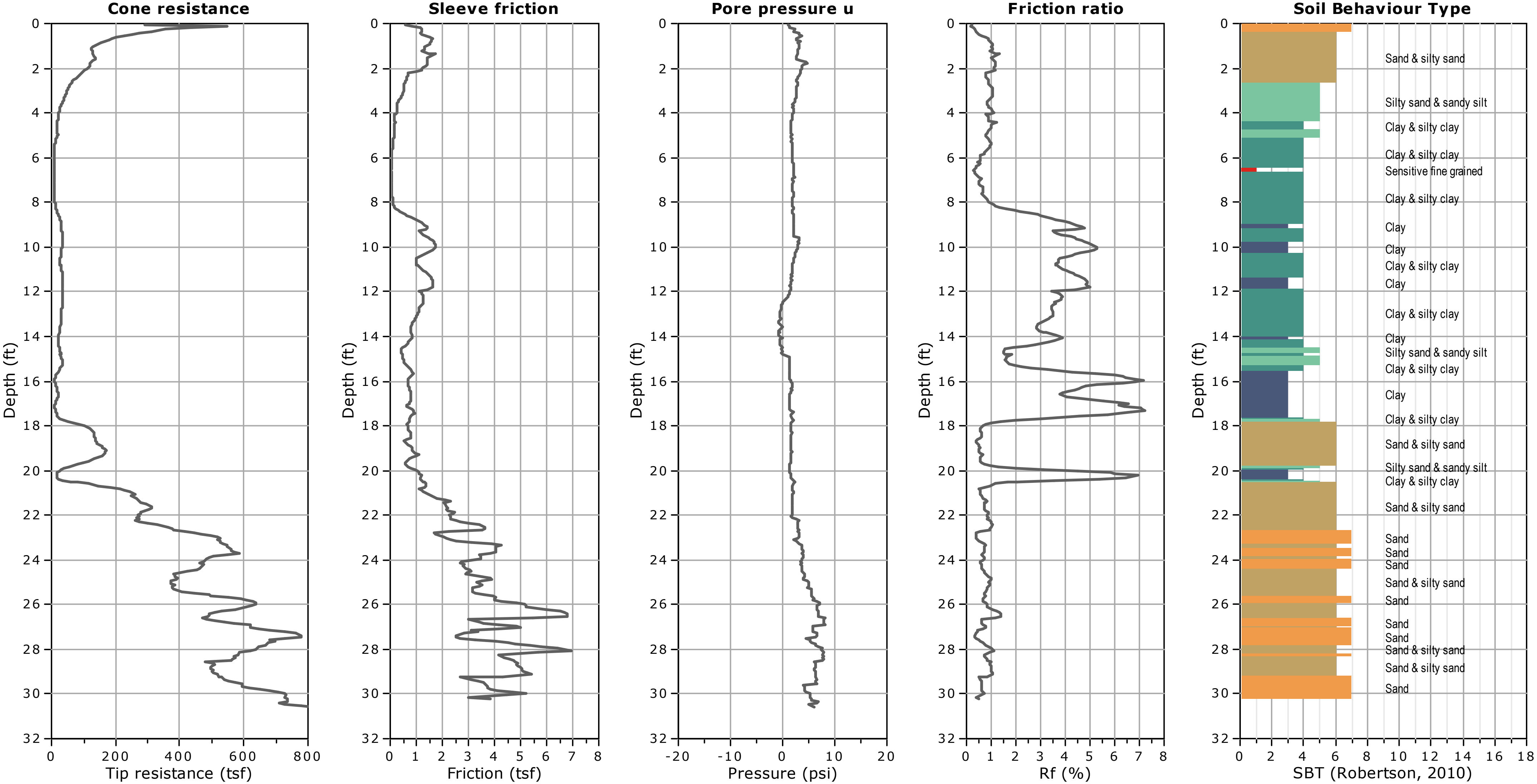
Project: **Southern California Geotechnical / Self Storage Facility**
Location: **El Monte, CA**

CPT-5
Total depth: 30.58 ft, Date: 10/13/2021



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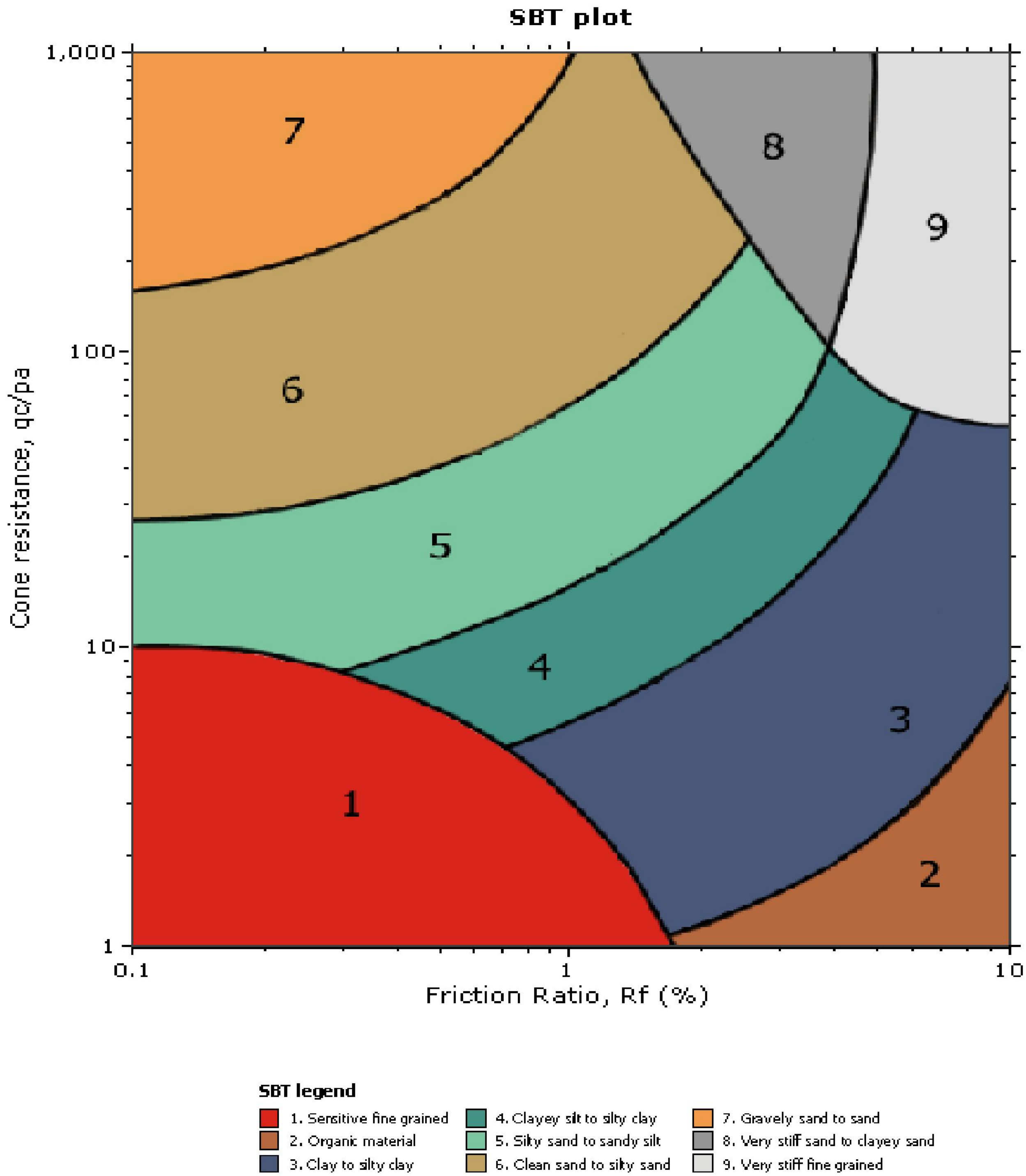

Southern California Geotechnical
Self Storage Facility
El Monte, CA

CPT Shear Wave Measurements

Location	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
CPT-1	5.02	4.02	4.49	5.44	825	
	10.04	9.04	9.26	12.96	714	634
	15.49	14.49	14.63	19.60	746	809
	20.01	19.01	19.11	25.48	750	763
	25.75	24.75	24.83	31.23	795	994
	30.05	29.05	29.12	34.56	843	1288
CPT-2	6.56	5.56	5.91	6.50	909	
	10.01	9.01	9.23	12.08	764	595
	15.03	14.03	14.17	17.76	798	870
	20.05	19.05	19.15	23.00	833	951
	25.03	24.03	24.11	26.88	897	1278
CPT-3	5.05	4.05	4.52	5.56	812	
	10.04	9.04	9.26	13.16	704	624
	15.03	14.03	14.17	18.92	749	853
	20.01	19.01	19.11	24.28	787	922
	25.03	24.03	24.11	29.12	828	1033
	29.36	28.36	28.43	32.44	876	1300

Shear Wave Source Offset - 2 ft

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

APPENDIX G

LIQUEFACTION EVALUATION

Project Name	Proposed Self-Storage Facility	MCE_G Design Acceleration	0.921 (g)
Project Location	El Monte, California	Design Magnitude	6.92
Project Number	21G226	Historic High Depth to Groundwater	20 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-1		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.92)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	20	10		120		1.3	1.05	1.1	1.56	0.75	0.0	0.0	1200	1200	1200	0.97	1.02	1.03	0.06	0.06	N/A	N/A	Above Water Table
19.5	20	22	21	17	120	10	1.3	1.05	1.26	0.94	0.95	26.0	27.1	2520	2458	2520	0.92	1.17	0.97	0.35	0.40	0.57	0.71	**Liquefiable**
24.5	22	27	24.5	28	120		1.3	1.05	1.3	0.91	0.95	43.0	43.0	2940	2659	2940	0.90	1.25	0.93	2.00	2.00	0.60	3.34	Nonliquefiable
29.5	27	32	29.5	46	120		1.3	1.05	1.3	0.94	0.95	72.5	72.5	3540	2947	3540	0.88	1.25	0.9	2.00	2.00	0.63	3.17	Nonliquefiable
34.5	32	37	34.5	50	120		1.3	1.05	1.3	0.95	1	84.1	84.1	4140	3235	4140	0.85	1.25	0.87	2.00	2.00	0.65	3.08	Nonliquefiable
39.5	37	42	39.5	58	120		1.3	1.05	1.3	0.99	1	102.3	102.3	4740	3523	4740	0.82	1.25	0.85	2.00	2.00	0.66	3.03	Nonliquefiable
44.5	42	47	44.5	50	120		1.3	1.05	1.3	0.92	1	81.6	81.6	5340	3811	5340	0.79	1.25	0.82	2.00	2.00	0.66	3.02	Nonliquefiable
49.5	47	50	48.5	22	120	30	1.3	1.05	1.267	0.70	1	26.7	32.1	5820	4042	5820	0.77	1.23	0.85	0.65	0.69	0.66	1.04	**Liquefiable**

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}
(2) Borehole Diameter Correction (Skempton, 1986)
(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)
(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)
(5) Rod Length Correction for Samples <10 m in depth
(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden
(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)
(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)
(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)
(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)
(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)
(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)
(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Self-Storage Facility
Project Location	El Monte, California
Project Number	21G226
Engineer	DWN

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	20	10	0.0	0.0	0.0	N/A	0.50	0.95	0.00	20.00		0.000		0.00	Above Water Table
19.5	20	22	21	26.0	1.1	27.1	0.71	0.07	0.10	0.07	2.00		0.015		0.35	**Liquefiable**
24.5	22	27	24.5	43.0	0.0	43.0	3.34	0.00	-1.04	0.00	5.00		0.000		0.00	Nonliquefiable
29.5	27	32	29.5	72.5	0.0	72.5	3.17	0.00	-3.52	0.00	5.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	84.1	0.0	84.1	3.08	0.00	-4.57	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	102.3	0.0	102.3	3.03	0.00	-6.29	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	81.6	0.0	81.6	3.02	0.00	-4.34	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	26.7	5.4	32.1	1.04	0.03	-0.23	0.03	3.00		0.006		0.22	**Liquefiable**
											Total Deformation (in)				0.57	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION

Project Name	Proposed Self-Storage Facility	MCE_G Design Acceleration	0.921 (g)
Project Location	El Monte, California	Design Magnitude	6.92
Project Number	21G226	Historic High Depth to Groundwater	20 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-2		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.92)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	20	10		120		1.3	1.05	1.1	1.56	0.75	0.0	0.0	1200	1200	1200	0.97	1.02	1.03	0.06	0.06	N/A	N/A	Above Water Table
19.5	20	22	21	10	120	10	1.3	1.05	1.135	0.92	0.95	13.5	14.7	2520	2458	2520	0.92	1.06	0.98	0.15	0.16	0.57	0.28	**Liquefiable**
24.5	22	27	24.5	43	120		1.3	1.05	1.3	0.95	0.95	69.1	69.1	2940	2659	2940	0.90	1.25	0.93	2.00	2.00	0.60	3.34	Nonliquefiable
29.5	27	32	29.5	37	120		1.3	1.05	1.3	0.90	0.95	56.0	56.0	3540	2947	3540	0.88	1.25	0.9	2.00	2.00	0.63	3.17	Nonliquefiable
34.5	32	37	34.5	60	120		1.3	1.05	1.3	1.01	1	107.3	107.3	4140	3235	4140	0.85	1.25	0.87	2.00	2.00	0.65	3.08	Nonliquefiable
39.5	37	42	39.5	48	120		1.3	1.05	1.3	0.92	1	78.3	78.3	4740	3523	4740	0.82	1.25	0.85	2.00	2.00	0.66	3.03	Nonliquefiable
44.5	42	47	44.5	34	120		1.3	1.05	1.3	0.79	1	47.7	47.7	5340	3811	5340	0.79	1.25	0.82	2.00	2.00	0.66	3.02	Nonliquefiable
49.5	47	50	48.5	35	120		1.3	1.05	1.3	0.78	1	48.2	48.2	5820	4042	5820	0.77	1.25	0.81	2.00	2.00	0.66	3.02	Nonliquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Self-Storage Facility
Project Location	El Monte, California
Project Number	21G226
Engineer	DWN

Boring No. B-2

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	20	10	0.0	0.0	0.0	N/A	0.50	0.95	0.00	20.00		0.000		0.00	Above Water Table
19.5	20	22	21	13.5	1.1	14.7	0.28	0.29	0.77	0.29	2.00		0.029		0.70	**Liquefiable**
24.5	22	27	24.5	69.1	0.0	69.1	3.34	0.00	-3.22	0.00	5.00		0.000		0.00	Nonliquefiable
29.5	27	32	29.5	56.0	0.0	56.0	3.17	0.00	-2.09	0.00	5.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	107.3	0.0	107.3	3.08	0.00	-6.77	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	78.3	0.0	78.3	3.03	0.00	-4.04	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	47.7	0.0	47.7	3.02	0.00	-1.40	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	48.2	0.0	48.2	3.02	0.00	-1.44	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				0.70	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION

Project Name	Proposed Self-Storage Facility	MCE_G Design Acceleration	0.921 (g)
Project Location	El Monte, California	Design Magnitude	6.92
Project Number	21G226	Historic High Depth to Groundwater	20 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-3		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.92)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	20	10		120		1.3	1.05	1.1	1.56	0.75	0.0	0.0	1200	1200	1200	0.97	1.02	1.03	0.06	0.06	N/A	N/A	Above Water Table
19.5	20	22	21	15	120	20	1.3	1.05	1.222	0.93	0.95	22.2	26.7	2520	2458	2520	0.92	1.17	0.97	0.34	0.38	0.57	0.68	**Liquefiable**
24.5	22	27	24.5	55	120		1.3	1.05	1.3	0.98	0.95	91.2	91.2	2940	2659	2940	0.90	1.25	0.93	2.00	2.00	0.60	3.34	Nonliquefiable
29.5	27	32	29.5	59	120		1.3	1.05	1.3	0.99	0.95	98.3	98.3	3540	2947	3540	0.88	1.25	0.9	2.00	2.00	0.63	3.17	Nonliquefiable
34.5	32	37	34.5	80	120		1.3	1.05	1.3	1.14	1	161.5	161.5	4140	3235	4140	0.85	1.25	0.87	2.00	2.00	0.65	3.08	Nonliquefiable
39.5	37	42	39.5	75	120		1.3	1.05	1.3	1.14	1	151.6	151.6	4740	3523	4740	0.82	1.25	0.85	2.00	2.00	0.66	3.03	Nonliquefiable
44.5	42	47	44.5	35	120		1.3	1.05	1.3	0.80	1	49.6	49.6	5340	3811	5340	0.79	1.25	0.82	2.00	2.00	0.66	3.02	Nonliquefiable
49.5	47	50	48.5	22	120		1.3	1.05	1.251	0.67	1	25.1	25.1	5820	4042	5820	0.77	1.15	0.89	0.29	0.30	0.66	0.45	**Liquefiable**

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Self-Storage Facility
Project Location	El Monte, California
Project Number	21G226
Engineer	DWN

Boring No. B-3

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	20	10	0.0	0.0	0.0	N/A	0.50	0.95	0.00	20.00		0.000		0.00	Above Water Table
19.5	20	22	21	22.2	4.5	26.7	0.68	0.07	0.13	0.07	2.00		0.016		0.38	**Liquefiable**
24.5	22	27	24.5	91.2	0.0	91.2	3.34	0.00	-5.23	0.00	5.00		0.000		0.00	Nonliquefiable
29.5	27	32	29.5	98.3	0.0	98.3	3.17	0.00	-5.91	0.00	5.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	161.5	0.0	161.5	3.08	0.00	-12.19	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	151.6	0.0	151.6	3.03	0.00	-11.18	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	49.6	0.0	49.6	3.02	0.00	-1.56	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	25.1	0.0	25.1	0.45	0.09	0.23	0.09	3.00		0.019		0.68	**Liquefiable**
											Total Deformation (in)				1.07	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION

Project Name	Proposed Self-Storage Facility	MCE_G Design Acceleration	0.921 (g)
Project Location	El Monte, California	Design Magnitude	6.92
Project Number	21G226	Historic High Depth to Groundwater	20 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-4		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.92)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	20	10		120		1.3	1.05	1.1	1.56	0.75	0.0	0.0	1200	1200	1200	0.97	1.02	1.03	0.06	0.06	N/A	N/A	Above Water Table
19.5	20	22	21	21	120		1.3	1.05	1.3	0.94	0.95	33.4	33.4	2520	2458	2520	0.92	1.25	0.96	0.81	0.97	0.57	1.71	Nonliquefiable
24.5	22	27	24.5	45	120		1.3	1.05	1.3	0.96	0.95	72.7	72.7	2940	2659	2940	0.90	1.25	0.93	2.00	2.00	0.60	3.34	Nonliquefiable
29.5	27	32	29.5	32	120		1.3	1.05	1.3	0.88	0.95	47.3	47.3	3540	2947	3540	0.88	1.25	0.9	2.00	2.00	0.63	3.17	Nonliquefiable
34.5	32	37	34.5	66	120		1.3	1.05	1.3	1.05	1	122.4	122.4	4140	3235	4140	0.85	1.25	0.87	2.00	2.00	0.65	3.08	Nonliquefiable
39.5	37	42	39.5	32	120		1.3	1.05	1.3	0.81	1	45.9	45.9	4740	3523	4740	0.82	1.25	0.85	2.00	2.00	0.66	3.03	Nonliquefiable
44.5	42	47	44.5	30	120		1.3	1.05	1.3	0.76	1	40.5	40.5	5340	3811	5340	0.79	1.25	0.82	2.00	2.00	0.66	3.02	Nonliquefiable
49.5	47	50	48.5	26	120		1.3	1.05	1.3	0.70	1	32.5	32.5	5820	4042	5820	0.77	1.24	0.85	0.70	0.74	0.66	1.11	**Liquefiable**

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Self-Storage Facility
Project Location	El Monte, California
Project Number	21G226
Engineer	DWN

Boring No.	B-4

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	20	10	0.0	0.0	0.0	N/A	0.50	0.95	0.00	20.00		0.000		0.00	Above Water Table
19.5	20	22	21	33.4	0.0	33.4	1.71	0.03	-0.32	0.01	2.00		0.000		0.00	Nonliquefiable
24.5	22	27	24.5	72.7	0.0	72.7	3.34	0.00	-3.54	0.00	5.00		0.000		0.00	Nonliquefiable
29.5	27	32	29.5	47.3	0.0	47.3	3.17	0.00	-1.37	0.00	5.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	122.4	0.0	122.4	3.08	0.00	-8.25	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	45.9	0.0	45.9	3.03	0.00	-1.26	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	40.5	0.0	40.5	3.02	0.01	-0.84	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	32.5	0.0	32.5	1.11	0.03	-0.26	0.03	3.00		0.005		0.19	**Liquefiable**
											Total Deformation (in)				0.19	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer
(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)
(3) Corrected $(N_1)_{60}$ for fines content
(4) Factor of Safety against Liquefaction, calculated previously for the individual layer
(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)
(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)
(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)
(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)