APPENDIX I: GEOTECHNICAL INVESTIGATION

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GEOTECHNICAL INVESTIGATION
PROPOSED PALOMINO BUSINESS PARK

NEC First Street and Pacific Avenue
Norco, California
for
Caprock Acquisitions, LLC

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

July 9, 2019

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Caprock Aquisions, LLC
1300 Dove Street, Suite 200
Newport Beach, California 92660

Attention: Mr. Patrick Daniels

Project No.: **17G105-3**

Subject: **Geotechnical Investigation**
Proposed Palomino Business Park
NEC First Street and Pacific Avenue
Norco, California

Gentlemen:

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

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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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 Riverside County GIS website indicates that the subject site is located within a zone of medium to high liquefaction susceptibility.
- Our site-specific liquefaction evaluation included seven (4) borings extended to depths of 50 to 51± feet and four (4) CPT soundings advanced to depths of 34 to 60± feet. Liquefiable soils were encountered five (5) of the borings and all four (4) of the CPT locations.
- The potential liquefaction-induced settlements range from 0.23± to 1.88± inches. Differential settlements of up to 1± inch are anticipated at the boring locations.
- Based on the estimated magnitude of the differential settlements, the proposed structures may be supported on shallow foundations. Additional design considerations related to the potentially liquefiable soils are presented in this report.
- The subject site is underlain by variable strength fill and near-surface native alluvial soils. These soils are not considered suitable, in their present condition, to support the foundations and floor slabs of the proposed structures. Remedial grading is recommended to remove the undocumented artificial fill soils and a portion of the near surface alluvial soils from the proposed building pad areas in order to replace them as compacted structural fill. The overexcavation and recompaction of these layers as structural fill will provide more consistent support characteristics for the proposed structures and help to mitigate against potential surface manifestations due to liquefaction.

Site Preparation

- The site is presently developed with numerous structures of varying age and type. It is assumed that all of these structures will be demolished as part of the proposed development. Demolition should include all foundations, floor slabs, utilities, and any other subsurface improvements that will not be utilized with the new development. Demolition of several paved areas will also be required. Concrete and asphalt debris resultant from demolition may be crushed to a maximum 2 inch particle size and mixed with on-site soils and reutilized as structural fill. Other demolition debris should be disposed of offsite in accordance with local regulations.
- Vegetation and organic materials within the existing landscaped areas should be disposed of off-site or in non-structural areas of the property.
- The existing soils within the proposed building areas should be overexcavated to a depth of 3 feet below existing grade and to a depth of at least 3 feet below proposed building pad subgrade elevation. The overexcavation should extend horizontally at least 5 feet beyond the building and foundation perimeters. The overexcavation should also extend to a sufficient depth to remove any artificial fill soils.
- The proposed foundation influence zones should be overexcavated to a depth of at least 2 feet below proposed foundation bearing grade.

- Following evaluation of the subgrade by the geotechnical engineer, the exposed subgrade soils should be scarified, moisture conditioned and/or flooded as necessary to achieve a moisture content of 2 to 4 percent above optimum, and recompacted. The resulting soils may be replaced as compacted structural fill.

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

Building Floor Slabs
- Conventional slabs-on-grade, 6 inches thick.
- Minimum reinforcement of the floor slab should consist of No. 3 bars at 16-inches on center in both directions, due to the presence of low to medium expansive soils and potentially liquefiable soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed slab loading and potential differential settlements.
- Modulus of Subgrade Reaction: 100 psi/in.

Pavements

ASPHALT PAVEMENTS (R = 25)					
Materials	**Thickness (inches)**				
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic		
			(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	4	7	9	11	12
Compacted Subgrade (90% minimum compaction)	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI =6.0)	(TI = 7.0)	(TI = 8.0)
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 Change Order Request No. 17G105-CO, dated May 24, 2019. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slabs, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. Based on the location of the subject site, this investigation also included a site-specific liquefaction evaluation. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the northeast corner of First Street and Pacific Avenue in Norco, California. The site is generally bounded to the north by Second Street, to the east by Mountain Avenue and commercial/industrial developments, to the south by First Street, and to the west by Pacific Avenue. The subject site also includes several additional parcels, which are located on the east side of Mountain Avenue and at the southwest corner of Mountain Avenue and First Street. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in Appendix A of this report.

The overall site consists of multiple rectangular- and irregular-shaped parcels, which total 118± acres in size. The parcels located adjacent to Pacific Avenue, First Street, and Second Street are developed with single-family residences or consist of vacant lots. The single-family residences appear to be of wood-frame and stucco construction and are assumed to be supported on conventional shallow foundations with concrete slab-on-grade floors. The ground surface cover in these areas consists of concrete driveways, concrete flatwork, and areas of exposed soil with moderate to dense native grass and weed growth. Several medium to large trees are present throughout these areas.

The existing parcels located on the east and west sides of Mountain Avenue are generally developed or consist of single-family residences, vacant lots, and a commercial/industrial building approximately 150,000 ft^2 in size. The commercial/industrial building appears to be a metal-framed structure and is assumed to be supported on conventional shallow foundations with a concrete slab-on-grade floor. The ground surface cover surrounding the building consists of asphaltic concrete in the parking and drive areas, and Portland cement concrete pavements in the loading dock areas. The single-family residences appear to consist of wood-frame and stucco construction and are assumed to be supported on conventional shallow foundations with concrete slab-on-grade floors. Several parcels in these areas contain concrete slabs from pre-existing structures. The ground surface cover in these areas consists of exposed soil with moderate to dense native grass and weed growth. In addition to the development described above, an asphalt parking lot, about 47,000 ft^2 in size, is located at the southeast corner of Second Street and Mountain Avenue. Several medium to large trees are also present throughout these parcels. A 5- to 7-foot wide drainage course is present in the parcel located at the northwest corner of Mountain Avenue and First Street. The draineage course is approxiamtly 2 to 4 feet deep.

Detailed topographic information was not available at the time of this report. Based on visual observations, the site topography within the subject site appears to be relatively level, sloping gently downward to the west-southwest at a gradient of less than 1± percent.

3.2 Proposed Development

A preliminary site plan (identified as Scheme 38) prepared by Carlile Coatsworth Architects, Inc. was provided to our office by the client. This plan indicates that the subject site will be developed with a total of thirty-six (36) commercial/industrial buildings (identified as Building 1 through Building 26). The new buildings will range from 4,095 to 157,275± ft² in size. Several of the buildings will share a common wall. The larger buildings will possess dock high doors on one side. The new buildings are expected to be surrounded by asphaltic concrete pavements in the parking and drive lane areas, Portland cement concrete pavements in the loading dock areas, concrete flatwork and landscaped planters located throughout the site.

Detailed structural information has not been provided. It is assumed that the new buildings will be single-story structures of tilt-up concrete construction, typically supported on conventional shallow foundation systems and concrete slab-on-grade floors. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 80 kips and 3 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 5± feet are expected to be necessary to achieve the proposed site grades within the proposed building areas. The proposed buildings are not expected to incorporate any significant below-grade construction such as basements or crawl spaces. It should be noted that this estimate does not include any remedial grading recommendations which are presented in a subsequent section of this report.

3.3 Previous Studies

Southern California Geotechnical, Inc. (SCG) previously prepared the following geotechnical reports for the subject site:

> Geotechnical Investigation and Liquefaction Evaluation, Proposed Norco Ranch Commerce Park, Mountain Avenue at 2nd Street, Norco, California, prepared by Southern California Geotechnical, Inc. (SCG) for Alere Property Group, LLC, SCG Project No. 11G114-1R, dated March 17, 2011.

> Supplementary Geotechnical Investigation, Proposed Norco Ranch Commerce Park- 6 Acre Parcel, Mountain Avenue at 2nd Street, Norco, California, prepared by Southern California Geotechnical, Inc. (SCG) for Alere Property Group, LLC, SCG Project No. 11G114-3, dated June 30, 2011.

> Geotechnical Feasibility Study, Proposed Commercial/Industrial Development, NEC First Street and Pacific Avenue, Norco, California, prepared by SCG for CapRock Partners, SCG Project No. 17G105-1, dated March 2, 2017

As part of these investigations, a total of thirteen (19) borings were advanced to depths of 20 to 51± feet. The 50 to 51±-foot deep borings were performed as part of the liquefaction evaluation.

Four (4) CPT soundings were also performed to depths ranging between 34 and 60± feet. CPT-1 and CPT-2 both encountered refusal conditions between depths of 34 and 35± feet.

The borings generally engounctered native alluvium at the ground surface, but artificial fill soils were encountered at the ground surface at one of the borings, Boring No. B-5, extending to a depth of 2½± feet below the existing grade. The fill soils generally consisted of medium dense silty fine sands. The alluvial soils extending from the ground surface or beneath the fill materials to depths of 8½ to 17± feet generally consist of loose to medium dense silty sand, clayey sands, and fine sandy silts. In addition, several zones of medium stiff to stiff clayey silts and silty clays are present in the upper 10± feet. At depths greater than 17± feet, the borings encountered medium dense to very dense fine to medium sands and fine to coarse sands with varying amounts of fine to coarse gravel content. At these depths, several strata of medium stiff to very stiff clayey silts, silty clays and sandy clays were also encountered. The native alluvial soils extended to at least the maximum depth explored of 51± feet excetpt at Boring No. B-16, which encountered La Sierra Tonalite bedrock at a depth of 37± feet. The bedrock consisted of very dense, phaneritic, highly weathered and friable tonalite. Free water was encountered during drilling Boring Nos. B-1, B-4, B-11, B-12, and B-13, B-16, B-17, and B-18, at depths ranging from 22 to 41± feet during the previous subsurface explorations.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration performed for this project consisted of a total of twenty-four (24) borings and four (4) cone penetration test (CPT) soundings. As discussed in the previous section of this report, nineteen (19) of these borings and all four of the CPT soundings were performed as a part of the referenced previous studies. Boring Nos. B-1 through B-15 were performed as a part of the referenced geotechnical investigation and supplemental investigation in 2011 (SCG Project Nos. 11G114-1 and 11G114-3). Boring Nos. B-16 through B-19 and CPT-1 through CPT-4 were performed as a part of the referenced feasibility study in 2017 (SCG Project No. 17G105-1). Five (5) additional borings, Boring Nos. B-20 through B-24, were performed recently as a part of this geotechnical report. 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 undisturbed soil samples were taken during drilling. Relatively undisturbed samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. Samples were also taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers were 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 geologist. The cone system used for this project was manufactured by Vertek. The CPT soundings were performed in general accordance with ASTM standards (D-5778). The cone penetrometers were pushed using 30-ton CPT rig. The cones used during the program recorded the cone resistance, sleeve friction, and dynamic core pressure at 2.5 centimeter depth intervals. The CPT soundings were advanced to depths of 34 to 60± feet. A more complete description of the CPT program as well as the results of the data interpretation are enclosed in Appendix G of this report. The CPT soundings do not result in any recovered soil samples. However, correlations have been developed that utilize the cone resistance and the sleeve friction to estimate the soil type that is present at each 2.5 centimeter interval in the subsurface profile. These soil classifications are presented graphically on the CPT output forms enclosed in Appendix G.

The raw data generated by the cone penetrometer equipment has been reduced using CPeT-IT, V1.6, 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 G 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 for recent Boring Nos. B-20 through B-24, 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 Boring Logs for the referenced previous studies, Boring Nos. B-1 through B-19, are included in Appendix H of this report.

4.2 Geotechnical Conditions

Pavements

Asphaltic concrete pavement was encountered at Boring No. B-5, which was performed for a referenced previous study. The pavement at this boring location consists of 3± inches of asphaltic concrete with 3½± inches of underlying aggregate base.

Artificial Fill

Artificial fill soils were encountered beneath the pavement at Boring No. B-5, extending to a depth of 2½± feet below the existing grade. The fill soils generally consist of medium dense silty fine sands.

Alluvium

The near surface soils at this site generally consist of interbedded sands, silts, and clays. Native alluvial soils were encountered beneath the fill soils at Boring No. B-5 and at the ground surface at all of the remaining boring locations. The alluvial soils within the upper 4 to 12± feet generally consist of loose to medium dense silty sands, clayey sands, and fine sandy silts. In addition, several zones of medium stiff to very stiff clayey silts, silty clays, and clayey silts are present in the upper 2½ to 8± feet.

At depths greater than 12± feet, the borings generally encountered medium dense to very dense fine to medium sands and fine to coarse sands with varying amounts of fine to coarse gravel content. At these depths, several strata of stiff to very stiff clayey silts, silty clays, and sandy clays were also encountered. The native alluvial soils extended to depths of 37± feet to at least the maximum depth explored of 51± feet.

La Sierra Tonalite

Bedrock was encountered at one boring performed for a referenced previous study (SCG Project No. 11G114-1). Boring No. B-16 encountered La Sierra Tonalite bedrock at a depth of 37± feet and extended to the maximum depth explored of 50± feet. The bedrock consists very dense, gray brown, phaneritic, highly weathered and friable tonalite.

Groundwater

Free water was not encountered during the drilling of any of the borings during the current subsurface exploration. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater is considered to have existed at a depth in excess of 20± feet at the time of the current subsurface exploration. It should be noted that free water was encountered at several of the boring locations, at depths ranging from 22 to 41± feet during the previous subsurface explorations.

Research from the Western Municipal Water District (WMWD) includes three wells that are located within approximately 1 mile of the subject site. Data obtained from these wells is presented below. It should be noted that the groundwater data for these wells extends from the present time back to the Spring of 1993.

WMWD Data

Well Number	Location	Elevation (ft)	Depth (ft)
03S/07W-13	0.3± miles E	605	26 to 34
03S/07W-14C	1,800± feet SW	569	21
03S/07W-12R	1,900± feet SW	620	30

Additionally, recent water level data was obtained from the California State Water Resources Control Board, GeoTracker, website, http://geotracker.waterboards.ca.gov/. Several monitoring wells in this database are located within a half mile from the subject site. Water level readings within these monitoring wells indicate groundwater levels as high as 25± feet below the ground surface, in January 2012.

Based on these data, the long-term groundwater depths in the vicinity of the subject site are expected to be in the range of 21 to 34± feet.

4.3 Geologic Conditions

Regional geologic conditions were obtained from the Geologic Map of the Corona North 7.5' Quadrangle, Riverside and San Bernardino Counties, California, by Douglas M. Morton and C.H. Gray, Jr., 2002. This map indicates that the site is underlain by very old alluvial channel deposits and La Sierra Tonalite (Map Symbol Klst). The Cretaceous age La Sierra Tonalite bedrock is described as a dark colored, massive, structure-less, medium to coarse grained, biotite tonalite. The La Sierra Tonalite is part of the composite Peninsular Ranges batholith that underlies the local region.

Based on the materials encountered at Boring No. B-16, it is our opinion the site is underlain by La Sierra formation tonalite. At this boring location, the bedrock consisted of very dense, medium to coarse grained, jointed, weathered tonalite bedrock. Therefore, the geologic conditions at the site are considered to be consistent with the mapped geologic conditions.

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. The results of consolidation testing performed for the referenced previous studies are presented in Appendix H of this report.

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.

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on a selected sample. This test is used to determine the Liquid Limit and Plastic Limit of the soil. The Plasticity Index is the difference between the two limits. Plasticity Index is a general indicator of the expansive potential of the soil, with higher numbers indicating higher expansive potential. Soils with a PI greater than 25 are considered to have a high expansion potential. Soils with a PI greater 18 are not considered to susceptible to liquefaction. Soils with a PI between 12 and 18 may possess a moderate susceptibility to liquefaction. The results of the Atterberg Limits testing are presented on the boring logs.

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 (%)**	**Sulfate Classification**
B-1 @ 0 to 5 feet	0.002	Not Applicable (S0)
B-5 @ 0 to 5 feet	0.003	Not Applicable (S0)
B-13 @ 0 to 5 feet	<0.001	Not Applicable (S0)
B-14 @ 0 to 5 feet	0.001	Not Applicable (S0)
B-16 @ 0 to 5 feet	0.005	Not Applicable (S0)
B-19 @ 0 to 5 feet	<0.001	Not Applicable (S0)
B-20 @ 0 to 5 feet	0.001	Not Applicable (S0)
B-24 @ 0 to 5 feet	0.002	Not Applicable (S0)

Maximum Dry Density and Optimum Moisture Content

Representative bulk samples were tested for their maximum dry densities and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil type or soil mixes may be necessary at a later date. The results of the testing are plotted on Plates C-9 and C-10 in Appendix C of this report. The results of testing performed for the referenced previous studies are presented in Appendix H of this report.

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829. The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample. The sample is initially remolded to 50 ± 1 percent saturation and then loaded with a surcharge equivalent to 144 pounds per square foot. The sample is then inundated with water, and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The results of the EI testing are as follows:

Sample Identification	Expansion Index	Expansive Potential
B-1 @ 0 to 5 feet	2	Very Low
B-9 @ 0 to 5 feet	51	Medium
B-13 @ 0 to 5 feet	5	Very Low
B-14 @ 0 to 5 feet	58	Medium
B-16 @ 0 to 5 feet	49	Low
B-19 @ 0 to 5 feet	52	Medium
B-23 @ 0 to 5 feet	37	Low

Direct Shear

Direct shear tests were performed on selected soil samples during the referenced geotechnical investigation 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 test are presented in Appendix H of this report.

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed 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 2016 California Building Code (CBC) was adopted by all municipalities within Southern California on January 1, 2017. The CBC provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

The 2016 CBC Seismic Design Parameters have been generated using SEAOC/OSHPD Seismic Design Maps Tool, a web-based software application developed by the United States Geological Survey. This software application, available at the USGS web site, calculates seismic design parameters in accordance with the 2016 CBC, utilizing a database of deterministic site accelerations at 0.01 degree intervals. The table below is a compilation of the data provided by the USGS application. A copy of the output generated from this program is included as Plate E-1 in Appendix E of this report. A copy of the Design Response Spectrum, as generated by the SEAOC/OSHPD application is also included in Appendix E. Based on this output, the following parameters may be utilized for the subject site:

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter		**Value**
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.606
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.635
Site Class	---	F*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.606
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.952
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.071
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.635

*The 2016 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 7-10. However, Section 20.3.1 of ASCE 7-10 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of the structure is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. Therefore, if the proposed structure has a fundamental period greater than 0.5 seconds, a site specific seismic hazards analysis would be required and additional subsurface exploration would be necessary.

Ground Motion Parameters

For the purposes of the liquefaction analysis performed for this study, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2016 CBC. The peak ground acceleration (PGA_M) was determined in accordance with Section 11.8.3 of ASCE 7-10. 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 7-10. The web-based software application SEAOC/OSHPD Seismic Design Maps Tool (described in the previous section) was used to determine PGA_M, based on ASCE 7-10 as the building code reference document. A portion of the program output is included as Plate E-1 in Appendix E of this report. As indicated on Plate E-1, the PGA_M for this site is 0.607g. An associated earthquake magnitude was obtained from the 2008 USGS Interactive Deaggregation application available on the USGS website. The deaggregated modal magnitude is 6.98, based on the peak ground acceleration and NEHRP soil classification D.

Liquefaction

Review of the Riverside County GIS website indicates that the subject site is located within a zone of medium to high liquefaction susceptibility. Therefore, the scope of this feasibility study included a detailed liquefaction evaluation in order to determine the site-specific liquefaction potential.

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and plasticity characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Non-sensitive clayey (cohesive) soils which possess a plasticity index of at least 18 (Bray and Sancio, 2006) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The liquefaction analysis was conducted in accordance with the requirements of Special Publication 117A (CDMG, 2008), and currently accepted practice (SCEC, 1997). The liquefaction potential of the subject site was evaluated using the empirical method developed by Boulanger and Idriss (Boulanger and Idriss, 2008, 2014). This method predicts the earthquake-induced liquefaction potential of the site based on a given design earthquake magnitude and peak ground acceleration at the subject site. This procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude). CRR is determined as a function of the corrected SPT N-value $(N_1)_{60\text{-}cs}$, adjusted for fines content or the corrected CPT tip stress, q_{c1Ncs}. 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 not considered to be susceptible to liquefaction. Non-sensitive soils with a PI greater than 18 are also considered non-liquefiable. Soils possessing a PI between 12 and 18, may also be moderately susceptible to liquefaction if the moisture content is greater than 85 percent of the liquid limit.

Liquefaction potential was evaluated for the subject site using field and laboratory data from the seven (7) boring locations that were drilled to depths of at least 50± feet and all four (4) of the CPT locations. The liquefaction potential was analyzed for the design level earthquake utilizing a PGA_M of 0.607g related to a 6.98 magnitude seismic event, assuming historic high groundwater levels of 21± feet.

The liquefaction analysis procedure for the seven (7) borings is tabulated on the spreadsheet forms included in Appendix F of this report. The liquefaction potential for the on-site soils was also determined using data obtained at the four CPT locations. This data was analyzed using the computer program Cliq V2.2, which was developed by Geologismiki, copyright 2006. The output of this computer program is also provided in Appendix F. The analysis method for both the boring and CPT data is based on Boulanger and Idriss, 2014.

If liquefiable soils are identified, the potential settlements that could occur as a result of liquefaction are determined using the equation for volumetric strain due to post-cyclic reconsolidation (Yoshimine et. al, 2006). This procedure uses an empirical relationship between the induced cyclic shear strain and the corrected N-value or CPT tip stress to determine the expected volumetric strain of saturated soils subjected to earthquake shaking. The settlement analysis is also provided on the spreadsheets and Cliq program output included in Appendix F.

Conclusions and Recommendations

The results of the liquefaction analysis have identified potentially liquefiable soils at five (5) of the seven 50- to 51±-foot-deep borings and at all four (4) of the CPT soundings performed at the site. No liquefiable soil strata were encountered at Boring Nos. B-1 and B-17. 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 32 and 49± feet at the borings are also considered to be non-liquefiable due to their cohesive characteristics and the results of the Atterberg limits testing with respect to the criteria of Bray and Sancio (2006). Settlement analyses were conducted for each of the potentially liquefiable strata. The results of the dynamic settlement analyses (also tabulated on the spreadsheets in Appendix F and the program output in Appendix H) are presented below:

- Boring No. B-1: 0.00± inches
- Boring No. B-4: 1.24± inches
- Boring No. B-11: 1.50± inches
- Boring No. B-13: 1.00± inches
- Boring No. B-16: 1.39± inches
- Boring No. B-17: 0.00± inches
- Boring No. B-18: 1.10± inches

Liquefiable soils were generally encountered at similar depths at the four CPT locations. Settlement analyses were conducted for each of the potentially liquefiable strata. It should be noted that CPT-1 and CPT-2 encountered refusal conditions between depthf of 34 and 35 feet. The total liquefaction settlement for each CPT is presented below:

- CPT-1: 0.23± inches
- CPT-2: 0.31± inches
- CPT-3: 1.88± inches
- CPT-4: 1.39± inches

Based on these total settlements, differential settlements of up to 1± 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 about 0.002 inches per inch.

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

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

The use of a shallow foundation system, as described in this report, is typical for buildings of this type, where they are underlain the extent of liquefiable soils encountered at this site. The post-liquefaction damage that could occur within the buildings 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

The subsurface conditions at this site generally consist of low to moderate strength alluvium, extending to depths of 3 to 12± feet, underlain by moderate to high strength alluvium and tonalite bedrock at greater depths. The results of laboratory testing indicate that some of the alluvium within the upper 3 to 6± feet possess minor to moderate collapse potentials. One of the borings for a referenced previous study, Boring No. B-5, also encountered artificial fill soils within the upper 2½± feet. Additional fill soils are expected to be encountered beneath and around the numerous structures which currently exist on the subject site. Based on the age of the existing developments, no documentation regarding the placement or compaction of any existing fill soils is expected to be available. Any fill soils are therefore considered to represent undocumented fill, not suitable for support of new structures. Based on these considerations, remedial grading will be necessary to remove and replace the existing undocumented fill soils as well as the surficial low strength alluvium.

Extensive demolition will be necessary since numerous structures currently occupy various parcels throughout the site. Demolition of these existing structures is expected to result in significant disturbance of the upper 2 to 4± feet of the existing soils.

As discussed in a 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 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.

Settlement

Laboratory testing indicates that the upper zone of native alluvium is potentially compressible and/or collapsible. The existing undocumented fill soils are also considered to be potentially compressible. The recommended remedial grading will remove all of these soils from within the proposed building areas. Provided that the recommended remedial grading is completed, the post-construction static settlements of the proposed structures are 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 to 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 consist of variable materials ranging from sands and silty sands to clayey silts and silty clays. Expansion index testing indicates that these materials possess very low to medium expansion potentials (EI = 2 to 58). Based on the presence of expansive soils, special care should be taken to properly moisture condition and maintain adequate moisture content within all subgrade soils as well as newly placed fill soils. The foundation and floor slab design recommendations contained within this report are made in consideration of the expansion index test results. 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 pads.

Shrinkage/Subsidence

Removal and recompaction of the near-surface alluvium and fill soils is estimated to result in an average shrinkage of 7 to 12 percent. However, the estimated shrinkage of the individual soil layers at the site is highly variable, locally ranging from 0 to 24 percent shrinkage. It should be noted that the potential shrinkage estimate is based on dry density testing performed on small-diameter samples taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

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

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

Grading and Foundation Plan Review

Granding and foundation plans were not available at the time of this report. It is therefore recommended that we be provided with copies of the preliminary grading and foundation 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

Initial site preparation will require demolition of numerous existing structures and pavements which occupy several areas of the subject site. Demolition of the existing structures should include all foundations, floor slabs, utilities and any other subsurface elements which will not remain in place for use with the new development. A representative of the geotechnical engineer should be present during the concluding stages of demolition to verify adequate removals. Concrete and asphalt debris resultant from demolition may be crushed to a maximum 2-inch particle size and mixed with the on-site soils for later use as structural fill.

Several areas of the site are landscaped with medium to large trees, shrubs, turf grass and other organic materials. Any organic materials that are currently present on the site should be removed and disposed of offsite or in non-structural areas of the property. The actual extent of site stripping should be determined by the geotechnical engineer at the time of grading, based on the organic content and the stability of the encountered materials.

Treatment of Existing Soils: Building Pads

Remedial grading should be performed within the proposed building areas in order to remove all of the undocumented fill soils, the upper portion of the alluvial soils, and any soils disturbed during the demolition of the existing site improvements. The fill soils were determined to extend to a depth of 2½± feet at Boring No. B-5. The fill soils are also expected to extend to similar or greater depths throughout the developed areas of the site. Based on the conditions encountered at the boring locations, the existing soils within the proposed building area are recommended to be overexcavated to a depth of at least 3 feet below the proposed building pad subgrade elevation and to a depth of at least 3 feet below existing grade, whichever is greater. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 2 feet below proposed foundation bearing grade.

Due to the extent and variety of the existing developments on the subject site, additional depths of undocumented fill soils are expected to be encountered during the grading process. Any undocumented fill soils that are encountered during grading should be removed in their entirety.

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 structures incorporate any exterior columns (such as for a canopy or overhang) the overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the building areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structures. 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. Any soils classified as possible fill soils on the boring logs should be evaluated at this time, in order to determine if they consist of undocumented fill materials. If so, then these undocumented fill soils should be removed as discussed above.

Based on the conditions encountered at the exploratory boring locations, some zones of very moist silty clays and clayey silts will be encountered at or near the base of the recommended overexcavation. Stabilization of the exposed overexcavation subgrade soils may be necessary in some localized areas. Scarification and air drying of these materials may be sufficient to obtain a stable subgrade. However, if highly unstable soils are identified, and if the construction schedule does not allow for delays associated with drying, mechanical stabilization, usually consisting of coarse crushed stone or geotextile, could be necessary. In this event, the geotechnical engineer should be contacted for supplementary recommendations.

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 to at least 90 percent of the ASTM D-1557 maximum dry density. Overexcavation bottoms should be thoroughly moisture conditioned to achieve a moisture content of 2 to 4 percent above the optimum moisture content, extending to a depth of 12 inches below the overexcavation subgrade. The previously excavated soils may then be replaced as compacted structural fill.

<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 2 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pads. Any undocumented fill soils within any of these 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 areas. The previously excavated soils may then be replaced as compacted structural fill. Please note that if the lateral and/or vertical extents of overexcavation are not achievable for the project retaining walls or site walls, 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: Parking and Drive Areas

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

Subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum, 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 parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within the proposed parking areas. The grading recommendations presented above do not completely mitigate the extent of existing undocumented fill soils and variable strength alluvium in the parking and drive areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking and drive areas should be overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill.

Treatment of Existing Soils: Flatwork Areas

The proposed development is expected to include numerous areas of Portland cement concrete flatwork. Based on conditions encountered at the boring locations, these areas of flatwork will be underlain by very low to medium to high expansive soils. The presence of medium expansive soils poses a significant risk of heave and damage to new flatwork, which will be relatively lightly loaded. In order to reduce the movement of the exterior flatwork areas to tolerable levels, it is recommended that the flatwork be underlain by a newly placed layer of very low to non-expansive (EI < 20) structural fill at least 1 foot in thickness. However, if it is desired to provide even greater protection against heaving of such flatwork, a thicker layer of non-expansive soils could be used.

In some cases, where slab performance is considered critical, non-expansive structural fill layers up to 3 feet in thickness have been utilized on similar projects.

Based on conditions encountered at the boring locations, most of the near surface soils at this site consist of very low expansive sands and silty sands. Therefore, the material used for the layer of very low expansive structural fill beneath the flatwork can be obtained from on-site soils.

It should be noted that elimination of the new layer of very low to non-expansive structural fill is a potential option. However, this option should only be utilized if the owner understands and accepts the risk of future distress to concrete walkways and paving, and that the associated maintenance is acceptable. If the layer of very low to non-expansive structural fill is eliminated, it is recommended that the flatwork thickness be increased and additional reinforcement be provided.

Whether the layer of very low to non-expansive fill is used or not, the geotechnical engineer should evaluate the subgrade materials after any necessary cuts have been made. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2016 CBC and the grading code of the city of Norco.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

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

Utility Trench Backfill

In general, all utility trench backfill 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 Norco. 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 silty sands, clayey sands, silty clays and clayey silts. Based on their composition, minor to moderate caving of shallow excavations may occur. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, temporary excavations should be laid back at a slope no steeper than 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.

Moisture Sensitive Subgrade Soils

The near surface soils include appreciable silt and clay content that may become unstable if exposed to significant moisture infiltration or disturbance by construction traffic. If grading occurs during a period of relatively wet weather, an increase in subgrade instability should also be expected. The site should, therefore, be graded to prevent ponding of surface water and to prevent water from running into excavations.

As discussed in Section 6.3 of this report, unstable subgrade soils may be encountered at the base of the overexcavations within the proposed building areas. The extent of unstable subgrade soils will to a large degree depend on methods used by the contractor to avoid adding additional moisture to these soils or disturbing soils which already possess high moisture contents. If grading occurs during a period of relatively wet weather, an increase in subgrade instability should also be expected. Due to the potential for subgrade instability, it is recommended that only tracked vehicles be utilized for grading or construction activities that require traffic over the exposed subgrade soils.

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 areas as well as the need for and/or the thickness of the crushed stone stabilization layer, discussed in Section 6.3 of this report.

Expansive Soils

The near surface on-site soils have been determined to possess very low to medium expansion potentials. Therefore, care should be given to proper moisture conditioning of all building pad subgrade soils to a moisture content of 2 to 4 percent above the Modified Proctor optimum during site grading. All imported fill soils should have low expansive (EI < 50) characteristics. **In addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintain moisture content of these soils at 2 to 4 percent above the Modified Proctor optimum. This will require the contractor to frequently moisture condition these soils throughout the grading process, unless grading occurs during a period of relatively wet weather**.

Due to the presence of expansive soils at this site, provisions should be made to limit the potential for surface water to penetrate the soils immediately adjacent to the structures. These provisions should include directing surface runoff into rain gutters and area drains, reducing the extent of landscaped areas around the structure, and sloping the ground surface away from the residence. Where possible, it is recommended that landscaped planters not be located immediately adjacent to the building. If landscaped planters around the building are necessary, it is recommended that drought tolerant plants or a drip irrigation system be utilized, to minimize the potential for deep moisture penetration around the structures. Presented below is a list of additional soil moisture control recommendations that should be considered by the owner, developer, and civil engineer:

- Ponding and areas of low flow gradients in unpaved walkways, grass and planter areas should be avoided. In general, minimum drainage gradients of 2 percent should be maintained in unpaved areas.
- Bare soil within five feet of proposed structures should be sloped at a minimum 5 percent gradient away from the structure (about three inches of fall in five feet), or the same area could be paved with a minimum surface gradient of one percent. Pavement is preferable.
- Decorative gravel ground cover tends to provide a reservoir for surface water and may hide areas of ponding or poor drainage. Decorative gravel is, therefore, not recommended and should not be utilized for landscaping unless equipped with a subsurface drainage system designed by a licensed landscape architect.
- Positive drainage devices, such as graded swales, paved ditches, and catch basins should be installed at appropriate locations within the area of the proposed development.
- Concrete walks and flatwork should not obstruct the free flow of surface water to the appropriate drainage devices.
- Area drains should be recessed below grade to allow free flow of water into the drain. Concrete or brick flatwork joints should be sealed with mortar or flexible mastic.
- Gutter and downspout systems should be installed to capture all discharge from roof areas. Downspouts should discharge directly into a pipe or paved surface system to be conveyed offsite.
- Enclosed planters adjoining, or in close proximity to proposed structures, should be sealed at the bottom and provided with subsurface collection systems and outlet pipes.
- Depressed planters should be raised with soil to promote runoff (minimum drainage gradient two percent or five percent, see above), and/or equipped with area drains to eliminate ponding.
- Drainage outfall locations should be selected to avoid erosion of slopes and/or properly armored to prevent erosion of graded surfaces. No drainage should be directed over or towards adjoining slopes.
- All drainage devices should be maintained on a regular basis, including frequent observations during the rainy season to keep the drains free of leaves, soil and other debris.

- Landscape irrigation should conform to the recommendations of the landscape architect and should be performed judiciously to preclude either soaking or excessive drying of the foundation soils. This should entail regular watering during the drier portions of the year and little or no irrigation during the rainy season. Automatic sprinkler systems should, therefore, be switched to manual operation during the rainy season. Good irrigation practice typically requires frequent application of limited quantities of water that are sufficient to sustain plant growth, but do not excessively wet the soils. Ponding and/or run-off of irrigation water are indications of excessive watering.

Other provisions, as determined by the landscape architect or civil engineer, may also be appropriate.

Groundwater

The static groundwater table at this site is considered to be present at a depths between 29 and 41± feet. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pads 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 2 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 structures may be supported on conventional shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom) 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 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

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

Estimated Foundation Settlements

Post-construction total and differential 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 slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 300 lbs/ft^3
- Friction Coefficient: 0.28

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 2500 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support the new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, and based on the design considerations presented in Section 6.1 of this report, the floors of the proposed structures may be constructed as conventional slabs-on-grade supported on newly placed structural fill, extending to a depth of at least 4 feet below proposed finished pad grade. Based on geotechnical considerations, the floor slabs 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 = 100 psi/in.

- Minimum slab reinforcement: Minimum slab reinforcement: No. 3 bars at 16 inches on-center, in both directions, due to the presence of low to medium expansive soils and 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 2 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

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

The actual design of the floor slabs should be completed by the structural engineer to verify adequate thickness and reinforcement. The steel reinforcement recommendations presented above are based on standard geotechnical practice, given the magnitude of predicted liquefaction-induced settlements, and the structure type proposed for the site. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements discussed in Section 6.1.

6.7 Retaining Wall Design and Construction

New retaining walls are expected to be necessary in the dock-high areas of the buildings Additionally, although not indicated on the site plan, the proposed development may require some small retaining walls (less than 5± feet in height) to facilitate the new site grades.

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 silty sands, clayey sands and sandy silts, as well as imported select granular material. Based on laboratory testing, the on-site soils possess ultimate friction angles of 28 to 31 degrees when compacted to 90 percent of the ASTM D-1557 maximum dry density. To account for variations in the quality of the on-site soils, the following retaining wall design parameters are based on a friction angle of 28 degrees. It is recommended that silty clays, sandy clays, and clayey silts, be excluded from retaining wall backfills since some these materials possess medium expansion potentials.

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

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type: On-Site Sands and Silts
Internal Friction Angle (ϕ)		28°
Unit Weight		130 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	47 lbs/ft^3
	Active Condition (2h:1v backfill)	82 lbs/ft^3
	At-Rest Condition (level backfill)	69 lbs/ft^3

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 accordance with the 2016 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill. 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 soils may be used to backfill the retaining walls. However, 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 properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls be used. If the drainage composite 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 drainage composite 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 one cubic foot gravel pocket surrounded by a suitable geotextile at each weep hole location.

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

6.8 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be supported on the existing fill and/or native soils. These materials generally consist of silty sands, sandy silts, sandy clays and occasional clayey silts. These materials are expected to exhibit fair to good pavement support characteristics, with estimated R-values of 25 to 40. Since R-value testing was not included in the scope of services for this feasibility study, the subsequent pavement design is based upon an assumed R-value of 25. 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 may be desirable to perform R-value testing after the completion of rough grading to verify the R-value of the as-graded parking subgrade.

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

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 = 25)					
Materials	**Thickness (inches)**				
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic (TI = 6.0)	Truck Traffic (TI = 7.0)	Truck Traffic (TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	4	7	9	11	12
Compacted Subgrade (90% minimum compaction)	12	12	12	12	12

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

Portland Cement Concrete

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

PORTLAND CEMENT CONCRETE PAVEMENTS				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI =6.0)	(TI = 7.0)	(TI = 8.0)
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. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness.

7.0 GENERAL COMMENTS

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

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

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

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

8.0 REFERENCES

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

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

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

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

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

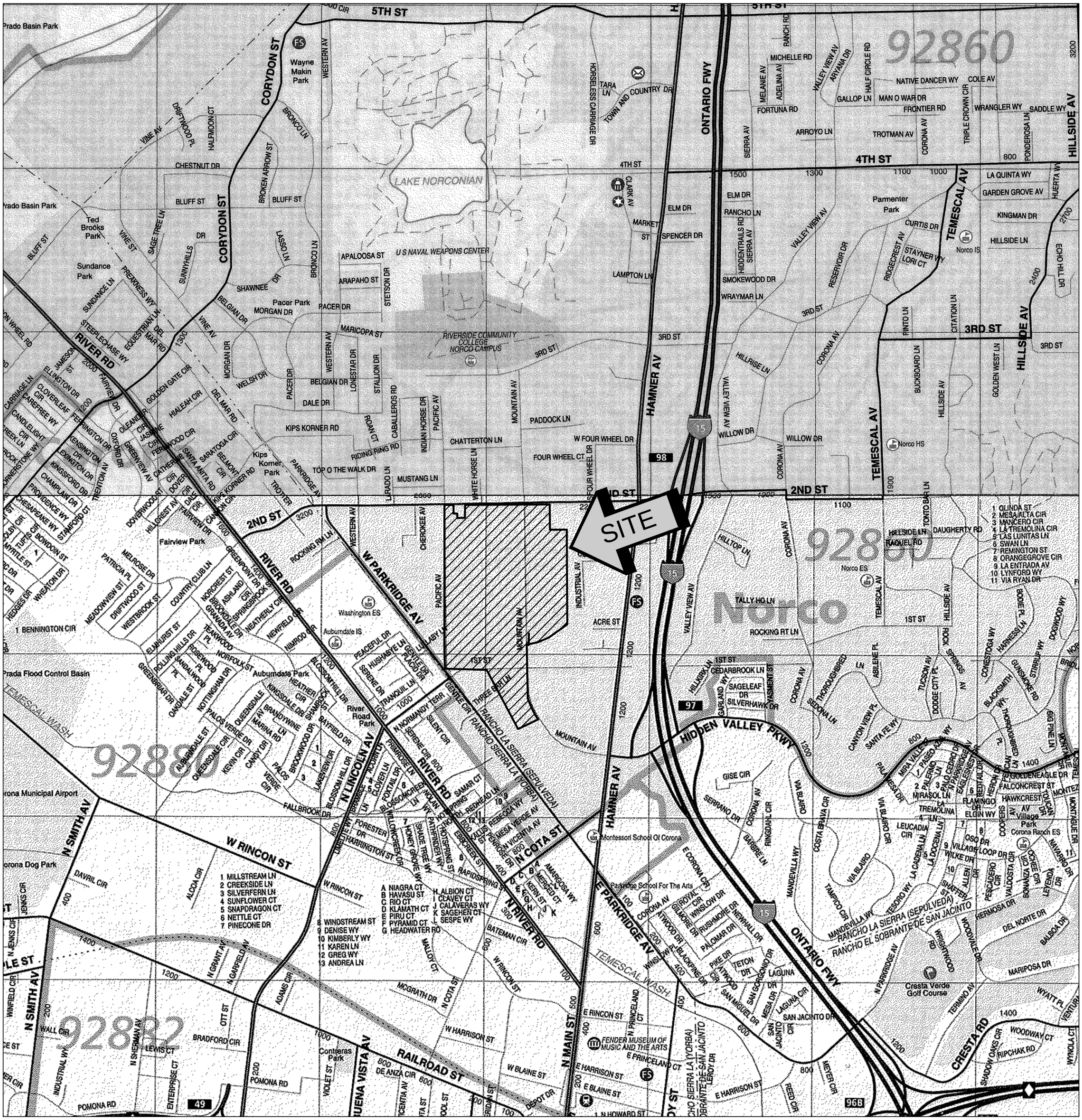
Southern California Earthquake Center (SCEC), University of Southern California, "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California," Committee formed 1997.

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

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

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

APPENDIX A



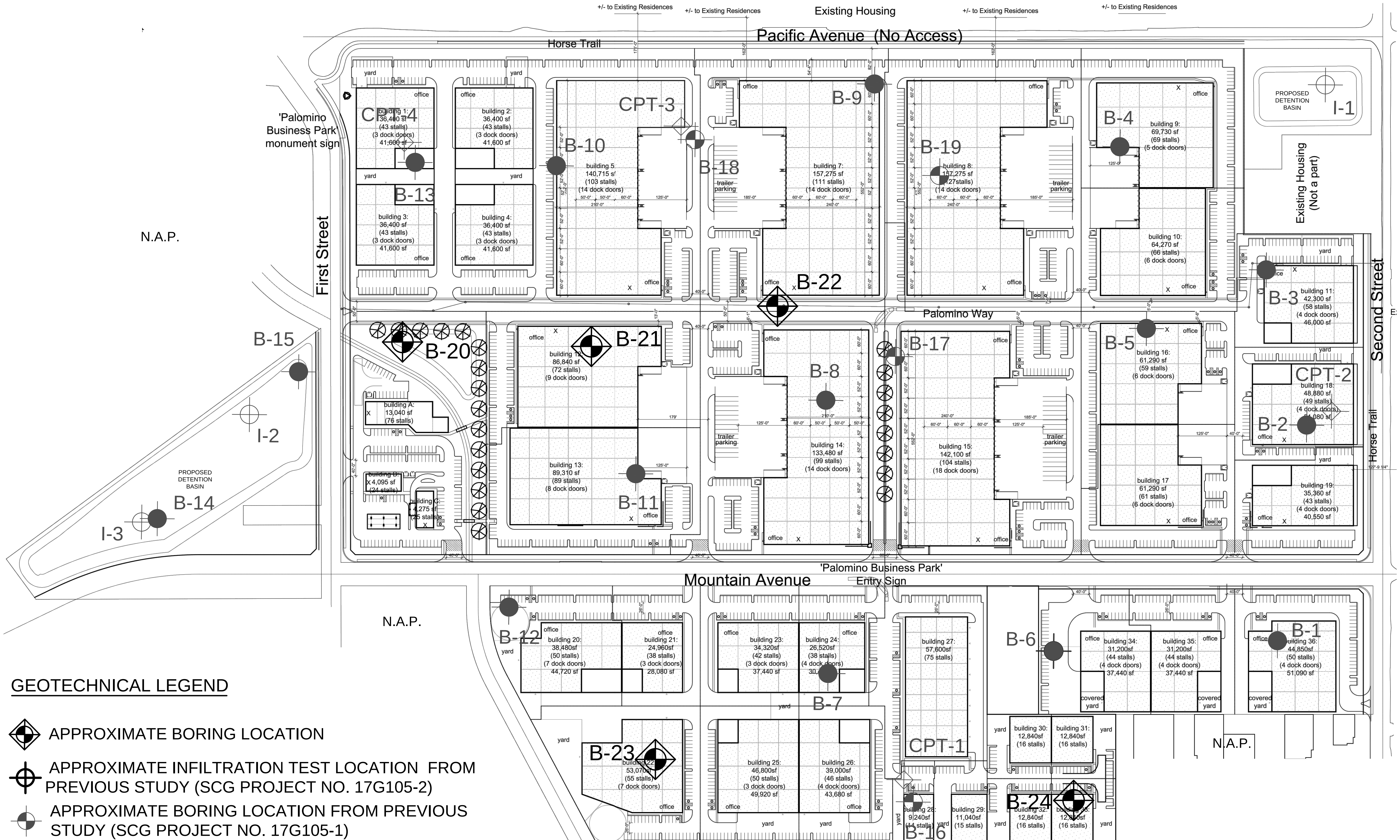
SOURCE: RIVERSIDE COUNTY
THOMAS GUIDE, 2013

SITE LOCATION MAP

PROPOSED PALOMINO BUSINESS PARK

NORCO, CALIFORNIA

SCALE: 1" = 2400'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JLL CHKD: RGT	
SCG PROJECT 17G105-3	
PLATE 1	



GEOTECHNICAL LEGEND

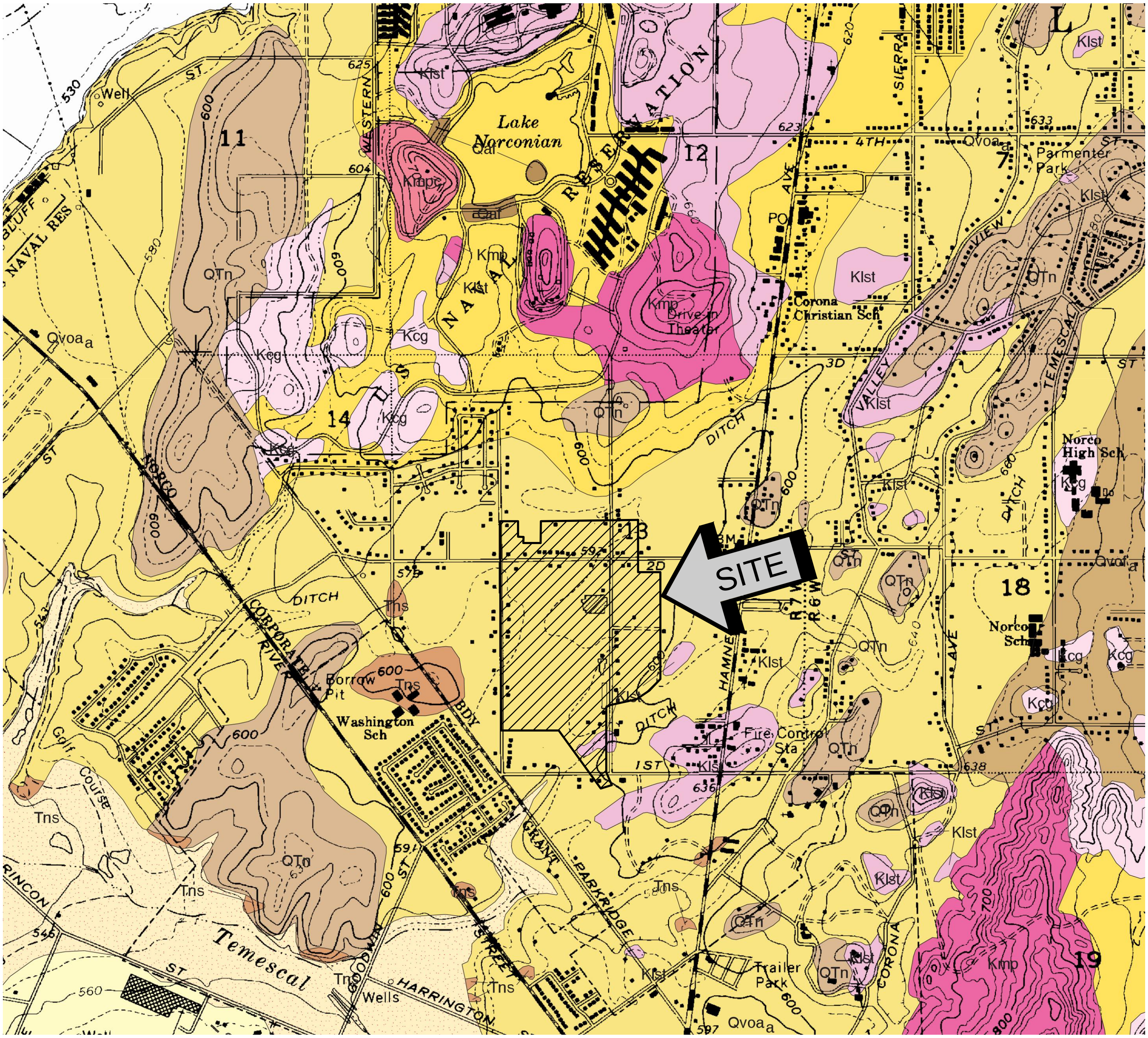
- APPROXIMATE BORING LOCATION
- APPROXIMATE INFILTRATION TEST LOCATION FROM PREVIOUS STUDY (SCG PROJECT NO. 17G105-2)
- APPROXIMATE BORING LOCATION FROM PREVIOUS STUDY (SCG PROJECT NO. 17G105-1)
- APPROXIMATE CPT LOCATION FROM PREVIOUS STUDY (SCG PROJECT NO. 17G105-1)
- APPROXIMATE BORING LOCATION FROM PREVIOUS STUDIES (SCG PROJECT NOS. 11G114-1 AND 11G114-3)

NOTE: SITE PLAN PREPARED BY CARLILE COATSWORTH ARCHITECTS, INC.

BORING AND CPT LOCATION PLAN
PROPOSED PALOMINO BUSINESS PARK
NORCO, CALIFORNIA

SCALE: 1" = 250'
DRAWN: JLL
CHKD: RGT
SCG PROJECT 17G105-3
PLATE 2

SOUTHERN CALIFORNIA GEOTECHNICAL



DESCRIPTION OF MAP UNITS

Qvof

Very old alluvial fan deposits (early Pleistocene)—Mostly well-dissected, well-indurated, reddish-brown sand deposits. Commonly contains duripans and locally silcretes. Forms large area east of Norco generally flanking steep bedrock slopes. North of Santa Ana River, forms broad area that is part of fans emanating from Puente Hills west of quadrangle

Qvoa

Very old alluvial channel deposits (early Pleistocene)—Gravel, sand, and silt; reddish-brown, well-indurated, surfaces well-dissected. Underlies large area between Santa Ana River and Temescal Wash

QTn

Late Cenozoic sedimentary rocks in Norco area (early Pleistocene to late Pliocene?)—Moderately indurated sandstone, conglomeratic sandstone, and conglomerate. In Norco area, unit includes locally derived clasts as well as clasts derived from San Bernardino Mountains. Found in and west of Norco, on both sides of Santa Ana River

Klst

La Sierra Tonalite (Cretaceous)—Massive biotite tonalite. Fairly dark-colored compared to other units in region containing no hornblende, but alteration found in much of rock tends to darken it. Medium- to coarse-grained; structureless. Much of tonalite is altered to secondary minerals, especially epidote and chlorite, and contains localized zones that are thoroughly altered to epidote, quartz, and chlorite; some highly altered rocks contain tourmaline and sulfide minerals. Large body exposed west of La Sierra and larger mass, partly covered by Quaternary deposits, underlies Norco area. Named by Larsen (1948) for exposures in vicinity of La Sierra.

SOURCE: "GEOLOGIC MAP OF THE CORONA NORTH 7.5' QUADRANGLE, RIVERSIDE AND SAN BERNARDINO COUNTIES, CALIFORNIA" MORTON AND GRAY, JR., 2002

GEOLOGIC MAP

PROPOSED PALOMINO BUSINESS PARK

NORCO, CALIFORNIA

SCALE: 1" = 2000'

DRAWN: JLL
CHKD: RGT

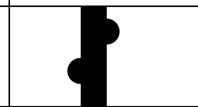
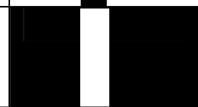
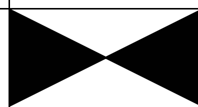
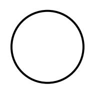
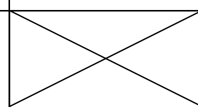
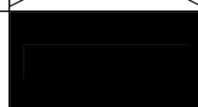
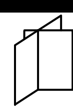
SCG PROJECT
17G105-3

PLATE 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

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

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

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

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

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

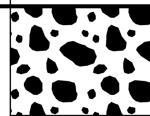
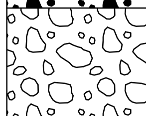
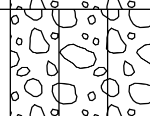
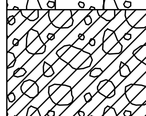
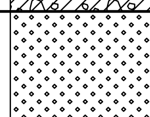
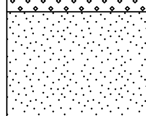
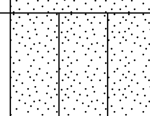
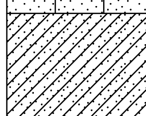
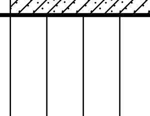
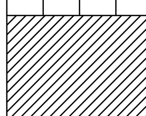
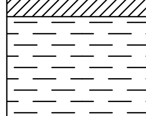
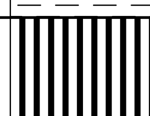
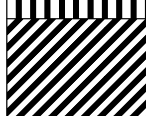
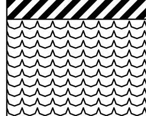
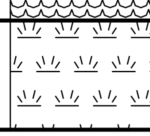
LIQUID LIMIT: The moisture content above which a soil behaves as a liquid.

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

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

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

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	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
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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-20

JOB NO.: 17G105-3
PROJECT: Proposed Palomino Business Park
LOCATION: Norco, California

DRILLING DATE: 6/18/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 16 feet
READING TAKEN: At Completion

FIELD RESULTS / LABORATORY RESULTS

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
					ALLUVIUM: Brown to Dark Brown Silty fine Sand, trace to little medium Sand, loose to medium dense-moist							
	X	16				111	7					
	X	9				111	9					
5	X	8				109	9					
	X	8				106	7					
					@ 8 to 12 feet, some medium Sand, trace coarse Sand							
	X	7				102	8					
10												
					Brown to Gray Brown fine to coarse Sand, trace fine Gravel, loose-damp							
	X	12				102	6					
15												
					Red Brown to Brown Clayey fine to coarse sand, dense-damp							
	X	49				119	6					
20												

Boring Terminated at 20'

TBL 17G105-3.GPJ SOCALGEO.GDT 7/9/19

BORING NO. B-21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 17G105-3
PROJECT: Proposed Palomino Business Park
LOCATION: Norco, California

DRILLING DATE: 6/18/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 15 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
					ALLUVIUM: Light Brown Silt, trace medium Sand, little Calcareous veining, medium dense-moist							
		17				106	9					
					Light Brown to Gray Brown Silty fine Sand to fine Sandy Silt, loose to medium dense-damp to moist							
		14				98	7					
5		27			@ 5 to 6 feet, slightly porous, trace Iron Oxide staining	94	7					
		28				106	6					
					Gray Brown fine Sand, little Silt, trace Iron Oxide staining, medium dense-damp							
		18				95	5					
10												
					Light Gray Brown to Gray Brown fine to coarse Sand, trace to little fine to coarse Gravel, medium dense-dry							
		17					2					
15												
					Light Gray Brown fine Sand, trace Iron Oxide staining, medium dense-damp							
		24					4					
20												
					Boring Terminated at 20'							

TBL 17G105-3.GPJ SOCALGEO.GDT 7/9/19

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-22

JOB NO.: 17G105-3
PROJECT: Proposed Palomino Business Park
LOCATION: Norco, California

DRILLING DATE: 6/18/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 14 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
					ALLUVIUM: Light Gray Brown to Brown Silt, trace fine Sand, trace to little fine Sand, trace Clay, some Calcareous nodules, loose to medium dense-very moist							
	X	6					14					
	X	10					19					
5					Gray Brown to Brown fine Sandy Silt, little Iron Oxide staining, medium dense-very moist		15					
	X	13			Light Gray Brown fine Sand, trace medium Sand, trace Silt, trace Iron Oxide staining, medium dense-damp		5					
	X	15			Light Gray to Light Gray Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, medium dense-dry		2					
10												
	X	24			@ 13½ to 20 feet, little fine to coarse Sand		1					
15												
	X	25					2					
20												
					Boring Terminated at 20'							

TBL 17G105-3.GPJ SOCALGEO.GDT 7/9/19

BORING NO.
B-23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 17G105-3
PROJECT: Proposed Palomino Business Park
LOCATION: Norco, California

DRILLING DATE: 6/18/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 16 feet
READING TAKEN: At Completion

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
					ALLUVIUM: Red Brown to Brown fine to medium Sandy Clay, stiff-very moist							
	X	15	4.5			116	14					EI = 37 @ 0 to 5'
					Brown Clayey fine Sand, trace Silt, medium dense-very moist							
	X	17				109	13					
5					Brown fine Sand, little Silt, trace medium Sand, loose-moist							
	X	12				105	8					
					Brown Silty fine Sand, trace to little medium to coarse Sand, trace Clay, slightly porous, medium dense-moist							
	X	18				112	12					
					Light Brown to Brown fine to medium Sand, trace coarse Sand, trace to little Silt, medium dense-moist							
	X	19				116	7					
10												
					Brown to Red Brown fine to medium Sandy Clay, hard-very moist							
	X	39					14					
15					Brown to Red Brown Silty fine to coarse sand, trace Clay, dense-moist		9					
					Gray Brown fine to coarse Sand, trace Silt, very dense-damp							
	X	50/3"					6					
20												

Boring Terminated at 20'

TBL 17G105-3.GPJ SOCALGEO.GDT 7/9/19

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-24

JOB NO.: 17G105-3
PROJECT: Proposed Palomino Business Park
LOCATION: Norco, California

DRILLING DATE: 6/18/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

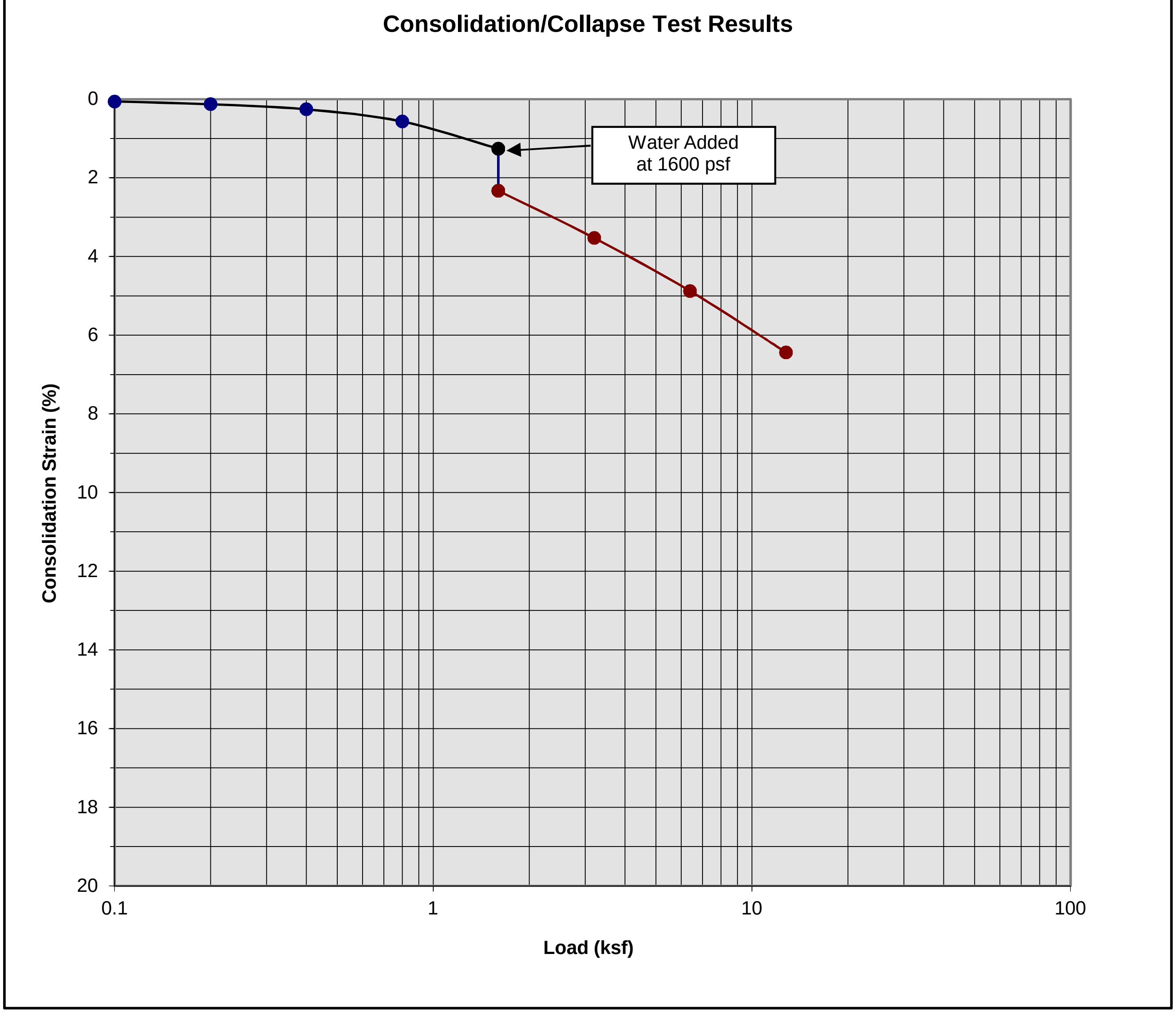
WATER DEPTH: Dry
CAVE DEPTH: 16 feet
READING TAKEN: At Completion

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
					ALLUVIUM: Dark Brown Silty Clay, trace fine Sand, stiff-very moist							
		14	4.5+			107	14					
					Light Brown Silt, little fine Sand, trace Clay, slightly porous, medium dense-damp							
		18				106	7					
5		20				95	7					
					Gray Brown Silty Clay, trace fine Sand, little Calcareous veining, trace Iron Oxide staining, stiff-moist							
		19	4.5+			100	10					
		21	4.5+			98	10					
10												
					Light Brown to Brown fine Sand, little Silt, trace medium Sand, medium dense-damp							
		20					4					
15												
					Gray Brown Silty fine Sand to fine Sandy Silt, trace Iron Oxide staining, medium dense-very moist							
		14					12					
20												
					Boring Terminated at 20'							

TBL 17G105-3.GPJ SOCALGEO.GDT 7/9/19

TEST BORING LOG

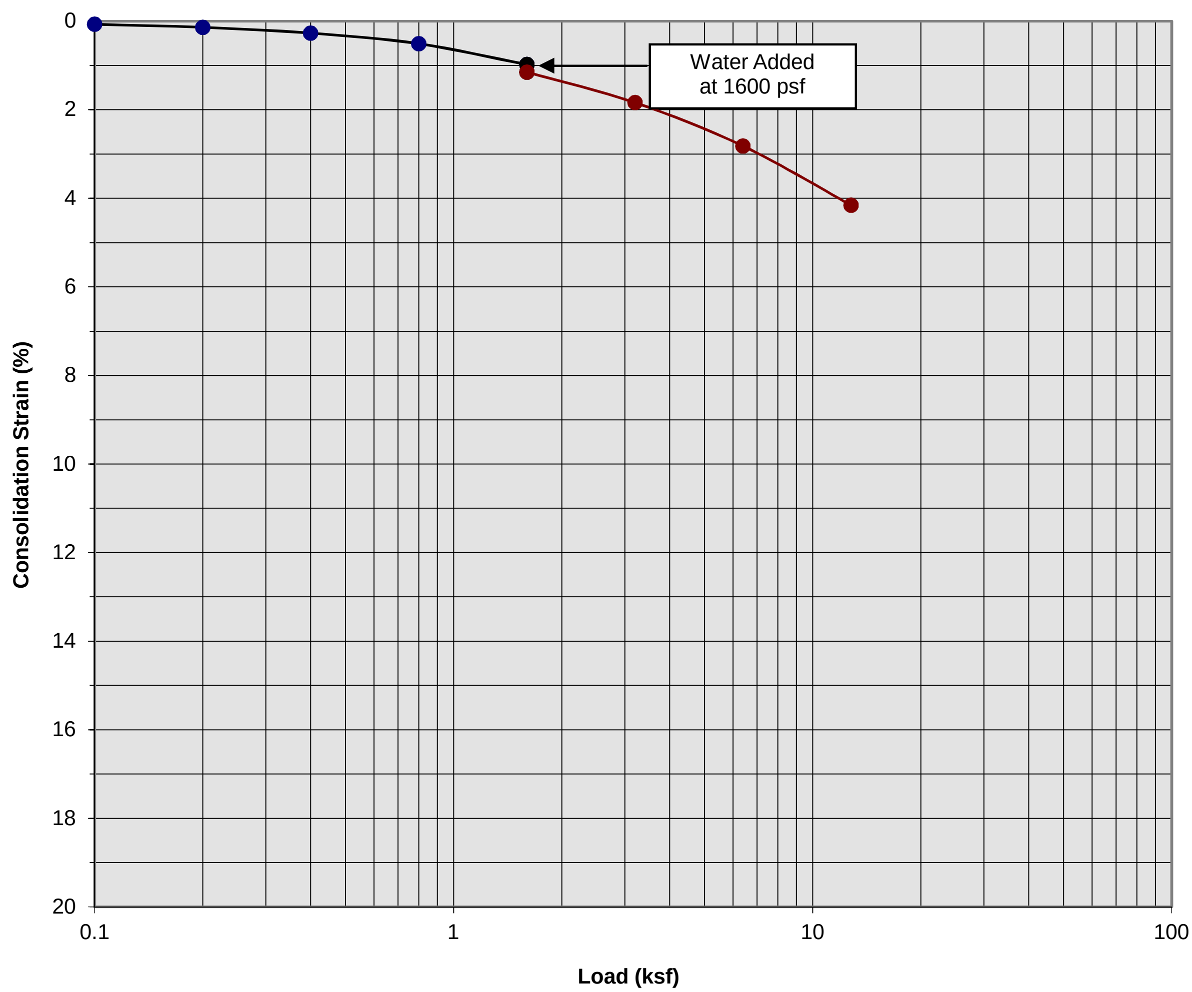
APPENDIX C



Classification: Brown to Dark Brown Silty fine Sand

Boring Number:	B-20	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	5 to 6	Initial Dry Density (pcf)	108.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.07

Consolidation/Collapse Test Results

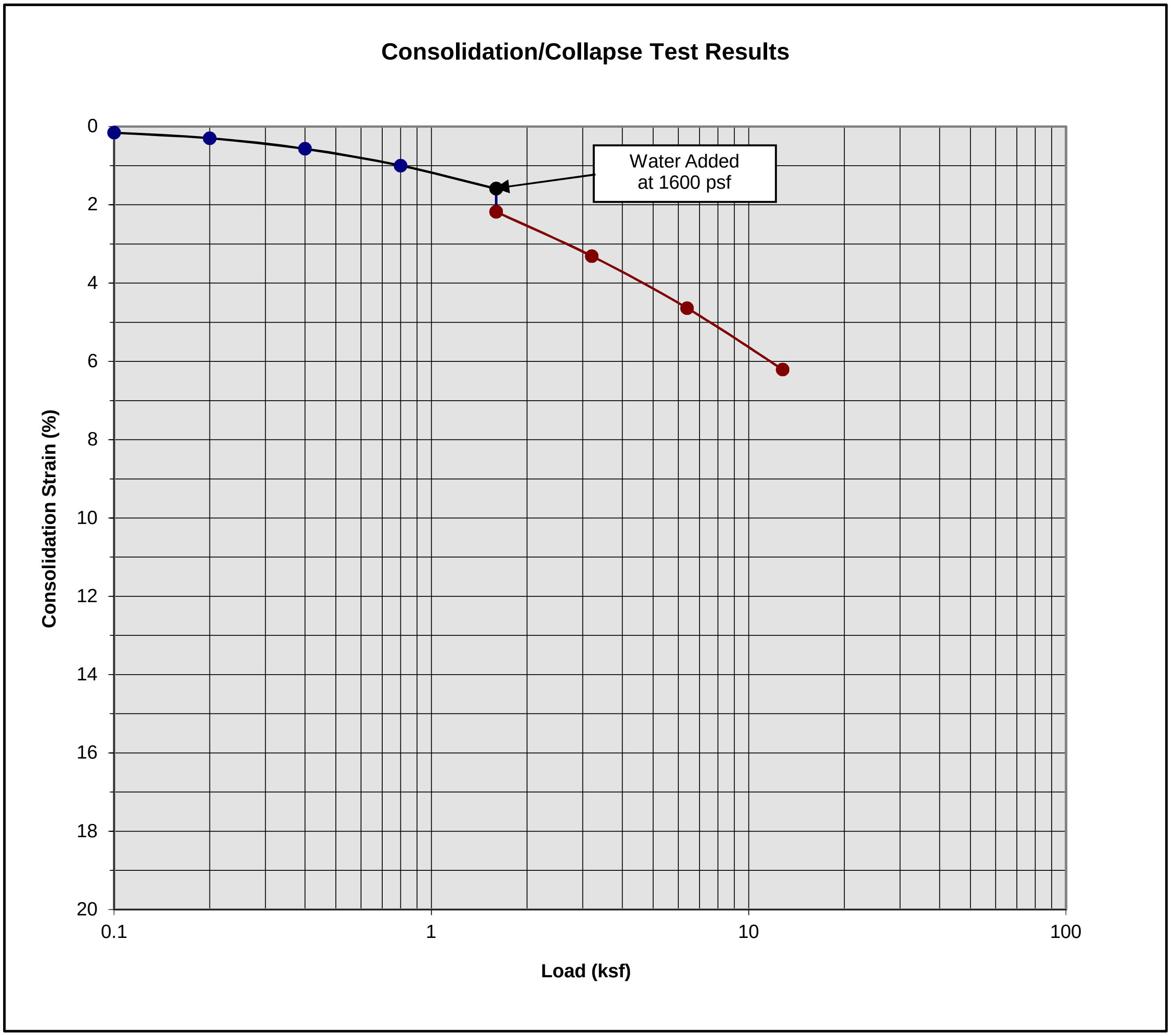


Classification: Brown to Dark Brown Silty fine Sand

Boring Number:	B-20	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	7 to 8	Initial Dry Density (pcf)	105.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.17

Proposed Palomino Business Park
Norco, California
Project No. 17G105-3
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



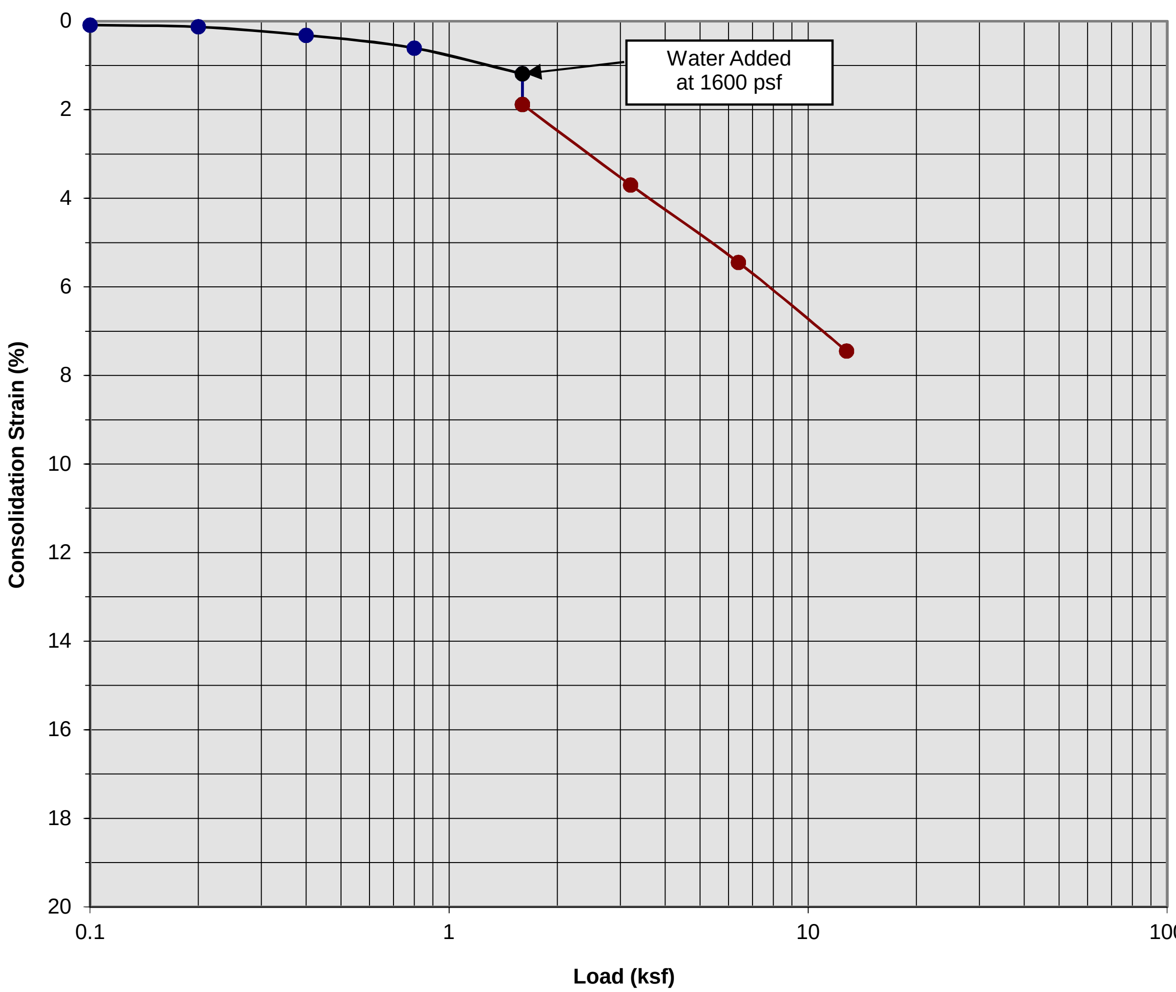
Classification: Brown to Dark Brown Silty fine Sand

Boring Number:	B-20	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	9 to 10	Initial Dry Density (pcf)	101.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.59

Proposed Palomino Business Park
Norco, California
Project No. 17G105-3
PLATE C- 3

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

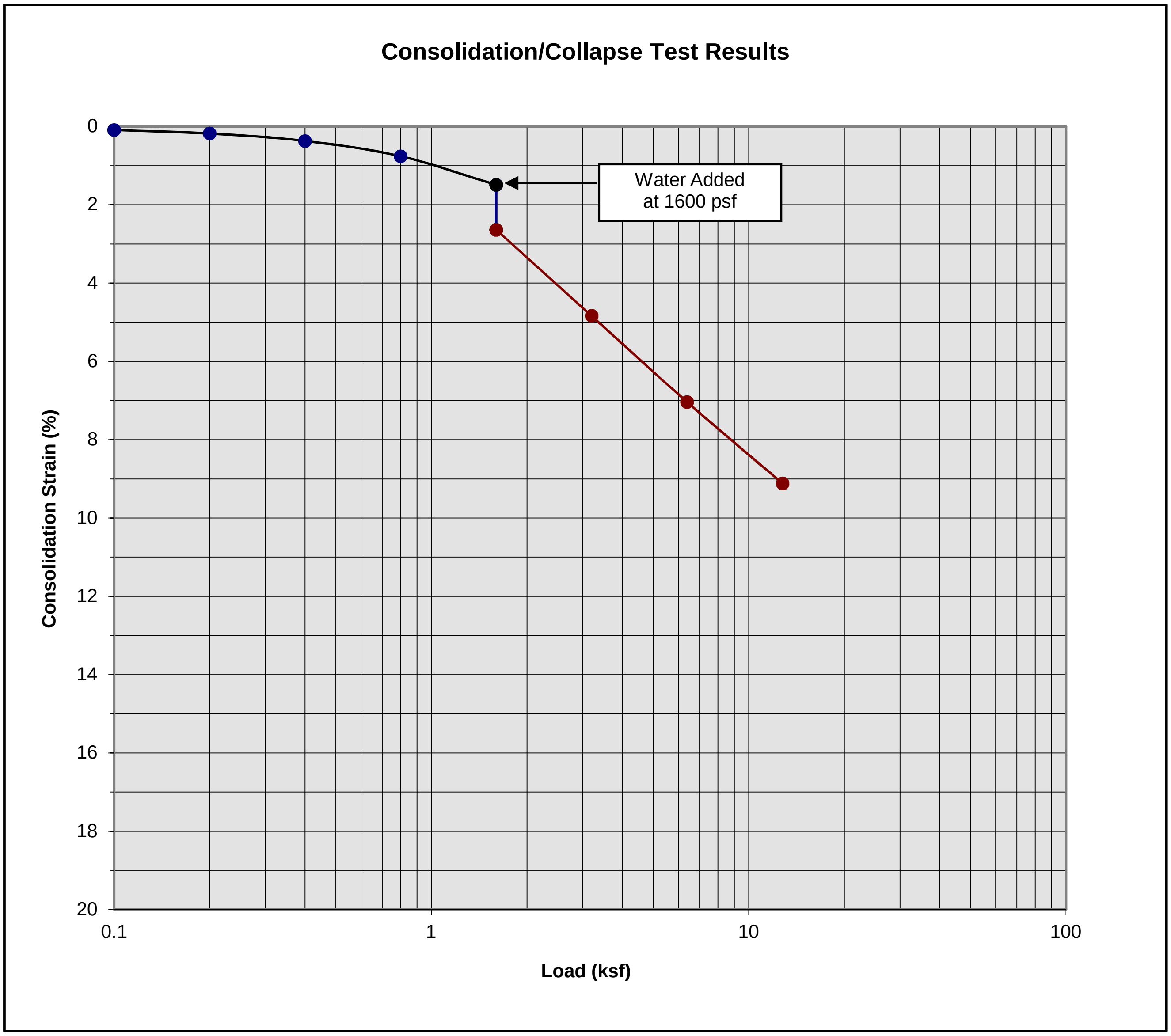


Classification: Brown to Gray Brown fine to coarse Sand

Boring Number:	B-20	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	14 to 15	Initial Dry Density (pcf)	101.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.69

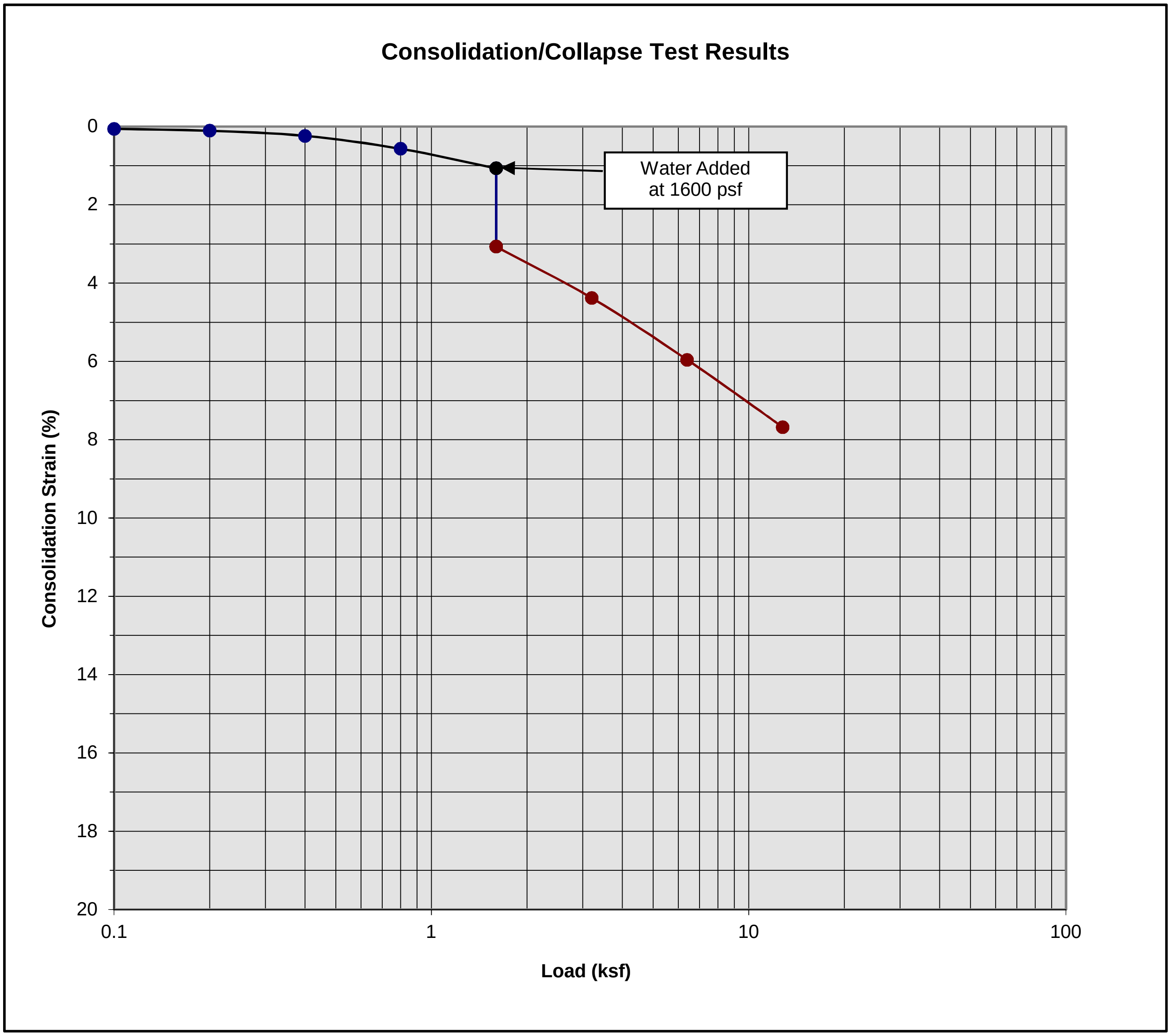
Proposed Palomino Business Park
Norco, California
Project No. 17G105-3
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: Brown Clayey fine Sand, trace Silt

Boring Number:	B-23	Initial Moisture Content (%)	13
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	3 to 4	Initial Dry Density (pcf)	109.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	120.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.15



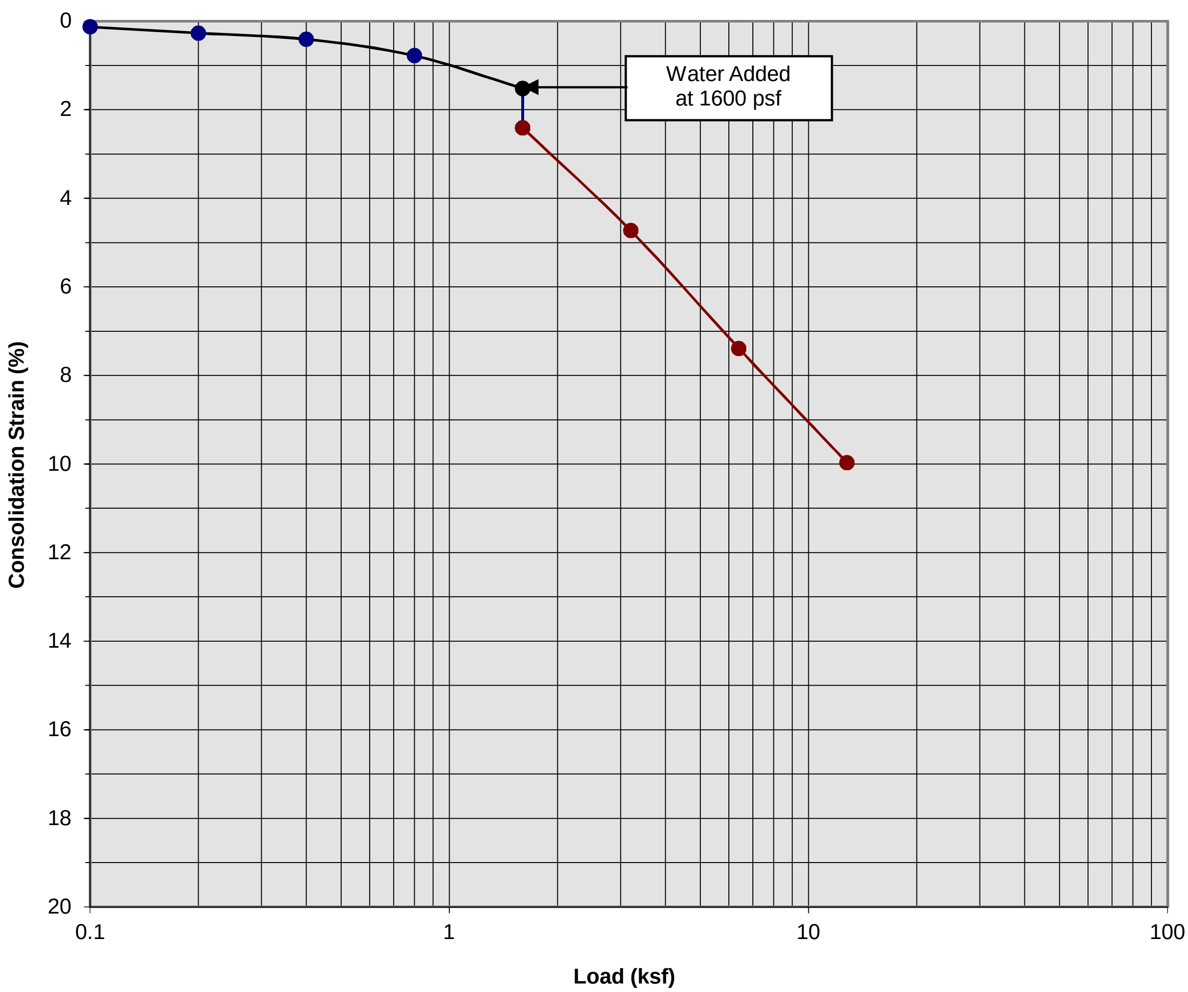
Classification: Brown fine Sand, little Silt

Boring Number:	B-23	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	5 to 6	Initial Dry Density (pcf)	104.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	113.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.00

Proposed Palomino Business Park
Norco, California
Project No. 17G105-3
PLATE C- 6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

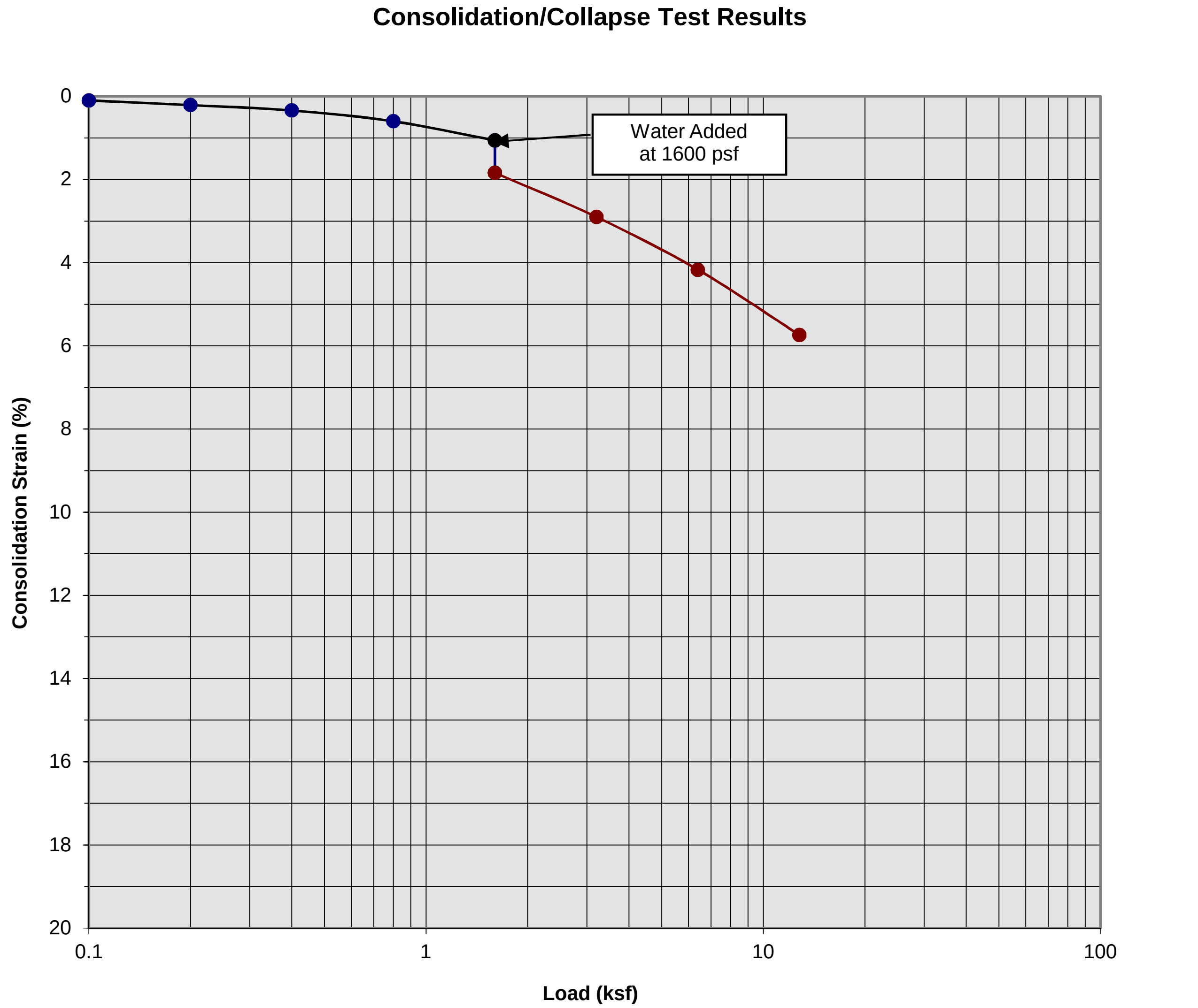


Classification: Brown Silty fine Sand

Boring Number:	B-23	Initial Moisture Content (%)	12
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	7 to 8	Initial Dry Density (pcf)	111.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	123.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.89

Proposed Palomino Business Park
Norco, California
Project No. 17G105-3
PLATE C- 7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

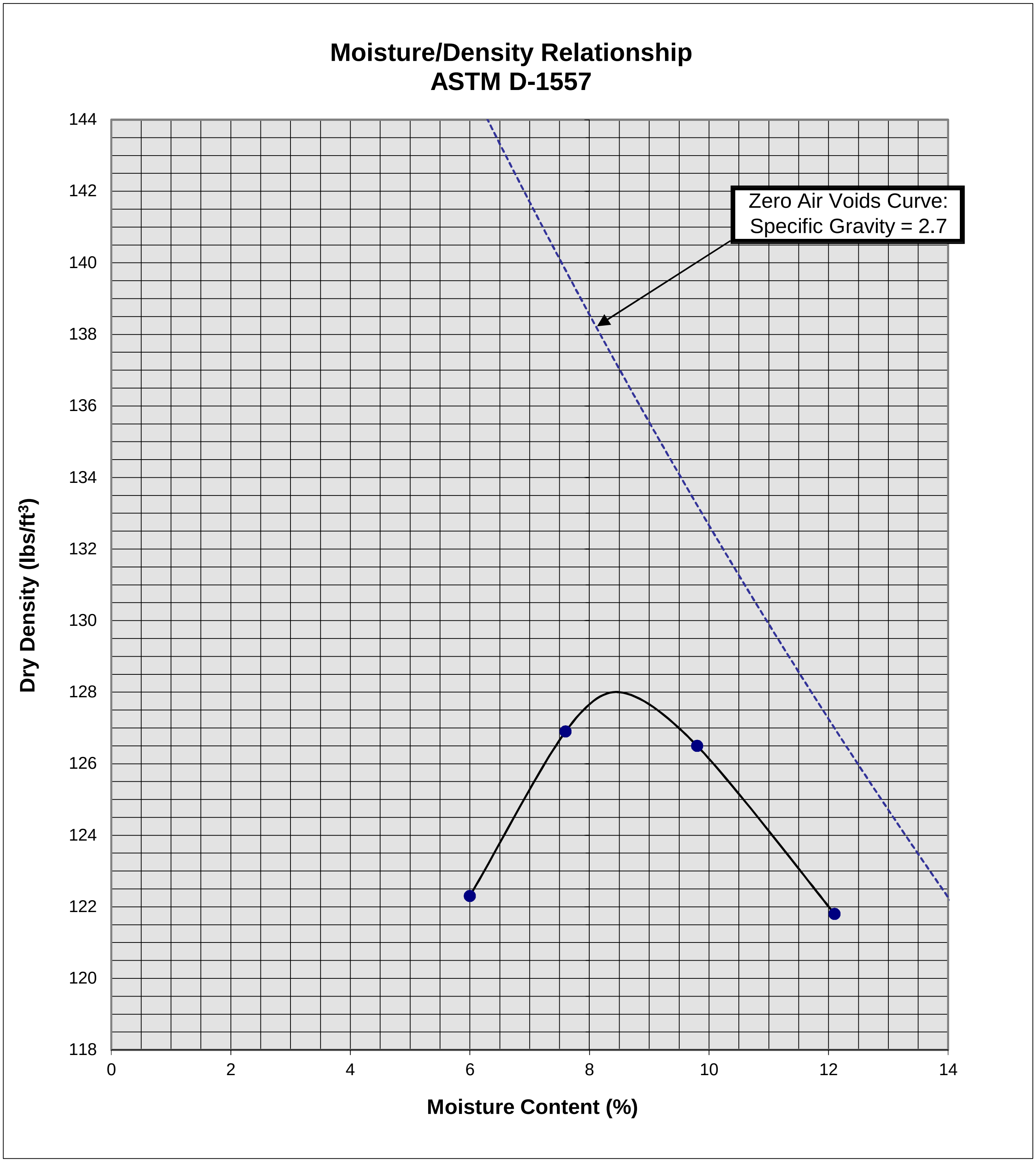


Classification: Light Brown to Brown fine to medium Sand

Boring Number:	B-23	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	9 to 10	Initial Dry Density (pcf)	115.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	122.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.78

Proposed Palomino Business Park
Norco, California
Project No. 17G105-3
PLATE C- 8

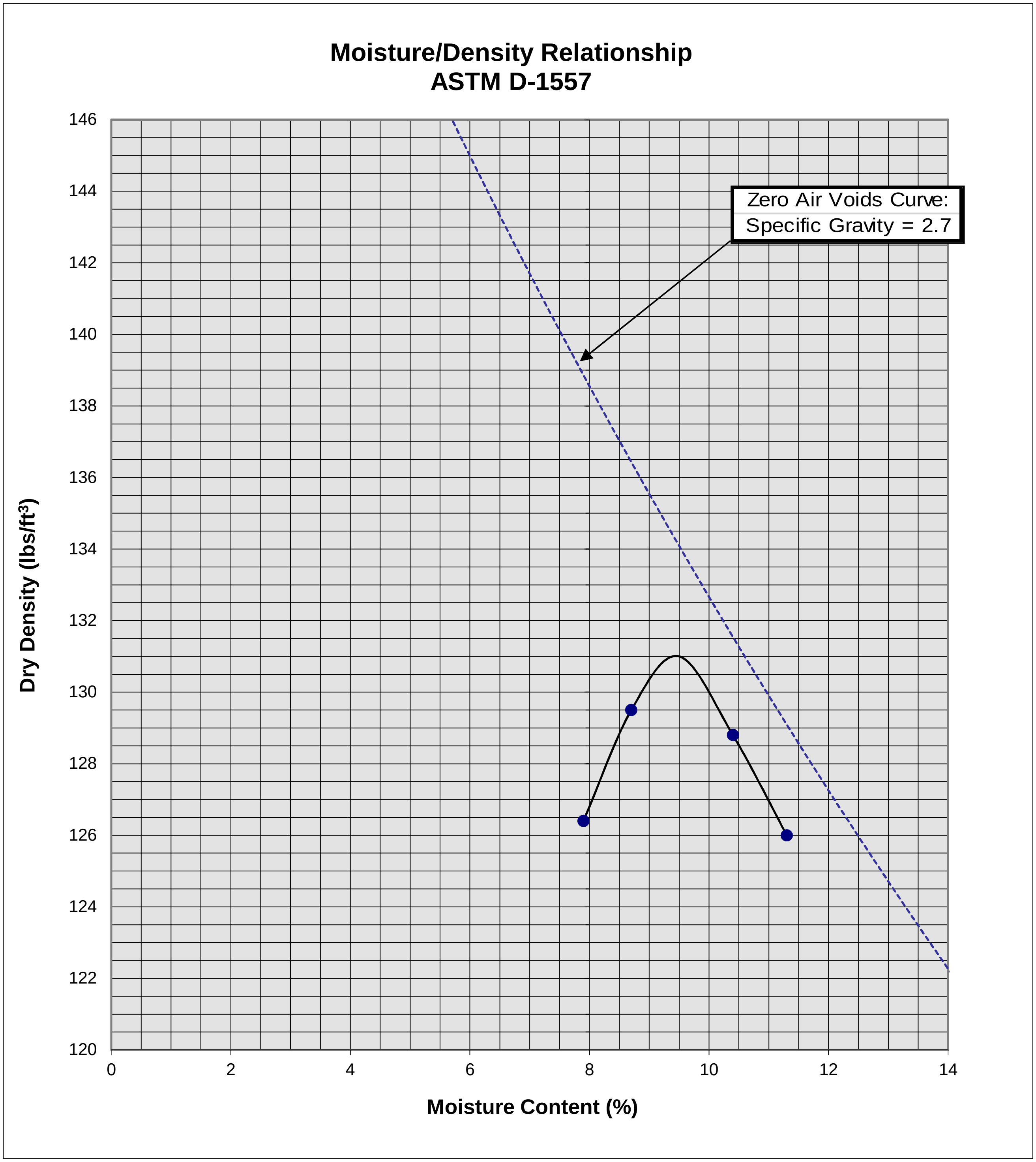
SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-20 @ 0-5'
Optimum Moisture (%)	8.5
Maximum Dry Density (pcf)	128
Soil Classification	Brown Silty fine Sand, trace to little medium Sand

Proposed Palomino Business Park
Norco, California
Project No. 17G105-3
PLATE C-9

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-23 @ 0-5'
Optimum Moisture (%)	9.5
Maximum Dry Density (pcf)	131
Soil Classification	Red Brown to Brown Clayey fine Sand to fine to medium Sandy Clay

Proposed Palomino Business Park
Norco, California
Project No. 17G105-3
PLATE C-10

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.

<u>Compacted Fills</u>

- 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.

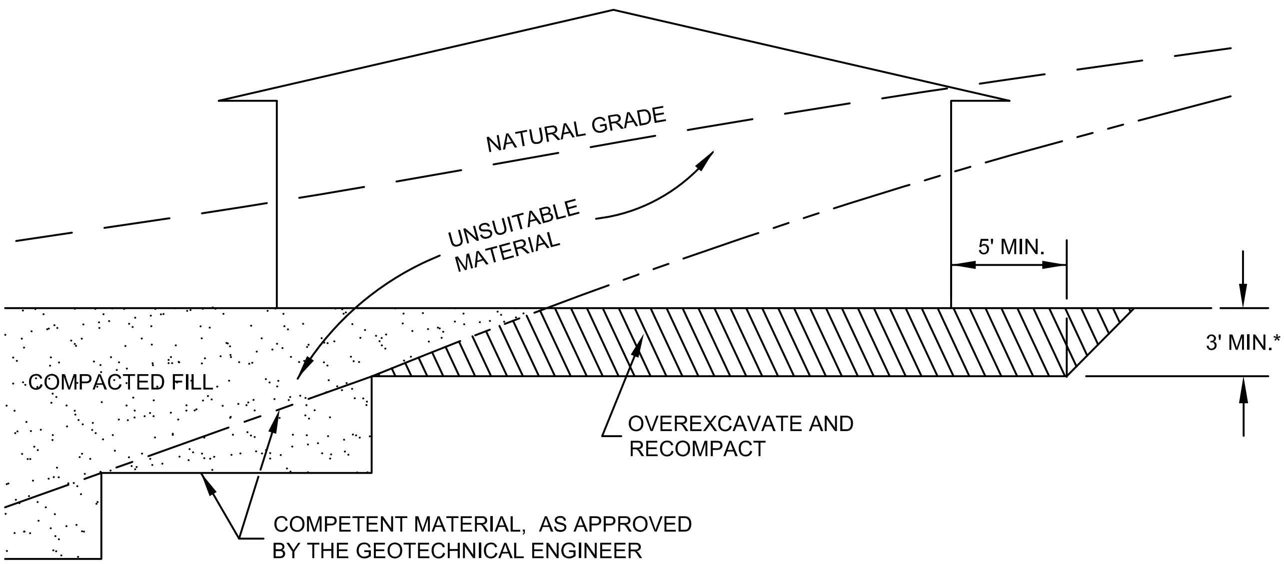
Subdrains

- 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.

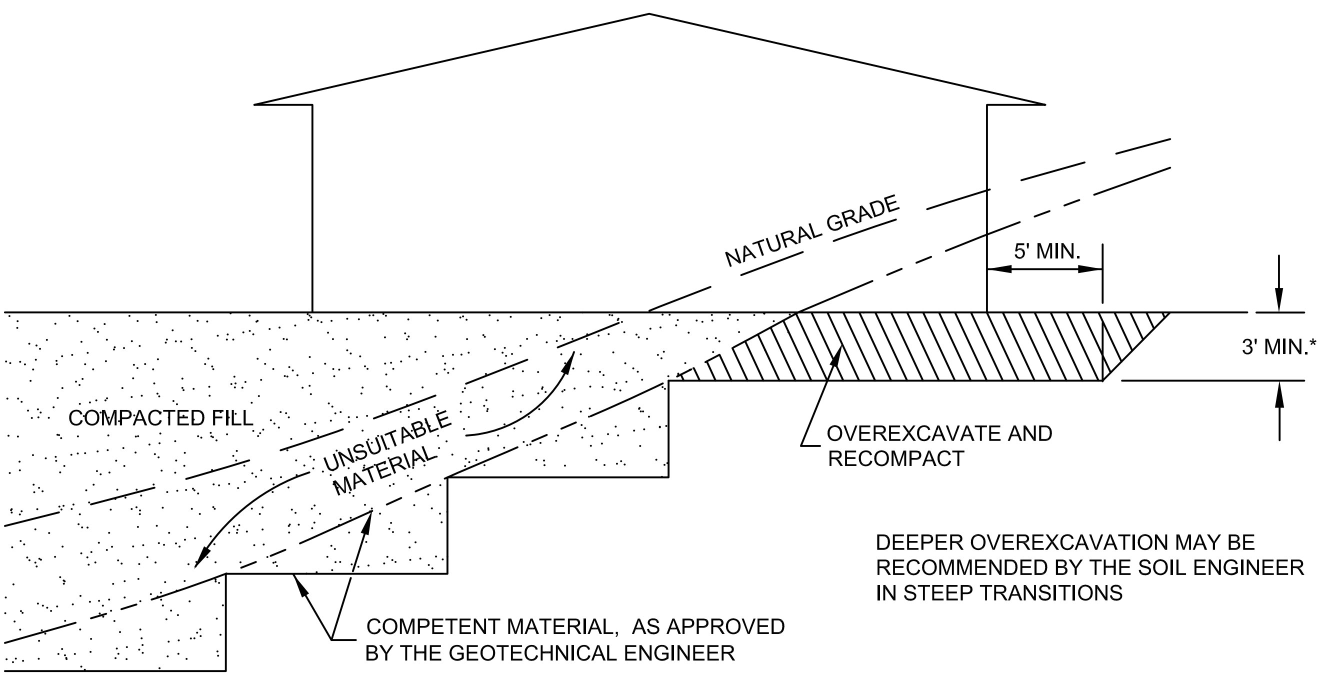
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- 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)



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

TRANSITION LOT DETAIL

GRADING GUIDE SPECIFICATIONS

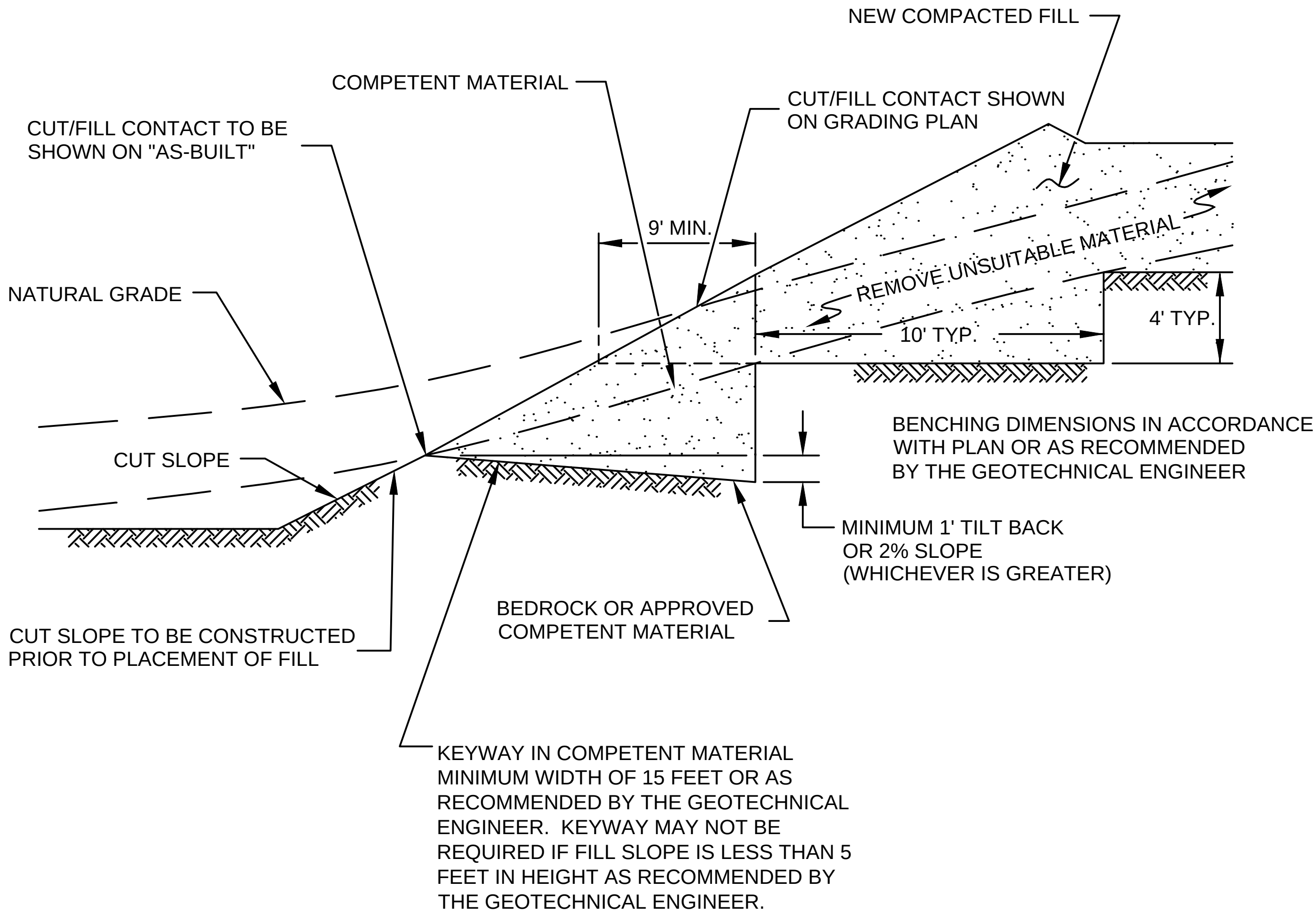
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo

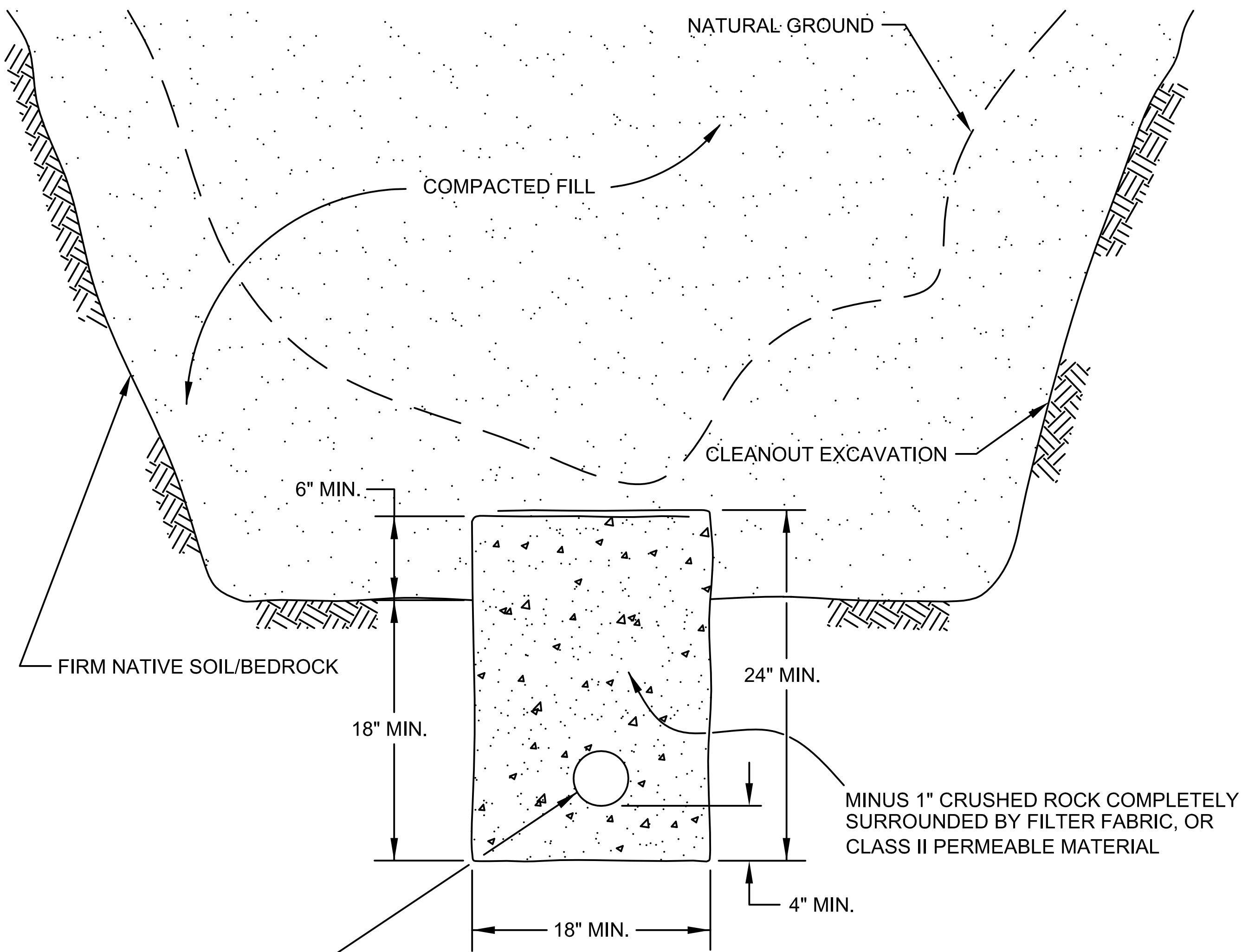
SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE	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

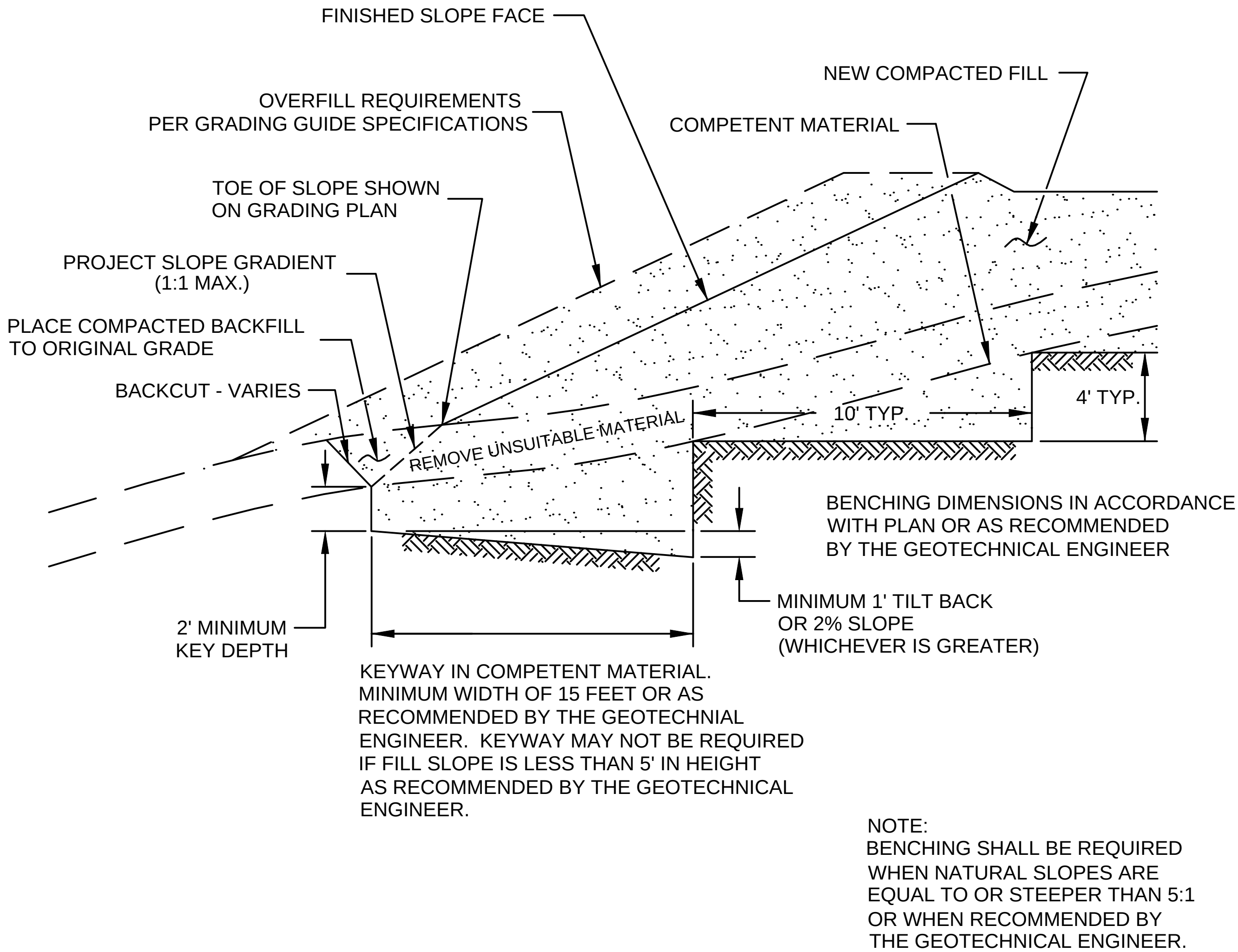
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

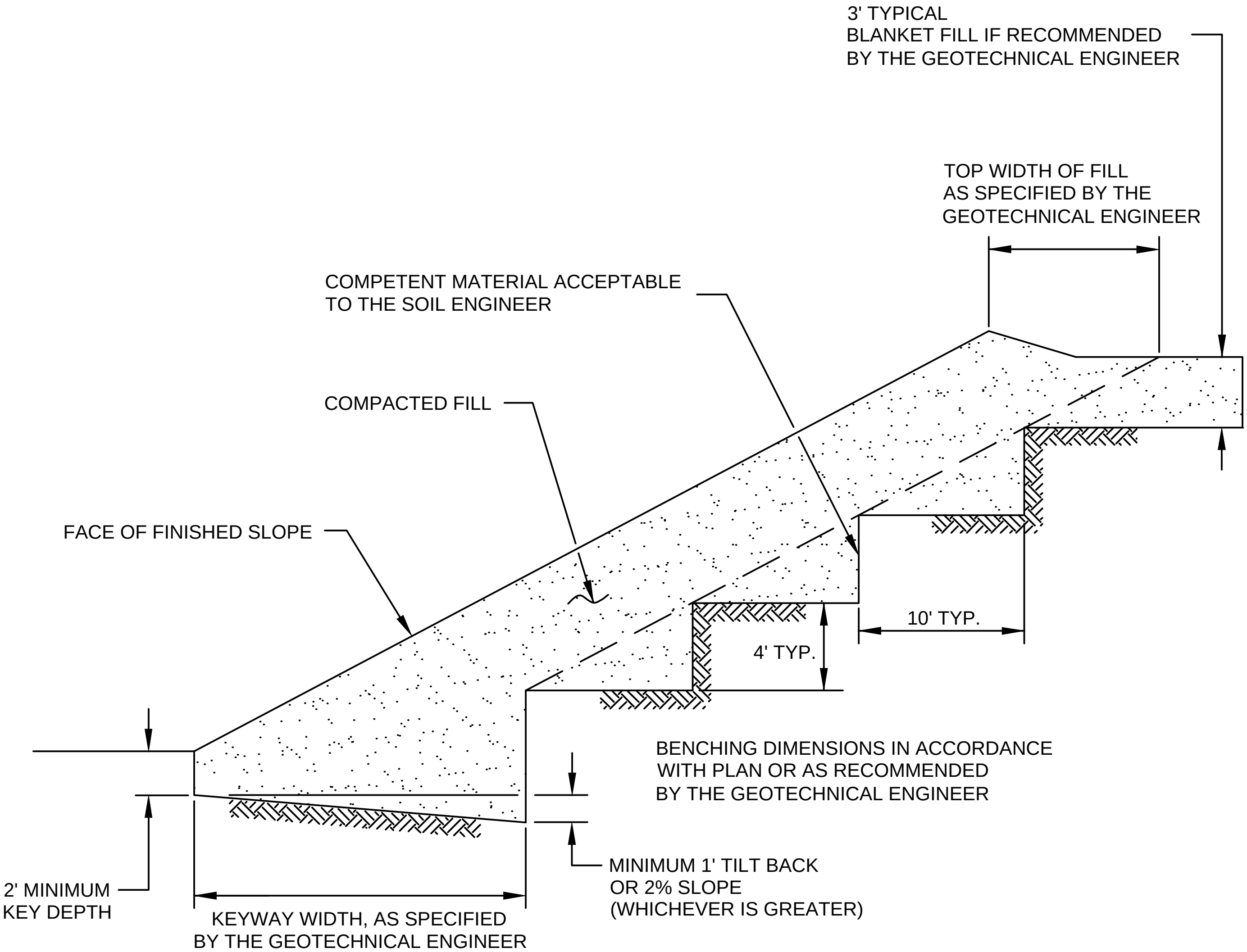
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

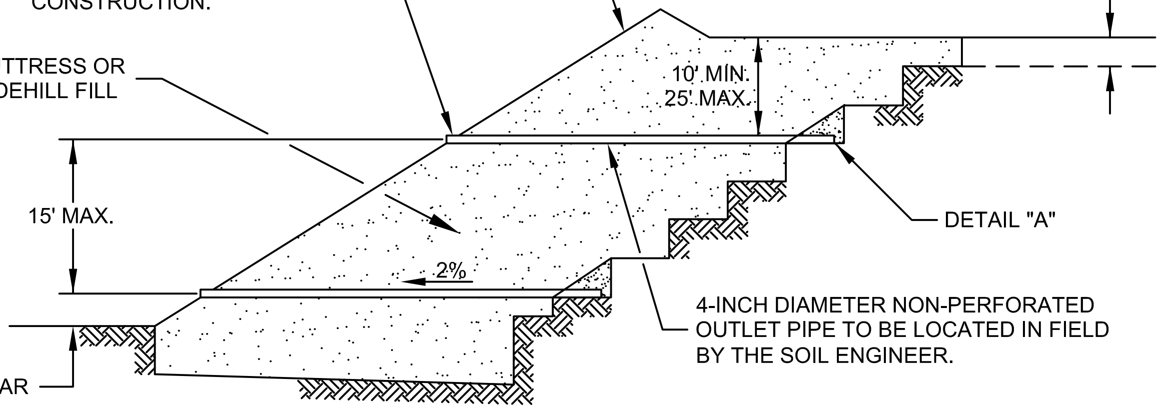
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SOUTHERN CALIFORNIA GEOTECHNICAL



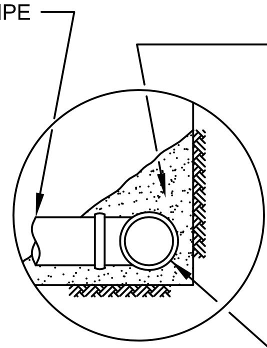
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50



DETAIL "A"

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

SLOPE FILL SUBDRAINS

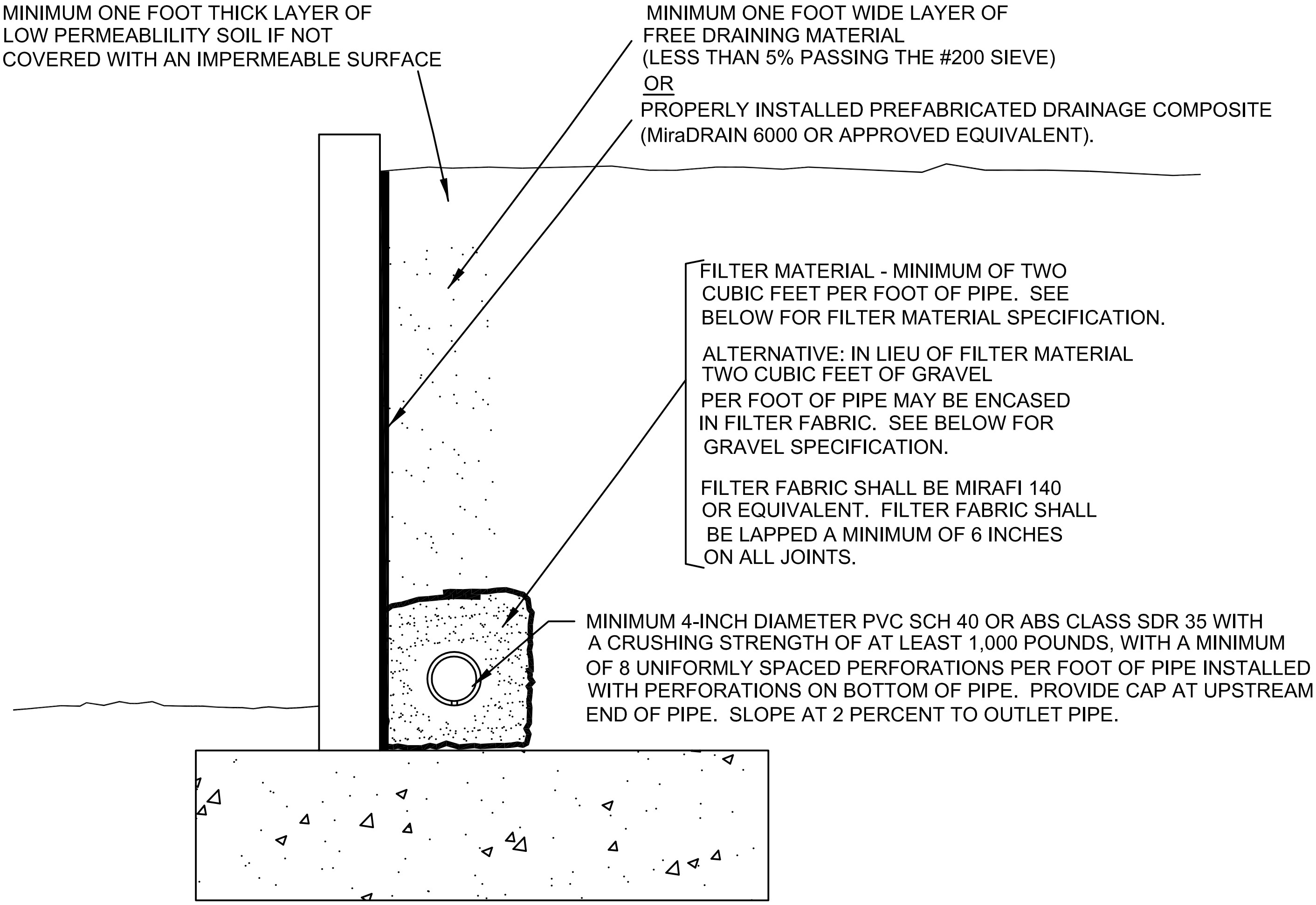
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

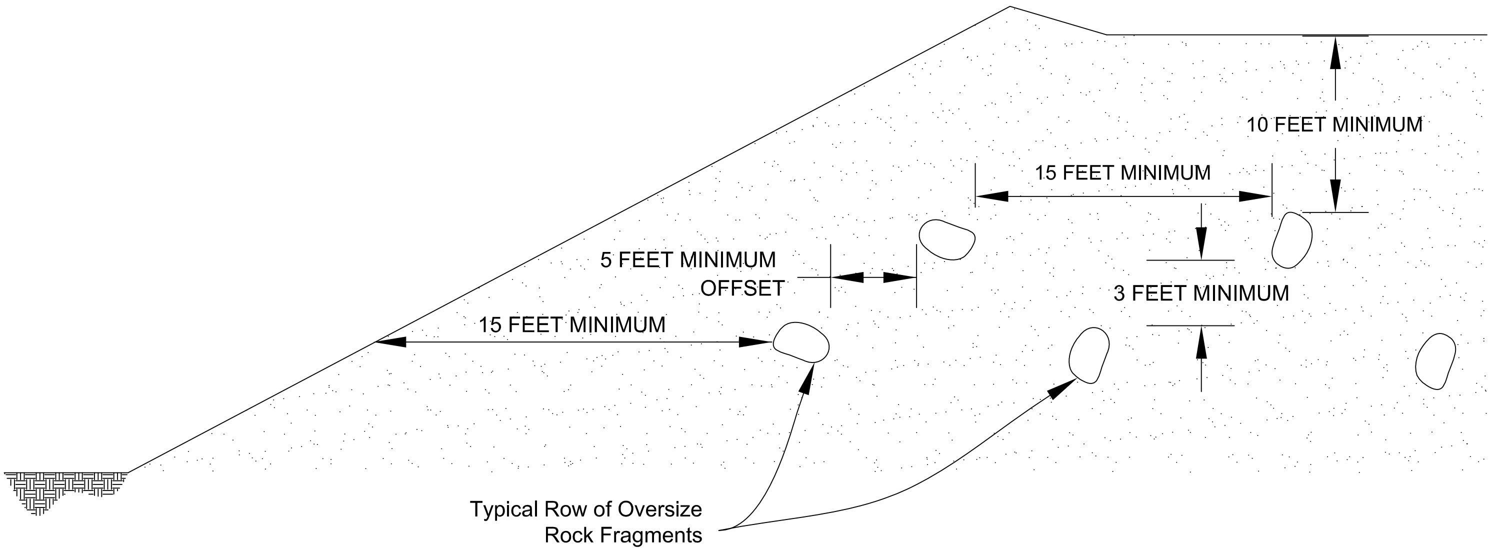
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

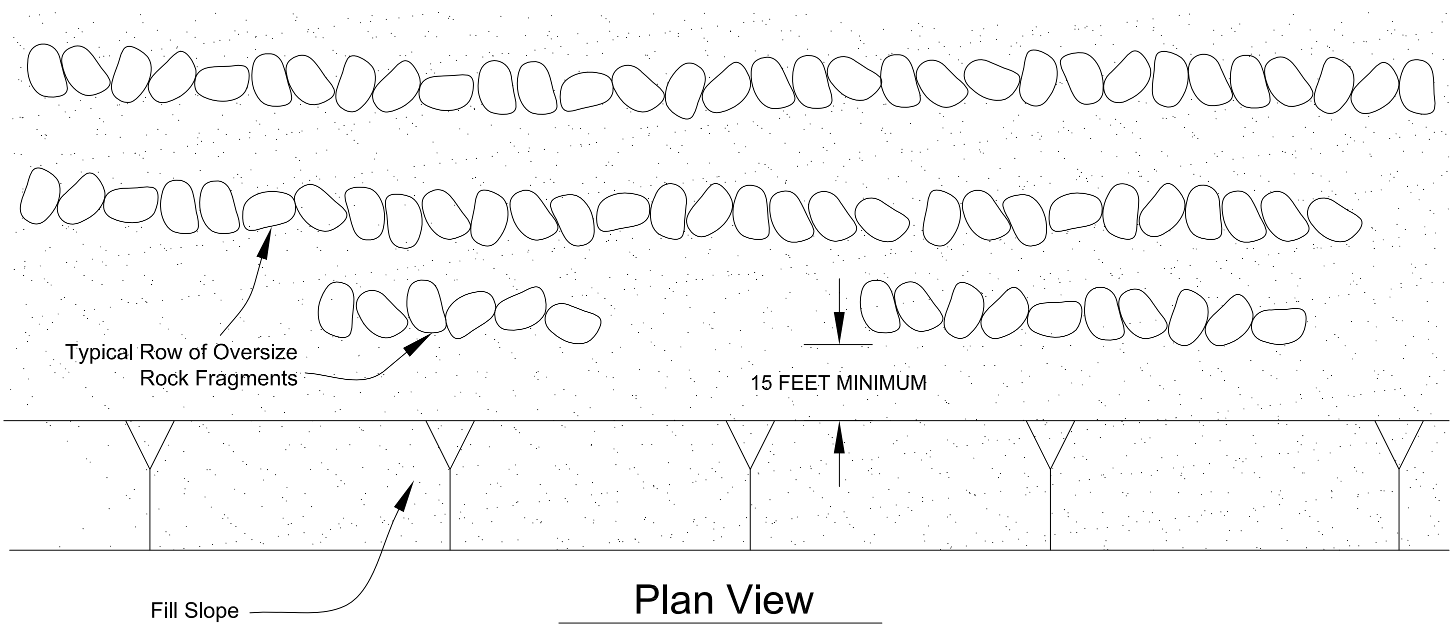
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

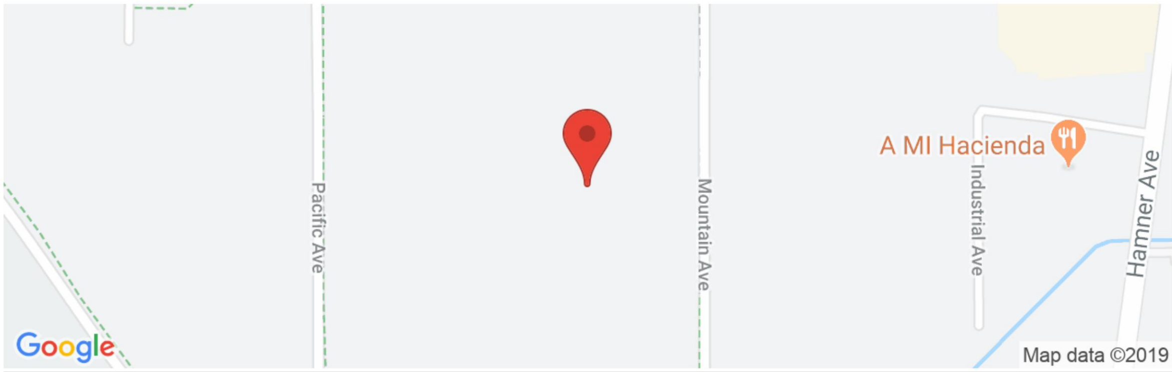
DRAWN: PM
CHKD: GKM

PLATE D-8

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

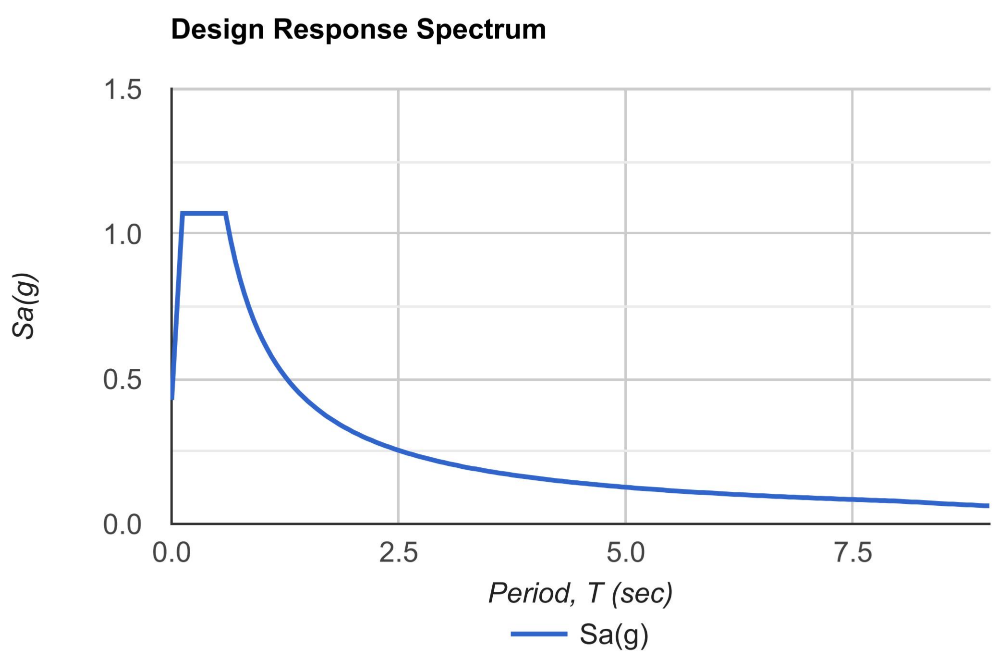
Latitude, Longitude: 33.906213, -117.568225



Date	7/1/2019, 1:57:41 PM
Design Code Reference Document	ASCE7-10
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.606	MCE_R ground motion. (for 0.2 second period)
S_1	0.635	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.606	Site-modified spectral acceleration value
S_{M1}	0.952	Site-modified spectral acceleration value
S_{DS}	1.071	Numeric seismic design value at 0.2 second SA
S_{D1}	0.635	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.607	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.607	Site modified peak ground acceleration



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

SEISMIC DESIGN PARAMETERS

PROPOSED PALOMINO BUSINESS PARK

NORCO, CALIFORNIA

DRAWN: JLL CHKD: RGT	SOUTHERN CALIFORNIA GEOTECHNICAL
SCG PROJECT 17G105-3	
PLATE E-1	

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Proposed Palomino Business Park	MCE_G Design Acceleration	0.607 (g)
Project Location	Norco, CA	Design Magnitude	6.98
Project Number	17G105-3	Historic High Depth to Groundwater	21 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	29.5 (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=6.98)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	0.97	1.02	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	27	120		1.3	1.05	1.3	0.95	0.95	43.1	43.1	2580	2549	2580	0.92	1.22	0.94	2.00	2.00	0.37	5.43	Non-Liquefiable
24.5	22	27	24.5	38	120		1.3	1.05	1.3	0.94	0.95	60.2	60.2	2940	2722	2940	0.91	1.22	0.92	2.00	2.00	0.39	5.18	Non-Liquefiable
29.5	27	29.5	28.3	35	120		1.3	1.05	1.3	0.90	0.95	53.1	53.1	3390	2938	3390	0.89	1.22	0.9	2.00	2.00	0.40	4.95	Non-Liquefiable
29.5	29.5	32	30.8	35	120		1.3	1.05	1.3	0.88	0.95	52.2	52.2	3690	3082	3612	0.87	1.22	0.89	2.00	2.00	0.41	4.85	Non-Liquefiable
34.5	32	37	34.5	38	120		1.3	1.05	1.3	0.89	1	60.3	60.3	4140	3298	3828	0.85	1.22	0.87	2.00	2.00	0.42	4.74	Non-Liquefiable
39.5	37	42	39.5	57	120		1.3	1.05	1.3	0.99	1	100.1	100.1	4740	3586	4116	0.82	1.22	0.84	2.00	2.00	0.43	4.65	Non-Liquefiable
44.5	42	47	44.5	71	120		1.3	1.05	1.3	1.09	1	137.1	137.1	5340	3874	4404	0.80	1.22	0.82	2.00	2.00	0.43	4.62	Non-Liquefiable
49.5	47	50	48.5	82	120		1.3	1.05	1.3	1.20	1	174.2	174.2	5820	4104	4634	0.77	1.22	0.8	2.00	1.96	0.43	4.52	Non-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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-3
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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	43.1	0.0	43.1	5.43	0.00	-1.04	0.00	1.00		0.000		0.00	Non-Liquefiable
24.5	22	27	24.5	60.2	0.0	60.2	5.18	0.00	-2.44	0.00	5.00		0.000		0.00	Non-Liquefiable
29.5	27	29.5	28.3	53.1	0.0	53.1	4.95	0.00	-1.84	0.00	2.50		0.000		0.00	Non-Liquefiable
29.5	29.5	32	30.8	52.2	0.0	52.2	4.85	0.00	-1.77	0.00	2.50		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	60.3	0.0	60.3	4.74	0.00	-2.45	0.00	5.00		0.000		0.00	Non-Liquefiable
39.5	37	42	39.5	100.1	0.0	100.1	4.65	0.00	-6.08	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	137.1	0.0	137.1	4.62	0.00	-9.71	0.00	5.00		0.000		0.00	**Non-Liquefiable**
											Total Deformation (in)				0.00	

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 Palomino Business Park		MCE_G Design Acceleration	0.607 (g)
Project Location	Norco, CA		Design Magnitude	6.98
Project Number	17G105-3		Historic High Depth to Groundwater	21 (ft)
Engineer	DWN		Depth to Groundwater at Time of Drilling	29.5 (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=6.98)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	0.97	1.02	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	24	120	5	1.3	1.05	1.3	0.94	0.95	38.0	38.0	2580	2549	2580	0.92	1.22	0.94	2.00	2.00	0.37	5.43	Non-Liquefiable
24.5	22	27	24.5	33	120	5	1.3	1.05	1.3	0.93	0.95	51.5	51.5	2940	2722	2940	0.91	1.22	0.92	2.00	2.00	0.39	5.18	Non-Liquefiable
29.5	27	32	29.5	43	120	5	1.3	1.05	1.3	0.92	0.95	66.9	66.9	3540	3010	3540	0.88	1.22	0.89	2.00	2.00	0.41	4.90	Non-Liquefiable
34.5	32	37	34.5	33	120	5	1.3	1.05	1.3	0.87	1	50.9	50.9	4140	3298	3828	0.85	1.22	0.87	2.00	2.00	0.42	4.74	Non-Liquefiable
39.5	37	42	39.5	18	120	39	1.3	1.05	1.24	0.78	1	23.8	29.4	4740	3586	4116	0.82	1.18	0.89	0.45	0.47	0.43	1.10	**Liquefiable**
44.5	42	44.5	43.3	13	120	6	1.3	1.05	1.14	0.70	1	14.2	14.3	5190	3802	4332	0.80	1.05	0.94	0.15	0.15	0.43	0.34	**Liquefiable**
44.5	44.5	47	45.8	13	120	74	1.3	1.05	1.15	0.72	1	14.6	20.2	5490	3946	4476	0.79	1.09	0.92	N/A	N/A	N/A	N/A	Non-Liq: PI>18
49.5	47	49	48	22	120		1.3	1.05	1.29	0.75	1	29.1	29.1	5760	4075	4606	0.78	1.17	0.87	N/A	N/A	N/A	N/A	Non-Liq: PI>18
49.5	49	50	49.5	22	120	19	1.3	1.05	1.3	0.77	1	29.9	34.2	5940	4162	4692	0.77	1.22	0.83	0.94	0.95	0.43	2.19	Non-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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-3
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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	38.0	0.0	38.0	5.43	0.01	-0.66	0.00	1.00		0.000		0.00	Non-Liquefiable
24.5	22	27	24.5	51.5	0.0	51.5	5.18	0.00	-1.71	0.00	5.00		0.000		0.00	Non-Liquefiable
29.5	27	32	29.5	66.9	0.0	66.9	4.90	0.00	-3.02	0.00	5.00		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	50.9	0.0	50.9	4.74	0.00	-1.66	0.00	5.00		0.000		0.00	Non-Liquefiable
39.5	37	42	39.5	23.8	5.6	29.4	1.10	0.05	-0.05	0.03	5.00		0.006		0.35	**Liquefiable**
44.5	42	44.5	43.3	14.2	0.0	14.3	0.34	0.30	0.78	0.30	2.50		0.030		0.89	**Liquefiable**
44.5	44.5	47	45.8	14.6	5.6	20.2	N/A	0.16	0.51	0.00	2.50		0.000		0.00	Non-Liq: PI>18
49.5	47	49	48	29.1	0.0	29.1	N/A	0.05	-0.03	0.00	2.00		0.000		0.00	Non-Liq: PI>18
49.5	49	50	49.5	29.9	4.3	34.2	2.19	0.03	-0.38	0.00	1.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				1.24	

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 Palomino Business Park	MCE_G Design Acceleration	0.607 (g)
Project Location	Norco, CA	Design Magnitude	6.98
Project Number	17G105-3	Historic High Depth to Groundwater	21 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	55 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-11		

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=6.98)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	0.97	1.02	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	20	120	6	1.3	1.05	1.3	0.93	0.95	31.4	31.5	2580	2549	2580	0.92	1.20	0.96	0.59	0.68	0.37	1.85	Non-Liquefiable
24.5	22	27	24.5	10	120	43	1.3	1.05	1.13	0.86	0.95	12.6	18.2	2940	2722	2940	0.91	1.08	0.97	0.19	0.19	0.39	0.50	**Liquefiable**
29.5	27	32	29.5	24	120	4	1.3	1.05	1.3	0.84	0.95	34.0	34.0	3540	3010	3540	0.88	1.22	0.91	0.92	1.02	0.41	2.49	Non-Liquefiable
34.5	32	37	34.5	43	120		1.3	1.05	1.3	0.91	1	69.2	69.2	4140	3298	4140	0.85	1.22	0.87	2.00	2.00	0.42	4.74	Non-Liquefiable
39.5	37	42	39.5	34	120		1.3	1.05	1.3	0.82	1	49.6	49.6	4740	3586	4740	0.82	1.22	0.84	2.00	2.00	0.43	4.65	Non-Liquefiable
44.5	42	47	44.5	44	120		1.3	1.05	1.3	0.87	1	67.9	67.9	5340	3874	5340	0.80	1.22	0.82	2.00	2.00	0.43	4.62	Non-Liquefiable
49.5	47	50	48.5	27	120	45	1.3	1.05	1.3	0.75	1	35.7	41.3	5820	4104	5820	0.77	1.22	0.8	2.00	1.96	0.43	4.52	Non-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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-3
Engineer	DWN

Boring No. B-11

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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	31.4	0.0	31.5	1.85	0.04	-0.19	0.00	1.00		0.001		0.01	Non-Liquefiable
24.5	22	27	24.5	12.6	5.6	18.2	0.50	0.20	0.61	0.20	5.00		0.025		1.49	**Liquefiable**
29.5	27	32	29.5	34.0	0.0	34.0	2.49	0.03	-0.37	0.00	5.00		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	69.2	0.0	69.2	4.74	0.00	-3.23	0.00	5.00		0.000		0.00	Non-Liquefiable
39.5	37	42	39.5	49.6	0.0	49.6	4.65	0.00	-1.56	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	67.9	0.0	67.9	4.62	0.00	-3.11	0.00	5.00		0.000		0.00	Non-Liquefiable
49.5	47	50	48.5	35.7	5.6	41.3	4.52	0.01	-0.90	0.00	3.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				1.50	

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 Palomino Business Park	MCE_G Design Acceleration	0.607 (g)
Project Location	Norco, CA	Design Magnitude	6.98
Project Number	17G105-3	Historic High Depth to Groundwater	21 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	22 (ft)
		Borehole Diameter	6 (in)

Boring No. B-13

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=6.98)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	0.97	1.02	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	20	120	4	1.3	1.05	1.3	0.93	0.95	31.4	31.4	2580	2549	2580	0.92	1.20	0.96	0.59	0.68	0.37	1.85	Non-Liquefiable
24.5	22	24	23	15	120	5	1.3	1.05	1.21	0.90	0.95	21.2	21.2	2760	2635	2698	0.91	1.10	0.97	0.22	0.24	0.38	0.63	**Liquefiable**
24.5	24	27	25.5	15	120	44	1.3	1.05	1.21	0.89	0.95	21.0	26.6	3060	2779	2842	0.90	1.15	0.95	0.33	0.36	0.39	0.93	**Liquefiable**
29.5	27	32	29.5	28	120	9	1.3	1.05	1.3	0.90	0.95	42.5	43.3	3540	3010	3072	0.88	1.22	0.89	2.00	2.00	0.41	4.90	Non-Liquefiable
34.5	32	37	34.5	8	120	59	1.3	1.05	1.1	0.80	1	9.6	15.2	4140	3298	3360	0.85	1.06	0.95	N/A	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL
39.5	37	39	38	17	120	46	1.3	1.05	1.24	0.83	1	23.7	29.3	4560	3499	3562	0.83	1.18	0.9	0.44	0.47	0.43	1.10	**Liquefiable**
39.5	39	42	40.5	17	120	76	1.3	1.05	1.23	0.81	1	23.2	28.8	4860	3643	3706	0.82	1.17	0.89	N/A	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL
44.5	42	47	44.5	10	120	74	1.3	1.05	1.11	0.75	1	11.4	16.9	5340	3874	3936	0.80	1.07	0.93	N/A	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL
49.5	47	50	48.5	13	120	61	1.3	1.05	1.15	0.75	1	15.2	20.8	5820	4104	4166	0.77	1.10	0.91	N/A	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL

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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-3
Engineer	DWN

Boring No. B-13

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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	31.4	0.0	31.4	1.85	0.04	-0.19	0.00	1.00		0.001		0.01	Non-Liquefiable
24.5	22	24	23	21.2	0.0	21.2	0.63	0.14	0.45	0.14	2.00		0.022		0.53	Liquefiable
24.5	24	27	25.5	21.0	5.6	26.6	0.93	0.07	0.13	0.04	3.00		0.009		0.33	Liquefiable
29.5	27	32	29.5	42.5	0.7	43.3	4.90	0.00	-1.05	0.00	5.00		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	9.6	5.6	15.2	N/A	0.27	0.75	0.00	5.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL
39.5	37	39	38	23.7	5.6	29.3	1.10	0.05	-0.04	0.03	2.00		0.006		0.14	Liquefiable
39.5	39	42	40.5	23.2	5.6	28.8	N/A	0.05	-0.01	0.00	3.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL
44.5	42	47	44.5	11.4	5.6	16.9	N/A	0.22	0.67	0.00	5.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL
49.5	47	50	48.5	15.2	5.6	20.8	N/A	0.14	0.47	0.00	3.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL

Total Deformation (in) 1.00

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 Palomino Business Park	MCE$_G$ Design Acceleration	0.607	(g)
Project Location	Norco, CA	Design Magnitude	6.98	
Project Number	17G105-3	Historic High Depth to Groundwater	21	(ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	41	(ft)
		Borehole Diameter	6	(in)
Boring No.	B-16			

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=6.98)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	0.97	1.02	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	36	120		1.3	1.05	1.3	0.96	0.95	58.4	58.4	2580	2549	2580	0.92	1.22	0.94	2.00	2.00	0.37	5.43	Non-Liquefiable
24.5	22	26	24	13	120	30	1.3	1.05	1.17	0.88	0.95	17.4	22.8	2880	2693	2880	0.91	1.11	0.96	0.25	0.26	0.38	0.69	**Liquefiable**
29.5	26	31	28.5	17	120	46	1.3	1.05	1.23	0.83	0.95	22.5	28.2	3420	2952	3420	0.89	1.16	0.94	0.39	0.43	0.40	1.05	**Liquefiable**
34.5	31	37	34	60	120		1.3	1.05	1.3	1.01	1	107.3	107.3	4080	3269	4080	0.86	1.22	0.87	2.00	2.00	0.42	4.75	Non-Liquefiable
39.5	37	50	43.5	50	120		1.3	1.05	1.3	0.93	1	82.2	82.2	5220	3816	5064	0.80	1.22	0.82	2.00	2.00	0.43	4.62	Non-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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-3
Engineer	DWN

Boring No. B-16

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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	58.4	0.0	58.4	5.43	0.00	-2.28	0.00	1.00		0.000		0.00	Non-Liquefiable
24.5	22	26	24	17.4	5.4	22.8	0.69	0.12	0.36	0.09	4.00		0.021		0.99	**Liquefiable**
29.5	26	31	28.5	22.5	5.6	28.2	1.05	0.06	0.03	0.03	5.00		0.007		0.40	**Liquefiable**
34.5	31	37	34	107.3	0.0	107.3	4.75	0.00	-6.77	0.00	6.00		0.000		0.00	Non-Liquefiable
39.5	37	50	43.5	82.2	0.0	82.2	4.62	0.00	-4.40	0.00	13.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				1.39	

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 Palomino Business Park	MCE_G Design Acceleration	0.607 (g)
Project Location	Norco, CA	Design Magnitude	6.98
Project Number	17G105-3	Historic High Depth to Groundwater	21 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	30 (ft)
		Borehole Diameter	6 (in)

Boring No. B-17

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=6.98)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	0.97	1.02	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	27	120		1.3	1.05	1.3	0.95	0.95	43.1	43.1	2580	2549	2580	0.92	1.22	0.94	2.00	2.00	0.37	5.43	Non-Liquefiable
24.5	22	27	24.5	46	120		1.3	1.05	1.3	0.96	0.95	74.5	74.5	2940	2722	2940	0.91	1.22	0.92	2.00	2.00	0.39	5.18	Non-Liquefiable
29.5	27	31	29	34	120		1.3	1.05	1.3	0.89	0.95	51.0	51.0	3480	2981	3480	0.88	1.22	0.9	2.00	2.00	0.41	4.92	Non-Liquefiable
34.5	31	37	34	10	120	59	1.3	1.05	1.12	0.76	1	11.5	17.2	4080	3269	3830	0.86	1.07	0.95	0.18	N/A	N/A	N/A	Non-liq: PI>18
39.5	37	42	39.5	15	120	51	1.3	1.05	1.18	0.76	1	18.4	24.1	4740	3586	4147	0.82	1.12	0.92	0.27	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL
44.5	42	47	44.5	12	120	45	1.3	1.05	1.13	0.72	1	13.3	18.9	5340	3874	4435	0.80	1.08	0.92	0.19	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL
49.5	47	50	48.5	23	120	39	1.3	1.05	1.3	0.78	1	31.8	37.4	5820	4104	4666	0.77	1.22	0.8	1.94	1.90	0.43	4.38	Non-Liq: PI>18

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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-3
Engineer	DWN

Boring No. B-17

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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	43.1	0.0	43.1	5.43	0.00	-1.04	0.00	1.00		0.000		0.00	Non-Liquefiable
24.5	22	27	24.5	74.5	0.0	74.5	5.18	0.00	-3.70	0.00	5.00		0.000		0.00	Non-Liquefiable
29.5	27	31	29	51.0	0.0	51.0	4.92	0.00	-1.67	0.00	4.00		0.000		0.00	Non-Liquefiable
34.5	31	37	34	11.5	5.6	17.2	N/A	0.22	0.66	0.00	6.00		0.000		0.00	Non-liq: PI>18
39.5	37	42	39.5	18.4	5.6	24.1	N/A	0.10	0.29	0.00	5.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL
44.5	42	47	44.5	13.3	5.6	18.9	N/A	0.18	0.57	0.00	5.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL
49.5	47	50	48.5	31.8	5.6	37.4	4.38	0.01	-0.61	0.00	3.00		0.000		0.00	Non-Liq: PI>18
												Total Deformation (in)			0.00	

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 Palomino Business Park	MCE_G Design Acceleration	0.607 (g)
Project Location	Norco, CA	Design Magnitude	6.98
Project Number	17G105-3	Historic High Depth to Groundwater	21 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	29 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-18		

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=6.98)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	0.97	1.02	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	50	120		1.3	1.05	1.3	0.98	0.95	82.9	82.9	2580	2549	2580	0.92	1.22	0.94	2.00	2.00	0.37	5.43	Non-Liquefiable
24.5	22	27	24.5	34	120		1.3	1.05	1.3	0.93	0.95	53.3	53.3	2940	2722	2940	0.91	1.22	0.92	2.00	2.00	0.39	5.18	Non-Liquefiable
29.5	27	32	29.5	42	120		1.3	1.05	1.3	0.92	0.95	65.2	65.2	3540	3010	3509	0.88	1.22	0.89	2.00	2.00	0.41	4.90	Non-Liquefiable
34.5	32	37	34.5	12	120	55	1.3	1.05	1.15	0.77	1	14.5	20.1	4140	3298	3797	0.85	1.09	0.94	0.21	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL
39.5	37	42	39.5	23	120	37	1.3	1.05	1.3	0.82	1	33.4	38.9	4740	3586	4085	0.82	1.22	0.84	2.00	2.00	0.43	4.65	Non-Liquefiable
44.5	42	47	44.5	17	120	59	1.3	1.05	1.21	0.76	1	21.3	26.9	5340	3874	4373	0.80	1.15	0.89	0.34	N/A	N/A	N/A	Non-Liq: PI>18
49.5	47	50	48.5	8	120	78	1.3	1.05	1.1	0.68	1	8.1	13.7	5820	4104	4603	0.77	1.05	0.93	0.15	0.14	0.43	0.33	**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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-3
Engineer	DWN

Boring No. B-18

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	$(N_1)_{60-CS}$	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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	82.9	0.0	82.9	5.43	0.00	-4.46	0.00	1.00		0.000		0.00	Non-Liquefiable
24.5	22	27	24.5	53.3	0.0	53.3	5.18	0.00	-1.86	0.00	5.00		0.000		0.00	Non-Liquefiable
29.5	27	32	29.5	65.2	0.0	65.2	4.90	0.00	-2.87	0.00	5.00		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	14.5	5.6	20.1	N/A	0.16	0.51	0.00	5.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL
39.5	37	42	39.5	33.4	5.5	38.9	4.65	0.01	-0.72	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	21.3	5.6	26.9	N/A	0.07	0.12	0.00	5.00		0.000		0.00	Non-Liq: PI>18
49.5	47	50	48.5	8.1	5.6	13.7	0.33	0.32	0.81	0.32	3.00		0.031		1.10	**Liquefiable**

Total Deformation (in)	1.10

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)

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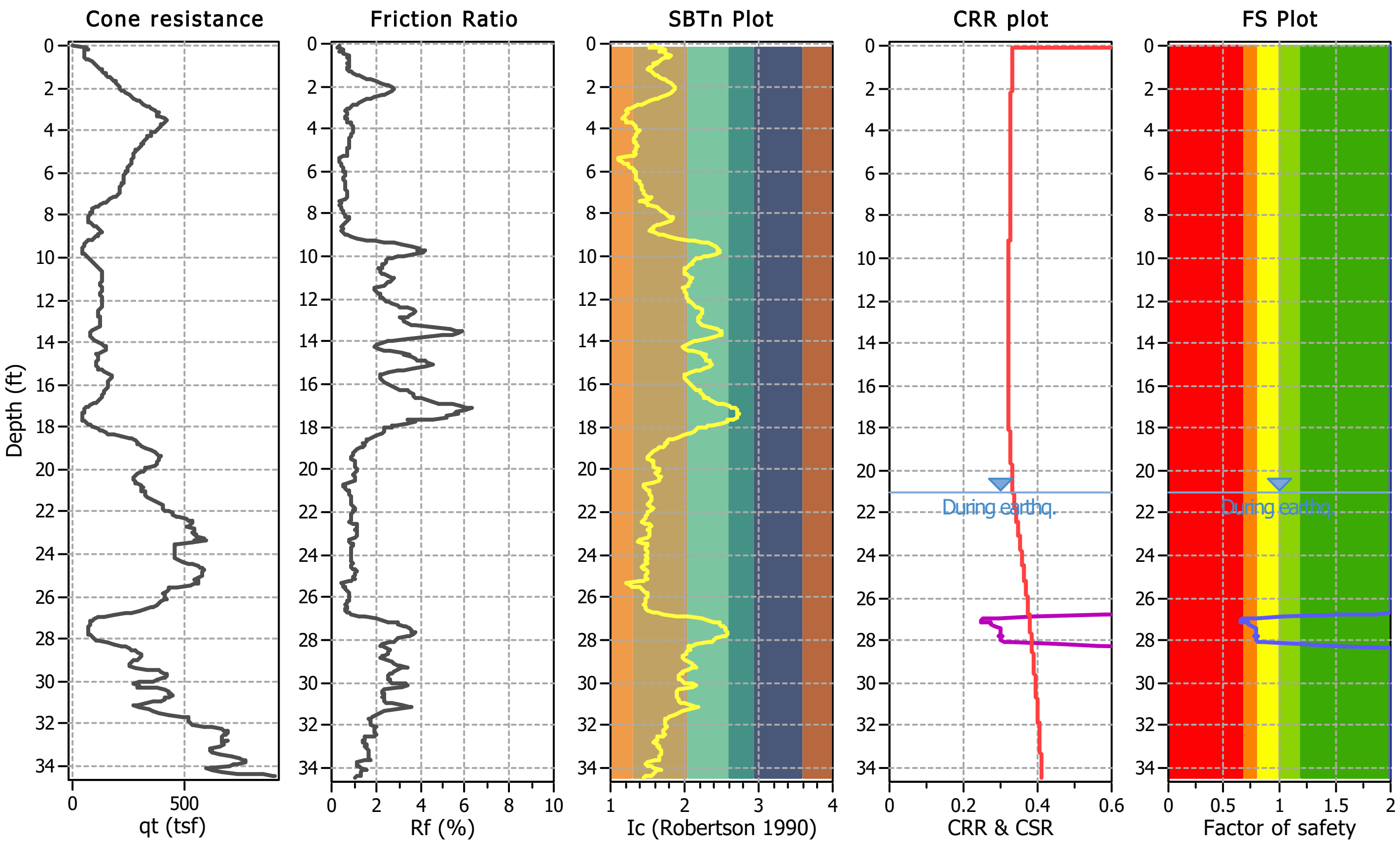
LIQUEFACTION ANALYSIS REPORT

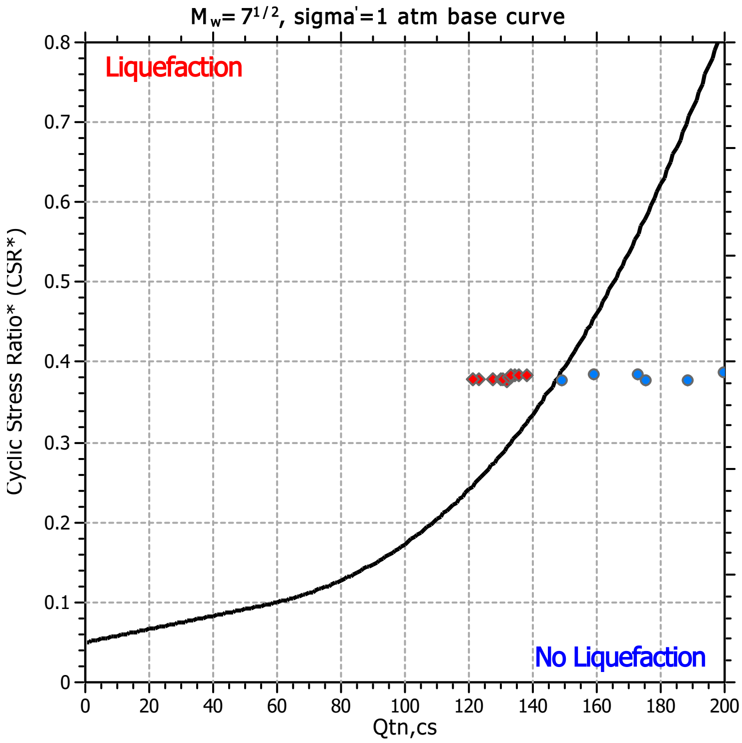
Project title : Proposed Palomino Business Park Location : Norco, California

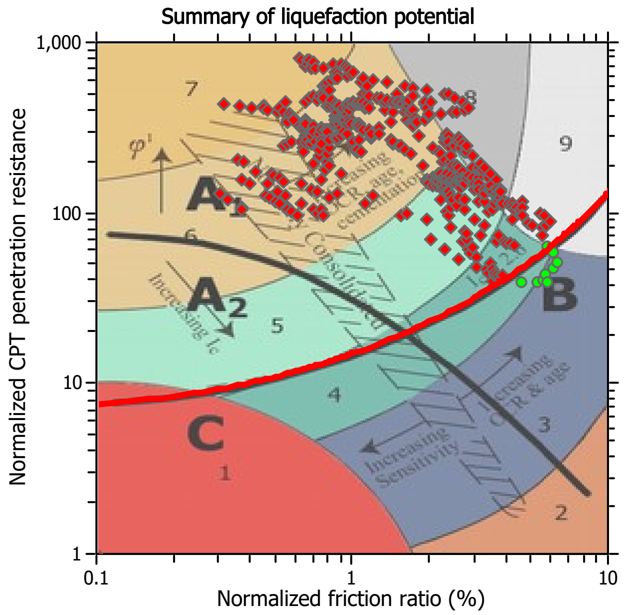
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	**NCEER (1998)**	**G.W.T. (in-situ):**	**35.00 ft**	**Use fill:**	**No**	**Clay like behavior applied:**	**Sands only**
Fines correction method:	**NCEER (1998)**	**G.W.T. (earthq.):**	**21.00 ft**	**Fill height:**	**N/A**	**Limit depth applie**	**No**
Points to test:	**Based on Ic value**	**Average results interva**	**1**	**Fill weight:**	**N/A**	**Limit depth:**	**N/A**
Earthquake magnitude M_w:	**6.98**	**Ic cut-off value:**	**2.60**	**Trans. detect. applied:**	**No**	**MSF method:**	**Method basec**
Peak ground acceleration:	**0.61**	**Unit weight calculatior**	**Based on SBT**	**K_σ applied:**	**Yes**		

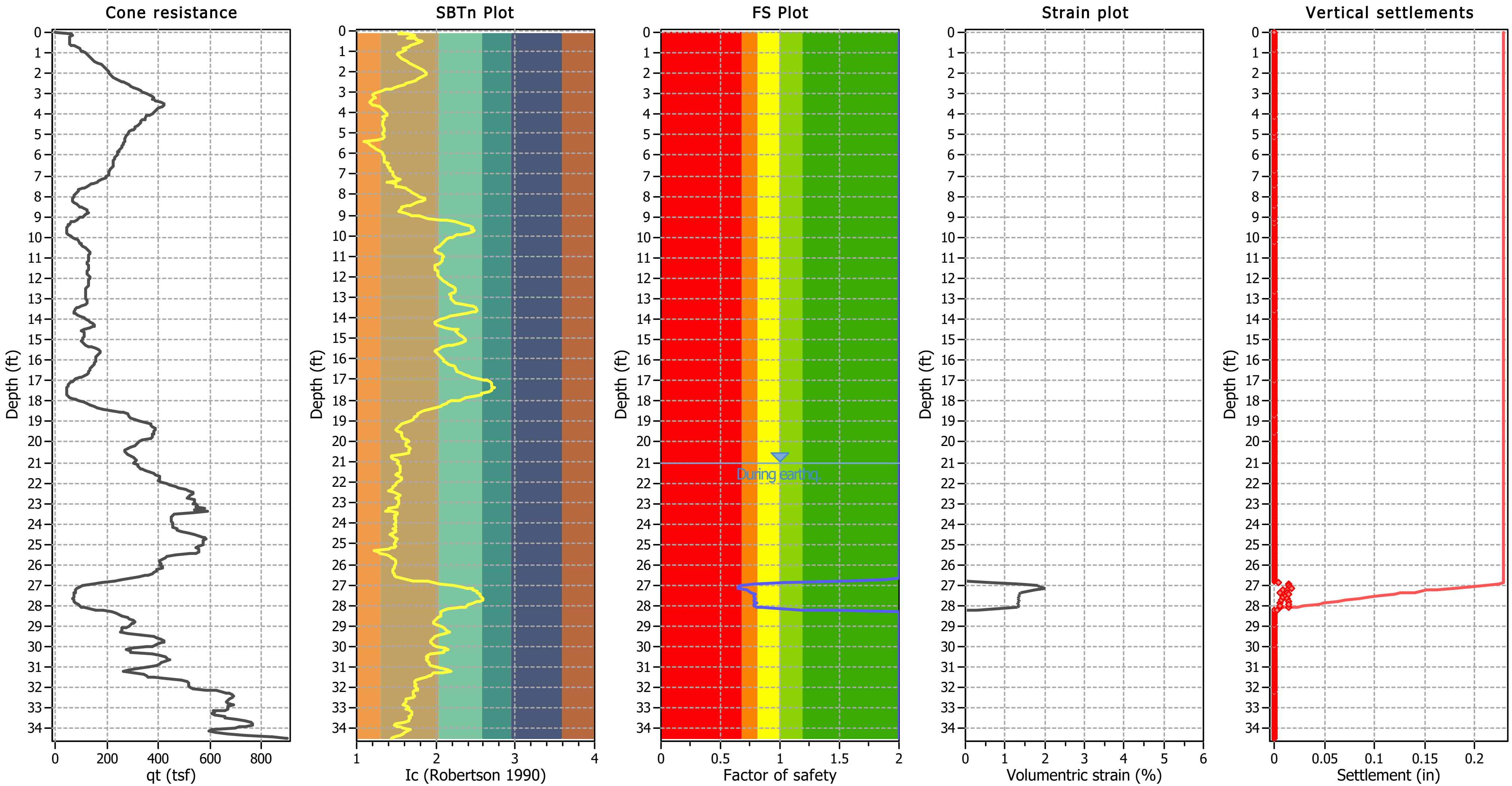






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and 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)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
21.00	254.86	2.00	0.00	1.00	0.00	21.07	266.67	2.00	0.00	1.00	0.00
21.15	268.31	2.00	0.00	1.00	0.00	21.22	269.69	2.00	0.00	1.00	0.00
21.29	275.29	2.00	0.00	1.00	0.00	21.33	278.43	2.00	0.00	1.00	0.00
21.40	286.26	2.00	0.00	1.00	0.00	21.48	301.11	2.00	0.00	1.00	0.00
21.55	315.92	2.00	0.00	1.00	0.00	21.59	318.63	2.00	0.00	1.00	0.00
21.67	331.82	2.00	0.00	1.00	0.00	21.74	332.27	2.00	0.00	1.00	0.00
21.81	329.68	2.00	0.00	1.00	0.00	21.86	327.12	2.00	0.00	1.00	0.00
21.92	334.81	2.00	0.00	1.00	0.00	22.00	351.35	2.00	0.00	1.00	0.00
22.07	361.91	2.00	0.00	1.00	0.00	22.13	363.18	2.00	0.00	1.00	0.00
22.19	378.05	2.00	0.00	1.00	0.00	22.25	385.72	2.00	0.00	1.00	0.00
22.34	411.71	2.00	0.00	1.00	0.00	22.38	419.89	2.00	0.00	1.00	0.00
22.44	431.57	2.00	0.00	1.00	0.00	22.52	429.70	2.00	0.00	1.00	0.00
22.59	419.46	2.00	0.00	1.00	0.00	22.66	413.36	2.00	0.00	1.00	0.00
22.73	408.37	2.00	0.00	1.00	0.00	22.80	432.52	2.00	0.00	1.00	0.00
22.86	429.99	2.00	0.00	1.00	0.00	22.93	432.38	2.00	0.00	1.00	0.00
23.00	426.89	2.00	0.00	1.00	0.00	23.03	432.73	2.00	0.00	1.00	0.00
23.10	439.27	2.00	0.00	1.00	0.00	23.18	434.03	2.00	0.00	1.00	0.00
23.24	461.22	2.00	0.00	1.00	0.00	23.31	428.34	2.00	0.00	1.00	0.00
23.39	467.36	2.00	0.00	1.00	0.00	23.44	422.65	2.00	0.00	1.00	0.00
23.50	364.82	2.00	0.00	1.00	0.00	23.57	357.51	2.00	0.00	1.00	0.00
23.63	354.30	2.00	0.00	1.00	0.00	23.70	354.11	2.00	0.00	1.00	0.00
23.76	353.60	2.00	0.00	1.00	0.00	23.84	353.34	2.00	0.00	1.00	0.00
23.90	352.09	2.00	0.00	1.00	0.00	23.97	356.85	2.00	0.00	1.00	0.00
24.03	355.31	2.00	0.00	1.00	0.00	24.11	354.63	2.00	0.00	1.00	0.00
24.18	352.99	2.00	0.00	1.00	0.00	24.23	367.26	2.00	0.00	1.00	0.00
24.29	367.70	2.00	0.00	1.00	0.00	24.35	379.79	2.00	0.00	1.00	0.00
24.42	398.04	2.00	0.00	1.00	0.00	24.50	414.00	2.00	0.00	1.00	0.00
24.56	430.50	2.00	0.00	1.00	0.00	24.64	439.45	2.00	0.00	1.00	0.00
24.69	443.08	2.00	0.00	1.00	0.00	24.75	448.36	2.00	0.00	1.00	0.00
24.82	440.77	2.00	0.00	1.00	0.00	24.90	439.67	2.00	0.00	1.00	0.00
24.96	437.12	2.00	0.00	1.00	0.00	25.03	438.23	2.00	0.00	1.00	0.00
25.10	423.47	2.00	0.00	1.00	0.00	25.16	412.89	2.00	0.00	1.00	0.00
25.20	423.70	2.00	0.00	1.00	0.00	25.27	423.67	2.00	0.00	1.00	0.00
25.34	423.60	2.00	0.00	1.00	0.00	25.41	411.68	2.00	0.00	1.00	0.00
25.46	395.20	2.00	0.00	1.00	0.00	25.54	349.69	2.00	0.00	1.00	0.00
25.60	324.03	2.00	0.00	1.00	0.00	25.67	322.14	2.00	0.00	1.00	0.00
25.74	312.29	2.00	0.00	1.00	0.00	25.80	301.68	2.00	0.00	1.00	0.00
25.88	305.00	2.00	0.00	1.00	0.00	25.95	306.73	2.00	0.00	1.00	0.00
25.99	304.32	2.00	0.00	1.00	0.00	26.09	310.28	2.00	0.00	1.00	0.00
26.12	308.22	2.00	0.00	1.00	0.00	26.19	292.10	2.00	0.00	1.00	0.00
26.26	290.32	2.00	0.00	1.00	0.00	26.33	284.11	2.00	0.00	1.00	0.00
26.41	278.79	2.00	0.00	1.00	0.00	26.48	266.77	2.00	0.00	1.00	0.00
26.52	258.86	2.00	0.00	1.00	0.00	26.59	240.27	2.00	0.00	1.00	0.00
26.67	216.90	2.00	0.00	1.00	0.00	26.74	188.92	1.88	0.00	1.00	0.00
26.78	175.83	1.56	0.00	1.00	0.00	26.84	149.62	1.04	0.61	1.00	0.00
26.93	132.18	0.78	1.35	1.00	0.01	26.99	122.85	0.67	1.84	1.00	0.01
27.07	121.44	0.65	1.87	1.00	0.02	27.14	121.24	0.65	2.00	1.00	0.02
27.19	127.29	0.72	1.75	1.00	0.01	27.24	127.20	0.72	1.75	1.00	0.01

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
27.33	130.20	0.75	1.38	1.00	0.01	27.36	130.37	0.75	1.38	1.00	0.01
27.45	133.20	0.79	1.34	1.00	0.01	27.53	133.15	0.79	1.34	1.00	0.01
27.58	133.19	0.79	1.34	1.00	0.01	27.67	133.34	0.79	1.33	1.00	0.01
27.72	133.19	0.78	1.34	1.00	0.01	27.80	132.51	0.78	1.35	1.00	0.01
27.84	134.59	0.80	1.32	1.00	0.01	27.93	133.37	0.78	1.33	1.00	0.01
27.97	133.12	0.78	1.34	1.00	0.01	28.06	135.40	0.81	1.31	1.00	0.01
28.10	137.95	0.84	1.27	1.00	0.01	28.19	159.24	1.18	0.29	1.00	0.00
28.23	172.93	1.46	0.00	1.00	0.00	28.31	199.92	2.00	0.00	1.00	0.00
28.36	212.06	2.00	0.00	1.00	0.00	28.44	227.86	2.00	0.00	1.00	0.00
28.49	235.09	2.00	0.00	1.00	0.00	28.57	245.33	2.00	0.00	1.00	0.00
28.61	250.13	2.00	0.00	1.00	0.00	28.70	256.40	2.00	0.00	1.00	0.00
28.78	255.65	2.00	0.00	1.00	0.00	28.82	251.71	2.00	0.00	1.00	0.00
28.87	246.88	2.00	0.00	1.00	0.00	28.96	246.07	2.00	0.00	1.00	0.00
29.03	241.91	2.00	0.00	1.00	0.00	29.08	238.47	2.00	0.00	1.00	0.00
29.16	239.98	2.00	0.00	1.00	0.00	29.21	243.62	2.00	0.00	1.00	0.00
29.29	251.80	2.00	0.00	1.00	0.00	29.33	264.38	2.00	0.00	1.00	0.00
29.42	290.35	2.00	0.00	1.00	0.00	29.50	314.61	2.00	0.00	1.00	0.00
29.55	320.60	2.00	0.00	1.00	0.00	29.61	328.98	2.00	0.00	1.00	0.00
29.69	334.67	2.00	0.00	1.00	0.00	29.74	334.07	2.00	0.00	1.00	0.00
29.80	329.71	2.00	0.00	1.00	0.00	29.87	321.58	2.00	0.00	1.00	0.00
29.95	306.24	2.00	0.00	1.00	0.00	29.99	294.65	2.00	0.00	1.00	0.00
30.06	273.55	2.00	0.00	1.00	0.00	30.13	263.16	2.00	0.00	1.00	0.00
30.20	265.00	2.00	0.00	1.00	0.00	30.25	262.65	2.00	0.00	1.00	0.00
30.34	297.38	2.00	0.00	1.00	0.00	30.39	312.63	2.00	0.00	1.00	0.00
30.47	325.97	2.00	0.00	1.00	0.00	30.53	330.36	2.00	0.00	1.00	0.00
30.60	334.99	2.00	0.00	1.00	0.00	30.65	337.07	2.00	0.00	1.00	0.00
30.72	328.06	2.00	0.00	1.00	0.00	30.78	314.92	2.00	0.00	1.00	0.00
30.86	312.85	2.00	0.00	1.00	0.00	30.93	307.45	2.00	0.00	1.00	0.00
30.98	293.05	2.00	0.00	1.00	0.00	31.05	278.16	2.00	0.00	1.00	0.00
31.11	266.51	2.00	0.00	1.00	0.00	31.17	259.25	2.00	0.00	1.00	0.00
31.25	259.09	2.00	0.00	1.00	0.00	31.31	268.97	2.00	0.00	1.00	0.00
31.38	271.07	2.00	0.00	1.00	0.00	31.44	272.67	2.00	0.00	1.00	0.00
31.50	283.85	2.00	0.00	1.00	0.00	31.56	309.34	2.00	0.00	1.00	0.00
31.65	342.41	2.00	0.00	1.00	0.00	31.72	357.77	2.00	0.00	1.00	0.00
31.77	359.13	2.00	0.00	1.00	0.00	31.83	360.05	2.00	0.00	1.00	0.00
31.90	358.81	2.00	0.00	1.00	0.00	31.96	365.38	2.00	0.00	1.00	0.00
32.02	377.62	2.00	0.00	1.00	0.00	32.10	408.77	2.00	0.00	1.00	0.00
32.15	433.53	2.00	0.00	1.00	0.00	32.22	457.67	2.00	0.00	1.00	0.00
32.29	461.36	2.00	0.00	1.00	0.00	32.35	470.33	2.00	0.00	1.00	0.00
32.42	477.47	2.00	0.00	1.00	0.00	32.48	468.72	2.00	0.00	1.00	0.00
32.55	449.21	2.00	0.00	1.00	0.00	32.61	443.03	2.00	0.00	1.00	0.00
32.68	442.24	2.00	0.00	1.00	0.00	32.75	442.11	2.00	0.00	1.00	0.00
32.81	457.11	2.00	0.00	1.00	0.00	32.87	441.40	2.00	0.00	1.00	0.00
32.95	442.95	2.00	0.00	1.00	0.00	33.01	441.40	2.00	0.00	1.00	0.00
33.08	434.33	2.00	0.00	1.00	0.00	33.14	410.22	2.00	0.00	1.00	0.00
33.21	412.31	2.00	0.00	1.00	0.00	33.28	407.36	2.00	0.00	1.00	0.00
33.34	412.73	2.00	0.00	1.00	0.00	33.41	433.09	2.00	0.00	1.00	0.00
33.47	434.33	2.00	0.00	1.00	0.00	33.54	462.53	2.00	0.00	1.00	0.00

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
33.62	485.04	2.00	0.00	1.00	0.00	33.67	494.51	2.00	0.00	1.00	0.00
33.73	499.98	2.00	0.00	1.00	0.00	33.80	498.94	2.00	0.00	1.00	0.00
33.87	479.40	2.00	0.00	1.00	0.00	33.92	466.30	2.00	0.00	1.00	0.00
33.99	452.28	2.00	0.00	1.00	0.00	34.07	397.22	2.00	0.00	1.00	0.00
34.12	387.98	2.00	0.00	1.00	0.00	34.19	404.59	2.00	0.00	1.00	0.00
34.26	432.21	2.00	0.00	1.00	0.00	34.33	477.42	2.00	0.00	1.00	0.00
34.39	547.30	2.00	0.00	1.00	0.00	34.45	578.23	2.00	0.00	1.00	0.00
34.46	581.30	2.00	0.00	1.00	0.00						

Total estimated settlement: 0.23

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

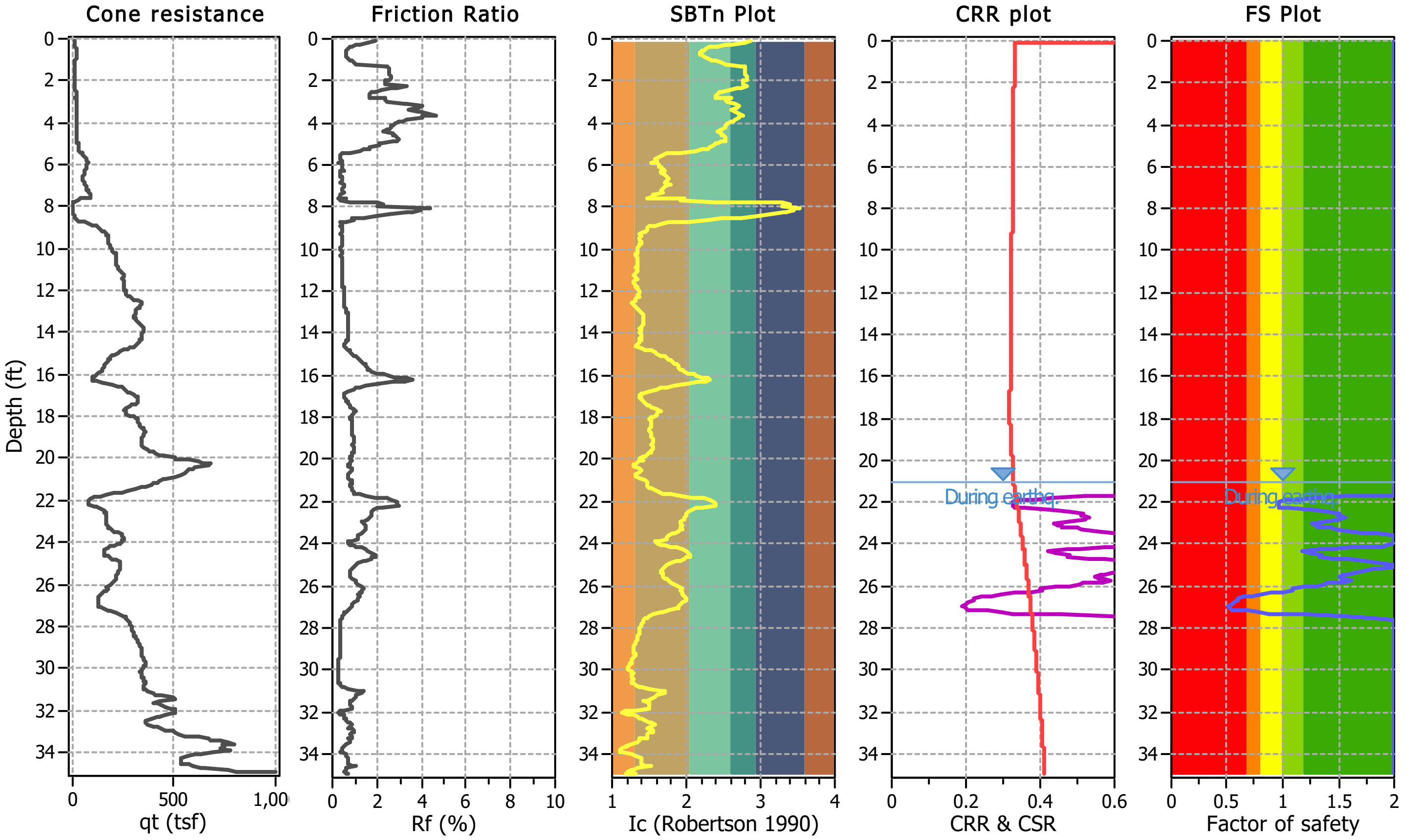
LIQUEFACTION ANALYSIS REPORT

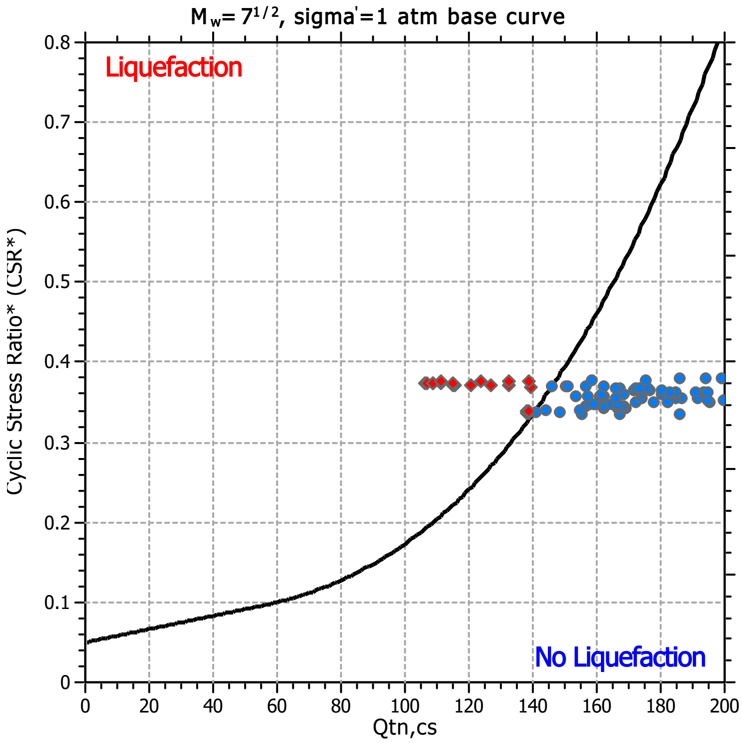
Project title : Proposed Palomino Business Park   Location : Norco, California

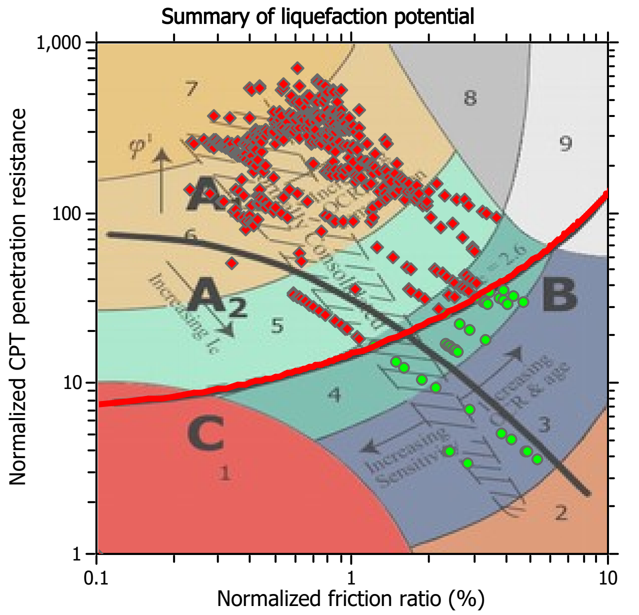
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	27.00 ft	**Use fill:**	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	21.00 ft	**Fill height:**	N/A	Limit depth applie	No
Points to test:	Based on Ic value	Average results interva	1	**Fill weight:**	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.98	Ic cut-off value:	2.60	**Trans. detect. applied:**	No	MSF method:	Method basec
Peak ground acceleration:	0.61	Unit weight calculatior	Based on SBT	**K_σ applied:**	Yes		

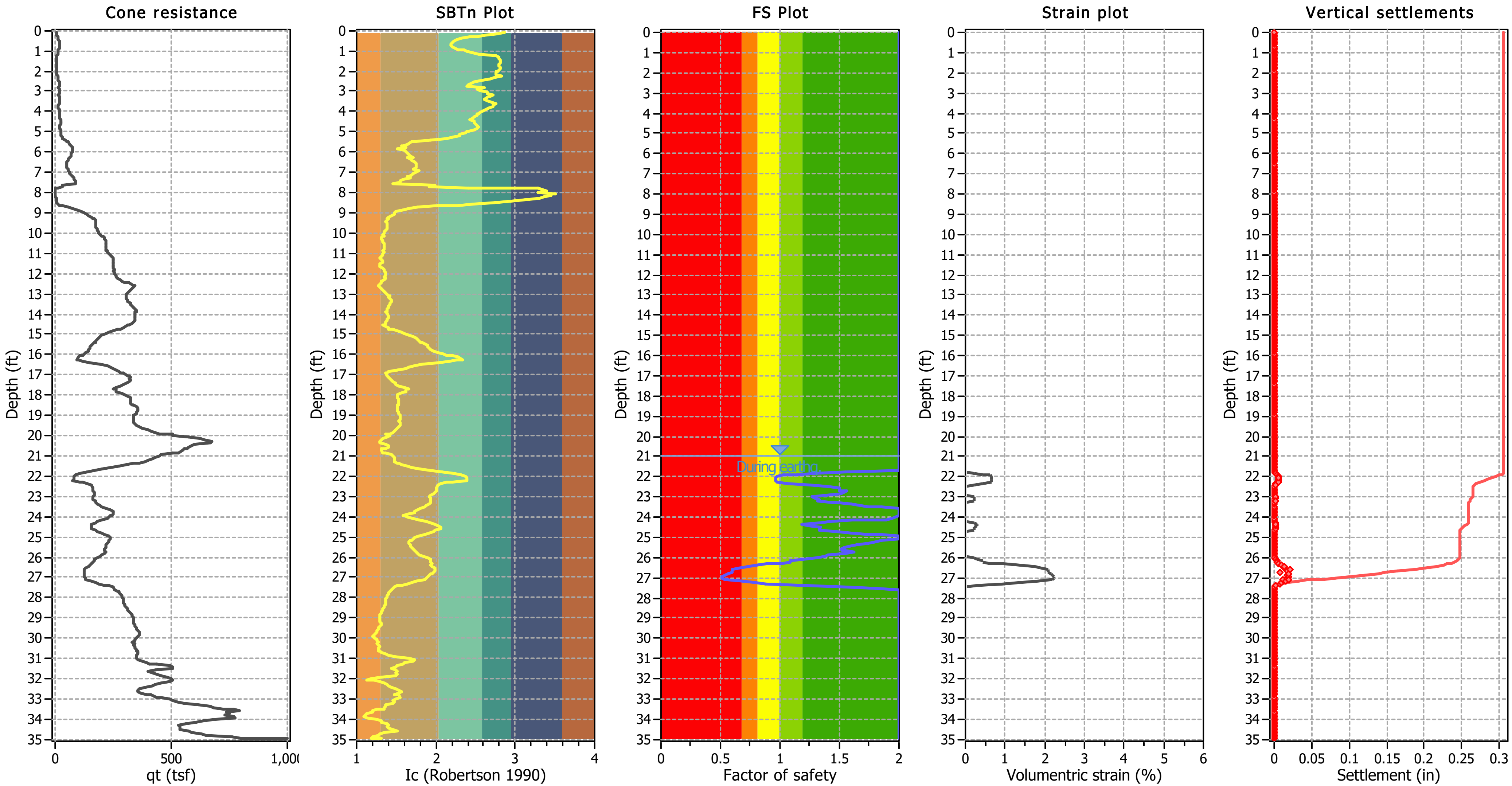






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and 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)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
21.00	388.63	2.00	0.00	1.00	0.00	21.07	370.89	2.00	0.00	1.00	0.00
21.14	361.52	2.00	0.00	1.00	0.00	21.20	344.84	2.00	0.00	1.00	0.00
21.27	331.12	2.00	0.00	1.00	0.00	21.34	309.19	2.00	0.00	1.00	0.00
21.40	287.63	2.00	0.00	1.00	0.00	21.47	262.35	2.00	0.00	1.00	0.00
21.53	233.35	2.00	0.00	1.00	0.00	21.60	208.20	2.00	0.00	1.00	0.00
21.66	186.01	2.00	0.00	1.00	0.00	21.74	167.80	1.55	0.00	1.00	0.00
21.80	155.67	1.29	0.21	1.00	0.00	21.87	148.65	1.15	0.43	1.00	0.00
21.93	141.44	1.02	0.64	1.00	0.00	21.99	138.50	0.97	0.65	1.00	0.00
22.07	138.38	0.97	0.65	1.00	0.01	22.14	138.21	0.96	0.65	1.00	0.01
22.20	138.12	0.96	0.65	1.00	0.01	22.27	138.88	0.97	0.65	1.00	0.01
22.31	144.37	1.06	0.43	1.00	0.00	22.39	155.07	1.26	0.21	1.00	0.00
22.46	162.57	1.41	0.00	1.00	0.00	22.56	167.12	1.51	0.00	1.00	0.00
22.60	166.72	1.50	0.00	1.00	0.00	22.67	167.23	1.50	0.00	1.00	0.00
22.71	169.50	1.56	0.00	1.00	0.00	22.78	168.89	1.54	0.00	1.00	0.00
22.85	167.95	1.51	0.00	1.00	0.00	22.93	162.25	1.39	0.00	1.00	0.00
23.00	156.74	1.27	0.21	1.00	0.00	23.07	158.24	1.30	0.21	1.00	0.00
23.10	159.82	1.33	0.21	1.00	0.00	23.18	159.62	1.32	0.21	1.00	0.00
23.24	165.22	1.44	0.00	1.00	0.00	23.31	166.81	1.47	0.00	1.00	0.00
23.37	172.24	1.59	0.00	1.00	0.00	23.45	178.23	1.74	0.00	1.00	0.00
23.49	182.79	1.86	0.00	1.00	0.00	23.56	195.38	2.00	0.00	1.00	0.00
23.63	204.87	2.00	0.00	1.00	0.00	23.71	209.68	2.00	0.00	1.00	0.00
23.78	211.88	2.00	0.00	1.00	0.00	23.85	200.29	2.00	0.00	1.00	0.00
23.92	194.84	2.00	0.00	1.00	0.00	23.95	191.83	2.00	0.00	1.00	0.00
24.03	186.63	1.94	0.00	1.00	0.00	24.10	185.01	1.89	0.00	1.00	0.00
24.17	174.26	1.61	0.00	1.00	0.00	24.21	165.94	1.42	0.00	1.00	0.00
24.28	161.56	1.33	0.21	1.00	0.00	24.36	154.02	1.18	0.30	1.00	0.00
24.43	157.34	1.24	0.30	1.00	0.00	24.50	162.60	1.34	0.20	1.00	0.00
24.58	162.26	1.33	0.20	1.00	0.00	24.61	162.08	1.33	0.21	1.00	0.00
24.69	169.05	1.48	0.00	1.00	0.00	24.75	174.41	1.60	0.00	1.00	0.00
24.83	180.52	1.74	0.00	1.00	0.00	24.87	184.89	1.86	0.00	1.00	0.00
24.94	193.46	2.00	0.00	1.00	0.00	25.01	195.13	2.00	0.00	1.00	0.00
25.08	191.22	2.00	0.00	1.00	0.00	25.17	185.21	1.85	0.00	1.00	0.00
25.20	183.39	1.80	0.00	1.00	0.00	25.27	180.92	1.74	0.00	1.00	0.00
25.34	177.03	1.64	0.00	1.00	0.00	25.42	174.70	1.58	0.00	1.00	0.00
25.49	172.90	1.54	0.00	1.00	0.00	25.56	171.57	1.51	0.00	1.00	0.00
25.62	172.33	1.52	0.00	1.00	0.00	25.67	173.58	1.55	0.00	1.00	0.00
25.73	176.51	1.62	0.00	1.00	0.00	25.81	173.74	1.55	0.00	1.00	0.00
25.87	167.32	1.41	0.00	1.00	0.00	25.95	166.08	1.38	0.00	1.00	0.00
25.99	162.61	1.30	0.20	1.00	0.00	26.06	156.98	1.19	0.30	1.00	0.00
26.13	150.67	1.08	0.42	1.00	0.00	26.20	151.44	1.09	0.42	1.00	0.00
26.28	146.47	1.01	0.62	1.00	0.01	26.33	139.22	0.89	0.96	1.00	0.01
26.39	132.59	0.80	1.35	1.00	0.01	26.46	126.83	0.73	1.75	1.00	0.01
26.52	120.47	0.65	1.89	1.00	0.01	26.61	115.46	0.60	2.08	1.00	0.02
26.68	115.27	0.60	2.08	1.00	0.02	26.71	115.21	0.60	2.08	1.00	0.01
26.79	111.02	0.56	2.14	1.00	0.02	26.86	108.85	0.54	2.18	1.00	0.02
26.93	106.08	0.51	2.23	1.00	0.02	26.99	106.60	0.51	2.22	1.00	0.02
27.07	108.69	0.53	2.18	1.00	0.02	27.12	111.04	0.55	2.14	1.00	0.01
27.19	123.99	0.68	1.81	1.00	0.01	27.24	132.49	0.79	1.35	1.00	0.01

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
27.30	138.80	0.87	0.97	1.00	0.01	27.37	158.85	1.20	0.29	1.00	0.00
27.43	175.38	1.54	0.00	1.00	0.00	27.51	186.53	1.81	0.00	1.00	0.00
27.59	194.34	2.00	0.00	1.00	0.00	27.65	199.65	2.00	0.00	1.00	0.00
27.72	204.00	2.00	0.00	1.00	0.00	27.79	207.87	2.00	0.00	1.00	0.00
27.82	209.61	2.00	0.00	1.00	0.00	27.90	213.39	2.00	0.00	1.00	0.00
27.97	215.84	2.00	0.00	1.00	0.00	28.03	217.12	2.00	0.00	1.00	0.00
28.10	218.31	2.00	0.00	1.00	0.00	28.16	219.52	2.00	0.00	1.00	0.00
28.23	220.62	2.00	0.00	1.00	0.00	28.31	222.03	2.00	0.00	1.00	0.00
28.38	224.62	2.00	0.00	1.00	0.00	28.44	227.29	2.00	0.00	1.00	0.00
28.48	229.08	2.00	0.00	1.00	0.00	28.55	231.52	2.00	0.00	1.00	0.00
28.62	233.55	2.00	0.00	1.00	0.00	28.68	237.69	2.00	0.00	1.00	0.00
28.75	241.12	2.00	0.00	1.00	0.00	28.82	244.63	2.00	0.00	1.00	0.00
28.90	246.62	2.00	0.00	1.00	0.00	28.96	248.20	2.00	0.00	1.00	0.00
29.03	248.82	2.00	0.00	1.00	0.00	29.10	248.27	2.00	0.00	1.00	0.00
29.16	248.20	2.00	0.00	1.00	0.00	29.21	249.03	2.00	0.00	1.00	0.00
29.27	250.88	2.00	0.00	1.00	0.00	29.34	251.74	2.00	0.00	1.00	0.00
29.40	253.60	2.00	0.00	1.00	0.00	29.48	255.99	2.00	0.00	1.00	0.00
29.54	258.07	2.00	0.00	1.00	0.00	29.61	260.85	2.00	0.00	1.00	0.00
29.68	265.01	2.00	0.00	1.00	0.00	29.74	266.78	2.00	0.00	1.00	0.00
29.81	265.84	2.00	0.00	1.00	0.00	29.89	261.73	2.00	0.00	1.00	0.00
29.92	260.18	2.00	0.00	1.00	0.00	29.99	255.94	2.00	0.00	1.00	0.00
30.05	252.33	2.00	0.00	1.00	0.00	30.13	249.92	2.00	0.00	1.00	0.00
30.21	240.68	2.00	0.00	1.00	0.00	30.27	249.84	2.00	0.00	1.00	0.00
30.34	246.46	2.00	0.00	1.00	0.00	30.40	249.69	2.00	0.00	1.00	0.00
30.46	252.78	2.00	0.00	1.00	0.00	30.53	255.61	2.00	0.00	1.00	0.00
30.60	257.29	2.00	0.00	1.00	0.00	30.67	258.67	2.00	0.00	1.00	0.00
30.73	258.73	2.00	0.00	1.00	0.00	30.81	257.41	2.00	0.00	1.00	0.00
30.84	257.01	2.00	0.00	1.00	0.00	30.91	254.93	2.00	0.00	1.00	0.00
30.98	257.02	2.00	0.00	1.00	0.00	31.04	270.28	2.00	0.00	1.00	0.00
31.11	279.11	2.00	0.00	1.00	0.00	31.17	286.22	2.00	0.00	1.00	0.00
31.25	296.06	2.00	0.00	1.00	0.00	31.30	316.81	2.00	0.00	1.00	0.00
31.38	356.52	2.00	0.00	1.00	0.00	31.43	368.11	2.00	0.00	1.00	0.00
31.51	369.04	2.00	0.00	1.00	0.00	31.56	310.33	2.00	0.00	1.00	0.00
31.64	289.40	2.00	0.00	1.00	0.00	31.70	301.26	2.00	0.00	1.00	0.00
31.77	317.11	2.00	0.00	1.00	0.00	31.84	329.96	2.00	0.00	1.00	0.00
31.91	347.60	2.00	0.00	1.00	0.00	31.97	361.44	2.00	0.00	1.00	0.00
32.04	366.44	2.00	0.00	1.00	0.00	32.10	363.05	2.00	0.00	1.00	0.00
32.17	353.54	2.00	0.00	1.00	0.00	32.23	327.16	2.00	0.00	1.00	0.00
32.30	309.28	2.00	0.00	1.00	0.00	32.36	299.49	2.00	0.00	1.00	0.00
32.43	278.91	2.00	0.00	1.00	0.00	32.49	259.57	2.00	0.00	1.00	0.00
32.55	255.73	2.00	0.00	1.00	0.00	32.61	255.72	2.00	0.00	1.00	0.00
32.68	265.08	2.00	0.00	1.00	0.00	32.76	282.67	2.00	0.00	1.00	0.00
32.82	295.25	2.00	0.00	1.00	0.00	32.90	314.48	2.00	0.00	1.00	0.00
32.95	327.71	2.00	0.00	1.00	0.00	33.02	351.96	2.00	0.00	1.00	0.00
33.08	359.44	2.00	0.00	1.00	0.00	33.16	372.53	2.00	0.00	1.00	0.00
33.21	396.45	2.00	0.00	1.00	0.00	33.27	442.78	2.00	0.00	1.00	0.00
33.33	476.29	2.00	0.00	1.00	0.00	33.40	485.72	2.00	0.00	1.00	0.00
33.47	514.80	2.00	0.00	1.00	0.00	33.53	548.27	2.00	0.00	1.00	0.00

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
33.61	565.38	2.00	0.00	1.00	0.00	33.66	524.05	2.00	0.00	1.00	0.00
33.75	532.38	2.00	0.00	1.00	0.00	33.79	518.40	2.00	0.00	1.00	0.00
33.88	545.02	2.00	0.00	1.00	0.00	33.94	548.95	2.00	0.00	1.00	0.00
34.00	486.44	2.00	0.00	1.00	0.00	34.06	448.68	2.00	0.00	1.00	0.00
34.12	427.98	2.00	0.00	1.00	0.00	34.20	394.95	2.00	0.00	1.00	0.00
34.27	376.16	2.00	0.00	1.00	0.00	34.33	376.66	2.00	0.00	1.00	0.00
34.40	377.15	2.00	0.00	1.00	0.00	34.46	377.64	2.00	0.00	1.00	0.00
34.52	378.15	2.00	0.00	1.00	0.00	34.59	402.96	2.00	0.00	1.00	0.00
34.66	414.62	2.00	0.00	1.00	0.00	34.73	444.71	2.00	0.00	1.00	0.00
34.78	457.58	2.00	0.00	1.00	0.00	34.85	535.18	2.00	0.00	1.00	0.00
34.91	562.22	2.00	0.00	1.00	0.00	34.94	701.96	2.00	0.00	1.00	0.00

Total estimated settlement: 0.31

Abbreviations

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

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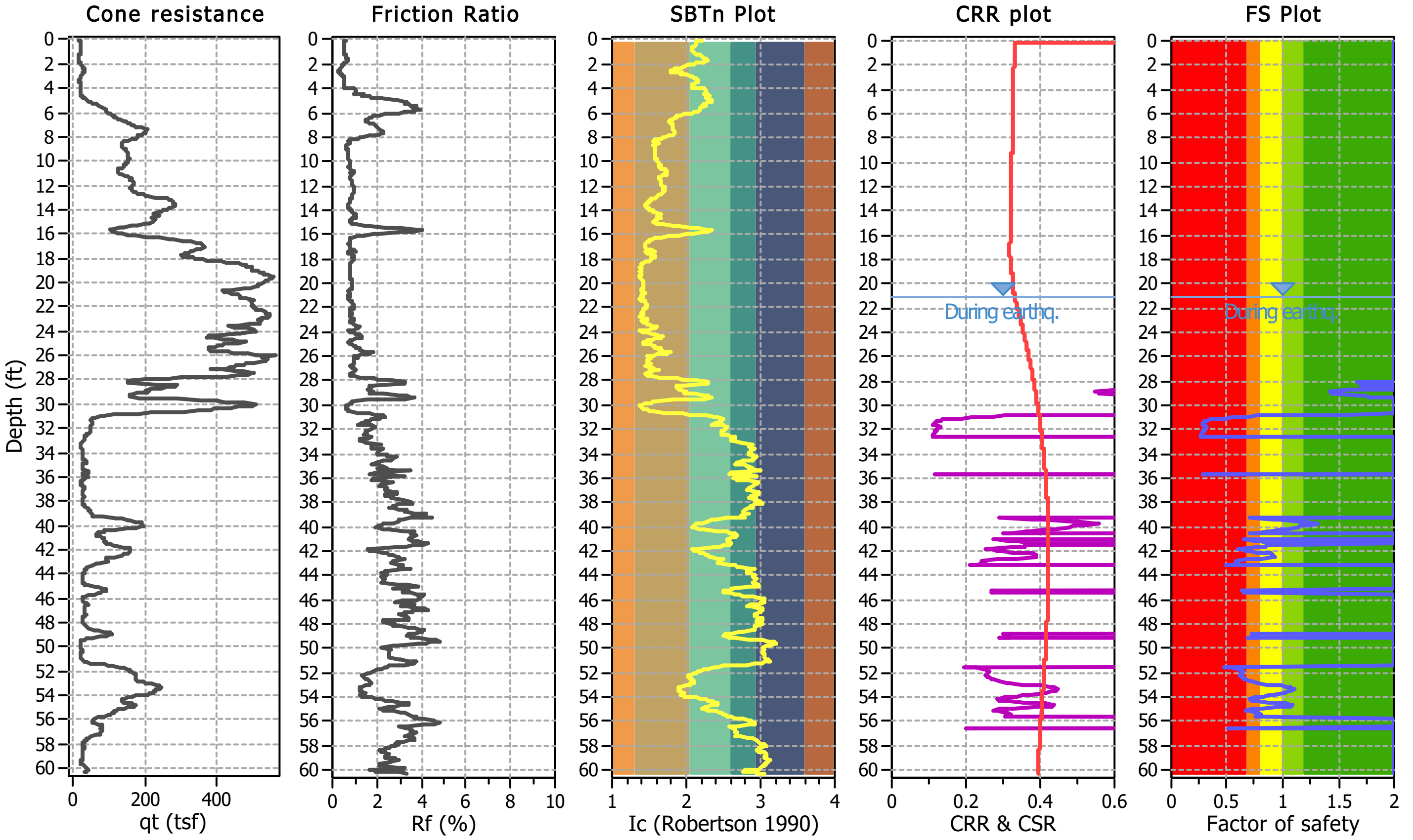
LIQUEFACTION ANALYSIS REPORT

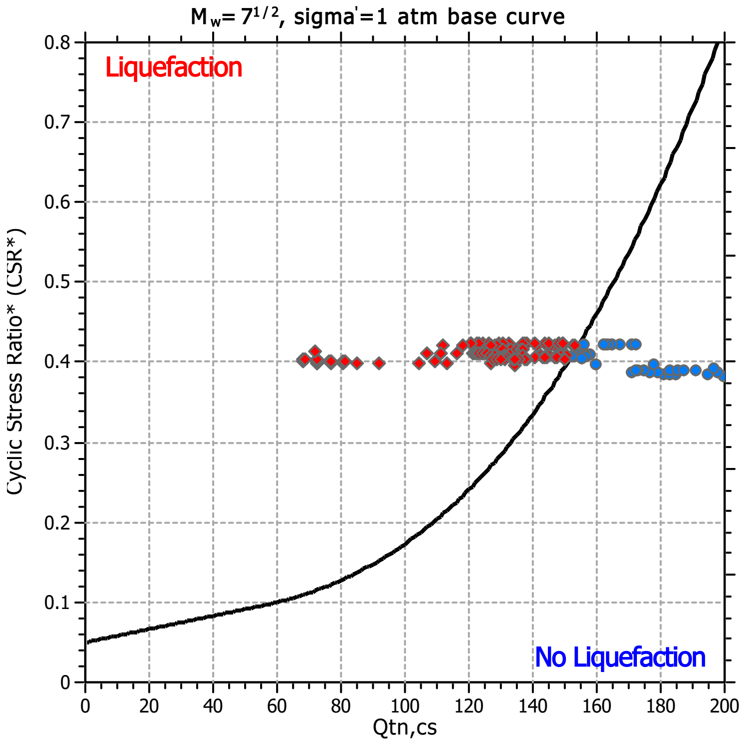
Project title : Proposed Palomino Business Park   Location : Norco, California

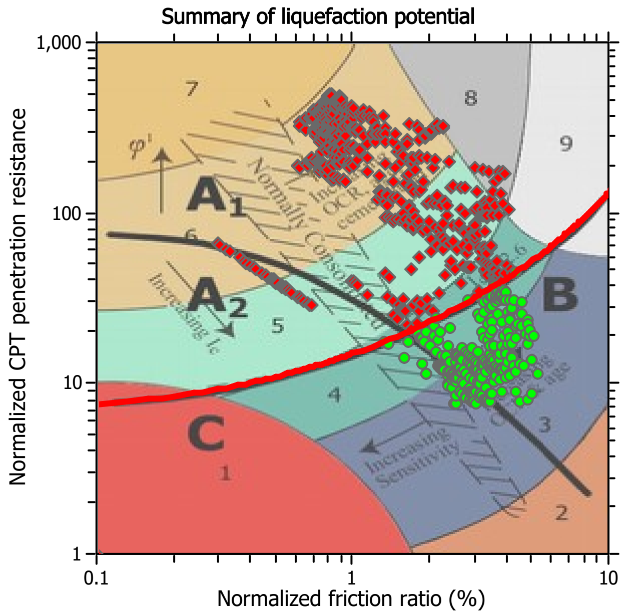
CPT file : CPT-3

Input parameters and analysis data

Analysis method:	**NCEER (1998)**	**G.W.T. (in-situ):**	**28.00 ft**	**Use fill:**	**No**	**Clay like behavior applied:**	**Sands only**
Fines correction method:	**NCEER (1998)**	**G.W.T. (earthq.):**	**21.00 ft**	**Fill height:**	**N/A**	**Limit depth applie**	**No**
Points to test:	**Based on Ic value**	**Average results interva**	**1**	**Fill weight:**	**N/A**	**Limit depth:**	**N/A**
Earthquake magnitude M_w:	**6.98**	**Ic cut-off value:**	**2.60**	**Trans. detect. applied:**	**No**	**MSF method:**	**Method basec**
Peak ground acceleration:	**0.61**	**Unit weight calculatior**	**Based on SBT**	**K_σ applied:**	**Yes**		

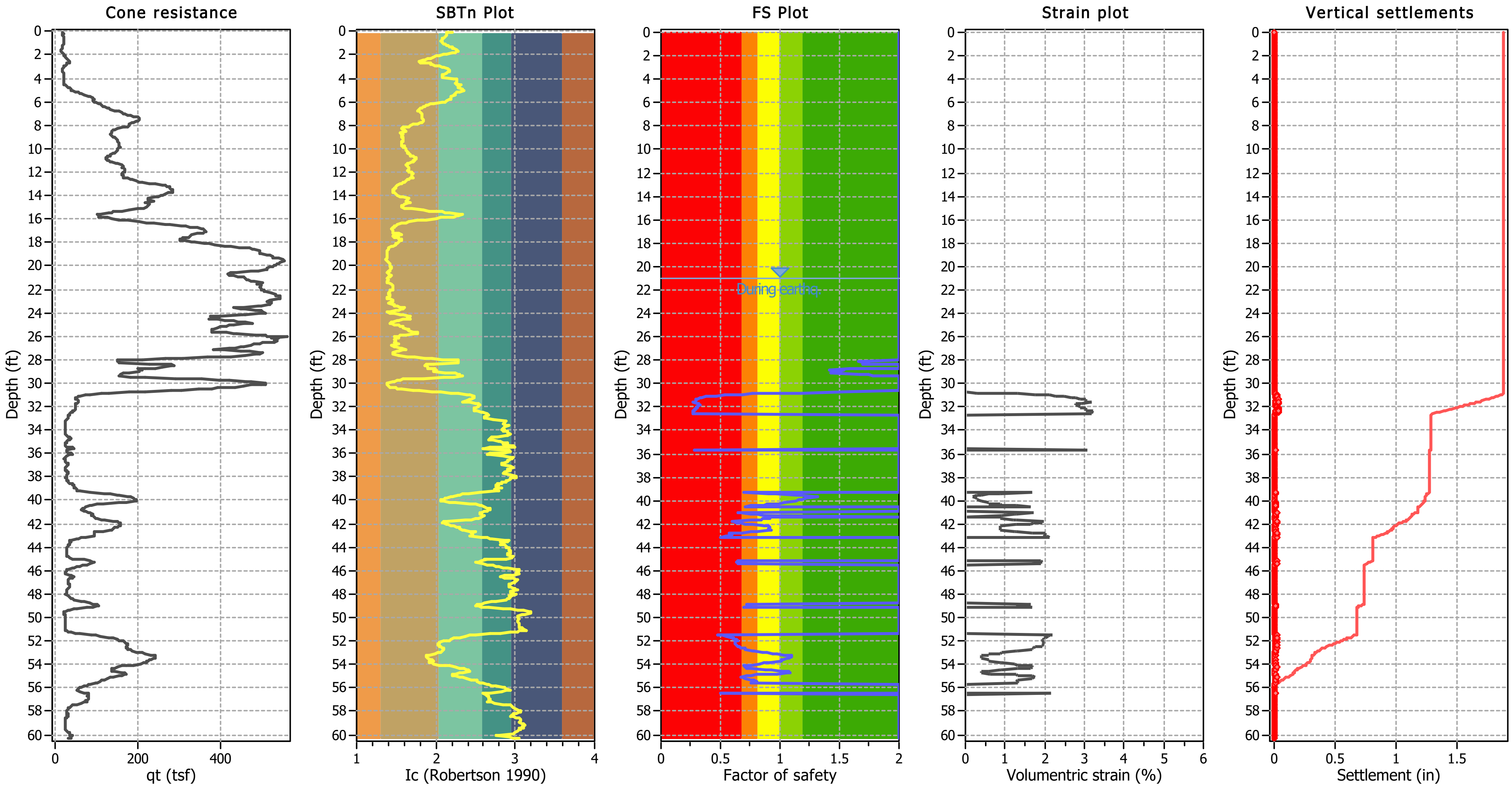






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and 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)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
21.00	389.46	2.00	0.00	1.00	0.00	21.09	392.52	2.00	0.00	1.00	0.00
21.13	393.15	2.00	0.00	1.00	0.00	21.23	394.46	2.00	0.00	1.00	0.00
21.27	399.79	2.00	0.00	1.00	0.00	21.33	405.54	2.00	0.00	1.00	0.00
21.42	423.91	2.00	0.00	1.00	0.00	21.46	417.21	2.00	0.00	1.00	0.00
21.53	411.32	2.00	0.00	1.00	0.00	21.62	418.16	2.00	0.00	1.00	0.00
21.66	418.10	2.00	0.00	1.00	0.00	21.75	414.26	2.00	0.00	1.00	0.00
21.80	414.14	2.00	0.00	1.00	0.00	21.89	416.86	2.00	0.00	1.00	0.00
21.94	413.99	2.00	0.00	1.00	0.00	22.03	414.47	2.00	0.00	1.00	0.00
22.07	415.75	2.00	0.00	1.00	0.00	22.12	418.71	2.00	0.00	1.00	0.00
22.20	417.96	2.00	0.00	1.00	0.00	22.25	421.41	2.00	0.00	1.00	0.00
22.33	431.32	2.00	0.00	1.00	0.00	22.39	438.43	2.00	0.00	1.00	0.00
22.47	438.85	2.00	0.00	1.00	0.00	22.51	438.63	2.00	0.00	1.00	0.00
22.60	444.59	2.00	0.00	1.00	0.00	22.64	444.76	2.00	0.00	1.00	0.00
22.74	444.16	2.00	0.00	1.00	0.00	22.77	444.67	2.00	0.00	1.00	0.00
22.86	438.44	2.00	0.00	1.00	0.00	22.91	423.35	2.00	0.00	1.00	0.00
23.00	415.29	2.00	0.00	1.00	0.00	23.04	422.95	2.00	0.00	1.00	0.00
23.13	422.93	2.00	0.00	1.00	0.00	23.18	420.29	2.00	0.00	1.00	0.00
23.27	419.29	2.00	0.00	1.00	0.00	23.32	416.30	2.00	0.00	1.00	0.00
23.41	400.84	2.00	0.00	1.00	0.00	23.45	393.20	2.00	0.00	1.00	0.00
23.50	365.11	2.00	0.00	1.00	0.00	23.58	344.57	2.00	0.00	1.00	0.00
23.66	358.74	2.00	0.00	1.00	0.00	23.69	367.66	2.00	0.00	1.00	0.00
23.78	384.62	2.00	0.00	1.00	0.00	23.83	397.05	2.00	0.00	1.00	0.00
23.89	399.81	2.00	0.00	1.00	0.00	23.96	399.18	2.00	0.00	1.00	0.00
24.03	401.89	2.00	0.00	1.00	0.00	24.08	392.08	2.00	0.00	1.00	0.00
24.15	363.65	2.00	0.00	1.00	0.00	24.22	357.57	2.00	0.00	1.00	0.00
24.29	317.36	2.00	0.00	1.00	0.00	24.36	300.25	2.00	0.00	1.00	0.00
24.45	311.00	2.00	0.00	1.00	0.00	24.48	295.28	2.00	0.00	1.00	0.00
24.57	307.41	2.00	0.00	1.00	0.00	24.61	321.64	2.00	0.00	1.00	0.00
24.69	336.83	2.00	0.00	1.00	0.00	24.76	344.68	2.00	0.00	1.00	0.00
24.83	358.01	2.00	0.00	1.00	0.00	24.90	370.52	2.00	0.00	1.00	0.00
24.97	354.75	2.00	0.00	1.00	0.00	25.02	333.46	2.00	0.00	1.00	0.00
25.10	324.84	2.00	0.00	1.00	0.00	25.13	318.73	2.00	0.00	1.00	0.00
25.22	308.74	2.00	0.00	1.00	0.00	25.27	303.29	2.00	0.00	1.00	0.00
25.35	293.69	2.00	0.00	1.00	0.00	25.39	290.45	2.00	0.00	1.00	0.00
25.47	289.31	2.00	0.00	1.00	0.00	25.54	291.51	2.00	0.00	1.00	0.00
25.61	300.62	2.00	0.00	1.00	0.00	25.70	336.14	2.00	0.00	1.00	0.00
25.74	347.43	2.00	0.00	1.00	0.00	25.79	348.72	2.00	0.00	1.00	0.00
25.87	367.88	2.00	0.00	1.00	0.00	25.95	385.88	2.00	0.00	1.00	0.00
25.99	387.97	2.00	0.00	1.00	0.00	26.06	425.32	2.00	0.00	1.00	0.00
26.12	395.73	2.00	0.00	1.00	0.00	26.19	407.48	2.00	0.00	1.00	0.00
26.25	400.23	2.00	0.00	1.00	0.00	26.33	405.31	2.00	0.00	1.00	0.00
26.39	398.82	2.00	0.00	1.00	0.00	26.46	403.18	2.00	0.00	1.00	0.00
26.52	397.42	2.00	0.00	1.00	0.00	26.59	376.34	2.00	0.00	1.00	0.00
26.66	369.08	2.00	0.00	1.00	0.00	26.71	359.73	2.00	0.00	1.00	0.00
26.78	352.35	2.00	0.00	1.00	0.00	26.87	338.55	2.00	0.00	1.00	0.00
26.91	332.80	2.00	0.00	1.00	0.00	26.98	321.86	2.00	0.00	1.00	0.00
27.06	313.44	2.00	0.00	1.00	0.00	27.11	283.04	2.00	0.00	1.00	0.00
27.19	300.38	2.00	0.00	1.00	0.00	27.26	331.42	2.00	0.00	1.00	0.00

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
27.30	347.86	2.00	0.00	1.00	0.00	27.38	364.71	2.00	0.00	1.00	0.00
27.45	369.44	2.00	0.00	1.00	0.00	27.52	364.87	2.00	0.00	1.00	0.00
27.58	354.32	2.00	0.00	1.00	0.00	27.65	337.92	2.00	0.00	1.00	0.00
27.71	311.86	2.00	0.00	1.00	0.00	27.78	280.25	2.00	0.00	1.00	0.00
27.84	248.43	2.00	0.00	1.00	0.00	27.91	226.46	2.00	0.00	1.00	0.00
27.98	199.92	2.00	0.00	1.00	0.00	28.05	183.36	1.71	0.00	1.00	0.00
28.09	181.38	1.66	0.00	1.00	0.00	28.17	183.23	1.70	0.00	1.00	0.00
28.25	185.06	1.74	0.00	1.00	0.00	28.29	183.10	1.70	0.00	1.00	0.00
28.36	194.86	2.00	0.00	1.00	0.00	28.43	224.92	2.00	0.00	1.00	0.00
28.48	230.28	2.00	0.00	1.00	0.00	28.55	226.04	2.00	0.00	1.00	0.00
28.63	211.73	2.00	0.00	1.00	0.00	28.71	198.29	2.00	0.00	1.00	0.00
28.75	179.16	1.59	0.00	1.00	0.00	28.82	171.52	1.42	0.00	1.00	0.00
28.90	177.16	1.54	0.00	1.00	0.00	28.94	174.80	1.49	0.00	1.00	0.00
29.01	173.28	1.45	0.00	1.00	0.00	29.09	172.30	1.43	0.00	1.00	0.00
29.17	182.99	1.67	0.00	1.00	0.00	29.21	185.85	1.74	0.00	1.00	0.00
29.29	187.50	1.78	0.00	1.00	0.00	29.36	191.15	1.87	0.00	1.00	0.00
29.40	196.67	2.00	0.00	1.00	0.00	29.48	202.99	2.00	0.00	1.00	0.00
29.55	207.94	2.00	0.00	1.00	0.00	29.62	225.71	2.00	0.00	1.00	0.00
29.69	254.54	2.00	0.00	1.00	0.00	29.76	282.88	2.00	0.00	1.00	0.00
29.80	300.39	2.00	0.00	1.00	0.00	29.87	331.13	2.00	0.00	1.00	0.00
29.93	339.45	2.00	0.00	1.00	0.00	30.00	363.41	2.00	0.00	1.00	0.00
30.08	362.99	2.00	0.00	1.00	0.00	30.12	355.13	2.00	0.00	1.00	0.00
30.19	337.85	2.00	0.00	1.00	0.00	30.26	324.92	2.00	0.00	1.00	0.00
30.33	309.39	2.00	0.00	1.00	0.00	30.41	279.34	2.00	0.00	1.00	0.00
30.45	269.18	2.00	0.00	1.00	0.00	30.53	245.51	2.00	0.00	1.00	0.00
30.61	222.64	2.00	0.00	1.00	0.00	30.65	212.68	2.00	0.00	1.00	0.00
30.73	178.28	1.53	0.00	1.00	0.00	30.79	160.27	1.17	0.29	1.00	0.00
30.88	134.38	0.77	1.32	1.00	0.01	30.92	127.09	0.68	1.75	1.00	0.01
31.00	112.88	0.54	2.12	1.00	0.02	31.05	104.21	0.47	2.26	1.00	0.01
31.14	91.83	0.38	2.51	1.00	0.03	31.18	84.71	0.34	2.68	1.00	0.01
31.23	80.85	0.32	2.78	1.00	0.02	31.31	76.96	0.31	2.90	1.00	0.03
31.39	72.63	0.29	3.04	1.00	0.03	31.43	72.51	0.29	3.04	1.00	0.01
31.53	72.41	0.29	3.04	1.00	0.04	31.58	68.13	0.27	3.20	1.00	0.02
31.67	76.56	0.30	2.91	1.00	0.03	31.71	80.80	0.32	2.78	1.00	0.02
31.80	81.09	0.32	2.77	1.00	0.03	31.84	81.24	0.32	2.77	1.00	0.01
31.93	77.01	0.30	2.89	1.00	0.03	31.98	77.00	0.30	2.90	1.00	0.02
32.03	76.98	0.30	2.90	1.00	0.02	32.11	72.59	0.29	3.04	1.00	0.03
32.16	72.51	0.29	3.04	1.00	0.02	32.24	72.45	0.29	3.04	1.00	0.03
32.29	72.46	0.29	3.04	1.00	0.02	32.39	68.03	0.27	3.20	1.00	0.04
32.42	68.18	0.27	3.20	1.00	0.02	32.52	68.45	0.27	3.19	1.00	0.04
32.55	68.61	0.27	3.18	1.00	0.01	32.65	68.89	2.00	0.00	1.00	0.00
32.70	69.01	2.00	0.00	1.00	0.00	32.78	64.10	2.00	0.00	1.00	0.00
32.83	58.57	2.00	0.00	1.00	0.00	32.92	58.55	2.00	0.00	1.00	0.00
32.97	58.54	2.00	0.00	1.00	0.00	33.01	58.62	2.00	0.00	1.00	0.00
33.10	58.97	2.00	0.00	1.00	0.00	33.14	58.97	2.00	0.00	1.00	0.00
33.21	64.45	2.00	0.00	1.00	0.00	33.31	59.07	2.00	0.00	1.00	0.00
33.35	59.06	2.00	0.00	1.00	0.00	33.44	59.04	2.00	0.00	1.00	0.00
33.48	59.02	2.00	0.00	1.00	0.00	33.57	59.00	2.00	0.00	1.00	0.00

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
33.62	58.98	2.00	0.00	1.00	0.00	33.66	64.25	2.00	0.00	1.00	0.00
33.75	64.32	2.00	0.00	1.00	0.00	33.80	64.30	2.00	0.00	1.00	0.00
33.89	64.28	2.00	0.00	1.00	0.00	33.94	64.27	2.00	0.00	1.00	0.00
34.03	64.23	2.00	0.00	1.00	0.00	34.08	64.22	2.00	0.00	1.00	0.00
34.12	69.03	2.00	0.00	1.00	0.00	34.22	68.99	2.00	0.00	1.00	0.00
34.27	68.98	2.00	0.00	1.00	0.00	34.32	73.34	2.00	0.00	1.00	0.00
34.43	73.42	2.00	0.00	1.00	0.00	34.47	73.43	2.00	0.00	1.00	0.00
34.51	73.43	2.00	0.00	1.00	0.00	34.61	73.30	2.00	0.00	1.00	0.00
34.66	73.03	2.00	0.00	1.00	0.00	34.71	72.88	2.00	0.00	1.00	0.00
34.81	68.32	2.00	0.00	1.00	0.00	34.86	68.56	2.00	0.00	1.00	0.00
34.95	68.75	2.00	0.00	1.00	0.00	35.01	68.75	2.00	0.00	1.00	0.00
35.06	68.75	2.00	0.00	1.00	0.00	35.15	68.65	2.00	0.00	1.00	0.00
35.20	68.63	2.00	0.00	1.00	0.00	35.24	63.84	2.00	0.00	1.00	0.00
35.34	72.93	2.00	0.00	1.00	0.00	35.39	77.13	2.00	0.00	1.00	0.00
35.44	81.11	2.00	0.00	1.00	0.00	35.53	80.66	2.00	0.00	1.00	0.00
35.59	80.09	2.00	0.00	1.00	0.00	35.63	71.95	0.28	3.06	1.00	0.01
35.73	72.98	2.00	0.00	1.00	0.00	35.78	77.00	2.00	0.00	1.00	0.00
35.88	80.87	2.00	0.00	1.00	0.00	35.94	81.05	2.00	0.00	1.00	0.00
35.99	76.51	2.00	0.00	1.00	0.00	36.04	76.24	2.00	0.00	1.00	0.00
36.09	72.16	2.00	0.00	1.00	0.00	36.15	67.98	2.00	0.00	1.00	0.00
36.22	68.33	2.00	0.00	1.00	0.00	36.30	68.09	2.00	0.00	1.00	0.00
36.35	68.05	2.00	0.00	1.00	0.00	36.44	63.18	2.00	0.00	1.00	0.00
36.49	63.22	2.00	0.00	1.00	0.00	36.58	63.22	2.00	0.00	1.00	0.00
36.63	63.29	2.00	0.00	1.00	0.00	36.72	68.08	2.00	0.00	1.00	0.00
36.78	72.58	2.00	0.00	1.00	0.00	36.82	72.57	2.00	0.00	1.00	0.00
36.92	72.52	2.00	0.00	1.00	0.00	36.97	72.51	2.00	0.00	1.00	0.00
37.01	72.49	2.00	0.00	1.00	0.00	37.11	72.24	2.00	0.00	1.00	0.00
37.16	72.19	2.00	0.00	1.00	0.00	37.25	72.15	2.00	0.00	1.00	0.00
37.30	67.80	2.00	0.00	1.00	0.00	37.35	67.83	2.00	0.00	1.00	0.00
37.44	67.82	2.00	0.00	1.00	0.00	37.49	72.09	2.00	0.00	1.00	0.00
37.54	72.28	2.00	0.00	1.00	0.00	37.63	76.39	2.00	0.00	1.00	0.00
37.68	76.38	2.00	0.00	1.00	0.00	37.77	80.03	2.00	0.00	1.00	0.00
37.82	80.02	2.00	0.00	1.00	0.00	37.87	79.99	2.00	0.00	1.00	0.00
37.96	79.96	2.00	0.00	1.00	0.00	38.00	79.94	2.00	0.00	1.00	0.00
38.10	79.90	2.00	0.00	1.00	0.00	38.14	75.94	2.00	0.00	1.00	0.00
38.19	75.95	2.00	0.00	1.00	0.00	38.28	76.03	2.00	0.00	1.00	0.00
38.33	76.07	2.00	0.00	1.00	0.00	38.43	76.03	2.00	0.00	1.00	0.00
38.47	76.05	2.00	0.00	1.00	0.00	38.52	79.92	2.00	0.00	1.00	0.00
38.62	90.41	2.00	0.00	1.00	0.00	38.66	96.87	2.00	0.00	1.00	0.00
38.75	96.69	2.00	0.00	1.00	0.00	38.81	99.74	2.00	0.00	1.00	0.00
38.89	105.65	2.00	0.00	1.00	0.00	38.95	111.34	2.00	0.00	1.00	0.00
38.98	111.20	2.00	0.00	1.00	0.00	39.08	108.13	2.00	0.00	1.00	0.00
39.12	108.11	2.00	0.00	1.00	0.00	39.21	119.05	2.00	0.00	1.00	0.00
39.26	128.73	2.00	0.00	1.00	0.00	39.31	131.59	0.69	1.66	1.00	0.01
39.40	147.86	0.90	0.88	1.00	0.01	39.44	152.23	0.97	0.60	1.00	0.00
39.54	162.87	1.14	0.40	1.00	0.00	39.58	164.97	1.18	0.29	1.00	0.00
39.67	171.24	1.30	0.20	1.00	0.00	39.71	172.28	1.32	0.20	1.00	0.00
39.81	167.33	1.22	0.28	1.00	0.00	39.86	164.92	1.18	0.29	1.00	0.00

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
39.90	163.96	1.16	0.29	1.00	0.00	39.99	162.30	1.13	0.40	1.00	0.00
40.04	162.44	1.14	0.40	1.00	0.00	40.09	162.24	1.13	0.40	1.00	0.00
40.18	156.55	1.04	0.58	1.00	0.01	40.23	153.39	0.99	0.59	1.00	0.00
40.31	146.98	0.89	0.89	1.00	0.01	40.36	145.33	0.87	0.90	1.00	0.00
40.45	137.24	0.76	1.28	1.00	0.01	40.50	132.86	0.71	1.64	1.00	0.01
40.59	123.85	2.00	0.00	1.00	0.00	40.63	121.86	2.00	0.00	1.00	0.00
40.72	119.54	2.00	0.00	1.00	0.00	40.76	119.64	2.00	0.00	1.00	0.00
40.82	116.88	2.00	0.00	1.00	0.00	40.91	121.52	2.00	0.00	1.00	0.00
40.95	123.60	2.00	0.00	1.00	0.00	41.05	127.88	0.65	1.73	1.00	0.02
41.09	129.92	0.67	1.70	1.00	0.01	41.17	136.56	0.75	1.29	1.00	0.01
41.22	141.08	0.81	1.23	1.00	0.01	41.31	145.44	0.87	0.90	1.00	0.01
41.36	149.66	2.00	0.00	1.00	0.00	41.45	149.19	0.92	0.87	1.00	0.01
41.50	148.84	0.92	0.87	1.00	0.00	41.54	144.30	0.85	0.91	1.00	0.01
41.63	143.77	0.84	1.20	1.00	0.01	41.68	137.38	0.76	1.28	1.00	0.01
41.77	122.30	0.59	1.98	1.00	0.02	41.82	124.29	0.61	1.96	1.00	0.01
41.91	126.11	0.63	1.93	1.00	0.02	41.95	129.60	0.67	1.70	1.00	0.01
42.00	132.80	0.70	1.64	1.00	0.01	42.08	138.41	0.77	1.26	1.00	0.01
42.17	145.05	0.86	0.90	1.00	0.01	42.21	147.96	0.90	0.88	1.00	0.00
42.26	149.16	0.92	0.87	1.00	0.00	42.35	148.16	0.91	0.88	1.00	0.01
42.40	147.94	0.90	0.88	1.00	0.01	42.49	149.47	0.92	0.86	1.00	0.01
42.54	149.20	0.92	0.87	1.00	0.00	42.63	140.73	0.80	1.23	1.00	0.01
42.67	136.70	0.75	1.29	1.00	0.01	42.72	130.40	0.68	1.69	1.00	0.01
42.82	120.91	0.58	2.00	1.00	0.02	42.85	120.87	0.58	2.00	1.00	0.01
42.95	123.16	0.60	1.97	1.00	0.02	43.00	120.74	0.58	2.00	1.00	0.01
43.05	118.30	0.55	2.04	1.00	0.01	43.15	111.94	0.50	2.13	1.00	0.02
43.20	107.30	2.00	0.00	1.00	0.00	43.26	102.18	2.00	0.00	1.00	0.00
43.31	96.62	2.00	0.00	1.00	0.00	43.42	87.69	2.00	0.00	1.00	0.00
43.47	87.46	2.00	0.00	1.00	0.00	43.52	84.27	2.00	0.00	1.00	0.00
43.58	77.44	2.00	0.00	1.00	0.00	43.68	73.67	2.00	0.00	1.00	0.00
43.73	73.64	2.00	0.00	1.00	0.00	43.79	69.70	2.00	0.00	1.00	0.00
43.84	69.63	2.00	0.00	1.00	0.00	43.94	69.53	2.00	0.00	1.00	0.00
43.99	69.43	2.00	0.00	1.00	0.00	44.04	65.29	2.00	0.00	1.00	0.00
44.10	65.16	2.00	0.00	1.00	0.00	44.21	65.07	2.00	0.00	1.00	0.00
44.25	65.05	2.00	0.00	1.00	0.00	44.30	65.00	2.00	0.00	1.00	0.00
44.36	65.01	2.00	0.00	1.00	0.00	44.47	64.97	2.00	0.00	1.00	0.00
44.52	64.95	2.00	0.00	1.00	0.00	44.58	65.05	2.00	0.00	1.00	0.00
44.62	65.09	2.00	0.00	1.00	0.00	44.73	72.91	2.00	0.00	1.00	0.00
44.78	76.62	2.00	0.00	1.00	0.00	44.82	80.09	2.00	0.00	1.00	0.00
44.88	89.75	2.00	0.00	1.00	0.00	44.98	98.85	2.00	0.00	1.00	0.00
45.01	100.72	2.00	0.00	1.00	0.00	45.09	113.81	2.00	0.00	1.00	0.00
45.19	127.12	0.64	1.92	1.00	0.02	45.24	126.71	0.64	1.92	1.00	0.01
45.30	128.64	0.66	1.72	1.00	0.01	45.34	128.61	0.66	1.72	1.00	0.01
45.44	127.02	0.64	1.92	1.00	0.02	45.50	127.44	2.00	0.00	1.00	0.00
45.54	125.66	2.00	0.00	1.00	0.00	45.66	116.47	2.00	0.00	1.00	0.00
45.70	109.13	2.00	0.00	1.00	0.00	45.75	101.20	2.00	0.00	1.00	0.00
45.80	95.43	2.00	0.00	1.00	0.00	45.91	86.18	2.00	0.00	1.00	0.00
45.96	82.92	2.00	0.00	1.00	0.00	46.01	79.53	2.00	0.00	1.00	0.00
46.12	72.34	2.00	0.00	1.00	0.00	46.16	68.47	2.00	0.00	1.00	0.00

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
46.22	68.45	2.00	0.00	1.00	0.00	46.27	72.28	2.00	0.00	1.00	0.00
46.37	85.93	2.00	0.00	1.00	0.00	46.43	89.34	2.00	0.00	1.00	0.00
46.47	89.41	2.00	0.00	1.00	0.00	46.53	89.38	2.00	0.00	1.00	0.00
46.62	89.26	2.00	0.00	1.00	0.00	46.68	88.84	2.00	0.00	1.00	0.00
46.73	88.89	2.00	0.00	1.00	0.00	46.83	88.82	2.00	0.00	1.00	0.00
46.88	85.73	2.00	0.00	1.00	0.00	46.93	82.51	2.00	0.00	1.00	0.00
47.03	82.47	2.00	0.00	1.00	0.00	47.08	79.09	2.00	0.00	1.00	0.00
47.14	79.10	2.00	0.00	1.00	0.00	47.19	79.25	2.00	0.00	1.00	0.00
47.29	82.46	2.00	0.00	1.00	0.00	47.34	82.45	2.00	0.00	1.00	0.00
47.39	82.31	2.00	0.00	1.00	0.00	47.49	78.93	2.00	0.00	1.00	0.00
47.54	78.95	2.00	0.00	1.00	0.00	47.59	75.46	2.00	0.00	1.00	0.00
47.65	75.41	2.00	0.00	1.00	0.00	47.75	63.88	2.00	0.00	1.00	0.00
47.81	63.86	2.00	0.00	1.00	0.00	47.85	67.88	2.00	0.00	1.00	0.00
47.91	63.84	2.00	0.00	1.00	0.00	48.01	67.84	2.00	0.00	1.00	0.00
48.04	67.83	2.00	0.00	1.00	0.00	48.10	71.60	2.00	0.00	1.00	0.00
48.19	75.40	2.00	0.00	1.00	0.00	48.24	78.71	2.00	0.00	1.00	0.00
48.33	85.15	2.00	0.00	1.00	0.00	48.39	91.10	2.00	0.00	1.00	0.00
48.45	96.80	2.00	0.00	1.00	0.00	48.49	102.22	2.00	0.00	1.00	0.00
48.59	114.65	2.00	0.00	1.00	0.00	48.65	119.20	2.00	0.00	1.00	0.00
48.70	123.46	2.00	0.00	1.00	0.00	48.79	129.27	2.00	0.00	1.00	0.00
48.84	133.08	0.72	1.64	1.00	0.01	48.89	134.69	0.74	1.61	1.00	0.01
49.00	136.44	0.76	1.29	1.00	0.02	49.05	136.63	0.76	1.29	1.00	0.01
49.10	130.79	0.69	1.68	1.00	0.01	49.16	125.06	2.00	0.00	1.00	0.00
49.26	111.79	2.00	0.00	1.00	0.00	49.30	103.87	2.00	0.00	1.00	0.00
49.36	96.11	2.00	0.00	1.00	0.00	49.41	90.26	2.00	0.00	1.00	0.00
49.51	77.30	2.00	0.00	1.00	0.00	49.56	73.70	2.00	0.00	1.00	0.00
49.61	70.13	2.00	0.00	1.00	0.00	49.71	62.81	2.00	0.00	1.00	0.00
49.77	62.88	2.00	0.00	1.00	0.00	49.81	62.97	2.00	0.00	1.00	0.00
49.92	58.72	2.00	0.00	1.00	0.00	49.97	58.71	2.00	0.00	1.00	0.00
50.02	58.69	2.00	0.00	1.00	0.00	50.12	62.98	2.00	0.00	1.00	0.00
50.17	63.01	2.00	0.00	1.00	0.00	50.22	63.01	2.00	0.00	1.00	0.00
50.27	63.02	2.00	0.00	1.00	0.00	50.38	62.97	2.00	0.00	1.00	0.00
50.42	62.95	2.00	0.00	1.00	0.00	50.48	62.92	2.00	0.00	1.00	0.00
50.56	62.89	2.00	0.00	1.00	0.00	50.62	62.87	2.00	0.00	1.00	0.00
50.67	62.86	2.00	0.00	1.00	0.00	50.77	62.83	2.00	0.00	1.00	0.00
50.83	62.82	2.00	0.00	1.00	0.00	50.88	66.71	2.00	0.00	1.00	0.00
50.92	66.66	2.00	0.00	1.00	0.00	51.02	70.33	2.00	0.00	1.00	0.00
51.07	73.71	2.00	0.00	1.00	0.00	51.14	77.47	2.00	0.00	1.00	0.00
51.18	80.72	2.00	0.00	1.00	0.00	51.29	89.70	2.00	0.00	1.00	0.00
51.34	92.98	2.00	0.00	1.00	0.00	51.39	98.10	2.00	0.00	1.00	0.00
51.48	106.95	0.47	2.21	1.00	0.02	51.54	111.40	0.51	2.14	1.00	0.01
51.59	115.98	0.55	2.07	1.00	0.01	51.69	121.19	0.60	2.00	1.00	0.03
51.74	121.95	0.60	1.99	1.00	0.01	51.79	122.80	0.61	1.97	1.00	0.01
51.89	124.56	0.63	1.95	1.00	0.02	51.94	125.37	0.64	1.94	1.00	0.01
51.98	124.72	0.63	1.95	1.00	0.01	52.05	124.08	0.63	1.96	1.00	0.01
52.14	123.06	0.62	1.97	1.00	0.02	52.19	124.01	0.63	1.96	1.00	0.01
52.24	123.27	0.62	1.97	1.00	0.01	52.34	124.58	0.63	1.95	1.00	0.02
52.40	124.20	0.63	1.96	1.00	0.01	52.44	125.35	0.64	1.94	1.00	0.01

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
52.50	126.68	0.66	1.76	1.00	0.01	52.59	128.00	0.67	1.73	1.00	0.02
52.64	131.14	0.71	1.67	1.00	0.01	52.69	131.50	0.71	1.67	1.00	0.01
52.80	135.73	0.76	1.30	1.00	0.02	52.84	136.84	0.78	1.29	1.00	0.01
52.94	142.66	0.86	0.93	1.00	0.01	52.98	144.87	0.89	0.91	1.00	0.00
53.04	148.65	0.94	0.87	1.00	0.01	53.09	150.64	0.97	0.60	1.00	0.00
53.18	154.80	1.04	0.59	1.00	0.01	53.24	156.29	1.07	0.41	1.00	0.00
53.28	157.84	1.09	0.41	1.00	0.00	53.38	158.17	1.10	0.41	1.00	0.00
53.43	156.99	1.08	0.41	1.00	0.00	53.48	155.68	1.06	0.41	1.00	0.00
53.58	153.49	1.02	0.59	1.00	0.01	53.63	153.11	1.02	0.59	1.00	0.00
53.68	153.09	1.02	0.59	1.00	0.00	53.78	146.77	0.92	0.89	1.00	0.01
53.84	144.42	0.88	0.91	1.00	0.01	53.88	143.32	0.87	0.92	1.00	0.01
53.94	140.49	0.83	1.24	1.00	0.01	54.04	134.75	0.76	1.31	1.00	0.02
54.07	130.77	0.71	1.68	1.00	0.01	54.14	129.89	0.70	1.70	1.00	0.01
54.24	131.08	0.71	1.67	1.00	0.02	54.29	131.95	0.72	1.66	1.00	0.01
54.34	135.03	0.76	1.31	1.00	0.01	54.44	143.92	0.88	0.91	1.00	0.01
54.49	147.40	0.93	0.88	1.00	0.00	54.55	152.57	1.01	0.60	1.00	0.00
54.60	156.01	1.07	0.41	1.00	0.00	54.70	156.69	1.08	0.41	1.00	0.01
54.75	155.76	1.07	0.41	1.00	0.00	54.80	149.92	0.97	0.61	1.00	0.00
54.89	138.31	0.81	1.27	1.00	0.01	54.94	133.94	0.75	1.33	1.00	0.01
54.99	131.16	0.72	1.67	1.00	0.01	55.10	127.44	0.67	1.74	1.00	0.02
55.15	128.70	0.69	1.72	1.00	0.01	55.19	130.25	0.71	1.69	1.00	0.01
55.29	133.85	0.75	1.33	1.00	0.02	55.35	135.74	0.77	1.30	1.00	0.01
55.40	137.68	0.80	1.27	1.00	0.01	55.45	135.84	0.78	1.30	1.00	0.01
55.55	134.09	0.75	1.32	1.00	0.02	55.60	134.24	0.76	1.32	1.00	0.01
55.70	126.71	2.00	0.00	1.00	0.00	55.74	122.73	2.00	0.00	1.00	0.00
55.79	120.75	2.00	0.00	1.00	0.00	55.84	118.75	2.00	0.00	1.00	0.00
55.95	118.51	2.00	0.00	1.00	0.00	55.99	116.30	2.00	0.00	1.00	0.00
56.05	113.92	2.00	0.00	1.00	0.00	56.15	111.30	2.00	0.00	1.00	0.00
56.19	109.29	2.00	0.00	1.00	0.00	56.25	109.44	2.00	0.00	1.00	0.00
56.30	109.72	2.00	0.00	1.00	0.00	56.41	109.66	2.00	0.00	1.00	0.00
56.45	109.39	2.00	0.00	1.00	0.00	56.50	109.19	0.50	2.17	1.00	0.01
56.60	115.84	2.00	0.00	1.00	0.00	56.66	115.87	2.00	0.00	1.00	0.00
56.71	118.02	2.00	0.00	1.00	0.00	56.76	117.95	2.00	0.00	1.00	0.00
56.86	117.89	2.00	0.00	1.00	0.00	56.91	119.97	2.00	0.00	1.00	0.00
56.96	122.02	2.00	0.00	1.00	0.00	57.06	117.85	2.00	0.00	1.00	0.00
57.11	113.61	2.00	0.00	1.00	0.00	57.16	111.31	2.00	0.00	1.00	0.00
57.25	106.98	2.00	0.00	1.00	0.00	57.30	102.40	2.00	0.00	1.00	0.00
57.40	97.41	2.00	0.00	1.00	0.00	57.45	92.16	2.00	0.00	1.00	0.00
57.50	89.13	2.00	0.00	1.00	0.00	57.56	83.62	2.00	0.00	1.00	0.00
57.66	77.83	2.00	0.00	1.00	0.00	57.71	74.75	2.00	0.00	1.00	0.00
57.76	71.52	2.00	0.00	1.00	0.00	57.86	68.10	2.00	0.00	1.00	0.00
57.90	68.01	2.00	0.00	1.00	0.00	58.01	64.31	2.00	0.00	1.00	0.00
58.05	64.32	2.00	0.00	1.00	0.00	58.11	64.37	2.00	0.00	1.00	0.00
58.16	64.42	2.00	0.00	1.00	0.00	58.20	64.42	2.00	0.00	1.00	0.00
58.30	64.40	2.00	0.00	1.00	0.00	58.35	60.55	2.00	0.00	1.00	0.00
58.40	60.57	2.00	0.00	1.00	0.00	58.50	60.53	2.00	0.00	1.00	0.00
58.55	60.49	2.00	0.00	1.00	0.00	58.60	60.42	2.00	0.00	1.00	0.00
58.70	60.32	2.00	0.00	1.00	0.00	58.76	60.28	2.00	0.00	1.00	0.00

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
58.80	60.30	2.00	0.00	1.00	0.00	58.91	60.33	2.00	0.00	1.00	0.00
58.95	60.33	2.00	0.00	1.00	0.00	59.01	64.01	2.00	0.00	1.00	0.00
59.11	63.89	2.00	0.00	1.00	0.00	59.16	63.81	2.00	0.00	1.00	0.00
59.20	63.78	2.00	0.00	1.00	0.00	59.26	60.13	2.00	0.00	1.00	0.00
59.36	60.05	2.00	0.00	1.00	0.00	59.40	59.97	2.00	0.00	1.00	0.00
59.46	56.06	2.00	0.00	1.00	0.00	59.56	56.00	2.00	0.00	1.00	0.00
59.61	56.03	2.00	0.00	1.00	0.00	59.71	60.21	2.00	0.00	1.00	0.00
59.81	70.93	2.00	0.00	1.00	0.00	59.86	77.15	2.00	0.00	1.00	0.00
59.91	77.09	2.00	0.00	1.00	0.00	60.02	60.87	2.00	0.00	1.00	0.00
60.06	64.53	2.00	0.00	1.00	0.00	60.10	68.22	2.00	0.00	1.00	0.00
60.20	77.14	2.00	0.00	1.00	0.00	60.25	79.91	2.00	0.00	1.00	0.00
60.31	73.89	2.00	0.00	1.00	0.00						

Total estimated settlement: 1.88

Abbreviations

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

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LIQUEFACTION ANALYSIS REPORT

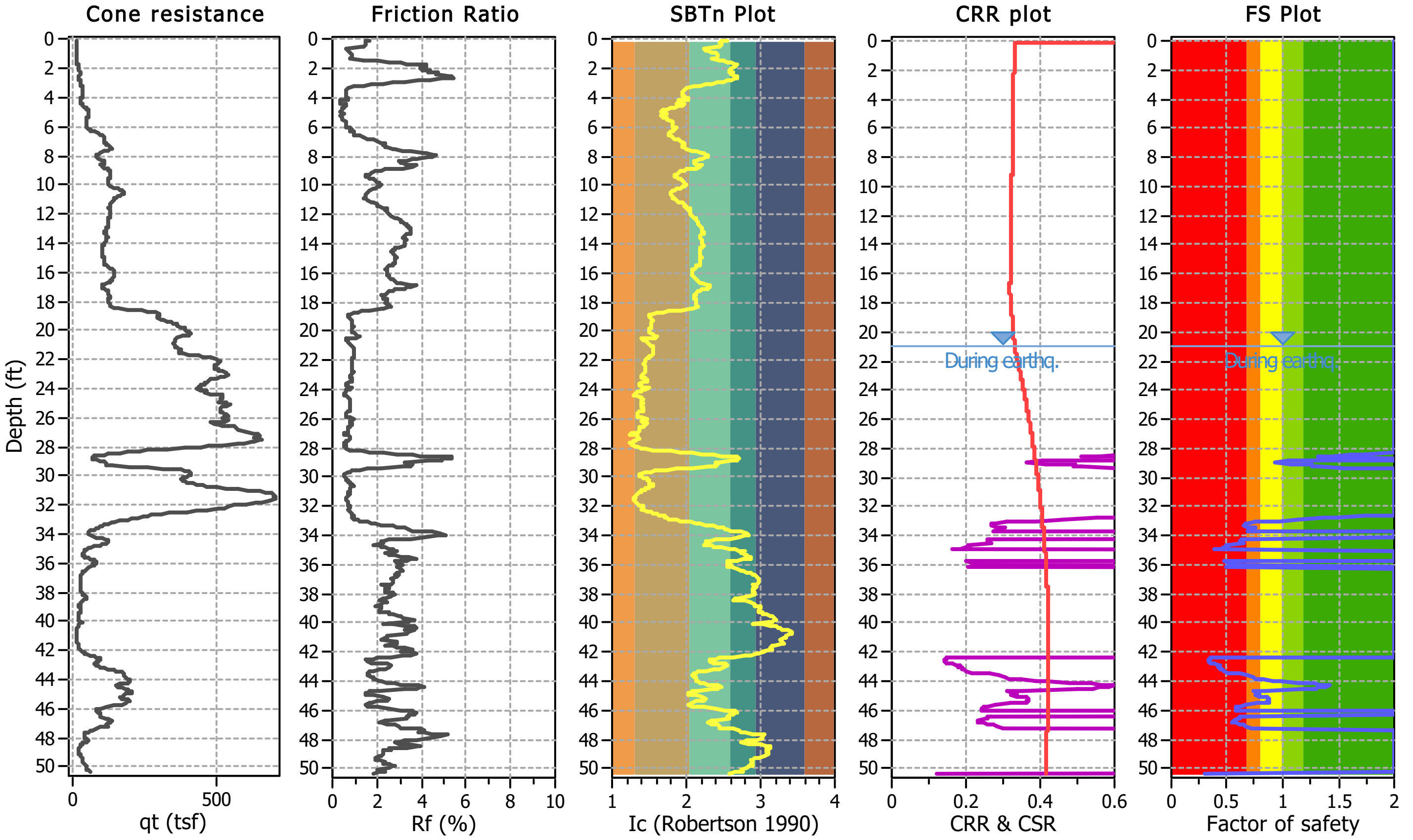
Project title : Proposed Palomino Business Park

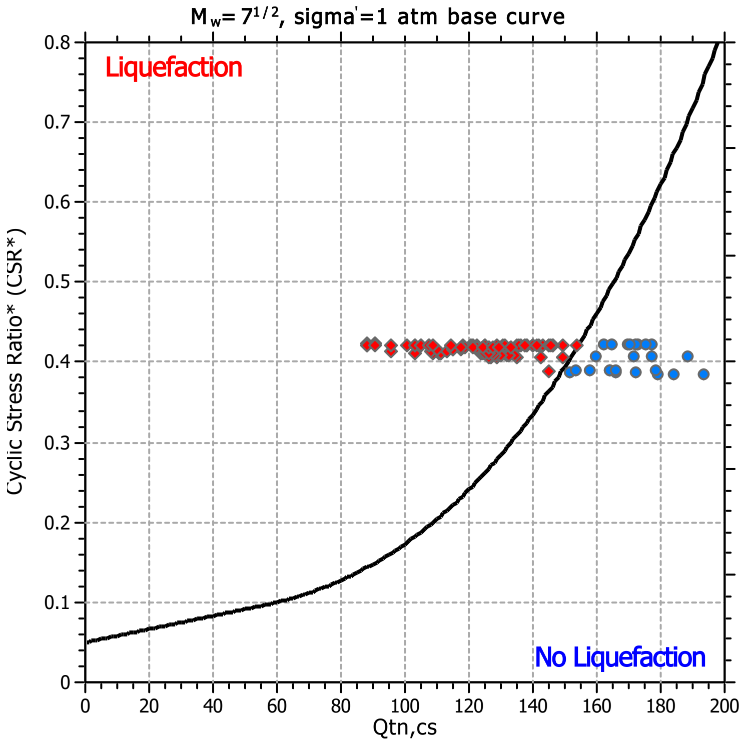
Location : Norco, California

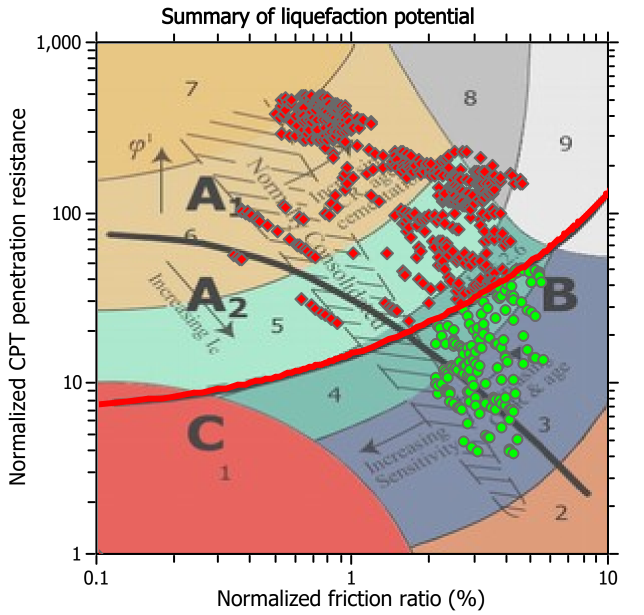
CPT file : CPT-4

Input parameters and analysis data

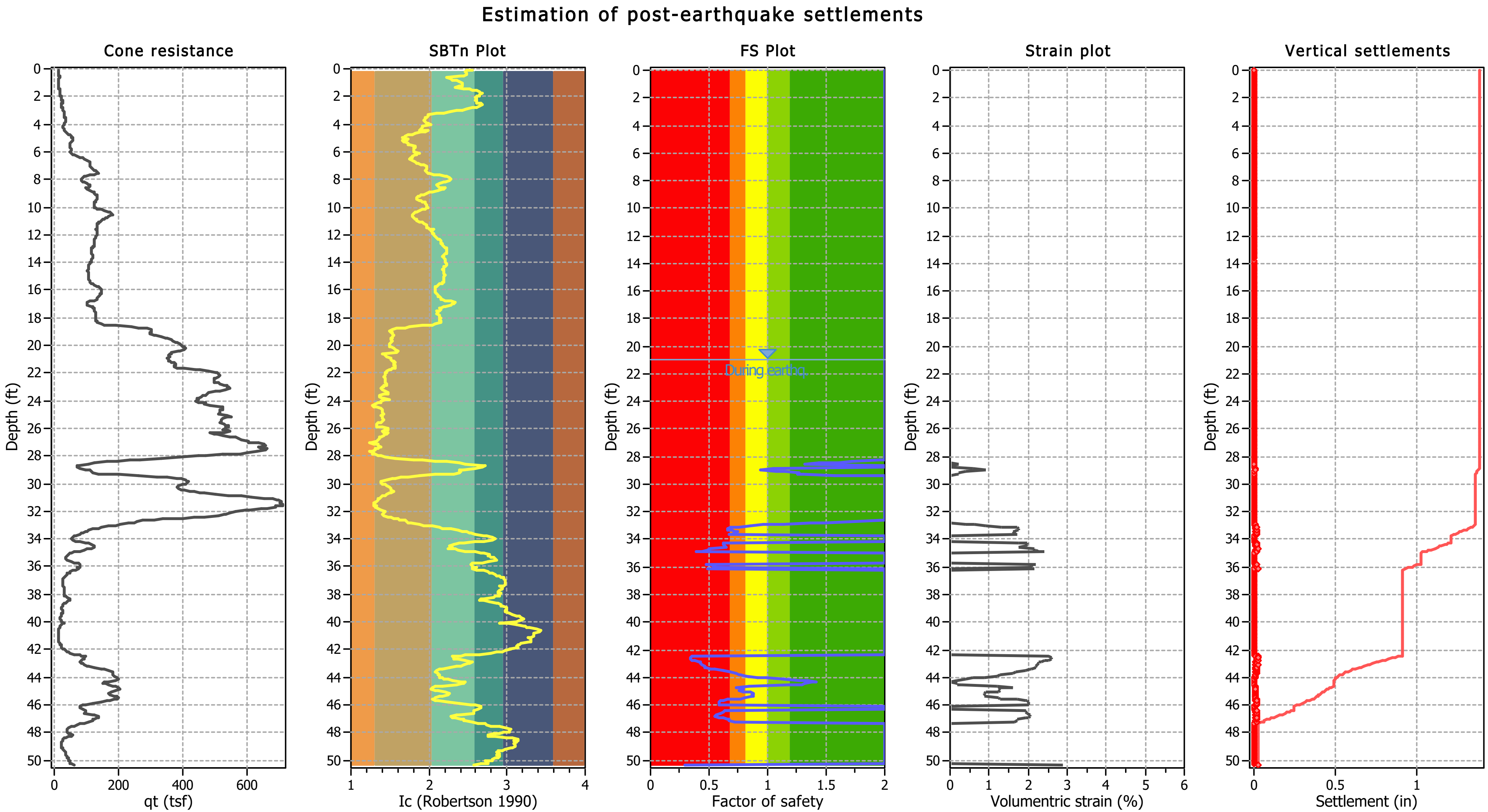
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	39.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	21.00 ft	Fill height:	N/A	Limit depth applie	No
Points to test:	Based on Ic value	Average results interva	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.98	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method basec
Peak ground acceleration:	0.61	Unit weight calculatior	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



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)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
21.02	301.07	2.00	0.00	1.00	0.00	21.10	302.06	2.00	0.00	1.00	0.00
21.14	303.26	2.00	0.00	1.00	0.00	21.21	306.03	2.00	0.00	1.00	0.00
21.28	313.68	2.00	0.00	1.00	0.00	21.33	316.69	2.00	0.00	1.00	0.00
21.40	315.55	2.00	0.00	1.00	0.00	21.48	312.61	2.00	0.00	1.00	0.00
21.53	312.89	2.00	0.00	1.00	0.00	21.60	320.61	2.00	0.00	1.00	0.00
21.67	338.65	2.00	0.00	1.00	0.00	21.76	365.25	2.00	0.00	1.00	0.00
21.79	377.47	2.00	0.00	1.00	0.00	21.88	399.27	2.00	0.00	1.00	0.00
21.95	415.12	2.00	0.00	1.00	0.00	21.98	418.74	2.00	0.00	1.00	0.00
22.07	422.56	2.00	0.00	1.00	0.00	22.14	423.92	2.00	0.00	1.00	0.00
22.18	424.03	2.00	0.00	1.00	0.00	22.26	420.62	2.00	0.00	1.00	0.00
22.34	411.81	2.00	0.00	1.00	0.00	22.39	408.34	2.00	0.00	1.00	0.00
22.45	407.48	2.00	0.00	1.00	0.00	22.54	406.35	2.00	0.00	1.00	0.00
22.57	405.80	2.00	0.00	1.00	0.00	22.65	404.81	2.00	0.00	1.00	0.00
22.72	415.06	2.00	0.00	1.00	0.00	22.77	417.32	2.00	0.00	1.00	0.00
22.84	426.84	2.00	0.00	1.00	0.00	22.92	433.73	2.00	0.00	1.00	0.00
22.97	435.59	2.00	0.00	1.00	0.00	23.04	435.52	2.00	0.00	1.00	0.00
23.11	438.62	2.00	0.00	1.00	0.00	23.20	429.43	2.00	0.00	1.00	0.00
23.26	412.62	2.00	0.00	1.00	0.00	23.31	404.03	2.00	0.00	1.00	0.00
23.39	384.89	2.00	0.00	1.00	0.00	23.47	379.50	2.00	0.00	1.00	0.00
23.51	380.07	2.00	0.00	1.00	0.00	23.58	373.41	2.00	0.00	1.00	0.00
23.62	369.16	2.00	0.00	1.00	0.00	23.70	364.98	2.00	0.00	1.00	0.00
23.78	357.05	2.00	0.00	1.00	0.00	23.83	353.33	2.00	0.00	1.00	0.00
23.90	349.44	2.00	0.00	1.00	0.00	23.97	353.17	2.00	0.00	1.00	0.00
24.05	345.04	2.00	0.00	1.00	0.00	24.09	353.32	2.00	0.00	1.00	0.00
24.17	368.37	2.00	0.00	1.00	0.00	24.25	366.34	2.00	0.00	1.00	0.00
24.28	364.41	2.00	0.00	1.00	0.00	24.37	378.45	2.00	0.00	1.00	0.00
24.44	406.08	2.00	0.00	1.00	0.00	24.48	408.79	2.00	0.00	1.00	0.00
24.57	409.24	2.00	0.00	1.00	0.00	24.63	408.54	2.00	0.00	1.00	0.00
24.70	399.78	2.00	0.00	1.00	0.00	24.77	400.02	2.00	0.00	1.00	0.00
24.81	398.62	2.00	0.00	1.00	0.00	24.90	397.85	2.00	0.00	1.00	0.00
24.95	396.29	2.00	0.00	1.00	0.00	25.02	414.49	2.00	0.00	1.00	0.00
25.10	415.24	2.00	0.00	1.00	0.00	25.14	424.32	2.00	0.00	1.00	0.00
25.21	414.21	2.00	0.00	1.00	0.00	25.29	401.91	2.00	0.00	1.00	0.00
25.33	397.17	2.00	0.00	1.00	0.00	25.40	396.40	2.00	0.00	1.00	0.00
25.48	395.63	2.00	0.00	1.00	0.00	25.55	392.18	2.00	0.00	1.00	0.00
25.60	394.20	2.00	0.00	1.00	0.00	25.67	400.68	2.00	0.00	1.00	0.00
25.75	405.99	2.00	0.00	1.00	0.00	25.82	410.21	2.00	0.00	1.00	0.00
25.86	410.72	2.00	0.00	1.00	0.00	25.94	403.46	2.00	0.00	1.00	0.00
26.01	402.35	2.00	0.00	1.00	0.00	26.09	395.52	2.00	0.00	1.00	0.00
26.12	403.60	2.00	0.00	1.00	0.00	26.21	411.01	2.00	0.00	1.00	0.00
26.27	398.87	2.00	0.00	1.00	0.00	26.34	364.92	2.00	0.00	1.00	0.00
26.40	373.37	2.00	0.00	1.00	0.00	26.44	377.85	2.00	0.00	1.00	0.00
26.52	390.99	2.00	0.00	1.00	0.00	26.59	408.89	2.00	0.00	1.00	0.00
26.64	419.80	2.00	0.00	1.00	0.00	26.74	428.04	2.00	0.00	1.00	0.00
26.77	427.21	2.00	0.00	1.00	0.00	26.85	431.24	2.00	0.00	1.00	0.00
26.92	448.11	2.00	0.00	1.00	0.00	26.99	448.45	2.00	0.00	1.00	0.00
27.07	468.90	2.00	0.00	1.00	0.00	27.13	481.71	2.00	0.00	1.00	0.00
27.18	478.57	2.00	0.00	1.00	0.00	27.25	484.40	2.00	0.00	1.00	0.00

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
27.32	471.21	2.00	0.00	1.00	0.00	27.37	466.00	2.00	0.00	1.00	0.00
27.44	472.00	2.00	0.00	1.00	0.00	27.50	486.08	2.00	0.00	1.00	0.00
27.57	480.10	2.00	0.00	1.00	0.00	27.63	462.37	2.00	0.00	1.00	0.00
27.70	460.91	2.00	0.00	1.00	0.00	27.76	438.13	2.00	0.00	1.00	0.00
27.82	420.92	2.00	0.00	1.00	0.00	27.91	380.68	2.00	0.00	1.00	0.00
27.98	343.20	2.00	0.00	1.00	0.00	28.02	325.51	2.00	0.00	1.00	0.00
28.09	283.87	2.00	0.00	1.00	0.00	28.18	240.75	2.00	0.00	1.00	0.00
28.26	204.53	2.00	0.00	1.00	0.00	28.29	193.74	1.97	0.00	1.00	0.00
28.37	184.26	1.72	0.00	1.00	0.00	28.42	179.63	1.61	0.00	1.00	0.00
28.50	172.34	1.44	0.00	1.00	0.00	28.55	166.41	1.32	0.20	1.00	0.00
28.65	165.33	2.00	0.00	1.00	0.00	28.69	161.06	2.00	0.00	1.00	0.00
28.77	153.54	2.00	0.00	1.00	0.00	28.81	153.45	2.00	0.00	1.00	0.00
28.89	151.75	1.05	0.60	1.00	0.01	28.96	144.83	0.93	0.91	1.00	0.01
29.04	153.60	1.07	0.42	1.00	0.00	29.09	157.96	1.15	0.41	1.00	0.00
29.16	164.39	1.27	0.20	1.00	0.00	29.25	164.14	1.26	0.20	1.00	0.00
29.28	166.52	1.31	0.20	1.00	0.00	29.35	178.84	1.57	0.00	1.00	0.00
29.42	204.62	2.00	0.00	1.00	0.00	29.49	232.62	2.00	0.00	1.00	0.00
29.54	246.27	2.00	0.00	1.00	0.00	29.61	268.81	2.00	0.00	1.00	0.00
29.67	278.76	2.00	0.00	1.00	0.00	29.73	288.27	2.00	0.00	1.00	0.00
29.81	290.93	2.00	0.00	1.00	0.00	29.88	292.57	2.00	0.00	1.00	0.00
29.93	290.73	2.00	0.00	1.00	0.00	30.02	285.38	2.00	0.00	1.00	0.00
30.06	280.17	2.00	0.00	1.00	0.00	30.14	270.91	2.00	0.00	1.00	0.00
30.19	269.60	2.00	0.00	1.00	0.00	30.26	265.70	2.00	0.00	1.00	0.00
30.34	267.37	2.00	0.00	1.00	0.00	30.41	274.69	2.00	0.00	1.00	0.00
30.45	280.03	2.00	0.00	1.00	0.00	30.53	291.20	2.00	0.00	1.00	0.00
30.60	301.90	2.00	0.00	1.00	0.00	30.68	314.43	2.00	0.00	1.00	0.00
30.72	323.64	2.00	0.00	1.00	0.00	30.80	347.40	2.00	0.00	1.00	0.00
30.86	372.99	2.00	0.00	1.00	0.00	30.93	401.29	2.00	0.00	1.00	0.00
30.97	422.33	2.00	0.00	1.00	0.00	31.05	449.52	2.00	0.00	1.00	0.00
31.12	466.48	2.00	0.00	1.00	0.00	31.18	477.03	2.00	0.00	1.00	0.00
31.24	477.65	2.00	0.00	1.00	0.00	31.31	484.19	2.00	0.00	1.00	0.00
31.37	484.26	2.00	0.00	1.00	0.00	31.44	479.23	2.00	0.00	1.00	0.00
31.52	484.88	2.00	0.00	1.00	0.00	31.56	484.92	2.00	0.00	1.00	0.00
31.65	467.98	2.00	0.00	1.00	0.00	31.71	469.01	2.00	0.00	1.00	0.00
31.78	451.16	2.00	0.00	1.00	0.00	31.85	436.50	2.00	0.00	1.00	0.00
31.92	413.43	2.00	0.00	1.00	0.00	31.96	402.50	2.00	0.00	1.00	0.00
32.03	382.40	2.00	0.00	1.00	0.00	32.11	369.98	2.00	0.00	1.00	0.00
32.19	359.02	2.00	0.00	1.00	0.00	32.23	355.25	2.00	0.00	1.00	0.00
32.31	345.04	2.00	0.00	1.00	0.00	32.38	313.15	2.00	0.00	1.00	0.00
32.42	292.84	2.00	0.00	1.00	0.00	32.50	267.22	2.00	0.00	1.00	0.00
32.58	224.80	2.00	0.00	1.00	0.00	32.62	208.02	2.00	0.00	1.00	0.00
32.69	188.82	1.74	0.00	1.00	0.00	32.77	177.67	1.48	0.00	1.00	0.00
32.81	171.85	1.36	0.00	1.00	0.00	32.88	160.20	1.14	0.41	1.00	0.00
32.97	149.23	0.96	0.61	1.00	0.01	33.01	142.46	0.86	0.93	1.00	0.00
33.07	135.03	0.76	1.31	1.00	0.01	33.17	128.97	0.69	1.71	1.00	0.02
33.22	126.67	0.66	1.76	1.00	0.01	33.29	126.06	0.65	1.77	1.00	0.02
33.38	129.57	0.69	1.70	1.00	0.02	33.42	131.80	0.72	1.66	1.00	0.01
33.48	134.32	0.75	1.62	1.00	0.01	33.57	132.28	0.72	1.65	1.00	0.02

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
33.61	130.11	0.70	1.69	1.00	0.01	33.68	128.27	0.68	1.73	1.00	0.01
33.75	129.04	2.00	0.00	1.00	0.00	33.80	129.30	2.00	0.00	1.00	0.00
33.87	127.07	2.00	0.00	1.00	0.00	33.95	126.92	2.00	0.00	1.00	0.00
33.99	126.85	2.00	0.00	1.00	0.00	34.06	126.71	2.00	0.00	1.00	0.00
34.14	126.11	2.00	0.00	1.00	0.00	34.22	127.28	0.66	1.75	1.00	0.02
34.28	123.75	0.62	1.96	1.00	0.02	34.32	123.61	0.62	1.96	1.00	0.01
34.41	123.99	0.63	1.96	1.00	0.02	34.48	124.59	0.63	1.95	1.00	0.02
34.51	124.73	0.63	1.95	1.00	0.01	34.59	126.36	0.65	1.76	1.00	0.02
34.67	110.38	0.50	2.15	1.00	0.02	34.75	111.30	0.51	2.14	1.00	0.02
34.79	108.51	0.48	2.19	1.00	0.01	34.87	103.40	0.44	2.27	1.00	0.02
34.92	95.88	0.39	2.42	1.00	0.02	35.01	93.36	2.00	0.00	1.00	0.00
35.05	90.52	2.00	0.00	1.00	0.00	35.13	87.55	2.00	0.00	1.00	0.00
35.21	77.23	2.00	0.00	1.00	0.00	35.24	77.25	2.00	0.00	1.00	0.00
35.33	77.21	2.00	0.00	1.00	0.00	35.37	80.72	2.00	0.00	1.00	0.00
35.45	84.00	2.00	0.00	1.00	0.00	35.53	90.25	2.00	0.00	1.00	0.00
35.57	96.18	2.00	0.00	1.00	0.00	35.66	104.35	2.00	0.00	1.00	0.00
35.70	106.58	2.00	0.00	1.00	0.00	35.77	108.16	0.48	2.19	1.00	0.02
35.86	112.90	0.52	2.12	1.00	0.02	35.90	115.28	0.54	2.08	1.00	0.01
35.98	117.57	0.56	2.05	1.00	0.02	36.06	115.14	0.53	2.08	1.00	0.02
36.09	113.11	2.00	0.00	1.00	0.00	36.18	110.18	0.49	2.16	1.00	0.02
36.22	105.43	2.00	0.00	1.00	0.00	36.31	100.63	2.00	0.00	1.00	0.00
36.38	95.23	2.00	0.00	1.00	0.00	36.42	92.36	2.00	0.00	1.00	0.00
36.51	86.24	2.00	0.00	1.00	0.00	36.59	82.93	2.00	0.00	1.00	0.00
36.63	79.56	2.00	0.00	1.00	0.00	36.70	75.92	2.00	0.00	1.00	0.00
36.75	75.70	2.00	0.00	1.00	0.00	36.83	71.89	2.00	0.00	1.00	0.00
36.92	67.99	2.00	0.00	1.00	0.00	36.95	67.98	2.00	0.00	1.00	0.00
37.04	67.92	2.00	0.00	1.00	0.00	37.07	67.90	2.00	0.00	1.00	0.00
37.16	67.84	2.00	0.00	1.00	0.00	37.24	67.78	2.00	0.00	1.00	0.00
37.28	67.76	2.00	0.00	1.00	0.00	37.36	67.68	2.00	0.00	1.00	0.00
37.40	63.84	2.00	0.00	1.00	0.00	37.49	67.70	2.00	0.00	1.00	0.00
37.56	67.62	2.00	0.00	1.00	0.00	37.60	67.62	2.00	0.00	1.00	0.00
37.69	67.75	2.00	0.00	1.00	0.00	37.77	67.71	2.00	0.00	1.00	0.00
37.81	67.69	2.00	0.00	1.00	0.00	37.88	67.65	2.00	0.00	1.00	0.00
37.93	71.21	2.00	0.00	1.00	0.00	38.01	71.30	2.00	0.00	1.00	0.00
38.09	74.79	2.00	0.00	1.00	0.00	38.16	74.99	2.00	0.00	1.00	0.00
38.20	78.39	2.00	0.00	1.00	0.00	38.29	81.42	2.00	0.00	1.00	0.00
38.33	81.30	2.00	0.00	1.00	0.00	38.40	77.91	2.00	0.00	1.00	0.00
38.49	74.67	2.00	0.00	1.00	0.00	38.53	74.76	2.00	0.00	1.00	0.00
38.61	71.03	2.00	0.00	1.00	0.00	38.65	67.21	2.00	0.00	1.00	0.00
38.73	62.96	2.00	0.00	1.00	0.00	38.81	58.58	2.00	0.00	1.00	0.00
38.85	58.47	2.00	0.00	1.00	0.00	38.93	58.33	2.00	0.00	1.00	0.00
39.01	58.29	2.00	0.00	1.00	0.00	39.05	58.29	2.00	0.00	1.00	0.00
39.13	58.27	2.00	0.00	1.00	0.00	39.21	58.26	2.00	0.00	1.00	0.00
39.26	58.24	2.00	0.00	1.00	0.00	39.33	62.50	2.00	0.00	1.00	0.00
39.38	62.47	2.00	0.00	1.00	0.00	39.46	62.34	2.00	0.00	1.00	0.00
39.54	62.05	2.00	0.00	1.00	0.00	39.58	61.85	2.00	0.00	1.00	0.00
39.66	61.63	2.00	0.00	1.00	0.00	39.70	61.62	2.00	0.00	1.00	0.00
39.78	65.34	2.00	0.00	1.00	0.00	39.86	65.60	2.00	0.00	1.00	0.00

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
39.90	65.73	2.00	0.00	1.00	0.00	39.98	65.99	2.00	0.00	1.00	0.00
40.06	66.62	2.00	0.00	1.00	0.00	40.10	66.31	2.00	0.00	1.00	0.00
40.19	65.90	2.00	0.00	1.00	0.00	40.26	65.10	2.00	0.00	1.00	0.00
40.31	65.01	2.00	0.00	1.00	0.00	40.39	60.66	2.00	0.00	1.00	0.00
40.44	56.11	2.00	0.00	1.00	0.00	40.51	54.87	2.00	0.00	1.00	0.00
40.56	54.86	2.00	0.00	1.00	0.00	40.64	49.43	2.00	0.00	1.00	0.00
40.72	49.60	2.00	0.00	1.00	0.00	40.76	49.68	2.00	0.00	1.00	0.00
40.83	45.34	2.00	0.00	1.00	0.00	40.92	45.32	2.00	0.00	1.00	0.00
40.96	45.31	2.00	0.00	1.00	0.00	41.04	45.29	2.00	0.00	1.00	0.00
41.08	45.64	2.00	0.00	1.00	0.00	41.16	46.12	2.00	0.00	1.00	0.00
41.24	51.02	2.00	0.00	1.00	0.00	41.29	51.07	2.00	0.00	1.00	0.00
41.37	50.92	2.00	0.00	1.00	0.00	41.41	50.91	2.00	0.00	1.00	0.00
41.49	51.25	2.00	0.00	1.00	0.00	41.57	52.16	2.00	0.00	1.00	0.00
41.61	56.99	2.00	0.00	1.00	0.00	41.69	61.50	2.00	0.00	1.00	0.00
41.77	65.44	2.00	0.00	1.00	0.00	41.81	69.03	2.00	0.00	1.00	0.00
41.89	69.03	2.00	0.00	1.00	0.00	41.94	69.49	2.00	0.00	1.00	0.00
42.01	76.26	2.00	0.00	1.00	0.00	42.09	85.37	2.00	0.00	1.00	0.00
42.13	88.14	2.00	0.00	1.00	0.00	42.21	91.33	2.00	0.00	1.00	0.00
42.26	91.52	2.00	0.00	1.00	0.00	42.34	93.81	2.00	0.00	1.00	0.00
42.41	90.38	0.35	2.54	1.00	0.02	42.45	90.36	0.35	2.54	1.00	0.01
42.53	88.26	0.34	2.59	1.00	0.02	42.61	88.13	0.34	2.59	1.00	0.03
42.66	88.00	0.34	2.60	1.00	0.01	42.74	90.32	0.35	2.54	1.00	0.02
42.81	95.44	0.38	2.43	1.00	0.02	42.85	100.58	0.41	2.33	1.00	0.01
42.93	103.09	0.43	2.28	1.00	0.02	43.01	102.83	0.43	2.28	1.00	0.02
43.05	105.09	0.44	2.24	1.00	0.01	43.13	104.93	0.44	2.25	1.00	0.02
43.20	107.22	0.46	2.21	1.00	0.02	43.25	107.40	0.46	2.20	1.00	0.01
43.33	108.92	0.47	2.18	1.00	0.02	43.40	114.12	0.52	2.10	1.00	0.02
43.44	118.02	0.55	2.04	1.00	0.01	43.52	125.12	0.62	1.94	1.00	0.02
43.60	130.68	0.68	1.68	1.00	0.02	43.64	130.96	0.68	1.68	1.00	0.01
43.71	134.73	0.73	1.61	1.00	0.01	43.79	136.55	0.75	1.29	1.00	0.01
43.84	138.18	0.77	1.27	1.00	0.01	43.91	142.04	0.82	1.22	1.00	0.01
43.99	149.14	0.92	0.87	1.00	0.01	44.03	153.57	0.99	0.59	1.00	0.00
44.11	162.54	1.14	0.40	1.00	0.00	44.19	170.08	1.27	0.20	1.00	0.00
44.23	173.36	1.34	0.20	1.00	0.00	44.31	177.38	1.42	0.00	1.00	0.00
44.39	175.56	1.38	0.00	1.00	0.00	44.46	172.38	1.32	0.20	1.00	0.00
44.50	170.86	1.29	0.20	1.00	0.00	44.58	165.13	1.18	0.29	1.00	0.00
44.65	143.15	0.84	1.20	1.00	0.01	44.70	135.40	0.74	1.60	1.00	0.01
44.77	138.18	0.77	1.27	1.00	0.01	44.85	140.04	0.80	1.24	1.00	0.01
44.90	140.15	0.80	1.24	1.00	0.01	44.96	137.27	0.76	1.28	1.00	0.01
45.01	139.96	0.80	1.24	1.00	0.01	45.10	144.75	0.86	0.91	1.00	0.01
45.17	146.11	0.88	0.89	1.00	0.01	45.22	145.95	0.88	0.90	1.00	0.01
45.29	145.00	0.86	0.90	1.00	0.01	45.37	146.23	0.88	0.89	1.00	0.01
45.42	145.34	0.87	0.90	1.00	0.00	45.50	141.04	0.81	1.23	1.00	0.01
45.54	137.61	0.77	1.27	1.00	0.01	45.62	128.71	0.66	1.72	1.00	0.02
45.70	124.70	0.62	1.95	1.00	0.02	45.73	122.21	0.59	1.98	1.00	0.01
45.81	121.10	0.58	2.00	1.00	0.02	45.89	120.71	0.58	2.00	1.00	0.02
45.97	120.74	0.58	2.00	1.00	0.02	46.01	120.99	0.58	2.00	1.00	0.01
46.09	119.21	2.00	0.00	1.00	0.00	46.13	119.25	2.00	0.00	1.00	0.00

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

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
46.21	121.21	2.00	0.00	1.00	0.00	46.29	124.97	2.00	0.00	1.00	0.00
46.37	126.55	0.64	1.93	1.00	0.02	46.41	126.50	0.64	1.93	1.00	0.01
46.49	124.50	0.62	1.95	1.00	0.02	46.52	124.48	0.62	1.95	1.00	0.01
46.60	122.52	0.60	1.98	1.00	0.02	46.68	118.21	0.56	2.04	1.00	0.02
46.73	118.50	0.56	2.03	1.00	0.01	46.80	117.57	0.55	2.05	1.00	0.02
46.89	121.22	0.59	2.00	1.00	0.02	46.92	124.50	0.62	1.95	1.00	0.01
47.00	127.74	0.65	1.74	1.00	0.02	47.08	129.51	0.67	1.70	1.00	0.02
47.11	129.49	0.67	1.70	1.00	0.01	47.19	133.14	0.72	1.64	1.00	0.02
47.27	133.32	0.72	1.63	1.00	0.02	47.32	133.67	2.00	0.00	1.00	0.00
47.39	126.34	2.00	0.00	1.00	0.00	47.47	114.40	2.00	0.00	1.00	0.00
47.52	112.22	2.00	0.00	1.00	0.00	47.60	107.36	2.00	0.00	1.00	0.00
47.64	100.21	2.00	0.00	1.00	0.00	47.72	100.24	2.00	0.00	1.00	0.00
47.80	97.93	2.00	0.00	1.00	0.00	47.84	97.91	2.00	0.00	1.00	0.00
47.92	90.59	2.00	0.00	1.00	0.00	48.01	87.88	2.00	0.00	1.00	0.00
48.08	90.82	2.00	0.00	1.00	0.00	48.12	91.21	2.00	0.00	1.00	0.00
48.20	91.29	2.00	0.00	1.00	0.00	48.24	88.71	2.00	0.00	1.00	0.00
48.31	83.15	2.00	0.00	1.00	0.00	48.38	79.70	2.00	0.00	1.00	0.00
48.43	79.58	2.00	0.00	1.00	0.00	48.51	73.24	2.00	0.00	1.00	0.00
48.59	66.73	2.00	0.00	1.00	0.00	48.63	59.67	2.00	0.00	1.00	0.00
48.71	59.50	2.00	0.00	1.00	0.00	48.76	55.55	2.00	0.00	1.00	0.00
48.83	55.50	2.00	0.00	1.00	0.00	48.92	55.47	2.00	0.00	1.00	0.00
48.95	55.43	2.00	0.00	1.00	0.00	49.04	55.44	2.00	0.00	1.00	0.00
49.11	55.57	2.00	0.00	1.00	0.00	49.15	55.62	2.00	0.00	1.00	0.00
49.24	55.72	2.00	0.00	1.00	0.00	49.31	59.86	2.00	0.00	1.00	0.00
49.35	60.13	2.00	0.00	1.00	0.00	49.44	63.89	2.00	0.00	1.00	0.00
49.48	63.94	2.00	0.00	1.00	0.00	49.56	67.28	2.00	0.00	1.00	0.00
49.64	67.44	2.00	0.00	1.00	0.00	49.68	70.60	2.00	0.00	1.00	0.00
49.75	73.73	2.00	0.00	1.00	0.00	49.84	76.80	2.00	0.00	1.00	0.00
49.88	76.92	2.00	0.00	1.00	0.00	49.96	77.02	2.00	0.00	1.00	0.00
50.01	77.03	2.00	0.00	1.00	0.00	50.08	70.94	2.00	0.00	1.00	0.00
50.16	74.03	2.00	0.00	1.00	0.00	50.20	77.03	2.00	0.00	1.00	0.00
50.28	76.96	2.00	0.00	1.00	0.00	50.36	76.72	0.29	2.90	1.00	0.03

Total estimated settlement: 1.39

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

APPENDIX G

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

Pacific Avenue & 2nd Street
Norco, CA
January 26, 2017

Prepared for:

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

APPENDIX

- CPT Plots
- CPT Classification/Soil Behavior Chart
- Interpretation Output (CPeT-IT)
- Pore Pressure Dissipation Graphs
- CPeT-IT Calculation Formulas

SUMMARY

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at Pacific Avenue & 2nd Street in Norco, California. The work was performed by Kehoe Testing & Engineering (KTE) on January 26, 2017. The scope of work was performed as directed by Southern California Geotechnical, Inc. personnel.

2. SUMMARY OF FIELD WORK

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

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	34	Refusal, hole open to 33 ft (dry)
CPT-2	34	Refusal, groundwater @ 27 ft
CPT-3	60	Groundwater @ 28 ft
CPT-4	50	Groundwater @ 39 ft

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 and recorded the following parameters at approximately 2.5 cm depth intervals:

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

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

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil classification on the CPT plots is derived from the attached CPT Classification Chart (Robertson) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (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.

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

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

Richard W. Koester, Jr.
General Manager

01/31/17-ms-8016

APPENDIX

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
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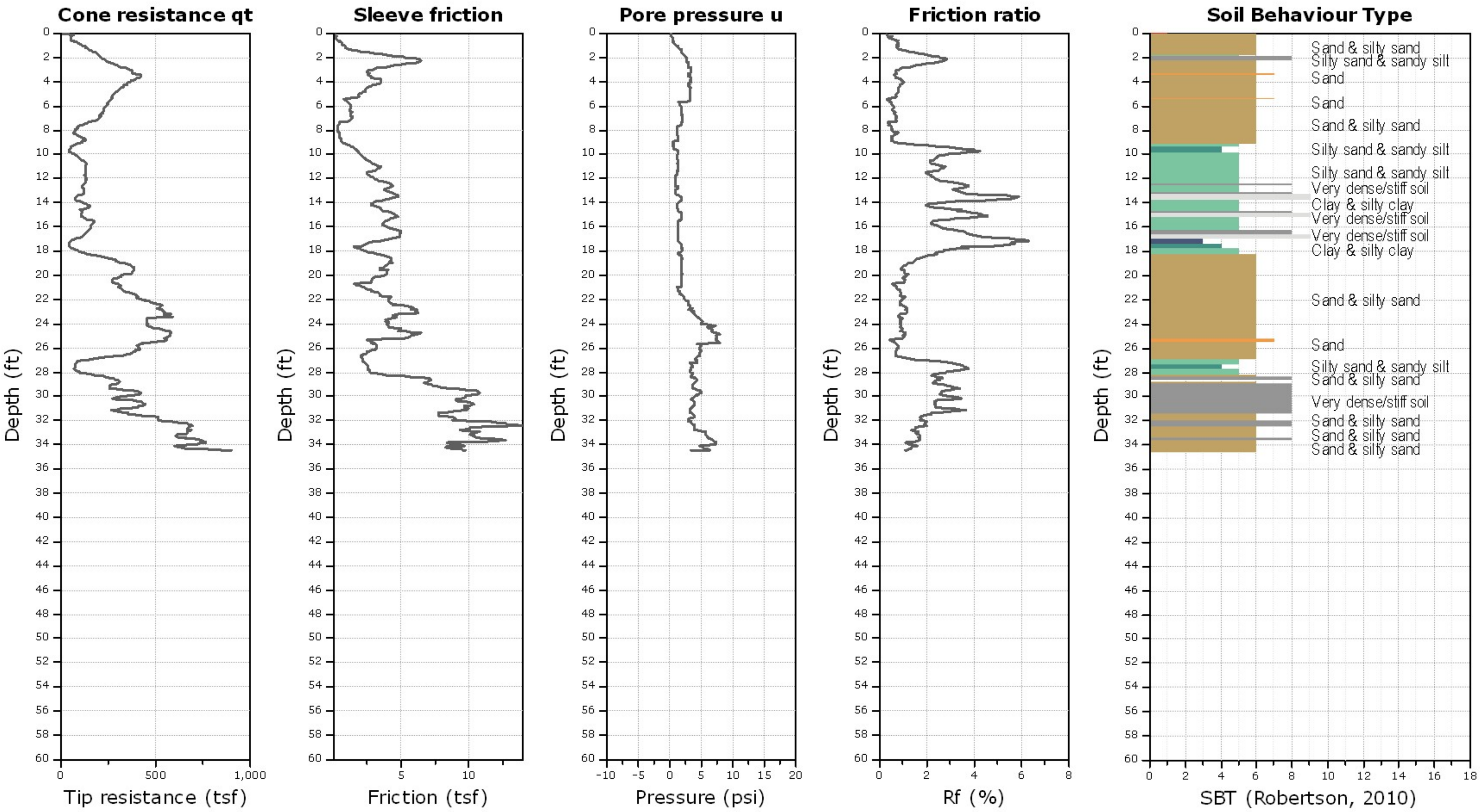
CPT-1

Project: Southern California Geotechnical, Inc.

Location: Pacific Ave & 2nd St Norco, CA

Total depth: 34.46 ft, Date: 1/26/2017

Cone Type: Vertek



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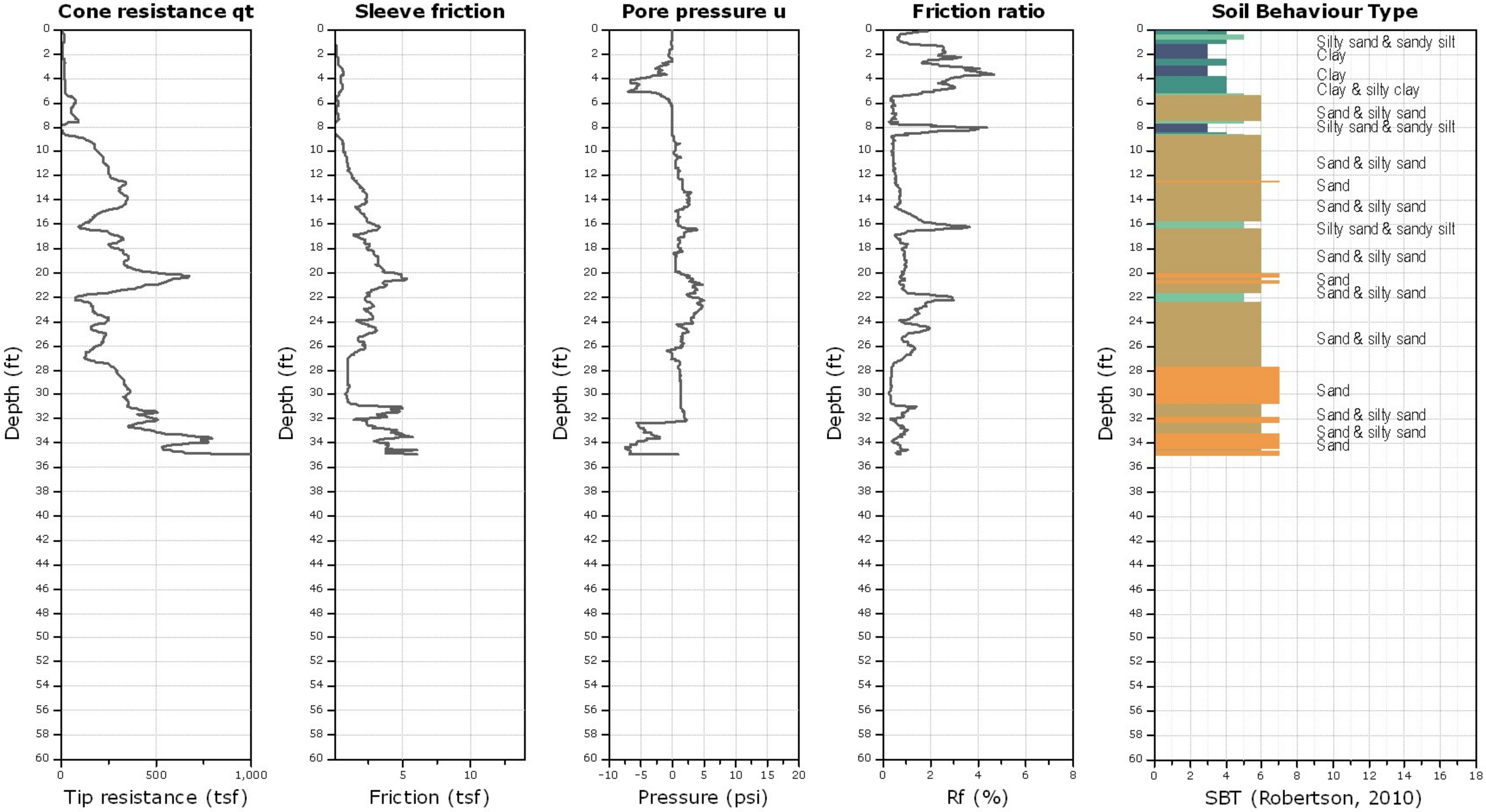
Project: **Southern California Geotechnical, Inc.**
Location: **Pacific Ave & 2nd St Norco, CA**

CPT-2
Total depth: 34.94 ft, Date: 1/26/2017
Cone Type: Vertek



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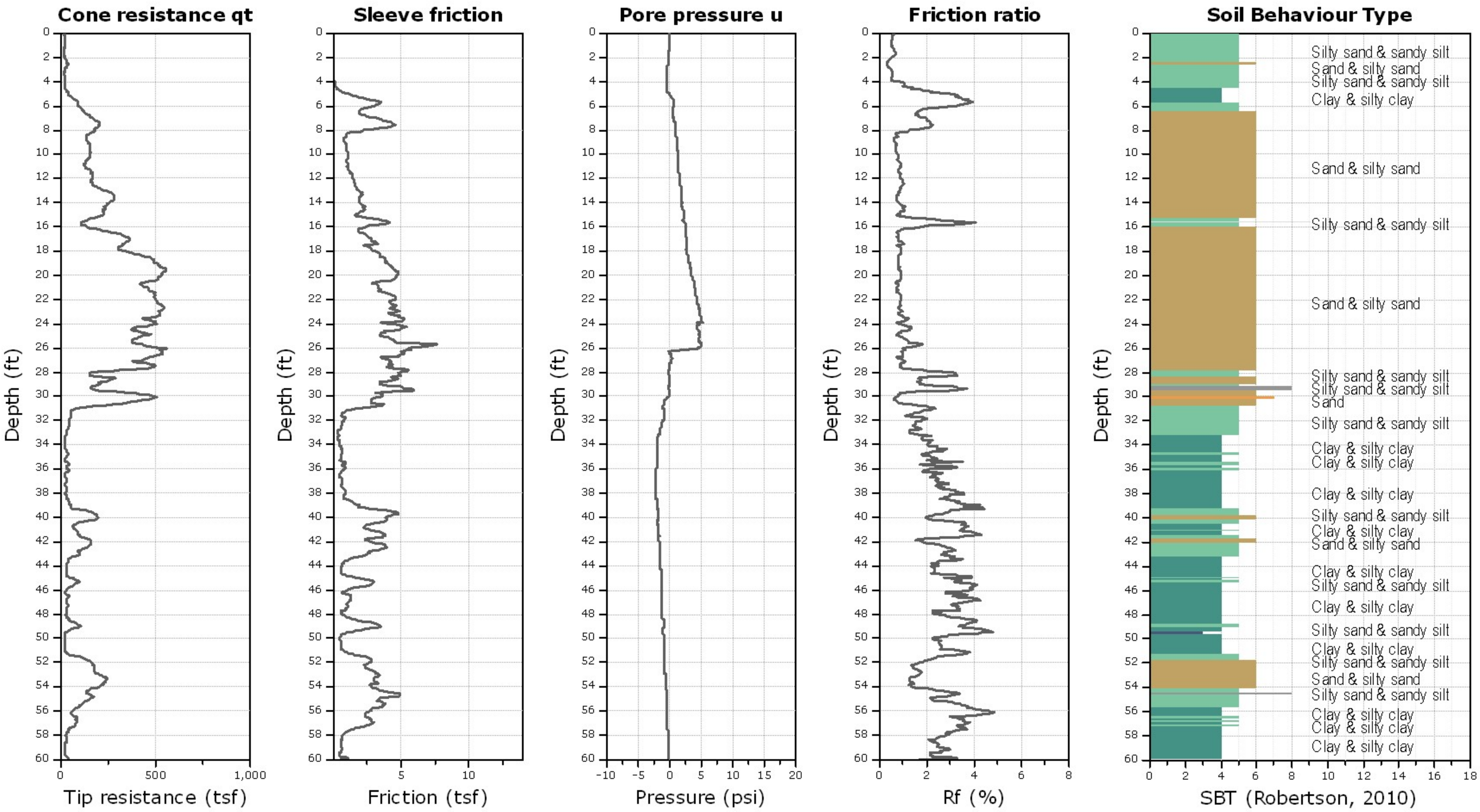
CPT-3

Project: Southern California Geotechnical, Inc.
Location: Pacific Ave & 2nd St Norco, CA

Total depth: 60.31 ft, Date: 1/26/2017
Cone Type: Vertek

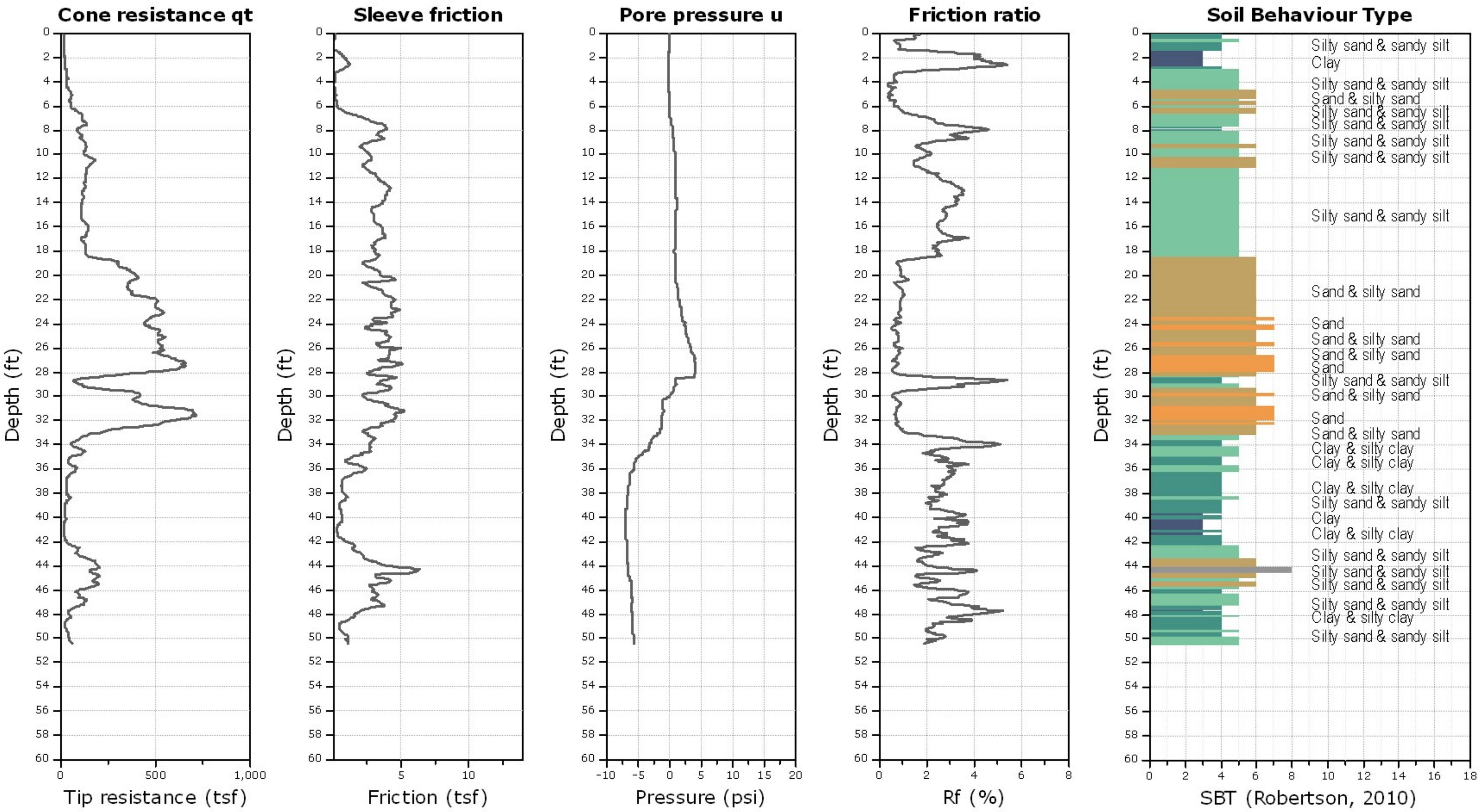


Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-4

Project: Southern California Geotechnical, Inc.
Location: Pacific Ave & 2nd St Norco, CA

Total depth: 50.36 ft, Date: 1/26/2017
Cone Type: Vertek



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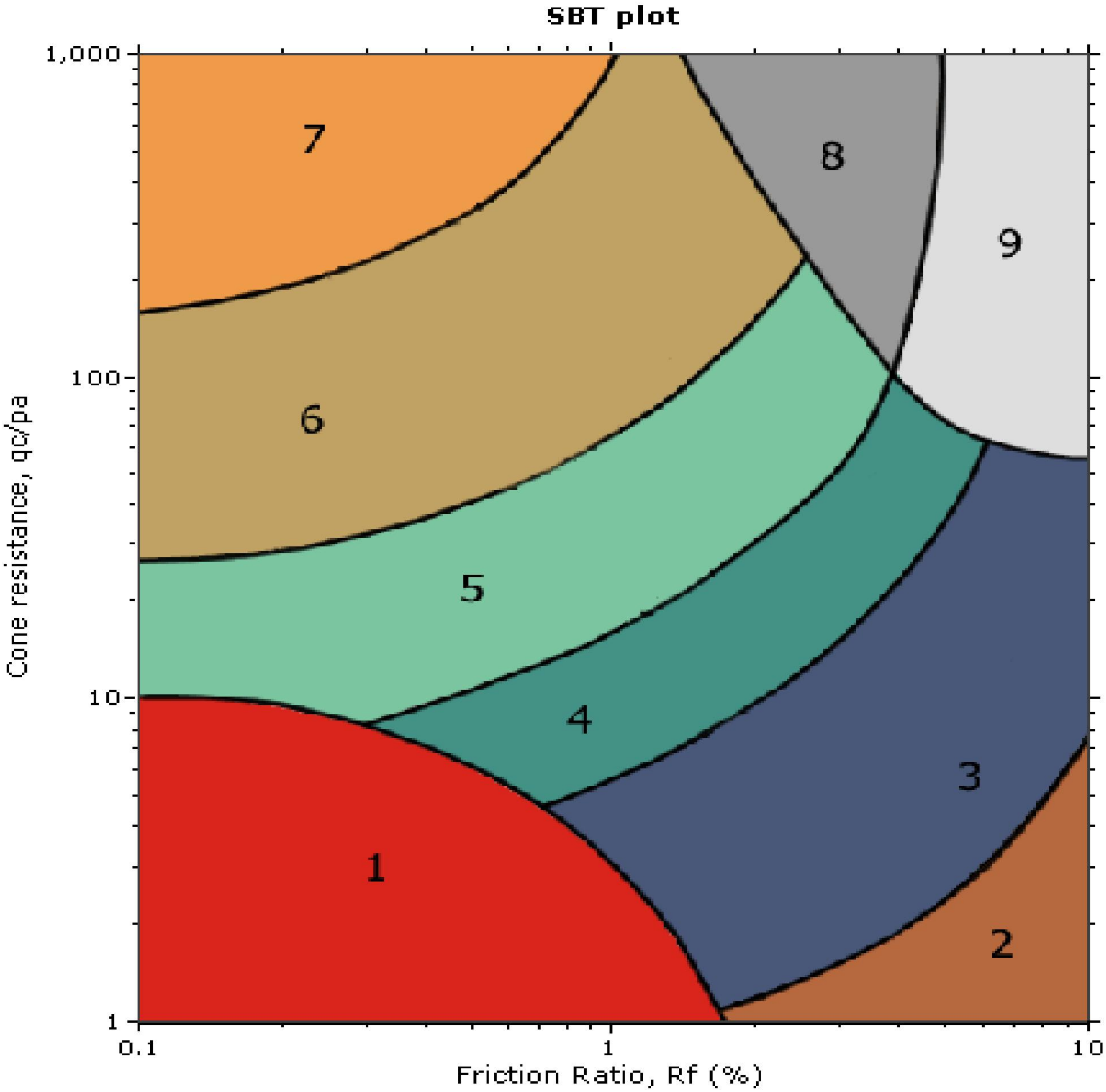


SBT legend

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

	CPT-1	In situ data									Basic output data												
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn	U2	I(B)	Mod. SBTn
1	116.02	0.94	1.18	0.5	116.03	0.81	6	1.82	121.41	0.06	0	0.06	1909.1	0.81	0	6	0.41	3.2	1.46	351.15	1.39	101.85	7
2	210.94	5.43	2.6	0.5	210.97	2.57	5	2.01	135.7	0.13	0	0.13	1639	2.58	0	8	0.53	3.05	1.77	607.82	1.46	37.78	7
3	361.21	2.92	3.44	0.4	361.26	0.81	6	1.47	132.49	0.19	0	0.19	1854.3	0.81	0	7	0.36	1.84	1.31	627.49	1.27	110.26	7
4	368.84	3.55	3.31	0.2	368.88	0.96	6	1.52	133.96	0.26	0	0.26	1408.4	0.96	0	6	0.39	1.73	1.39	601.01	0.91	94.16	7
5	279.86	1.98	3.22	0.1	279.9	0.71	6	1.5	129.03	0.33	0	0.33	856.84	0.71	0	6	0.39	1.57	1.37	415.88	0.71	116.63	7
6	238.2	1.36	1.68	0.1	238.22	0.57	6	1.48	125.86	0.39	0	0.39	610.98	0.57	0	6	0.39	1.47	1.36	330.85	0.31	131.68	7
7	205.72	1.46	2.02	0	205.75	0.71	6	1.59	126.04	0.45	0	0.45	453.87	0.71	0	6	0.44	1.45	1.48	280.84	0.32	107.72	7
8	73.83	0.42	1.2	0	73.84	0.57	6	1.89	114.38	0.51	0	0.51	143.97	0.57	0	6	0.54	1.49	1.75	102.99	0.17	87.82	7
9	108.29	0.73	0.55	0.2	108.3	0.67	6	1.8	119.4	0.57	0	0.57	189.3	0.68	0	6	0.52	1.38	1.69	140.58	0.07	91.05	7
10	68.71	1.98	1.25	0.2	68.73	2.89	5	2.36	125.6	0.63	0	0.63	107.76	2.91	0	5	0.74	1.46	2.25	94.15	0.14	30.25	5
11	127.3	3.45	1.38	0	127.31	2.71	5	2.16	131.15	0.7	0	0.7	181.51	2.72	0	5	0.68	1.33	2.09	158.7	0.14	33.61	7
12	133.25	3.13	0.97	0	133.26	2.35	5	2.1	130.56	0.76	0	0.76	173.68	2.36	0	5	0.66	1.24	2.04	155.62	0.09	37.82	7
13	121.03	3.97	0.91	0	121.04	3.28	5	2.24	132.05	0.83	0	0.83	145.06	3.3	0	5	0.73	1.19	2.19	135.63	0.08	28.13	5
14	118.52	3.03	1.28	0	118.54	2.55	5	2.16	130.03	0.89	0	0.89	131.63	2.57	0	5	0.7	1.13	2.13	125.22	0.1	34.47	7
15	105.37	4.59	1.75	-0.1	105.39	4.36	9	2.37	132.79	0.96	0	0.96	108.75	4.4	0	9	0.79	1.08	2.36	106.59	0.13	21.63	3
16	155.7	4.07	1.47	-0.2	155.72	2.62	5	2.09	132.86	1.03	0	1.03	150.67	2.63	0	5	0.7	1.02	2.09	149.29	0.1	34.4	7
17	72.68	4.07	1.29	-0.1	72.7	5.6	9	2.56	131	1.09	0	1.09	65.56	5.69	0	9	0.88	0.97	2.57	65.8	0.08	17.06	3
18	90.96	2.4	1.73	-0.1	90.98	2.64	5	2.25	127.68	1.16	0	1.16	77.71	2.67	0	5	0.77	0.93	2.28	79.29	0.11	31.66	5
19	330.83	4.18	1.82	-0.1	330.85	1.26	6	1.64	134.88	1.22	0	1.22	269.43	1.27	0	6	0.54	0.92	1.66	287.94	0.11	68.51	7
20	322.89	3.76	1.84	-0.1	322.91	1.16	6	1.62	134.05	1.29	0	1.29	249.23	1.17	0	6	0.54	0.9	1.65	273.04	0.1	72.73	7
21	305.97	2.3	1.25	-0.3	305.99	0.75	6	1.49	130.32	1.36	0	1.36	224.71	0.75	0	6	0.5	0.88	1.53	254.57	0.07	100.99	7
22	431.39	4.07	2.39	-0.4	431.42	0.94	6	1.47	135.34	1.42	0	1.42	302.09	0.95	0	6	0.49	0.86	1.51	351.07	0.12	89.7	7
23	536.44	6.06	3.76	-0.7	536.49	1.13	6	1.48	137.28	1.49	0	1.49	358.61	1.13	0	6	0.5	0.84	1.53	425.51	0.18	78.94	7
24	457.7	4.07	5.39	-1.7	457.77	0.89	6	1.44	135.49	1.56	0	1.56	292.51	0.89	0	6	0.49	0.83	1.49	356.43	0.25	94.39	7
25	574.56	5.85	7.4	-2.2	574.65	1.02	6	1.43	137.28	1.63	0	1.63	351.91	1.02	0	6	0.49	0.81	1.48	438.13	0.33	86.66	7
26	409.15	3.13	4.87	-2.4	409.21	0.77	6	1.41	133.3	1.7	0	1.7	240.42	0.77	0	6	0.49	0.79	1.48	305.07	0.21	103.46	7
27	108.6	2.19	4.32	-2.5	108.66	2.02	5	2.11	127.45	1.76	0	1.76	60.78	2.05	0	5	0.79	0.67	2.24	67.69	0.18	37.2	7
28	96.91	2.72	3.28	-2.6	96.95	2.8	5	2.25	128.74	1.82	0	1.82	52.18	2.85	0	5	0.85	0.63	2.4	56.59	0.13	28.76	5
29	279.97	7.21	3.58	-2.5	280.01	2.57	8	1.94	137.28	1.89	0	1.89	147.02	2.59	0	5	0.72	0.66	2.05	173.11	0.14	35.32	7
30	349.1	9.82	4.02	-2	349.15	2.81	8	1.92	137.28	1.96	0	1.96	177.11	2.83	0	8	0.71	0.64	2.03	211.18	0.15	33.16	7
31	352.44	9.92	3.43	-2	352.48	2.81	8	1.92	137.28	2.03	0	2.03	172.72	2.83	0	8	0.72	0.63	2.03	207.28	0.12	33.08	7
32	532.89	10.03	3.03	-1.6	532.93	1.88	8	1.68	137.28	2.1	0	2.1	253.05	1.89	0	6	0.63	0.65	1.78	326.96	0.1	49.01	7
33	669.48	10.34	5.77	-1.1	669.55	1.54	6	1.56	137.28	2.17	0	2.17	308.09	1.55	0	6	0.58	0.66	1.65	416.31	0.19	59.63	7
34	684.1	9.5	6.85	-1	684.19	1.39	6	1.51	137.28	2.23	0	2.23	305.14	1.39	0	6	0.57	0.65	1.6	421.78	0.22	65.65	7

	CPT-2	In situ data										Basic output data											
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn	U2	I(B)	Mod. SBTn
1	13.37	0.1	-0.18	0.1	13.36	0.78	4	2.62	100.06	0.05	0	0.05	265.94	0.78	0	6	0.59	6.11	1.94	76.85	-0.26	66.67	7
2	8.67	0.21	-0.64	0.2	8.66	2.41	3	3.02	104.08	0.1	0	0.1	83.78	2.44	-0.01	5	0.77	6.06	2.4	49	-0.45	31.12	5
3	16.6	0.42	-2.01	0.4	16.58	2.52	4	2.79	110.73	0.16	0	0.16	104.36	2.54	-0.01	5	0.74	4.13	2.33	64.1	-0.92	31.8	5
4	18.27	0.52	-4.98	0.4	18.21	2.87	4	2.79	112.6	0.21	0	0.21	84.23	2.9	-0.02	5	0.77	3.43	2.39	58.36	-1.68	28.57	5
5	24.44	0.52	-6.44	0.4	24.36	2.14	4	2.62	113.3	0.27	0	0.27	89.08	2.17	-0.02	5	0.73	2.72	2.29	61.91	-1.71	35.22	7
6	72.16	0.31	-0.37	0.4	72.15	0.43	6	1.85	112.21	0.33	0	0.33	219.98	0.44	0	6	0.49	1.78	1.63	120.52	-0.08	106.49	7
7	61.51	0.31	0.09	0.4	61.51	0.51	6	1.94	111.83	0.38	0	0.38	159.82	0.51	0	6	0.53	1.72	1.74	99.27	0.02	90.4	7
8	2.4	0	0	0.5	2.4	0	0	0	87.36	0.43	0	0.43	4.64	0	0	0	1	2.48	4.06	4.64	0	20.91	0
9	124.48	0.52	0.2	0.5	124.48	0.42	6	1.63	117.28	0.48	0	0.48	255.81	0.42	0	6	0.45	1.42	1.51	166.09	0.03	125.83	7
10	191.62	0.73	0.4	0.4	191.63	0.38	6	1.45	120.8	0.55	0	0.55	350.51	0.38	0	6	0.39	1.3	1.36	234.53	0.05	153.1	7
11	229.11	1.04	0.37	0.4	229.12	0.46	6	1.44	123.84	0.61	0	0.61	376.38	0.46	0	6	0.4	1.25	1.36	269.31	0.04	144.67	7
12	256.79	1.36	1.01	0.4	256.8	0.53	6	1.44	126.04	0.67	0	0.67	382.18	0.53	0	6	0.41	1.2	1.38	291.52	0.11	134.3	7
13	307.64	1.98	1.56	0.4	307.66	0.64	6	1.44	129.26	0.73	0	0.73	417.79	0.65	0	6	0.42	1.16	1.4	337.68	0.15	120.6	7
14	344.61	2.3	2.39	0.3	344.64	0.67	6	1.42	130.61	0.8	0	0.8	429.81	0.67	0	6	0.42	1.12	1.38	364.96	0.22	119.47	7
15	213.45	1.98	0.86	0.3	213.46	0.93	6	1.66	128.37	0.86	0	0.86	246	0.93	0	6	0.51	1.11	1.64	222.95	0.07	83.77	7
16	118.52	2.82	0.92	0.3	118.54	2.38	5	2.14	129.5	0.93	0	0.93	126.59	2.4	0	5	0.7	1.1	2.12	121.75	0.07	36.41	7
17	307.85	1.67	1.74	0.4	307.87	0.54	6	1.39	128	0.99	0	0.99	309.03	0.54	0	6	0.42	1.03	1.38	297.92	0.13	132.61	7
18	299.6	2.4	1.28	0.3	299.62	0.8	6	1.52	130.59	1.06	0	1.06	282.15	0.8	0	6	0.48	1	1.52	282.15	0.09	98.37	7
19	338.55	3.24	0.52	0.2	338.56	0.96	6	1.54	133.07	1.12	0	1.12	300.01	0.96	0	6	0.49	0.97	1.55	309.43	0.03	87.07	7
20	506.37	4.28	1.2	0.2	506.38	0.85	6	1.39	136.1	1.19	0	1.19	423.51	0.85	0	6	0.44	0.95	1.41	452.77	0.07	101.99	7
21	449.45	3.86	3.3	0.1	449.5	0.86	6	1.43	135.06	1.26	0	1.26	355.62	0.86	0	6	0.46	0.92	1.45	390.68	0.19	98.5	7
22	79.26	2.3	3.21	0.2	79.3	2.9	5	2.32	127.02	1.32	0	1.32	58.89	2.95	0	5	0.82	0.83	2.38	61.31	0.17	28.45	5
23	164.06	2.19	4.3	0.4	164.11	1.34	6	1.86	128.46	1.39	0	1.39	117.23	1.35	0	6	0.65	0.84	1.92	129.07	0.22	57.01	7
24	235.07	1.78	3.04	0.4	235.1	0.76	6	1.57	127.79	1.45	0	1.45	160.92	0.76	0	6	0.54	0.84	1.63	186.19	0.15	92.78	7
25	236.42	2.51	1.69	0.4	236.44	1.06	6	1.68	130.32	1.52	0	1.52	154.84	1.07	0	6	0.59	0.81	1.74	179.8	0.08	72.49	7
26	191.83	2.19	1.48	0.6	191.85	1.14	6	1.76	128.84	1.58	0	1.58	120.3	1.15	0	6	0.63	0.78	1.84	139.76	0.07	64.81	7
27	125.21	1.04	0.07	0.6	125.21	0.83	6	1.8	122.37	1.64	0	1.64	75.22	0.85	0	6	0.66	0.75	1.91	87.52	0	67.74	7
28	289.89	1.04	1.19	0.7	289.9	0.36	7	1.29	124.42	1.7	0.03	1.67	172.19	0.36	0	6	0.45	0.81	1.37	221.54	0.03	154.08	7
29	335.73	1.04	1.28	0.8	335.75	0.31	7	1.2	124.77	1.77	0.06	1.7	195.89	0.31	0	7	0.42	0.82	1.28	258.7	0.02	178.08	7
30	348.26	0.84	1.46	0.8	348.28	0.24	7	1.13	123.23	1.83	0.09	1.74	199.64	0.24	0	7	0.39	0.82	1.2	270.02	0.01	207.25	7
31	353.69	4.28	1.37	0.9	353.71	1.21	6	1.61	135.23	1.9	0.12	1.77	198.56	1.22	0	6	0.58	0.74	1.69	246.72	-0.01	69.34	7
32	504.59	1.57	2.11	1.9	504.62	0.31	7	1.06	128.73	1.96	0.16	1.81	278.48	0.31	0	7	0.37	0.82	1.13	390.57	0	208.94	7
33	483.81	4.49	-4.4	2.4	483.76	0.93	6	1.44	136.34	2.03	0.19	1.84	261.54	0.93	0	6	0.51	0.75	1.51	342.47	-0.27	90.56	7
34	689.01	3.86	-5.76	1.5	688.94	0.56	7	1.17	136.1	2.1	0.22	1.88	365.58	0.56	0	7	0.41	0.79	1.23	513.47	-0.34	145.87	7
35	-9999	-9999	-9999	0	0	0	0	0	120.9	2.16	0.25	1.91	0	0	0	0	1	0.55	0	-1.13	0	12.67	0

	CPT-3	In situ data										Basic output data											
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn	U2	I(B)	Mod. SBTn
1	20.47	0.1	0	-0.2	20.47	0.51	5	2.37	101.1	0.05	0	0.05	403.62	0.51	0	6	0.53	4.94	1.76	95.34	0	88.7	7
2	21.41	0.1	-0.18	-0.1	21.41	0.49	5	2.35	101.21	0.1	0	0.1	210.46	0.49	0	6	0.55	3.67	1.84	73.91	-0.13	78.99	7
3	22.24	0.1	-0.39	-0.1	22.24	0.47	5	2.33	101.31	0.15	0	0.15	145.55	0.47	0	6	0.58	3.06	1.89	63.91	-0.19	73.75	7
4	21.3	0.1	-0.46	-0.1	21.3	0.49	5	2.35	101.2	0.2	0	0.2	104.24	0.5	0	6	0.61	2.73	1.96	54.5	-0.16	66.51	7
5	38.53	1.15	-0.03	-0.2	38.53	2.98	4	2.55	120.19	0.26	0	0.26	145.79	3	0	5	0.72	2.72	2.25	98.47	-0.01	29.67	5
6	94.82	3.03	0.64	-0.2	94.83	3.19	5	2.3	129.48	0.33	0	0.33	288.73	3.2	0	8	0.66	2.17	2.09	194.18	0.14	29.49	5
7	164.58	3.13	0.64	-0.2	164.59	1.9	6	1.97	131.07	0.39	0	0.39	417.92	1.91	0	6	0.56	1.75	1.82	271.1	0.12	47.87	7
8	179.61	2.3	0.83	-0.1	179.62	1.28	6	1.82	129.02	0.46	0	0.46	391.87	1.28	0	6	0.52	1.54	1.69	261.29	0.13	66.98	7
9	142.02	0.94	1.1	-0.2	142.03	0.66	6	1.7	121.91	0.52	0	0.52	273.09	0.66	0	6	0.48	1.41	1.59	188.18	0.15	101.64	7
10	151.63	1.15	1.29	-0.1	151.64	0.76	6	1.71	123.53	0.58	0	0.58	260.45	0.76	0	6	0.49	1.35	1.62	192.12	0.16	93.54	7
11	126.57	1.15	1.38	0	126.58	0.91	6	1.82	123.09	0.64	0	0.64	196.29	0.91	0	6	0.54	1.31	1.74	156.14	0.15	78.21	7
12	166.04	1.46	1.62	0	166.06	0.88	6	1.73	125.52	0.7	0	0.7	234.74	0.88	0	6	0.52	1.23	1.66	192.84	0.17	84.34	7
13	238.51	1.98	1.84	0	238.53	0.83	6	1.6	128.64	0.77	0	0.77	309.37	0.83	0	6	0.48	1.17	1.55	261.79	0.17	94.22	7
14	255.53	1.98	2.01	0	255.56	0.78	6	1.55	128.81	0.83	0	0.83	305.79	0.78	0	6	0.47	1.12	1.52	269.33	0.17	99.84	7
15	218.46	1.78	2.21	0	218.49	0.81	6	1.62	127.61	0.9	0	0.9	242.62	0.82	0	6	0.5	1.09	1.59	223.35	0.18	92.52	7
16	143.48	2.4	2.48	0	143.51	1.67	6	1.97	128.8	0.96	0	0.96	148.29	1.68	0	6	0.64	1.06	1.95	143.26	0.19	49.22	7
17	361.94	2.82	2.67	-0.1	361.98	0.78	6	1.45	132.23	1.03	0	1.03	351.31	0.78	0	6	0.45	1.01	1.45	345.69	0.19	104.6	7
18	339.49	3.13	2.76	-0.1	339.53	0.92	6	1.53	132.84	1.09	0	1.09	309.44	0.93	0	6	0.49	0.98	1.53	314.75	0.18	89.87	7
19	503.34	3.97	3.13	-0.1	503.38	0.79	6	1.37	135.53	1.16	0	1.16	432.39	0.79	0	6	0.43	0.96	1.38	455.94	0.19	108.29	7
20	529.34	4.7	3.49	-0.2	529.38	0.89	6	1.4	136.89	1.23	0	1.23	429.4	0.89	0	6	0.45	0.93	1.42	466.58	0.2	98.24	7
21	455.82	3.34	3.95	-0.2	455.87	0.73	6	1.37	134.03	1.3	0	1.3	350.47	0.74	0	6	0.44	0.91	1.4	392.56	0.22	112.27	7
22	498.12	4.59	4.38	-0.4	498.17	0.92	6	1.43	136.58	1.37	0	1.37	363.86	0.92	0	6	0.47	0.89	1.46	416.44	0.23	93.69	7
23	512.53	4.8	4.69	-0.6	512.59	0.94	6	1.43	136.97	1.43	0	1.43	356.53	0.94	0	6	0.48	0.87	1.46	418.1	0.24	92.48	7
24	505.53	3.97	4.52	-0.9	505.59	0.78	6	1.37	135.54	1.5	0	1.5	335.72	0.79	0	6	0.46	0.85	1.41	405.82	0.22	106.77	7
25	441.73	3.45	4.69	-1.3	441.78	0.78	6	1.4	134.18	1.57	0	1.57	280.64	0.78	0	6	0.48	0.83	1.45	344.73	0.22	104.37	7
26	517.96	5.53	4.44	-1.2	518.01	1.07	6	1.47	137.28	1.64	0	1.64	315.38	1.07	0	6	0.51	0.8	1.53	390.7	0.2	81.98	7
27	431.18	4.18	0.28	-1	431.18	0.97	6	1.48	135.53	1.71	0	1.71	251.87	0.97	0	6	0.52	0.78	1.55	316.59	0.01	86.42	7
28	196.43	5.12	-0.28	-0.9	196.42	2.61	5	2.03	135.1	1.77	0	1.77	109.82	2.63	0	5	0.75	0.68	2.14	125.02	-0.01	33.87	7
29	204.16	3.45	-0.09	-0.9	204.15	1.69	6	1.87	132.3	1.84	0.03	1.81	111.93	1.7	0	6	0.69	0.69	1.98	132.11	-0.02	48.17	7
30	508.77	3.45	-0.31	-0.4	508.77	0.68	7	1.31	134.52	1.91	0.06	1.84	274.93	0.68	0	6	0.47	0.77	1.39	369.96	-0.05	118.17	7
31	85	1.98	-1.1	-0.3	84.99	2.33	5	2.23	126.12	1.97	0.09	1.88	44.27	2.39	0	5	0.85	0.61	2.4	48.16	-0.09	31.42	5
32	47.62	0.84	-1.2	-0.3	47.6	1.75	5	2.33	118.38	2.03	0.12	1.9	23.94	1.83	0	4	0.91	0.59	2.54	25.24	-0.11	30.31	5
33	32.27	0.42	-1.63	-0.1	32.25	1.3	5	2.39	112.36	2.08	0.16	1.93	15.64	1.38	-0.01	4	0.95	0.57	2.64	16.14	-0.14	28.31	4
34	25.58	0.52	-2.02	0.2	25.56	2.04	4	2.59	113.42	2.14	0.19	1.95	11.99	2.23	-0.01	4	1	0.54	2.86	11.99	-0.17	22.73	4
35	29.24	0.63	-2.11	0.3	29.21	2.14	4	2.55	115.08	2.2	0.22	1.98	13.64	2.32	-0.01	4	1	0.53	2.82	13.64	-0.19	23.26	4
36	40	0.84	-2.17	0.4	39.97	2.09	5	2.44	117.95	2.26	0.25	2.01	18.78	2.22	-0.01	4	0.97	0.54	2.69	19.14	-0.2	25.93	4
37	30.6	0.73	-2.3	0.4	30.57	2.39	4	2.57	116.32	2.32	0.28	2.04	13.88	2.59	-0.02	4	1	0.52	2.84	13.88	-0.22	22.55	4
38	26.52	0.94	-2.21	0.4	26.5	3.55	4	2.72	117.81	2.37	0.31	2.06	11.69	3.9	-0.02	3	1	0.51	3.01	11.69	-0.23	18.77	2
39	50.86	1.98	-2.05	0.5	50.83	3.9	4	2.55	124.87	2.44	0.34	2.09	23.11	4.1	-0.01	3	1	0.51	2.79	23.11	-0.23	20.1	3
40	197.16	3.86	-1.93	0.7	197.13	1.96	6	1.93	133.05	2.5	0.37	2.13	91.41	1.99	0	5	0.75	0.59	2.09	109.2	-0.24	41.56	7
41	78.53	2.72	-1.84	0.8	78.51	3.46	5	2.38	128.22	2.57	0.41	2.16	35.12	3.58	-0.01	4	0.94	0.51	2.6	36.53	-0.25	23.19	5
42	156.33	2.82	-1.7	0.9	156.31	1.8	6	1.97	130.18	2.63	0.44	2.2	69.97	1.83	0	5	0.77	0.57	2.15	82.64	-0.25	41.8	7
43	95.76	2.61	-1.56	1	95.74	2.73	5	2.24	128.42	2.7	0.47	2.23	41.74	2.81	-0.01	4	0.9	0.51	2.47	45.12	-0.26	28.04	5
44	29.24	0.63	-1.56	1.2	29.22	2.14	4	2.55	115.08	2.75	0.5	2.26	11.73	2.37	-0.02	4	1	0.47	2.88	11.73	-0.27	22.23	4
45	54.82	1.78	-1.45	1.2	54.81	3.24	4	2.47	124.24	2.82	0.53	2.29	22.74	3.41	-0.01	4	1	0.46	2.75	22.74	-0.28	22.17	5
46	28.2	1.04	-1.47	1.2	28.18	3.71	4	2.72	118.73	2.88	0.56	2.31	10.93	4.13	-0.03	3	1	0.46	3.05	10.93	-0.29	18.18	2
47	31.85	1.15	-1.38	1.2	31.83	3.61	4	2.67	119.73	2.94	0.59	2.34	12.33	3.98	-0.02	3	1	0.45	2.99	12.33	-0.3	18.76	2
48	28.09	0.73	-1.38	1.3	28.07	2.6	4	2.62	116.11	2.99	0.62	2.37	10.58	2.91	-0.03	3	1	0.45	2.97	10.58	-0.31	20.41	2
49	106.41	3.55	-1.01	1.4	106.4	3.34	5	2.28	130.93	3.06	0.66	2.4	42.98	3.44	-0.01	4	0.93	0.47	2.52	45.71	-0.3	24.54	5
50	23.39	0.52	-1.01	1.5	23.38	2.23	4	2.64	113.2	3.12	0.69	2.43	8.34	2.58	-0.04	3	1	0.44	3.03	8.34	-0.31	20.05	2
51	24.54	0.84	-0.92	1.4	24.53	3.41	4	2.74	116.76	3.17	0.72	2.46	8.69	3.91	-0.04	3	1	0.43	3.11	8.69	-0.32	17.97	2
52	161.03	2.72	-0.83	1.4	161.02	1.69	6	1.94	129.97	3.24	0.75	2.49	63.35	1.72	-0.01	5	0.79	0.51	2.16	75.87	-0.32	42.82	7
53	204.68	3.45	-0.74	1.4	204.67	1.68	6	1.87	132.3	3.31	0.78	2.53	79.73	1.71	0	5	0.76	0.52	2.07	98.31	-0.33	45.46	7
54	197.16	2.92	-0.64	1.5	197.15	1.48	6	1.84	131.01	3.37	0.81	2.56	75.7	1.51	0	5	0.75	0.51	2.05	94.29	-0.33	49.13	7
55	154.13	3.34	-0.46	1.5	154.13	2.17	5	2.03	131.39	3.44	0.84	2.59	58.08	2.22	-0.01	5	0.84	0.47	2.27	67.2	-0.34	35.25	7
56	58.9	2.72	-0.46	1.5	58.89	4.61	4	2.55	127.52	3.5	0.87	2.63	21.09	4.9	-0.02	3	1	0.4	2.87	21.09	-0.35	17.93	3
57	78.95	2.92	-0.37	1.6	78.94	3.7	4	2.4	128.78	3.57	0.9	2.66	28.34	3.88	-0.01	4	1	0.4	2.71	28.34	-0.35	21.31	3
58	26.42	0.73	-0.28	1.6	26.42	2.77	4	2.66	115.96	3.62	0.94	2.69	8.48	3.21	-0.04	3	1	0.39	3.07	8.48	-0.36	19.01	2
59	26.11	0.73	-0.18	1.6	26.1	2.8	4	2.66	115.93	3.68	0.97	2.71	8.26	3.26	-0.04	3	1	0.39	3.09	8.26	-0.36	18.84	2
60	37.07	0.21	-0.18	1.7	37.07	0.56	5	2.16	107.62	3.73	1	2.74	12.18	0.63	-0.03	5	0.96	0.4	2.58	12.63	-0.37	29.04	4

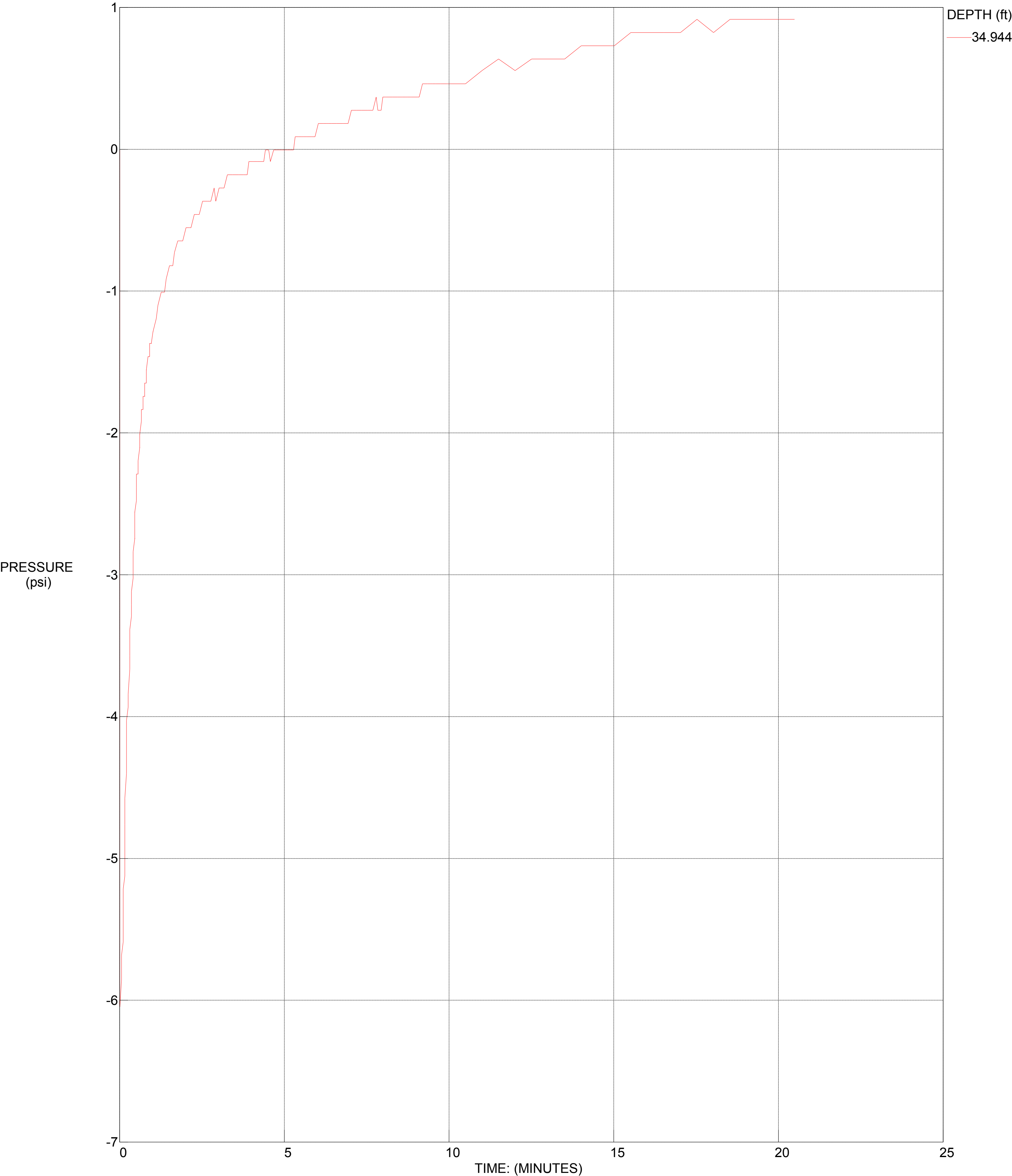
CPT-4 In situ data

Basic output data

Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn	U2	I(B)	Mod. SBTn
1	11.9	0.1	-0.09	0	11.9	0.88	4	2.68	99.78	0.05	0	0.05	237.44	0.88	0	6	0.61	6.48	1.99	72.55	-0.13	61.64	7
2	21.51	0.84	-0.18	0.1	21.51	3.88	3	2.82	116.44	0.11	0	0.11	197.83	3.9	0	8	0.74	5.36	2.31	108.46	-0.12	24.01	5
3	27.36	0.63	-0.18	0.2	27.36	2.29	4	2.59	114.92	0.17	0	0.17	164.31	2.3	0	5	0.69	3.6	2.18	92.5	-0.08	36.2	7
4	30.91	0.21	-0.18	0.2	30.91	0.68	5	2.26	107.18	0.22	0	0.22	140.06	0.68	0	6	0.59	2.54	1.92	73.68	-0.06	69.65	7
5	56.29	0.21	-0.09	0.2	56.29	0.37	6	1.91	108.64	0.27	0	0.27	204.82	0.37	0	6	0.5	1.95	1.66	103.43	-0.02	104.48	7
6	51.06	0.31	0	0.1	51.06	0.61	6	2.05	111.37	0.33	0	0.33	154.12	0.62	0	6	0.56	1.92	1.81	91.89	0	80.39	7
7	112.68	2.3	0.12	0.1	112.68	2.04	5	2.1	127.88	0.39	0	0.39	285.58	2.05	0	6	0.61	1.82	1.94	193.25	0.02	43.67	7
8	83.75	3.76	0.48	0.2	83.76	4.49	4	2.44	130.76	0.46	0	0.46	181.72	4.51	0	9	0.74	1.86	2.28	146.4	0.08	21.4	3
9	126.36	2.72	0.74	0.2	126.37	2.15	5	2.09	129.38	0.52	0	0.52	240.56	2.16	0	6	0.62	1.55	1.97	184.53	0.1	41.55	7
10	124.79	2.72	0.83	0.3	124.8	2.18	5	2.09	129.35	0.59	0	0.59	211.3	2.19	0	5	0.64	1.45	1.99	170.64	0.1	40.78	7
11	145.05	2.3	0.92	0.2	145.06	1.58	6	1.95	128.5	0.65	0	0.65	221.44	1.59	0	6	0.59	1.33	1.87	181.77	0.1	53.39	7
12	131.47	3.34	1.01	0.2	131.49	2.54	5	2.13	131	0.72	0	0.72	182.21	2.56	0	5	0.67	1.3	2.06	160.26	0.1	35.51	7
13	117.48	4.18	1.01	0.2	117.49	3.56	5	2.27	132.36	0.78	0	0.78	148.93	3.58	0	8	0.73	1.25	2.22	137.43	0.09	26.24	5
14	112.47	3.65	1.1	0.2	112.48	3.25	5	2.25	131.27	0.85	0	0.85	131.43	3.27	0	5	0.73	1.18	2.21	123.97	0.09	28.15	5
15	107.04	3.03	1.01	0.3	107.05	2.83	5	2.22	129.78	0.91	0	0.91	116.09	2.85	0	5	0.73	1.11	2.2	111.61	0.08	31.31	5
16	146.62	3.55	0.92	0.3	146.63	2.42	5	2.08	131.71	0.98	0	0.98	148.59	2.44	0	5	0.69	1.05	2.07	145.06	0.07	36.6	7
17	102.86	3.55	0.88	0.3	102.87	3.45	5	2.3	130.84	1.05	0	1.05	97.38	3.49	0	5	0.78	1.01	2.3	97.12	0.06	26.21	5
18	126.98	3.03	0.74	0.2	126.99	2.38	5	2.12	130.19	1.11	0	1.11	113.35	2.41	0	5	0.72	0.97	2.13	114.92	0.05	36.06	7
19	301.69	2.19	0.92	0.2	301.7	0.73	6	1.48	129.94	1.18	0	1.18	255.64	0.73	0	6	0.48	0.95	1.5	270.1	0.06	104.87	7
20	396.09	3.55	0.92	0.2	396.1	0.9	6	1.48	134.13	1.24	0	1.24	317.74	0.9	0	6	0.48	0.93	1.5	345.47	0.05	93.39	7
21	354.74	3.03	1.27	0.1	354.75	0.85	6	1.49	132.7	1.31	0	1.31	269.99	0.86	0	6	0.49	0.9	1.52	300.87	0.07	94.84	7
22	506.58	4.49	1.49	0.2	506.59	0.89	6	1.41	136.45	1.38	0	1.38	366.8	0.89	0	6	0.46	0.88	1.44	422.44	0.08	97.08	7
23	538.95	4.49	1.88	0.3	538.97	0.83	6	1.37	136.6	1.45	0	1.45	371.87	0.84	0	6	0.46	0.87	1.41	440.75	0.09	102.87	7
24	443.92	3.24	2.3	0.2	443.95	0.73	6	1.37	133.73	1.51	0	1.51	292.54	0.73	0	6	0.46	0.85	1.42	354.34	0.11	110.65	7
25	529.13	3.97	2.73	0.4	529.17	0.75	6	1.34	135.65	1.58	0	1.58	333.86	0.75	0	6	0.45	0.83	1.39	415.68	0.12	111.24	7
26	532.06	4.91	3.31	0.5	532.1	0.92	6	1.41	137.22	1.65	0	1.65	321.69	0.93	0	6	0.49	0.81	1.47	403.9	0.14	93.28	7
27	607.14	3.34	3.86	0.4	607.19	0.55	7	1.2	134.73	1.72	0	1.72	352.77	0.55	0	7	0.41	0.82	1.25	469.88	0.16	145.71	7
28	461.99	2.72	4.1	0	462.04	0.59	7	1.29	132.54	1.78	0	1.78	258.22	0.59	0	6	0.45	0.79	1.36	343.43	0.17	129.65	7
29	105.26	3.45	1.06	-0.1	105.28	3.27	5	2.28	130.68	1.85	0	1.85	55.97	3.33	0	4	0.86	0.62	2.43	60.44	0.04	25.96	5
30	409.25	2.3	0.09	0	409.25	0.56	7	1.31	131.03	1.91	0	1.91	212.89	0.56	0	6	0.47	0.76	1.4	290.81	0	128.54	7
31	625.73	4.7	-1.2	-0.1	625.71	0.75	7	1.3	137.28	1.98	0	1.98	314.69	0.75	0	6	0.47	0.75	1.37	439.66	-0.04	112.07	7
32	576.44	4.39	-1.47	0	576.42	0.76	6	1.32	136.59	2.05	0	2.05	280.13	0.76	0	6	0.48	0.73	1.41	394.3	-0.05	108.95	7
33	194.03	2.4	-1.56	0.3	194.01	1.24	6	1.78	129.53	2.11	0	2.11	90.73	1.25	0	6	0.69	0.62	1.94	112.64	-0.05	58.13	7
34	54.3	2.72	-3.22	0.3	54.26	5	4	2.6	127.32	2.18	0	2.18	23.91	5.21	0	3	1	0.49	2.85	23.91	-0.11	17.42	3
35	54.51	1.46	-5.01	0.3	54.45	2.69	5	2.41	122.8	2.24	0	2.24	23.31	2.8	-0.01	4	0.98	0.48	2.68	23.73	-0.16	24.72	5
36	80.72	2.4	-5.79	0.5	80.65	2.98	5	2.32	127.39	2.3	0	2.3	34.01	3.07	-0.01	4	0.94	0.48	2.57	35.71	-0.18	25.47	5
37	27.05	0.73	-6.44	0.7	26.97	2.71	4	2.64	116.01	2.36	0	2.36	10.42	2.97	-0.02	3	1	0.45	2.98	10.42	-0.2	20.23	2
38	31.95	0.84	-6.62	0.8	31.87	2.62	4	2.58	117.4	2.42	0	2.42	12.17	2.84	-0.02	3	1	0.44	2.91	12.17	-0.2	21.21	2
39	24.02	0.52	-6.85	1	23.93	2.18	4	2.63	113.26	2.48	0	2.48	8.66	2.43	-0.02	3	1	0.43	3	8.66	-0.2	20.49	2
40	25.06	0.73	-6.99	1.1	24.98	2.93	4	2.69	115.83	2.54	0.03	2.5	8.96	3.26	-0.02	3	1	0.42	3.06	8.96	-0.21	19.12	2
41	12.74	0.31	-7.08	1.1	12.65	2.48	3	2.89	107.97	2.59	0.06	2.53	3.98	3.11	-0.06	3	1	0.42	3.34	3.98	-0.23	16.97	2
42	29.14	1.04	-6.93	1.3	29.05	3.59	4	2.7	118.81	2.65	0.09	2.55	10.33	3.96	-0.02	3	1	0.41	3.05	10.33	-0.23	18.34	2
43	82.81	2.09	-6.8	1.3	82.73	2.52	5	2.26	126.43	2.71	0.12	2.59	30.93	2.61	-0.01	4	0.95	0.43	2.55	32.48	-0.24	27.45	5
44	195.28	4.18	-6.62	1.3	195.2	2.14	6	1.96	133.6	2.78	0.16	2.62	73.37	2.17	0	5	0.81	0.48	2.18	87.57	-0.24	37.51	7
45	189.95	3.45	-6.34	1.4	189.88	1.81	6	1.91	132.12	2.84	0.19	2.66	70.38	1.84	0	5	0.79	0.48	2.14	85.34	-0.24	41.96	7
46	92.21	3.03	-6.16	1.4	92.13	3.29	5	2.31	129.41	2.91	0.22	2.69	33.16	3.39	-0.01	4	0.97	0.4	2.61	34.02	-0.25	23.73	5
47	123.64	3.45	-5.98	1.4	123.57	2.79	5	2.18	131.07	2.97	0.25	2.73	44.25	2.86	-0.01	4	0.91	0.42	2.45	48.02	-0.25	28	5
48	41.77	1.57	-5.96	1.4	41.7	3.76	4	2.6	122.65	3.04	0.28	2.76	14.03	4.05	-0.02	3	1	0.38	2.96	14.03	-0.26	18.94	3
49	22.66	0.52	-5.88	1.5	22.59	2.31	4	2.66	113.12	3.09	0.31	2.78	7.01	2.68	-0.04	3	1	0.38	3.1	7.01	-0.26	19.16	2
50	45.74	1.15	-5.79	1.5	45.67	2.52	5	2.45	120.61	3.15	0.34	2.81	15.13	2.7	-0.02	4	1	0.38	2.82	15.13	-0.27	22.66	4

DISSIPATION

TEST ID: CPT-2
LOCATION: Norco
TEST DATE: Thu 26/Jan/2017
CLIENT: Southern California Geotechnical, Inc



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

:: Unit Weight, g (kN/m³) ::

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

where g_w = water unit weight

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

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

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

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

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

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

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

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

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

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

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

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

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

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

If $I_c > 2.20$

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

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

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

If $I_c \leq 2.20$

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

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

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

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

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

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

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

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

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

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

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

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

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

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

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

:: Soil Sensitivity, S_t ::

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

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

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

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

(applicable for $0.10 < B_q < 1.00$)

References

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

APPENDIX H

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 11G114
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California

DRILLING DATE: 2/24/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 17 feet
READING TAKEN: 8 hours

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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Brown Silty fine Sand, medium dense-damp							
	X	14					5					EI = 2 @ 0 to 5'
					Gray Brown Clayey fine Sand, medium dense-dry to damp							
	X	14					9					
5					Light Gray fine Sandy Clay, trace calcareous nodules, medium stiff to stiff-damp							
	X	7					7					
	X	8					6					
10												
	X	22					7					
15												
					Light Brown to Brown fine to coarse Sand, trace fine Gravel, medium dense to dense-dry to wet							
	X	27					1			4		
20												
	X	38					1					
25												
	X	35			@ 29 feet, Groundwater encountered during drilling		10					
30												
	X	38					11					

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

TEST BORING LOG

BORING NO. B-1

JOB NO.: 11G114	DRILLING DATE: 2/24/11	WATER DEPTH: Dry
PROJECT: Norco Ranch Commerce Park	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Norco, California	LOGGED BY: Daryl Kas	READING TAKEN: 8 hours

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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					@ 35 feet, 2- 1 inch lenses of Gray Silty Clay, soft-wet Light Brown to Brown fine to coarse Sand, trace fine Gravel, medium dense to dense-dry to wet							
					ALLUVIUM: Brown fine to coarse Sand, fine to coarse Gravel, dense-wet							
		57					9					
40												
		71					8					
45												
		82					10					
50												
					Boring Terminated at 50'							

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 11G114
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California

DRILLING DATE: 2/24/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 12 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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
		10			ALLUVIUM: Red Brown Clayey fine Sand, loose-moist	108	9					
		8			Gray Brown Silty fine Sand, calcareous veining and nodules, loose-moist	103	16					
5		8			Gray Brown Silty fine Sand to fine Sandy Silt, loose-moist	103	16					
		11			Brown Silty fine Sand, loose-moist	110	8					
10		14			Brown fine to coarse Sand, trace fine Gravel, loose to medium dense-dry to damp	110	6					
15		22					2					
20		32					4					
					Boring Terminated at 20'							

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 11G114
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California

DRILLING DATE: 2/24/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 14 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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Gray Brown Silty fine Sand, slightly porous, calcareous veining and nodules, loose-moist							
	X	8				107	15					
					Light Gray fine Sandy Silt, Iron oxide staining, slightly porous, loose-moist							
	X	11				98	18					
5	X	20				116	11					
					Gray Brown to Light Gray Brown Silty fine to medium Sand, trace fine Gravel, Iron oxide staining, medium dense-dry							
	X	30				112	2					
					Light Brown fine to medium Sand, little fine to coarse Gravel, medium dense-dry							
	X	25				107	2					
10												
					Light Brown fine Sand, medium dense-dry							
	X	22				101	2					
15												
					Brown fine to coarse Sand, trace fine Gravel, medium dense-moist							
	X	43				112	6					
20												
					Boring Terminated at 20'							

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 11G114
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California

DRILLING DATE: 2/24/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 16.5 feet
READING TAKEN: 8 hours

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)		SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
					ALLUVIUM: Gray to Brown Clayey fine Sand, calcareous veining and nodules, loose-moist							
	X	4					13					
	X	3			Gray Silty fine Sand to fine Sandy Silt, trace calcareous nodules, loose-moist		15					
5												
	X	21			Light Gray fine Sandy Silt, calcareous nodules and veining, trace Iron oxide staining, medium dense to dense-damp		20					
	X	31					11					
10												
	X	21					19					
15					Light Brown Silty fine Sand, Iron oxide staining, medium dense-dry		3					
					Light Brown to Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, Iron oxide staining, medium dense to dense-dry to wet							
	X	24					2			5		
20												
	X	33					8			5		
25												
					@ 27 feet, Groundwater encountered during drilling							
	X	43					10			5		
30												
	X	33					14			5		

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 11G114	DRILLING DATE: 2/24/11	WATER DEPTH: Dry
PROJECT: Norco Ranch Commerce Park	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 16.5 feet
LOCATION: Norco, California	LOGGED BY: Daryl Kas	READING TAKEN: 8 hours

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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Light Brown to Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, Iron oxide staining, medium dense to dense-dry to wet							
					Gray Brown fine Sandy Silt, abundant Iron oxide staining, medium dense-very moist to wet							
	X	18					29			39		
40												
					`Brown fine to coarse Sand, fine to coarse Gravel, medium dense-wet							
	X	13					10			6		
45					Gray Brown fine Sandy Clay, trace Silt, stiff-very moist		23			74		
					Gray Brown Silty Clay, Iron oxide staining, stiff-very moist							
	X	22					25	40	19			
					Gray Brown Silty fine Sand, trace fine Sandy Silt, Iron oxide staining, medium dense-very moist		21			19		
50												

Boring Terminated at 50'

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 11G114
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California
DRILLING DATE: 2/24/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas
WATER DEPTH: Dry
CAVE DEPTH: 15 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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					3± inches Asphaltic concrete, 3½± inches Aggregate base							
		22			FILL: Gray Brown Silty fine Sand, trace medium Sand, medium dense-damp	113	7					
		5			ALLUVIUM: Brown Clayey fine Sand, loose-moist	100	13					
5		6				99	15					
					Gray Brown Clayey Silt, little fine Sand, calcareous veining and nodules, medium stiff-moist							
		20			Gray Brown Silty fine to medium Sand, trace Iron oxide staining, medium dense-moist	118	10					
		13				101	11					
10					Brown to Light Brown fine to coarse Sand, little fine Gravel, loose to dense-damp to moist							
		22					2					
15												
		49					4					
20												
					Boring Terminated at 20'							

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 11G114
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California
DRILLING DATE: 2/24/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas
WATER DEPTH: Dry
CAVE DEPTH: 12 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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Red Brown Silty fine Sand, loose-damp							
	X	9				103	9					
					Gray Brown Silty fine Sand, trace medium Sand, loose-damp							
	X	6				107	11					
					Gray Brown fine Sandy Silt, loose-moist							
					Gray Brown Silty fine Sand, trace calcareous nodules/veining, loose-moist							
5	X	7				93	16					
					Light Gray fine Sandy Silt to Silty fine Sand, trace calcareous nodules and veining, trace Iron oxide staining, medium dense-damp							
	X	20				103	6					
	X	16				95	4					
10												
	X	14					12					
15												
					Light Gray fine Sand, medium dense-dry							
	X	26					3					
20												

Boring Terminated at 20'

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

BORING NO. B-7

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 11G114
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California

DRILLING DATE: 2/25/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 13 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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Red Brown Silty fine to medium Sand, medium dense-damp							
		19				120	8					
					Red Brown fine to medium Sandy Clay, stiff-moist							
		20	4.5+			123	11					
					Red Brown Clayey fine to coarse Sand, medium dense-moist							
5		22			Orange Brown fine to coarse Sand, trace fine Gravel, Iron oxide staining, little fine to coarse Gravel, medium dense-dry to damp	114	5					
		22				109	3					
		18				112	3					
10												
					Gray Brown fine Sandy Silt, trace Clay, Iron oxide staining, loose-moist							
		9					16					
15												
		26					21					
					Gray Brown Silty fine Sand, Iron oxide staining, medium dense-moist		10					
20												

Boring Terminated at 20'

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 11G114
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California

DRILLING DATE: 2/25/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 15 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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Red Brown Clayey fine Sand to fine Sandy Clay, loose to medium stiff-moist							
	X	12	4.5+			121	16					
					Brown Silty fine Sand, loose-moist							
	X	9				105	13					
					Gray Brown fine Sandy Silt, loose-moist							
5	X	9			Gray Clayey Silt, medium stiff to stiff-moist	100	21					
	X	14	2.0			99	24					
					Gray Brown Silty fine Sand, trace Iron oxide staining, loose-moist							
					Light Brown to Brown fine to coarse Sand, little fine Gravel, Iron oxide staining, medium dense to dense-damp							
	X	26				108	3					
10												
	X	33				107	3					
15												
	X	41				115	4					
20												

Boring Terminated at 20'

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-9

JOB NO.: 11G114	DRILLING DATE: 2/25/11	WATER DEPTH: Dry
PROJECT: Norco Ranch Commerce Park	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH:
LOCATION: Norco, California	LOGGED BY: Daryl Kas	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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Red Brown Clayey fine Sand, loose-moist							
		7				111	12					EI = 51 @ 0 to 5'
		13	3.75			112	18					
					Gray Brown Silty Clay, stiff-moist							
5		14	3.5		Light Gray Brown Clayey Silt, abundant calcareous nodules and veining, very stiff-moist	96	25					
		12			Light Brown Silty fine Sand, loose-damp	102	6					
10		13				99	4					
					Brown fine Sand, Iron oxide staining, medium dense-dry to damp							
15		10					5					
					Brown fine to coarse Sand, little fine to coarse Gravel, Iron oxide staining, medium dense-dry to damp							
		14					3					
20					Gray Silty Clay, medium stiff-moist		20					
					Boring Terminated at 20'							

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-10

JOB NO.: 11G114
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California

DRILLING DATE: 2/25/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 13 feet
READING TAKEN: At Completion

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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Gray Brown fine Sandy Silt to Silty fine Sand, trace calcareous veining, medium dense-damp							
		20				105	15					
		15			Gray Brown fine Sandy Silt, trace Clay, Iron oxide staining, calcareous nodules and veining, loose to medium dense-damp to moist	107	18					
5		16				101	22					
		18	4.5+		Light Gray Silt, Iron oxide staining, medium dense-damp	95	12					
		30			Light Brown fine Sand, medium dense-dry to damp	103	3					
10												
					Gray Brown Silty fine to medium Sand, medium dense-damp							
		17					7					
15												
					Light Brown fine Sand, trace medium Sand, dense-dry							
		40					2					
20												

Boring Terminated at 20'

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-11

JOB NO.: 11G114
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California

DRILLING DATE: 2/25/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 22 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS: DEPTH (FEET)	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	LABORATORY RESULTS: DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	10		ALLUVIUM: Brown Silty fine Sand, slightly porous, loose to medium dense-dry		6					
	6		Brown fine Sand, little Silt, loose-damp		9					
5										
	13		Light Gray Silty fine Sand, trace medium Sand, medium dense-damp		13					
	16		Light Gray to Gray Brown fine Sandy Silt, trace Clay, Iron oxide staining, medium dense-moist		17					
10										
	23				31					
15			Gray Brown Silty fine Sand, trace medium Sand, trace Iron oxide staining, medium dense-damp		10					
	20				7			6		
20										
	10		Interbedded Silty Clay with Silty fine Sand, Iron oxide staining, stiff to medium dense-very moist		25	29	17	43		
25										
	22		Brown fine to coarse Sand, trace Silt, medium dense to dense-wet @ 27 feet, Groundwater encountered during drilling		12			4		
30										
	43				11					

SOUTHERN CALIFORNIA GEOTECHNICAL
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BORING NO. B-11

JOB NO.: 11G114	DRILLING DATE: 2/25/11	WATER DEPTH: Dry
PROJECT: Norco Ranch Commerce Park	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 22 feet
LOCATION: Norco, California	LOGGED BY: Daryl Kas	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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Brown fine to coarse Sand, trace Silt, medium dense to dense-wet							
	X	34					15					
40												
	X	44					16					
45												
					Red Brown Clayey fine Sand, medium dense-moist							
	X	27					15			45		
50												
					Boring Terminated at 50'							

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-12

JOB NO.: 11G114
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California

DRILLING DATE: 2/25/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 12 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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Red Brown Silty fine Sand, trace medium Sand, loose-damp							
	X	9				117	10					
	X	6				104	10					
5	X	6				105	8					
	X	7				103	6					
	X	9				106	7					
10												
					Brown fine to medium Sand, loose-damp to moist							
	X	11				98	4					
15					Light Gray Clayey Silt, trace fine Sand, Iron oxide staining, medium stiff-moist							
					Gray Brown Silty fine Sand, medium dense-wet							
					@ 18 feet, Groundwater encountered during drilling							
	X	20				96	24					
20												

Boring Terminated at 20'

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-13

JOB NO.: 11G114
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California

DRILLING DATE: 2/25/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: 22 feet
CAVE DEPTH: 24 feet
READING TAKEN: At Completion

FIELD RESULTS | LABORATORY RESULTS

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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Orange Brown Silty fine Sand, loose-damp							
	X	4					5					EI = 5 @ 0 to 5'
					Red Brown fine Sandy Clay, trace medium Sand, medium stiff-damp to moist							
	X	8					15					
5												
					Red Brown Silty fine to medium Sand, trace coarse Sand, loose-damp							
	X	5					7					
					Orange Brown Silty fine Sand, loose-damp							
	X	5					16					
					Light Gray Brown fine Sandy Silt, trace calcareous veining, loose-moist		8					
10												
					Brown fine to coarse Sand, little fine to coarse Gravel, medium dense-dry							
	X	22					2					
15												
	X	20					2			4		
20												
					@ 23 feet, Groundwater encountered during drilling							
	X	15					14			5		
					Gray fine Sandy Silt, medium dense-very moist to wet		24			44		
25												
					Gray Brown fine Sand, medium dense-wet							
	X	28					19			9		
30												
					Gray Silty Clay thinly interbedded with Gray Brown fine Sandy Clay, medium stiff to stiff-very moist							
	X	8					24			59		

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

TEST BORING LOG

BORING NO. B-13

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 11G114
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California

DRILLING DATE: 2/25/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: 22 feet
CAVE DEPTH: 24 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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Gray Silty Clay thinly interbedded with Gray Brown fine Sandy Clay, medium stiff to stiff-very moist							
					Gray Brown fine Sandy Silt, little Clay, loose-moist							
		17			Gray Silt Clay, Iron oxide staining, very stiff-very moist		18 26			46 76		
40												
		10					21	33	19	74		
45												
					Gray Brown Clayey Silt, trace fine Sand, Iron oxide staining, medium dense-very moist							
		13					21	29	17	61		
50												
					Boring Terminated at 50'							

TBL 11G114.GPJ SOCALGEO.GDT 3/17/11

SOUTHERN CALIFORNIA GEOTECHNICAL

A California Corporation

BORING NO. B-14

JOB NO.: 11G114-3
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California

DRILLING DATE: 6/15/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Danny Mourad

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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Red Brown Clayey fine Sand to fine Sandy Clay, medium dense to stiff-damp							
	X	14				116	12					EI = 58 @ 0 to 5'
					Brown Clayey fine Sand, trace calcareous nodules and veining, slightly porous, medium dense-damp							
	X	20				118	12					
5					Light Brown to Brown Silty fine Sand, trace Clay, slightly porous, medium dense-damp							
	X	16				116	11					
	X	20				112	7					
					Light Brown fine to medium Sand, trace coarse Sand, loose to medium dense-dry							
	X	14				101	1					
10												
					LIght Gray Brown Clayey Silt, Iron oxide staining, stiff-moist							
	X	19					35					
15												
					Gray Brown fine Sandy Silt, little Clay, dense-moist							
	X	63					24					
20												
					Boring Terminated at 20'							

TBL 11G114-3.GPJ SOCALGEO.GDT 6/30/11

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-15

JOB NO.: 11G114-3
PROJECT: Norco Ranch Commerce Park
LOCATION: Norco, California

DRILLING DATE: 6/15/11
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Danny Mourad

WATER DEPTH: Dry
CAVE DEPTH: 9 feet
READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)		SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
					ALLUVIUM: Red Brown Silty fine Sand, medium dense-dry to damp							
	X	23				108	9					
	X	16			Red Brown fine to coarse Sand, trace fine Gravel, medium dense-dry to damp	110	3					
5	X	15			Red Brown fine Sand, little medium Sand, trace coarse Sand, trace Silt, loose to medium dense-damp	105	5					
	X	34			Light Brown fine to coarse Sand, trace fine Gravel, medium dense-dry	109	3					
	X	51					2					Disturbed Sample
10												
					Light Brown fine to medium Sand, trace Silt, medium dense-damp							
	X	23					1					
15												
	X	28					2					
20												
					Boring Terminated at 20'							

TBL 11G114-3.GPJ SOCALGEO.GDT 6/30/11

TEST BORING LOG

PLATE B-15

SOUTHERN CALIFORNIA GEOTECHNICAL
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BORING NO. B-16

JOB NO.: 17G105	DRILLING DATE: 2/2/17	WATER DEPTH: 41 feet
PROJECT: Proposed C/I Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 27 feet
LOCATION: Norco, California	LOGGED BY: Anthony Luna	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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	26	4.5		ALLUVIUM: Orange Brown Clayey fine Sand to fine Sandy Clay, little medium Sand, medium dense to very stiff-damp		9					EI = 49 @ 0 to 5'
	X	10			Dark Orange Brown Silty fine to coarse Sand, trace fine Gravel, trace Clay, noduels, loose to medium dense-moist		8					
5												
	X	16			Light Orange Brown fine to coarse Sand, trace Silt, trace fine to coarse Gravel, medium dense-dry to damp		3					
	X	24					2					
10												
					Gray Brown fine Sandy Silt, trace Clay, medium dense-moist							
	X	14					13					
15												
					Light Gray fine to coarse Sand, trace Silt, trace fine Gravel, dense-dry to damp							
	X	36					2					
20												
					Gray Brown Silty fine Sand, little Clay nodules, trace Iron oxide staining, medium dense-very moist							
	X	13					29			30		
25												
					Brown Clayey fine to medium Sand, medium dense-moist to very moist							
	X	17					15			46		
30												
					Brown Clayey fine to coarse Sand, very dense-damp							
	X	60					6					

TBL 17G105.GPJ SOCALGEO.GDT 3/2/17

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-16

JOB NO.: 17G105
PROJECT: Proposed C/I Development
LOCATION: Norco, California

DRILLING DATE: 2/2/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Anthony Luna

WATER DEPTH: 41 feet
CAVE DEPTH: 27 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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Brown Clayey fine to coarse Sand, very dense-damp							
					LA SIERRA TONALITE (Klst) BEDROCK: Gray Brown Tonalite Bedrock, highly weathered, friable, phaneritic, very dense-damp to wet							
	X	78/3"					7					
40												
	X	50/4"					16					
45												
	X	50/5"					12					
50												
					Boring Terminated at 50'							

TBL 17G105.GPJ SOCALGEO.GDT 3/2/17

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-17

JOB NO.: 17G105
PROJECT: Proposed C/I Development
LOCATION: Norco, California
DRILLING DATE: 2/2/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Anthony Luna
WATER DEPTH: 30 feet
CAVE DEPTH: 20 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS: DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS: DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	8			ALLUVIUM: Gray Brown fine Sandy Silt, loose-very moist		17					
5	X	10	1.0		Light Gray Brown Silty Clay, abundant calcareous veining, stiff-very moist		17					
	X	10			Brown Silty fine Sand, loose to medium dense-moist		10					
10	X	11			Light Brown fine Sand, little Silt, medium dense-damp		7					
15	X	24			Light Gray fine to medium Sand, medium dense-dry to damp		3					
20	X	27			Light Gray fine to coarse Sand, trace fine Gravel, trace Silt, medium dense to dense-dry to damp		3					
25	X	46					4					
30	X	34			@ 28½ feet, wet		11					
	X	10	1.0		Dark Brown Silty Clay, little Silt, trace fine Sand, stiff-very moist to wet		22	37	19	59		

TBL 17G105.GPJ SOCALGEO.GDT 3/2/17

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-17

JOB NO.: 17G105
PROJECT: Proposed C/I Development
LOCATION: Norco, California

DRILLING DATE: 2/2/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Anthony Luna

WATER DEPTH: 30 feet
CAVE DEPTH: 20 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Dark Brown Silty Clay, little Silt, trace fine Sand, stiff-very moist to wet							
					Gray Brown Clayey fine Sand to fine Sandy Clay, mottled, stiff to very stiff-very moist							
	X	15	3.5				15	25	12	51		
40												
	X	12	2.5				18			45		
45												
					Light Brown Clayey fine Sand, trace Iron oxide staining, very stiff-wet							
	X	23					17			39		
50												
					Boring Terminated at 50'							

SOUTHERN CALIFORNIA GEOTECHNICAL
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BORING NO. B-18

JOB NO.: 17G105
PROJECT: Proposed C/I Development
LOCATION: Norco, California

DRILLING DATE: 2/2/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Anthony Luna

WATER DEPTH: 29 feet
CAVE DEPTH: 23 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION (SURFACE ELEVATION: --- MSL)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
				ALLUVIUM: Brown Silty fine Sand, slightly porous, medium dense-dry to damp							
	X	16				3					
				Brown Clayey fine Sand to fine Sandy Clay, dense to hard-damp							
	X	34	2.5			6					
5											
	X	30	2.5			5					
				Brown Clayey fine Sand, medium dense to dense-damp							
	X	30				7					
10											
				Brown Clayey fine to coarse Sand, medium dense-damp							
	X	24				7					
15											
				Brown fine to coarse Sand, trace Clay nodules, trace fine Gravel, very dense-damp							
	X	50				5					
20											
				Gray Brown Gravelly fine to coarse Sand, little Clay nodules, dense-damp to moist							
	X	34				8					
25											
				Gray fine to medium Sand, little coarse Sand, trace Silt, dense-wet							
	X	42				14					
30											
				Gray fine Sandy Clay, stiff-wet							
	X	12	2.0			21	31	17	55		

TBL 17G105.GPJ SOCALGEO.GDT 3/2/17

TEST BORING LOG PLATE B-3a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-18

JOB NO.: 17G105
PROJECT: Proposed C/I Development
LOCATION: Norco, California

DRILLING DATE: 2/2/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Anthony Luna

WATER DEPTH: 29 feet
CAVE DEPTH: 23 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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Gray fine Sandy Clay, stiff-wet							
					Gray Brown Clayey fine Sand, trace fine Gravel, medium dense-wet							
		23					17			37		
40												
					Gray Brown fine Sandy Clay, little Silt, very stiff-very moist to wet							
		17	3.0				22			59		
45												
					Brown Silty Clay, trace fine Sand, medium stiff to stiff-wet							
		8	0.5				31	34	17	78		
50		13				92	33					
					Boring Terminated at 50'							

TBL 17G105.GPJ SOCALGEO.GDT 3/2/17

TEST BORING LOG **PLATE B-3b**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-19

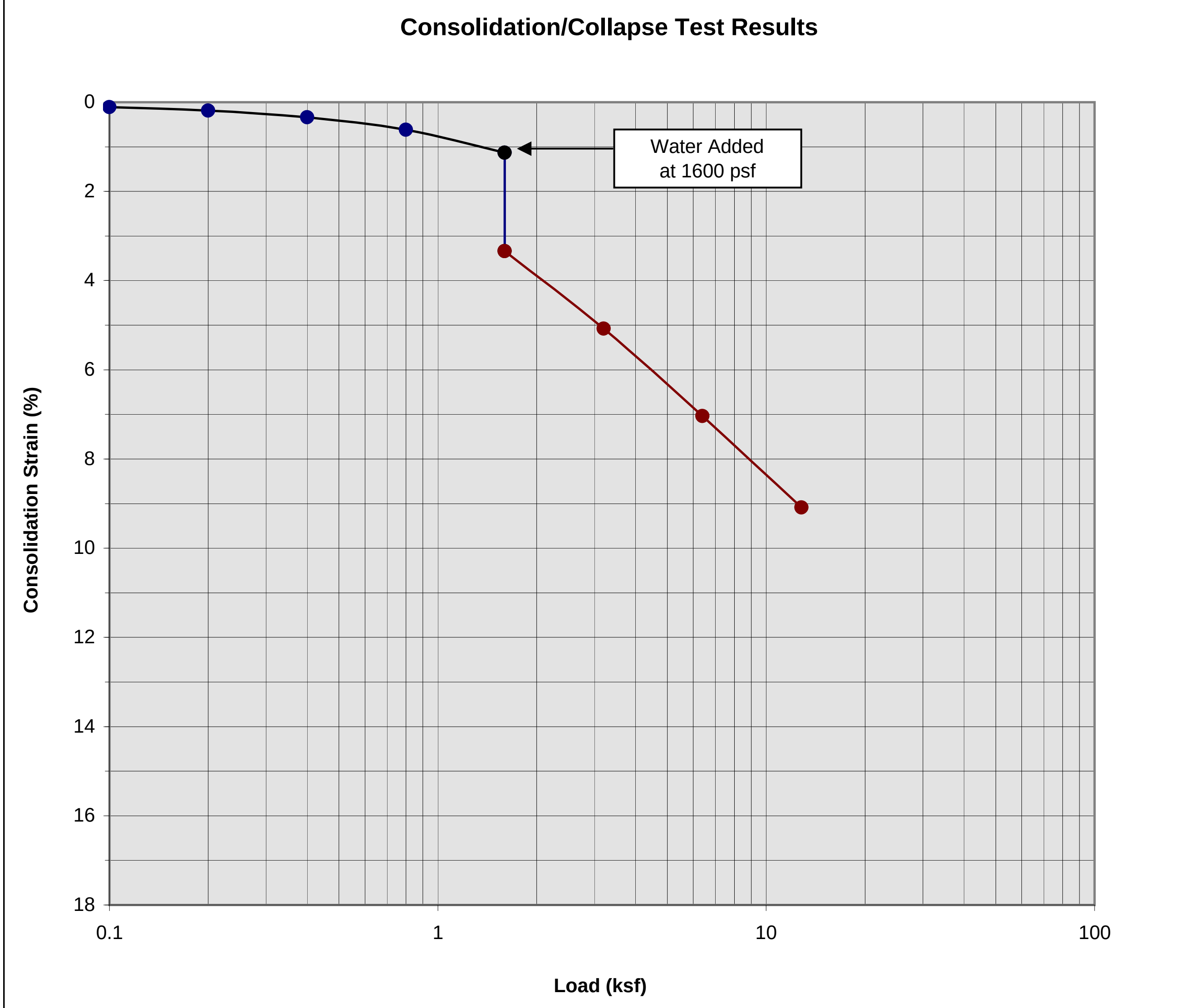
JOB NO.: 17G105
PROJECT: Proposed C/I Development
LOCATION: Norco, California

DRILLING DATE: 2/2/17
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Anthony Luna

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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Dark Brown Silty Clay, medium stiff-moist to very moist							
		7				103	20					EI = 52 @ 0 to 5'
					Gray Brown Silty Clay, abundant calcareous veining, stiff to very stiff-very moist							
		20				104	20					
5					Light Gray to White Silt, little Clay, medium dense-damp							
		23				105	7					
					Light Gray fine to medium Sand, trace coarse Sand, trace fine Gravel, medium dense-dry							
		18										No Sample Recovered
		24				126	1					
10												
					Gray Brown fine to medium Sand, little coarse Sand, trace fine Gravel, trace Clay nodules, medium dense-damp							
		20					5					
15												
					Light Gray fine to coarse Sand, little fine Gravel, dense-dry							
		40					1					
20												
					Boring Terminated at 20'							

TBL 17G105.GPJ SOCALGEO.GDT 3/2/17



Classification: Red Brown Clayey fine Sand

Boring Number:	B-2	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	1 to 2	Initial Dry Density (pcf)	107.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	118.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.21

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C- 1

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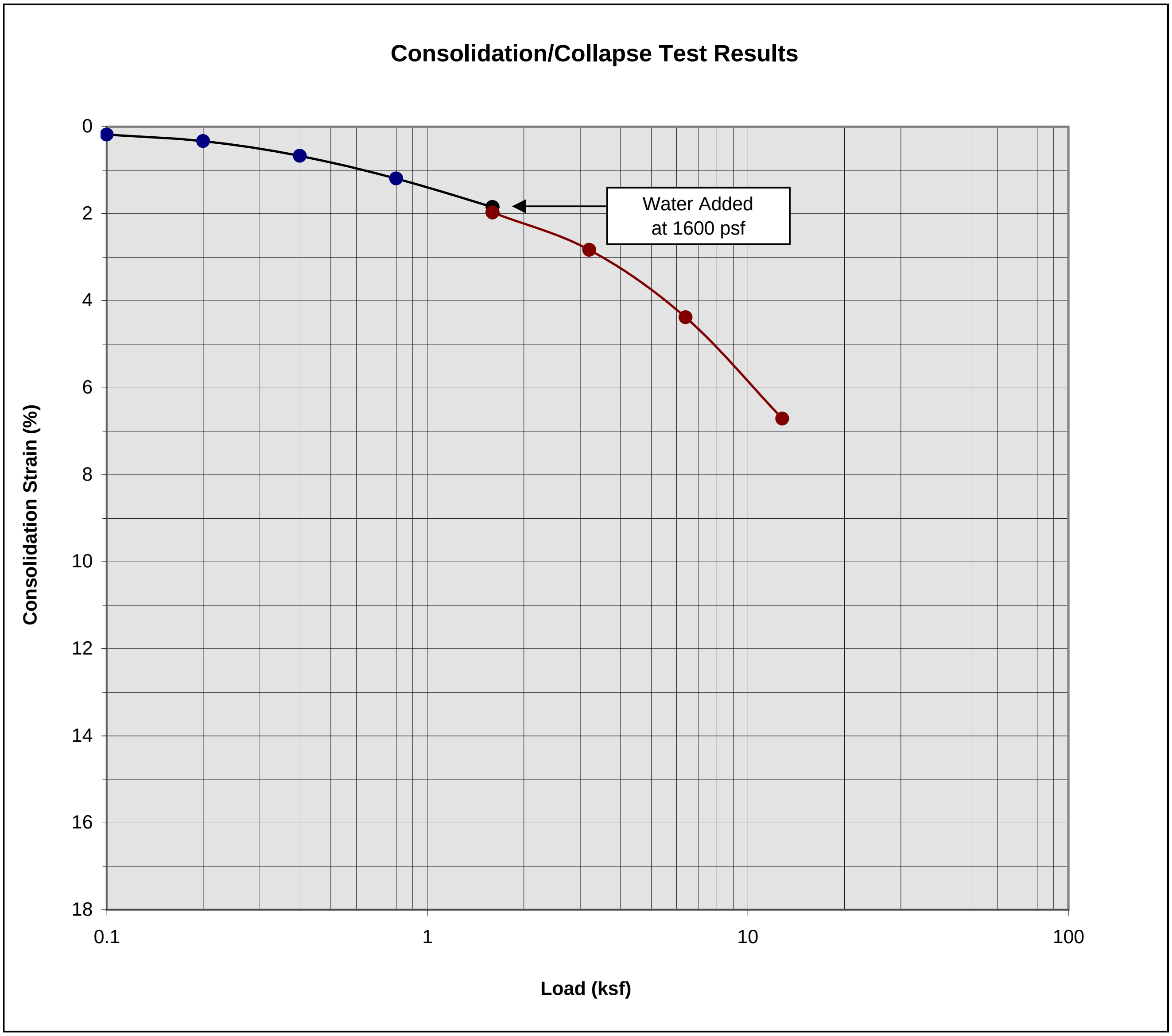

Classification: Gray Brown Silty fine Sand

Boring Number:	B-2	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	3 to 4	Initial Dry Density (pcf)	104.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.12

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C- 2

SOUTHERN CALIFORNIA GEOTECHNICAL
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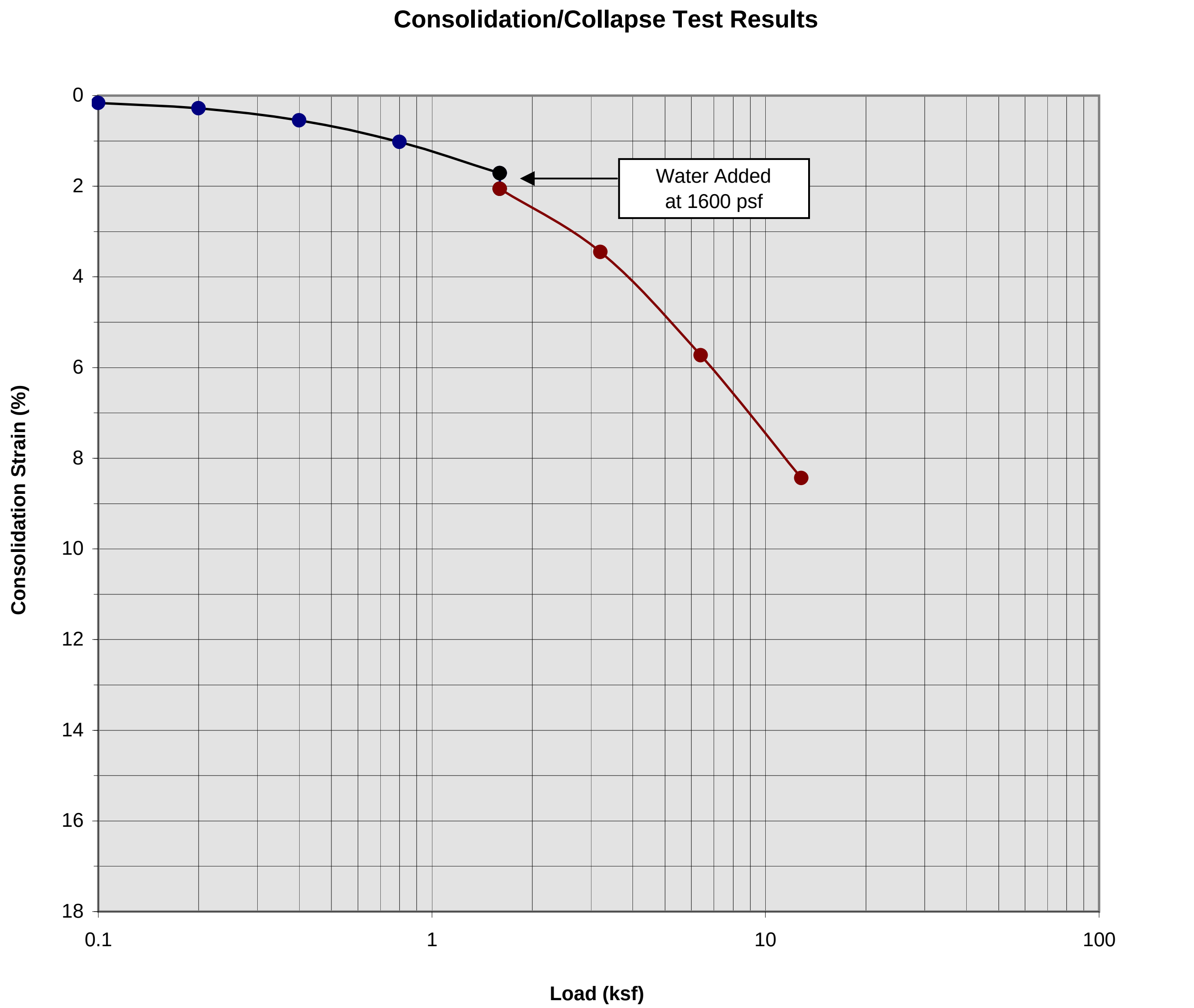

Classification: Gray Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-2	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	5 to 6	Initial Dry Density (pcf)	101.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.35

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C- 3

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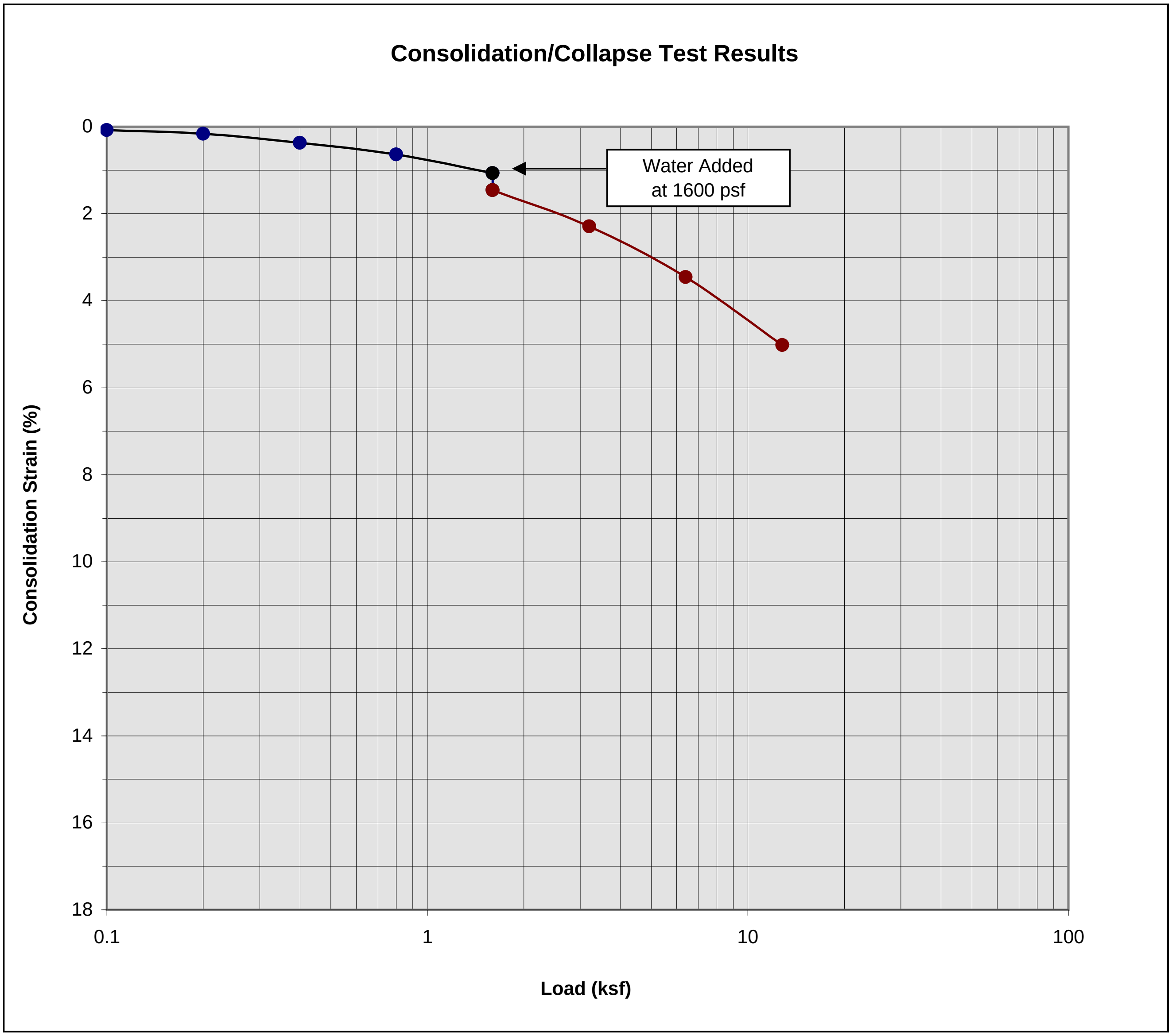

Classification: Brown Silty fine Sand

Boring Number:	B-2	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	7 to 8	Initial Dry Density (pcf)	109.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.39

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C- 4

SOUTHERN CALIFORNIA GEOTECHNICAL
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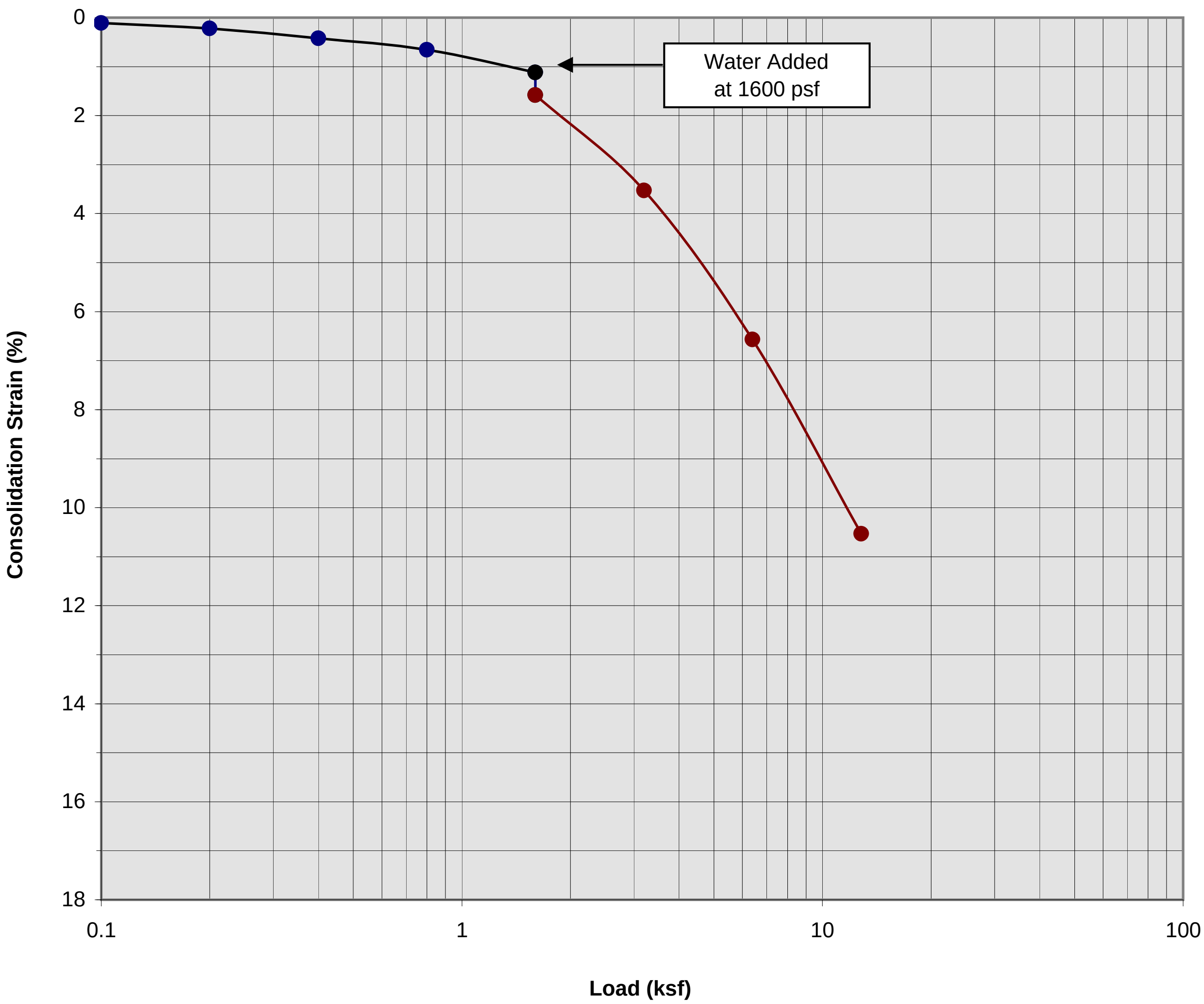
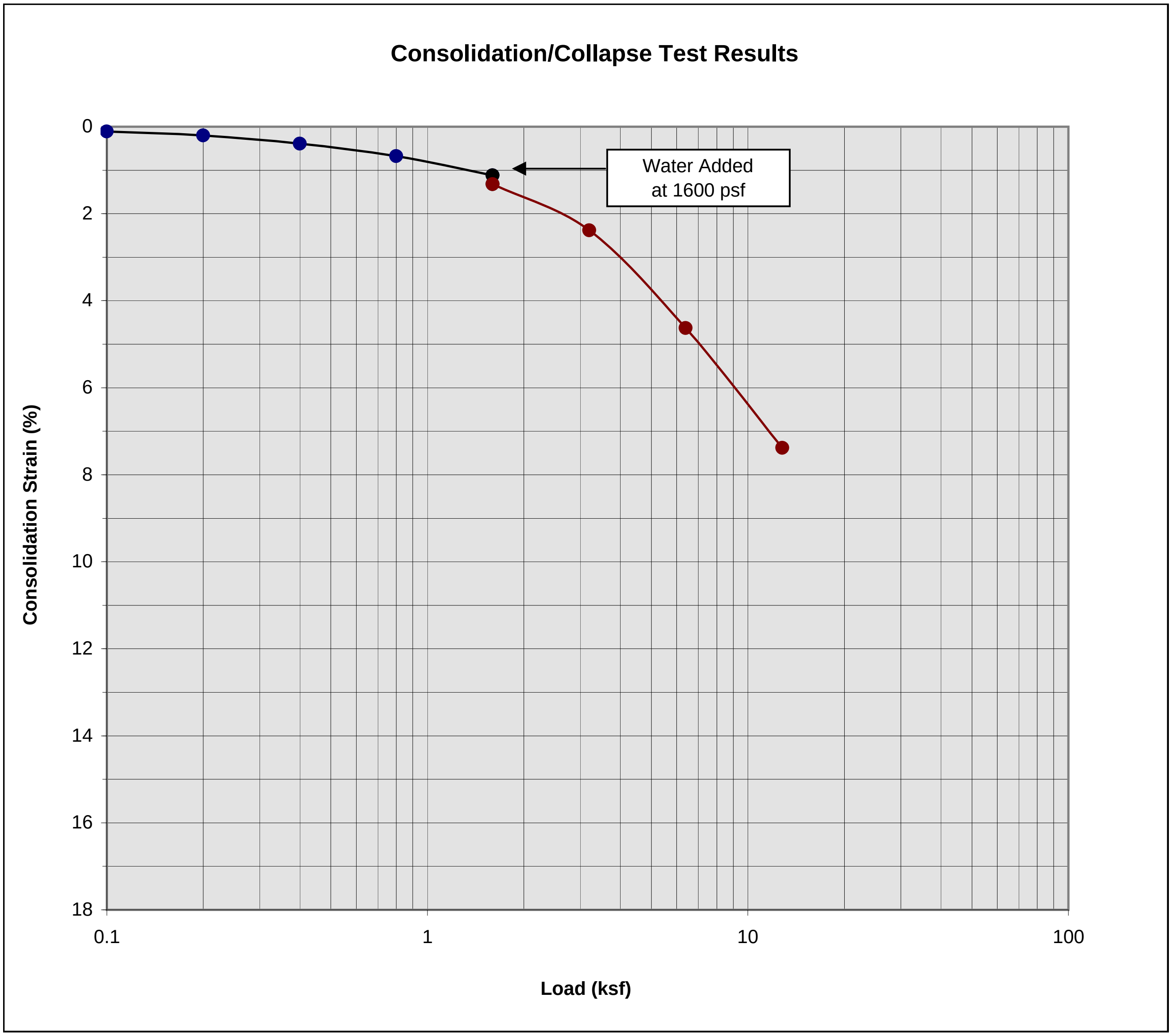
Consolidation/Collapse Test Results



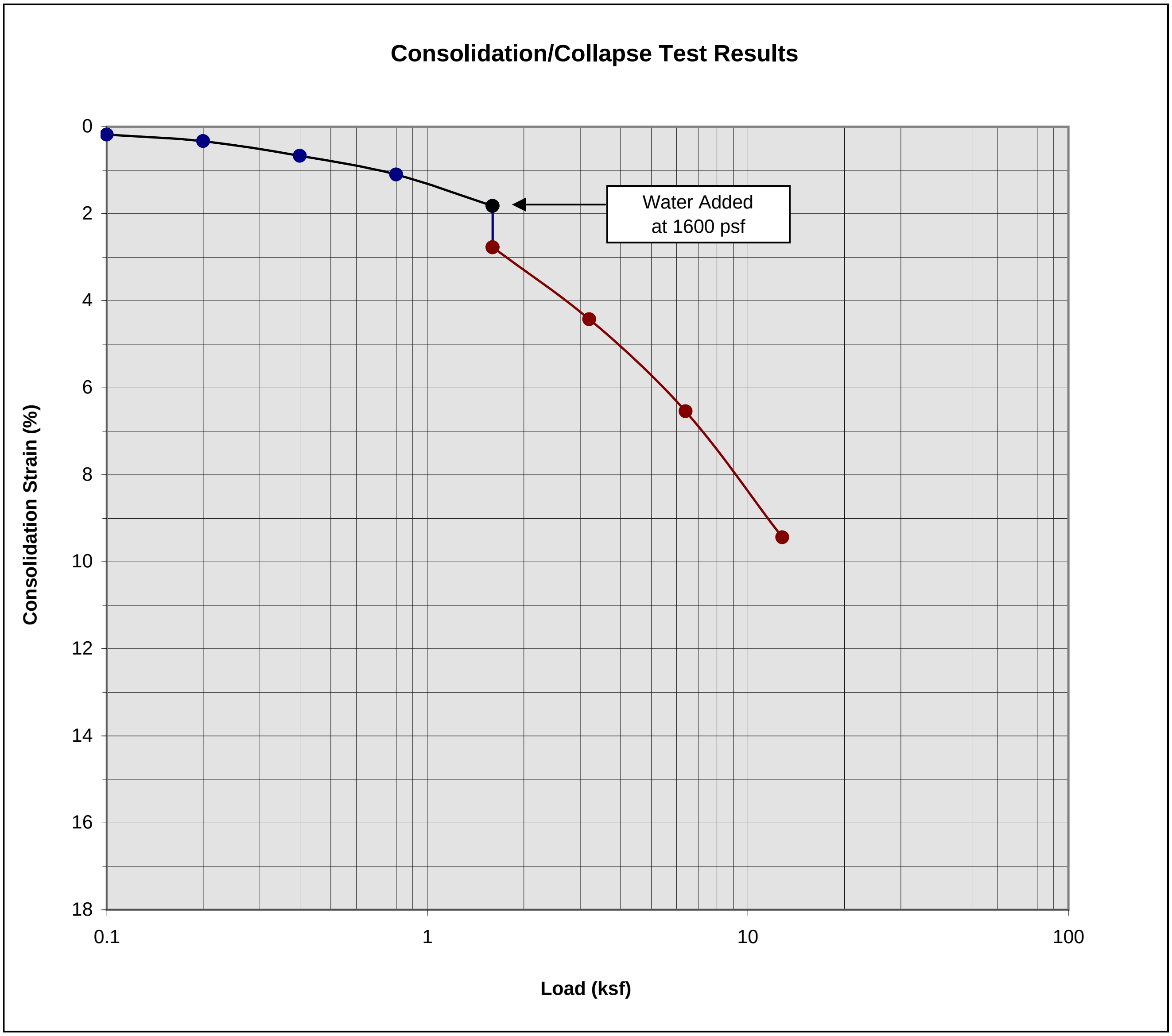
Classification: Red Brown Silty fine Sand

Boring Number:	B-6	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	1 to 2	Initial Dry Density (pcf)	104.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	117.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.46



Classification: Gray Brown fine Sandy Silt

Boring Number:	B-6	Initial Moisture Content (%)	12
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	3 to 4	Initial Dry Density (pcf)	108.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	117.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.20



Classification: Gray Brown Silty fine Sand

Boring Number:	B-6	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	25
Depth (ft)	5 to 6	Initial Dry Density (pcf)	94.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	104.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.95

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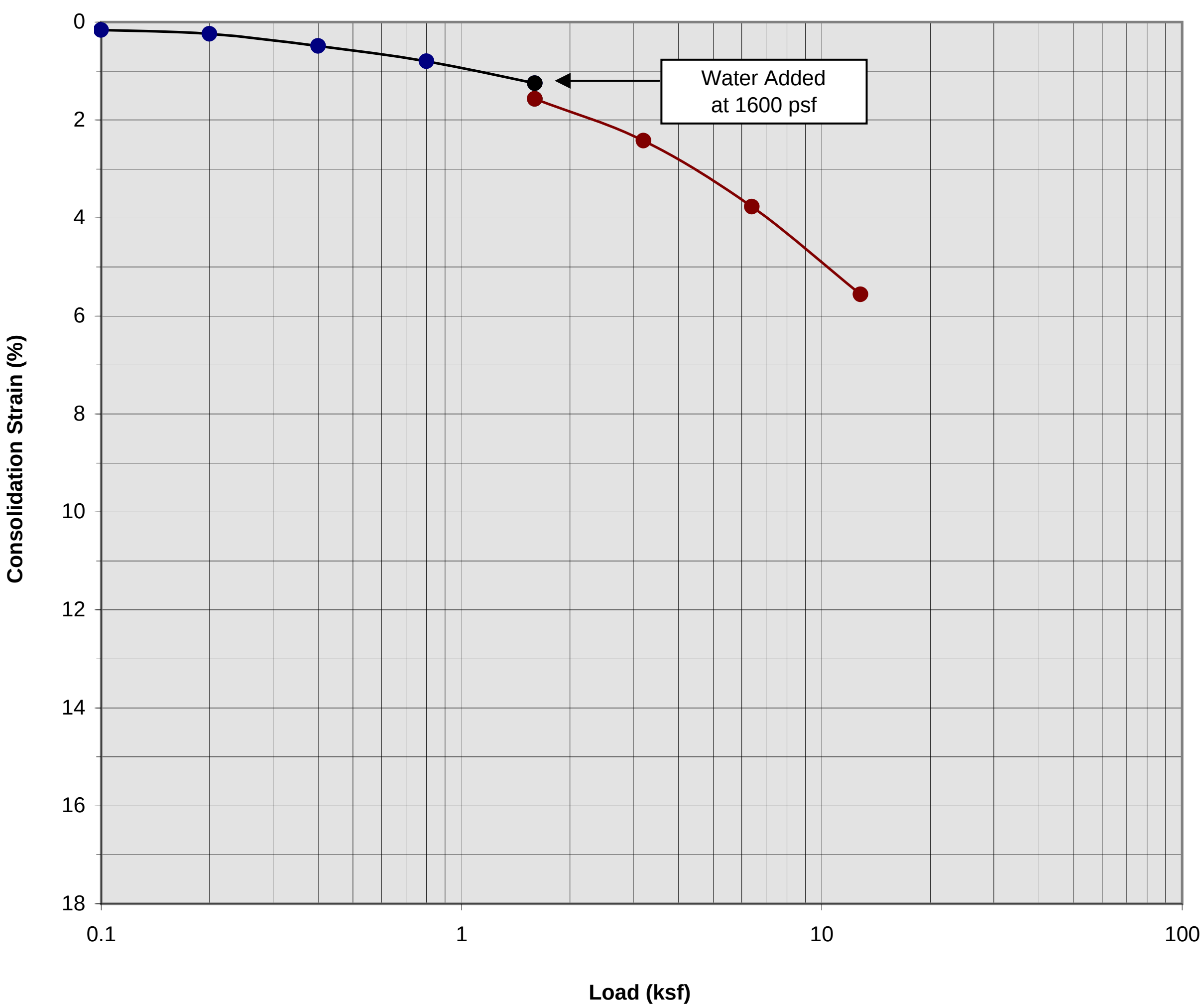
Consolidation/Collapse Test Results



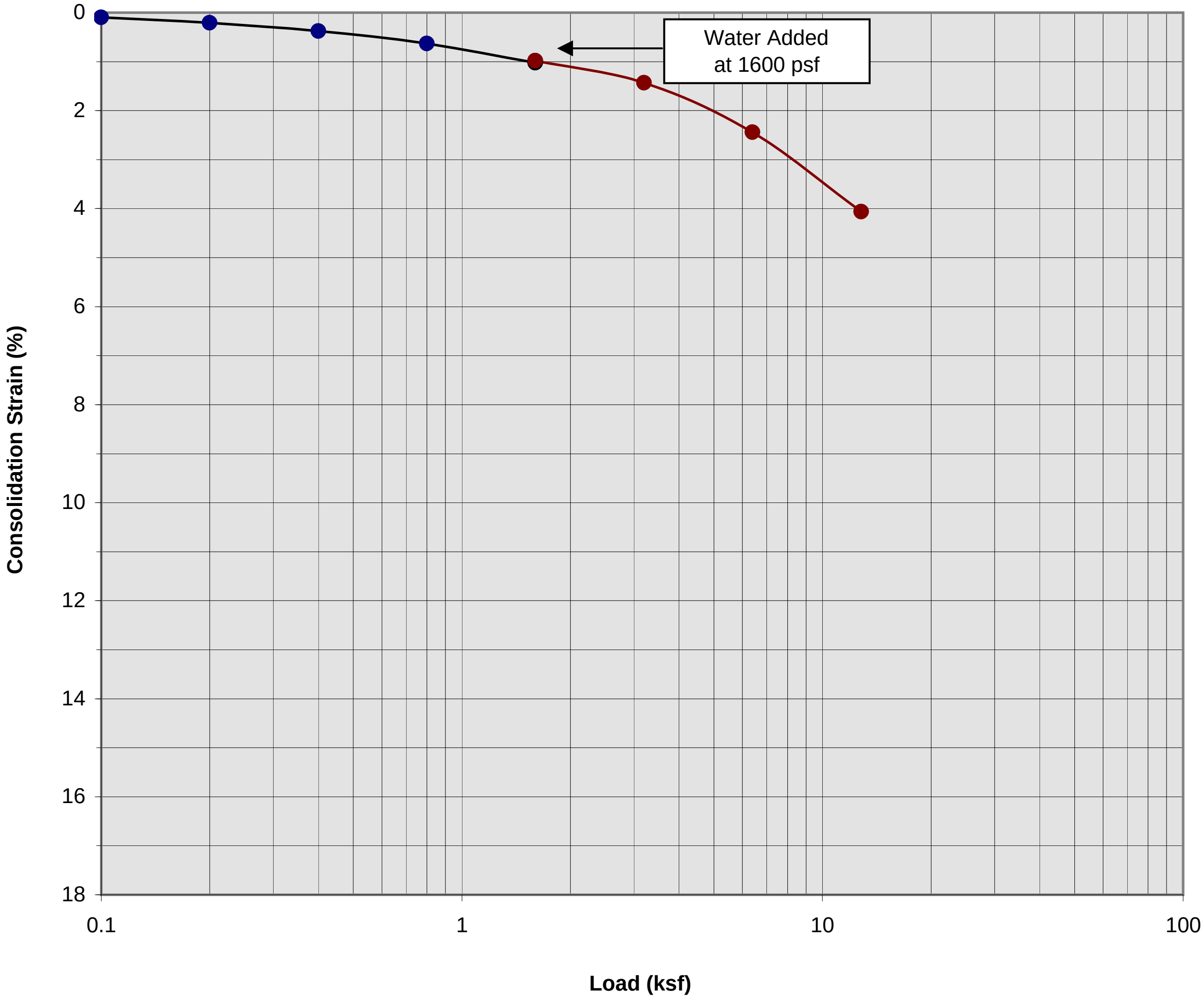
Classification: Gray Brown Silty fine Sand

Boring Number:	B-6	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	26
Depth (ft)	7 to 8	Initial Dry Density (pcf)	99.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	104.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.32

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C- 8

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

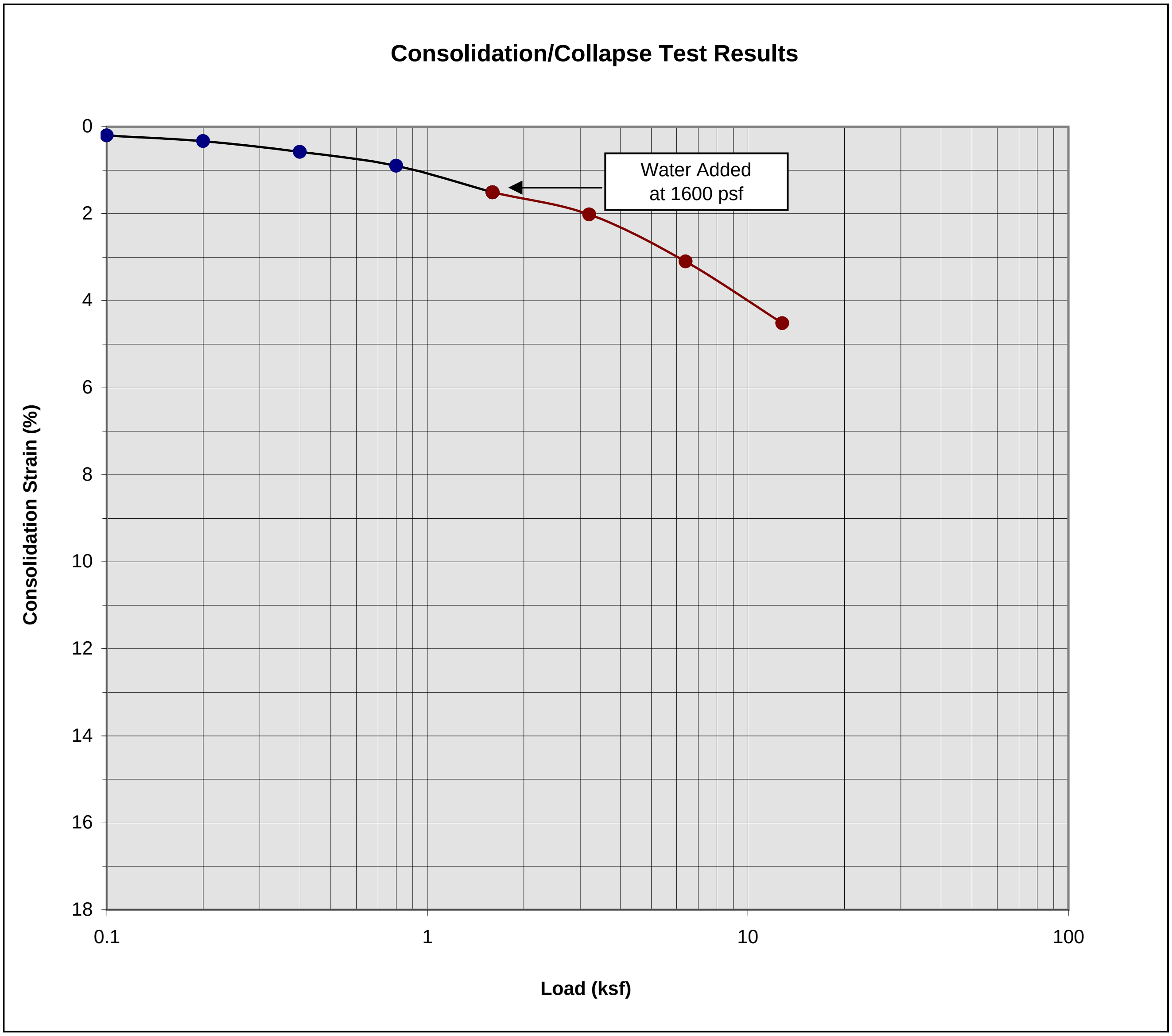


Classification: Gray Brown fine Sandy Silt to Silty fine Sand

Boring Number:	B-10	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	1 to 2	Initial Dry Density (pcf)	104.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.04

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C- 9

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

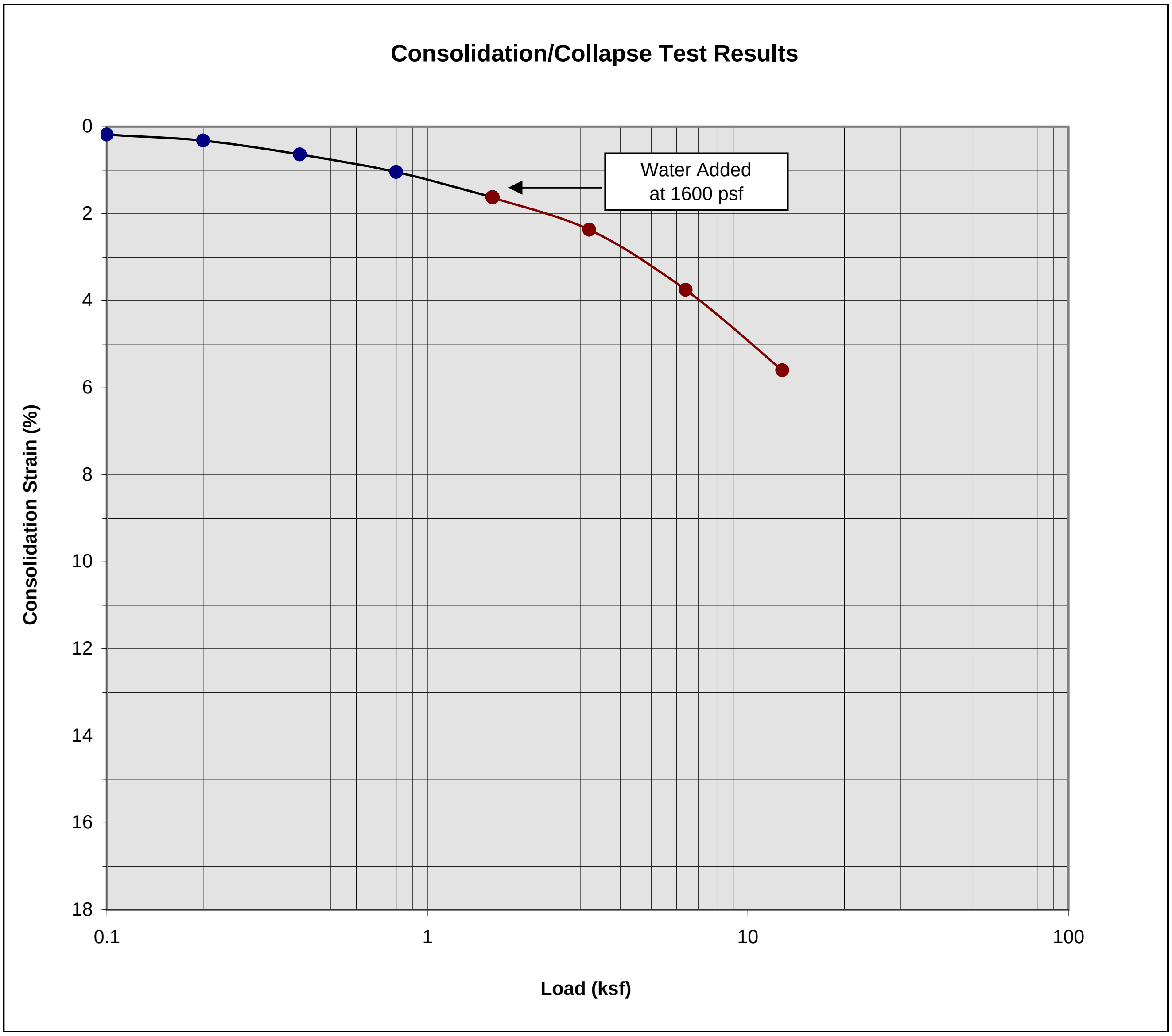


Classification: Gray Brown fine Sandy Silt, trace Clay

Boring Number:	B-10	Initial Moisture Content (%)	18
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	3 to 4	Initial Dry Density (pcf)	109.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	114.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.00

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C- 10

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

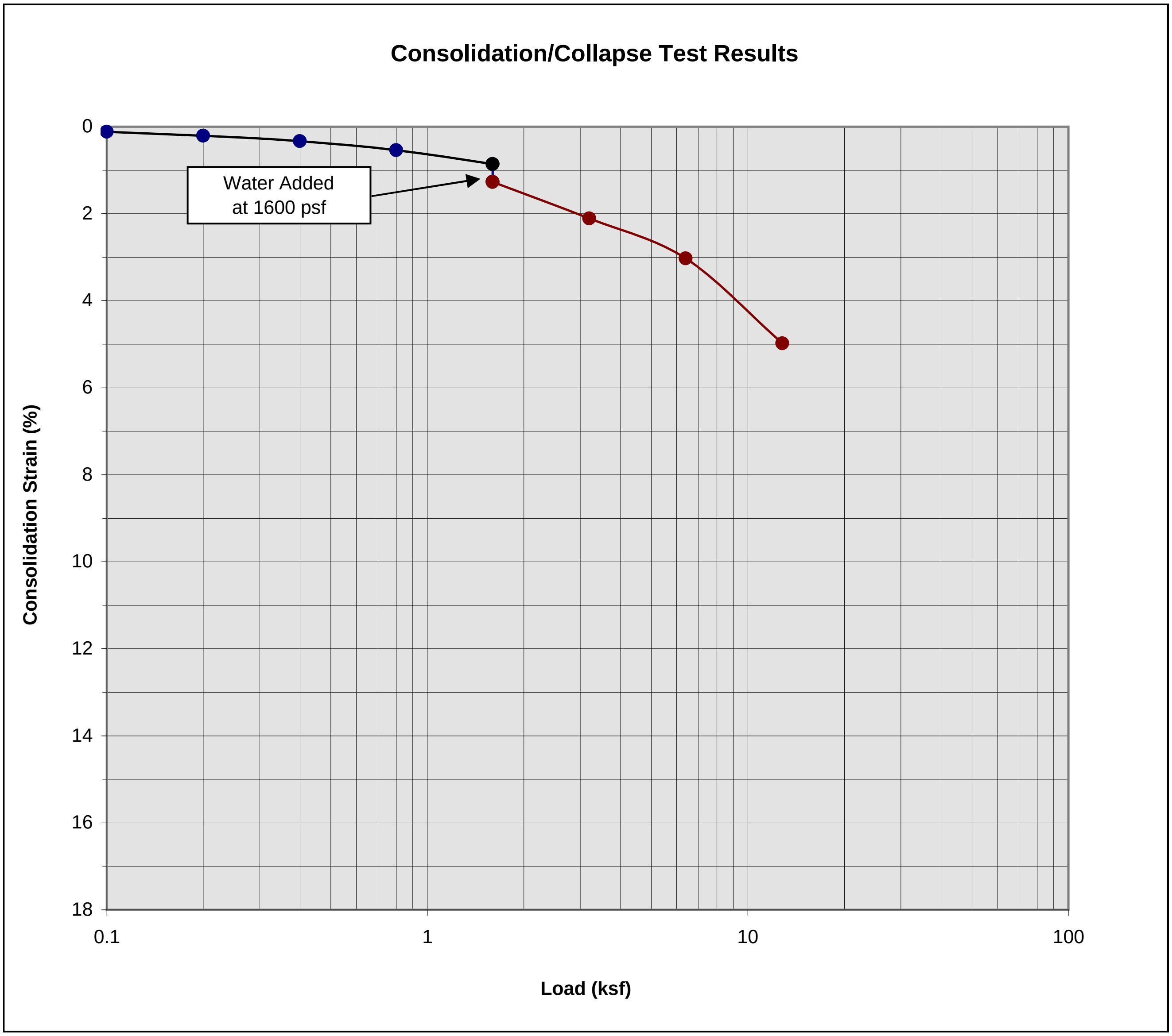


Classification: Gray Brown fine Sandy Silt, trace Clay

Boring Number:	B-10	Initial Moisture Content (%)	21
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	5 to 6	Initial Dry Density (pcf)	103.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.01

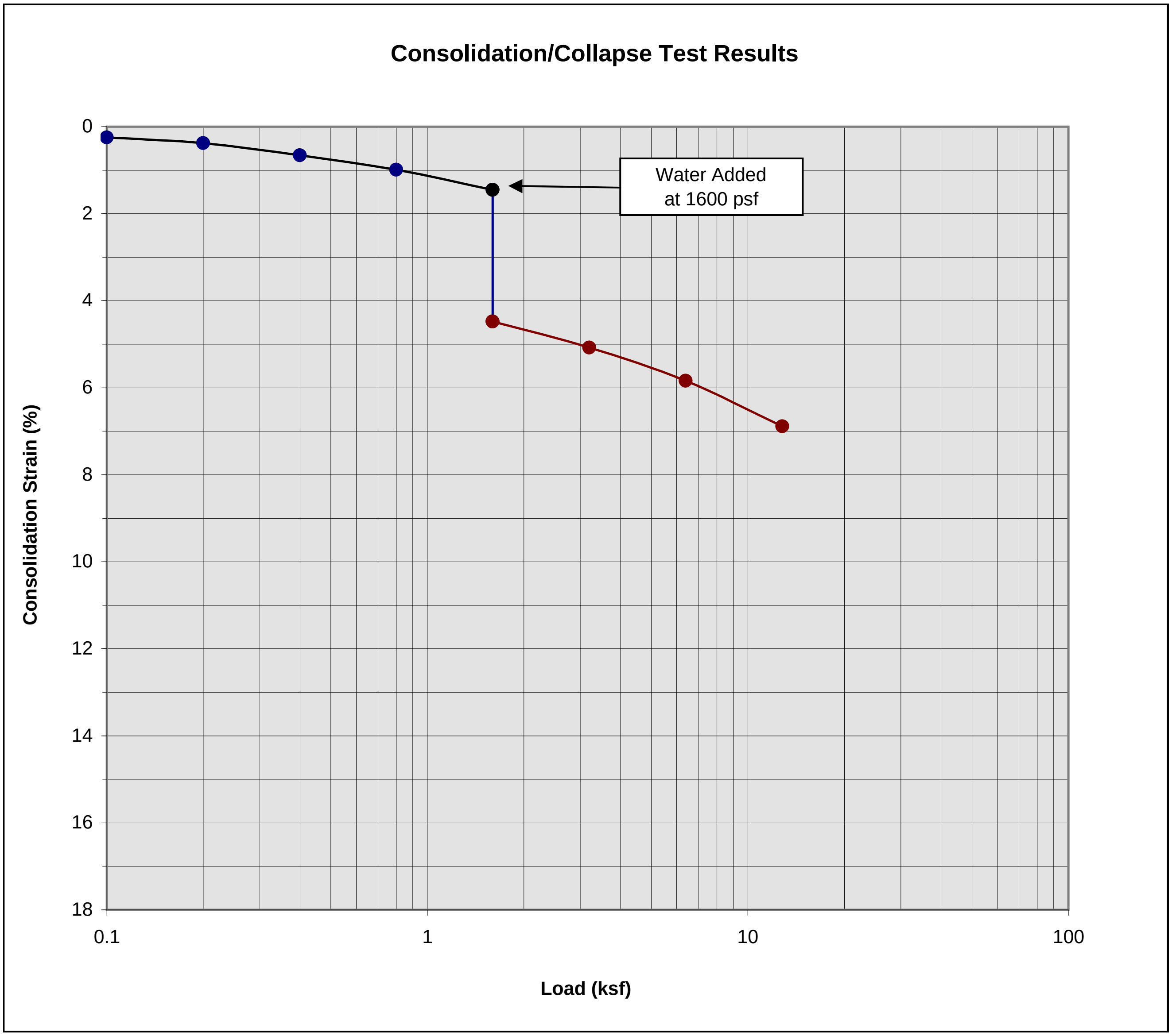
Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C- 11

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: Light Gray Silt

Boring Number:	B-10	Initial Moisture Content (%)	12
Sample Number:	---	Final Moisture Content (%)	27
Depth (ft)	7 to 8	Initial Dry Density (pcf)	96.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	101.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.41

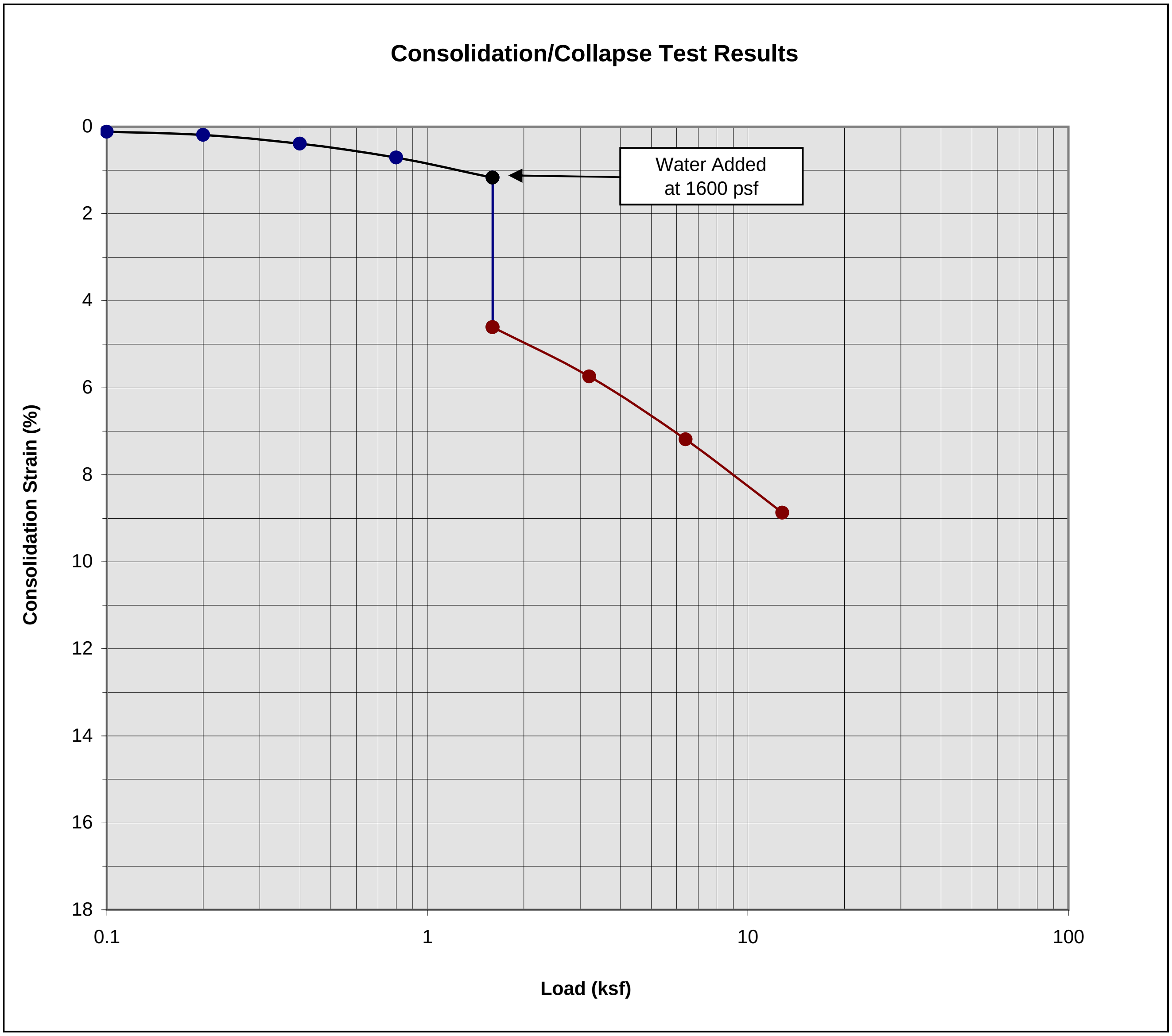


Classification: Red Brown Silty fine Sand

Boring Number:	B-12	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	11
Depth (ft)	1 to 2	Initial Dry Density (pcf)	119.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	128.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	3.03

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C- 13

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

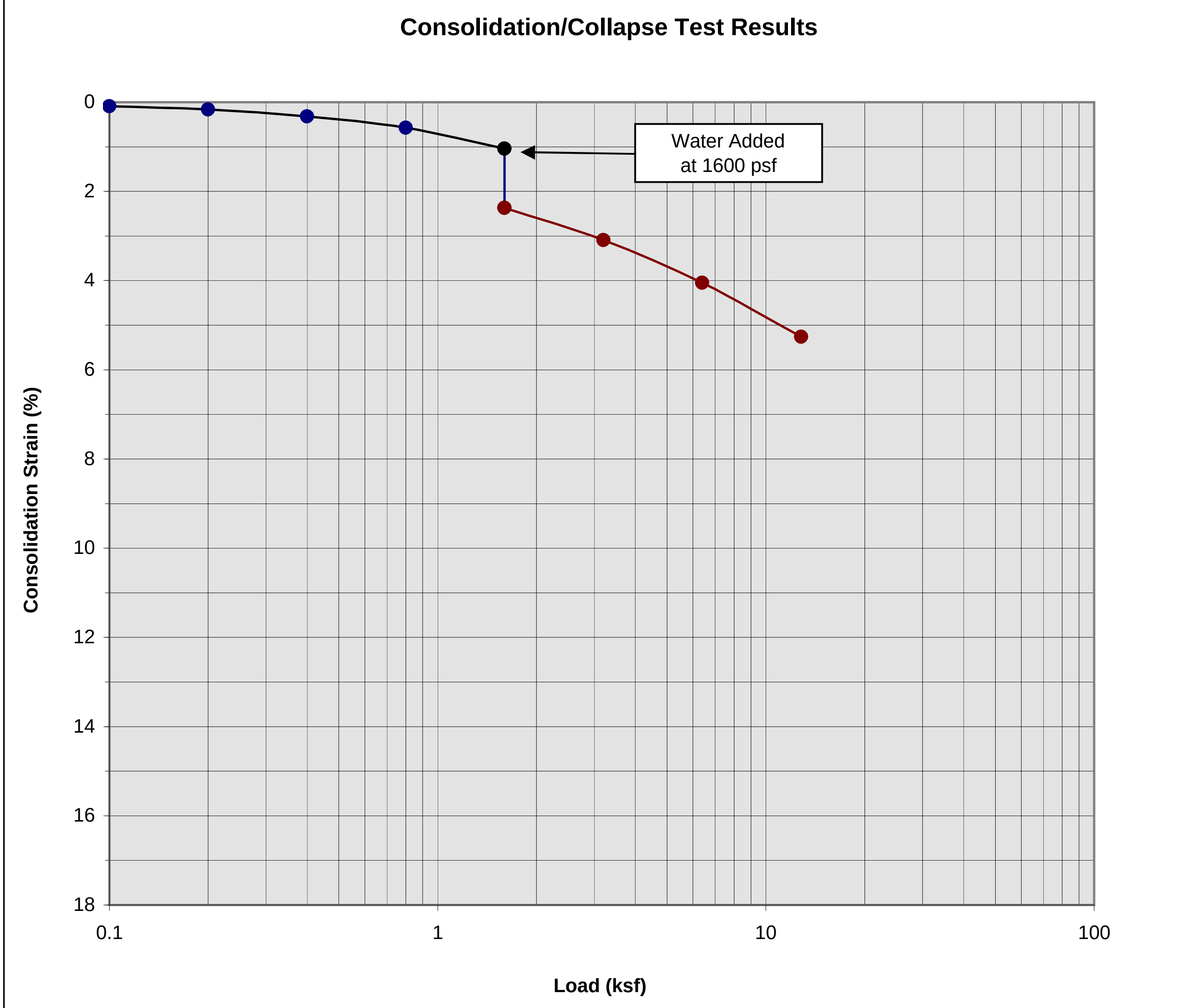


Classification: Red Brown Silty fine Sand

Boring Number:	B-12	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	3 to 4	Initial Dry Density (pcf)	105.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	116.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	3.44

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C- 14

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL A California Corporation

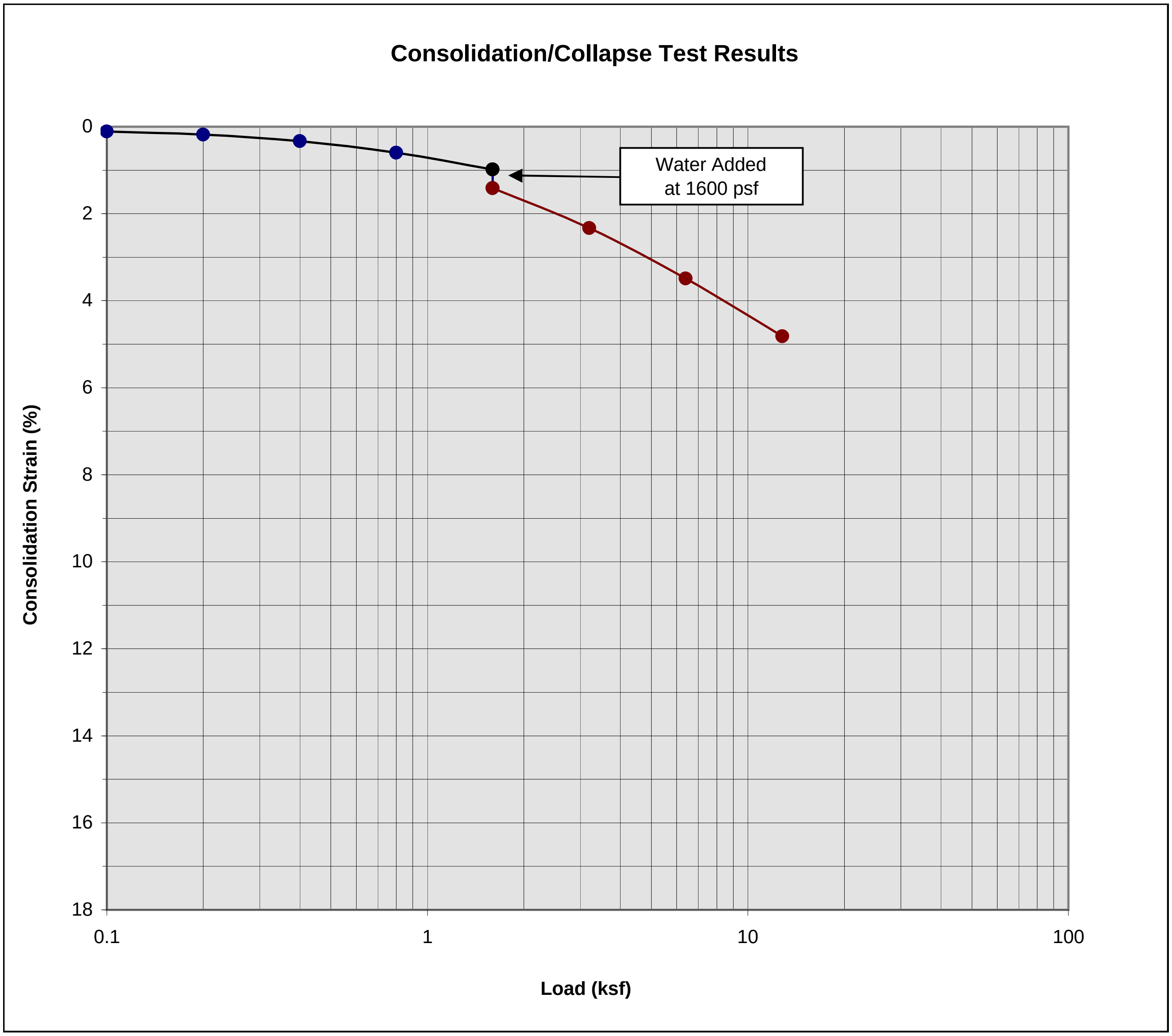


Classification: Red Brown Silty fine Sand

Boring Number:	B-12	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	5 to 6	Initial Dry Density (pcf)	104.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	110.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.33

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C- 15

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

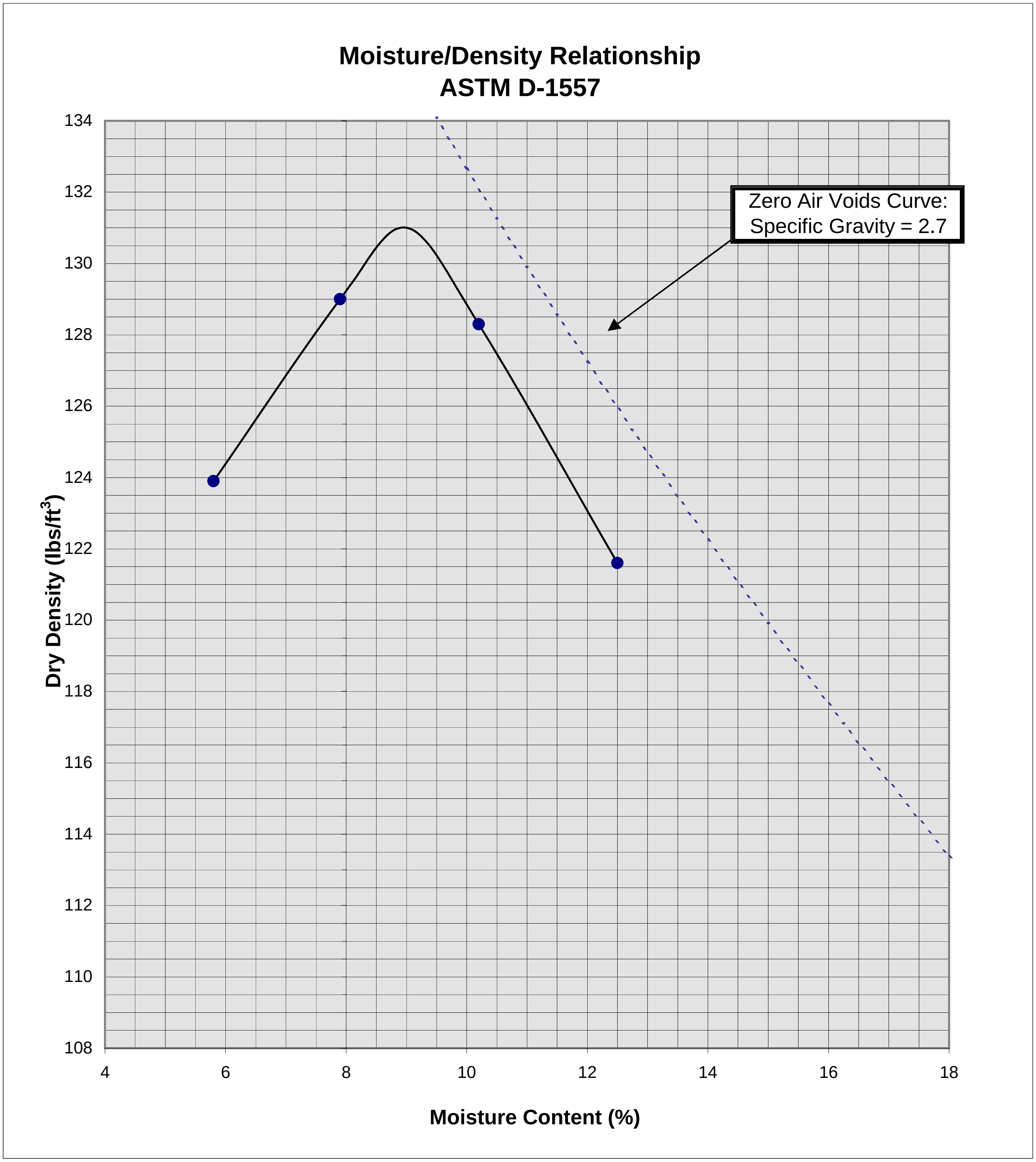


Classification: Red Brown Silty fine Sand

Boring Number:	B-12	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	7 to 8	Initial Dry Density (pcf)	101.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	106.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.43

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C- 16

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number		B-1 @ 0 to 5'
Optimum Moisture (%)		9
Maximum Dry Density (pcf)		131
Soil Classification	Brown Silty fine Sand	

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C-17

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
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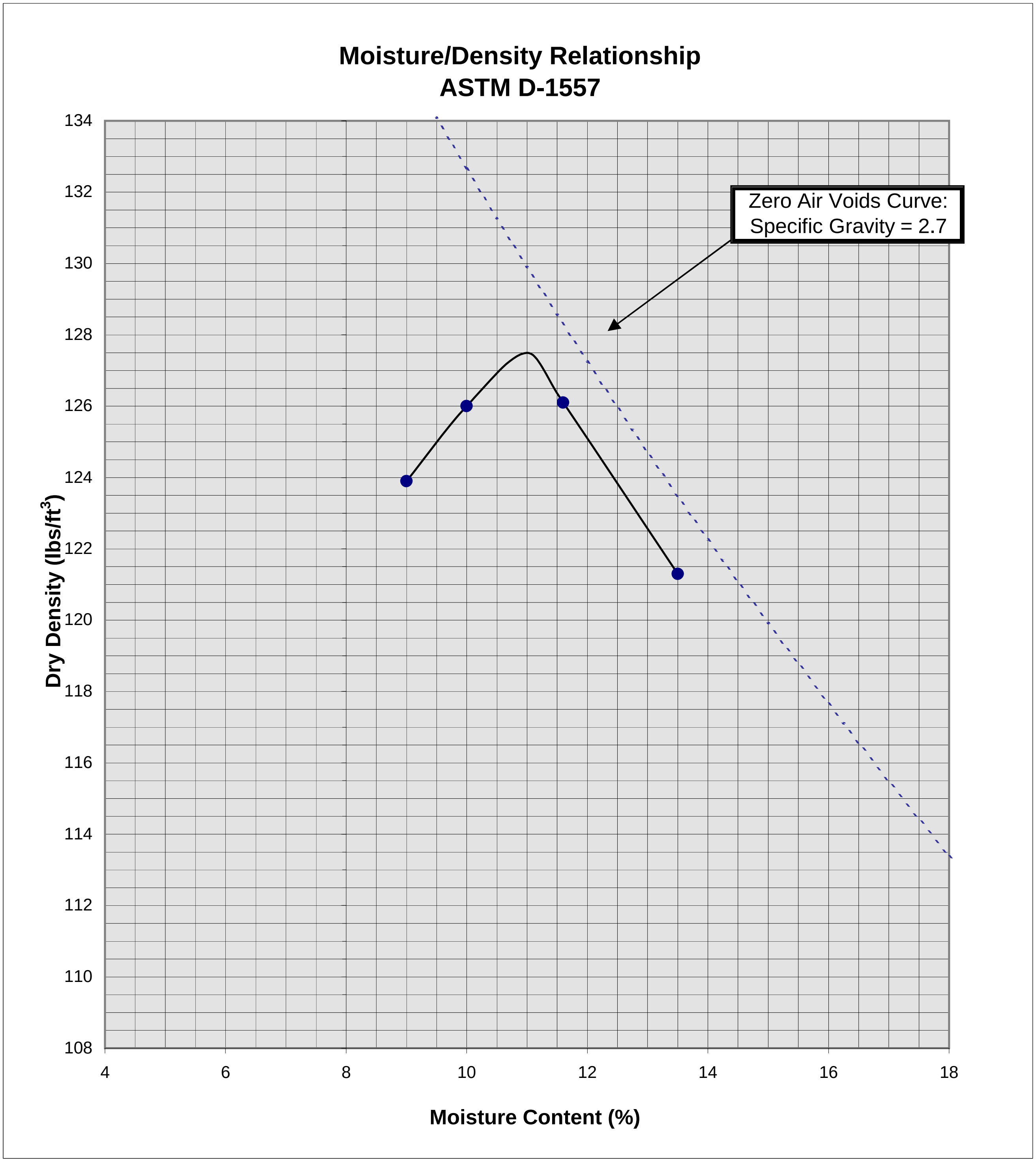
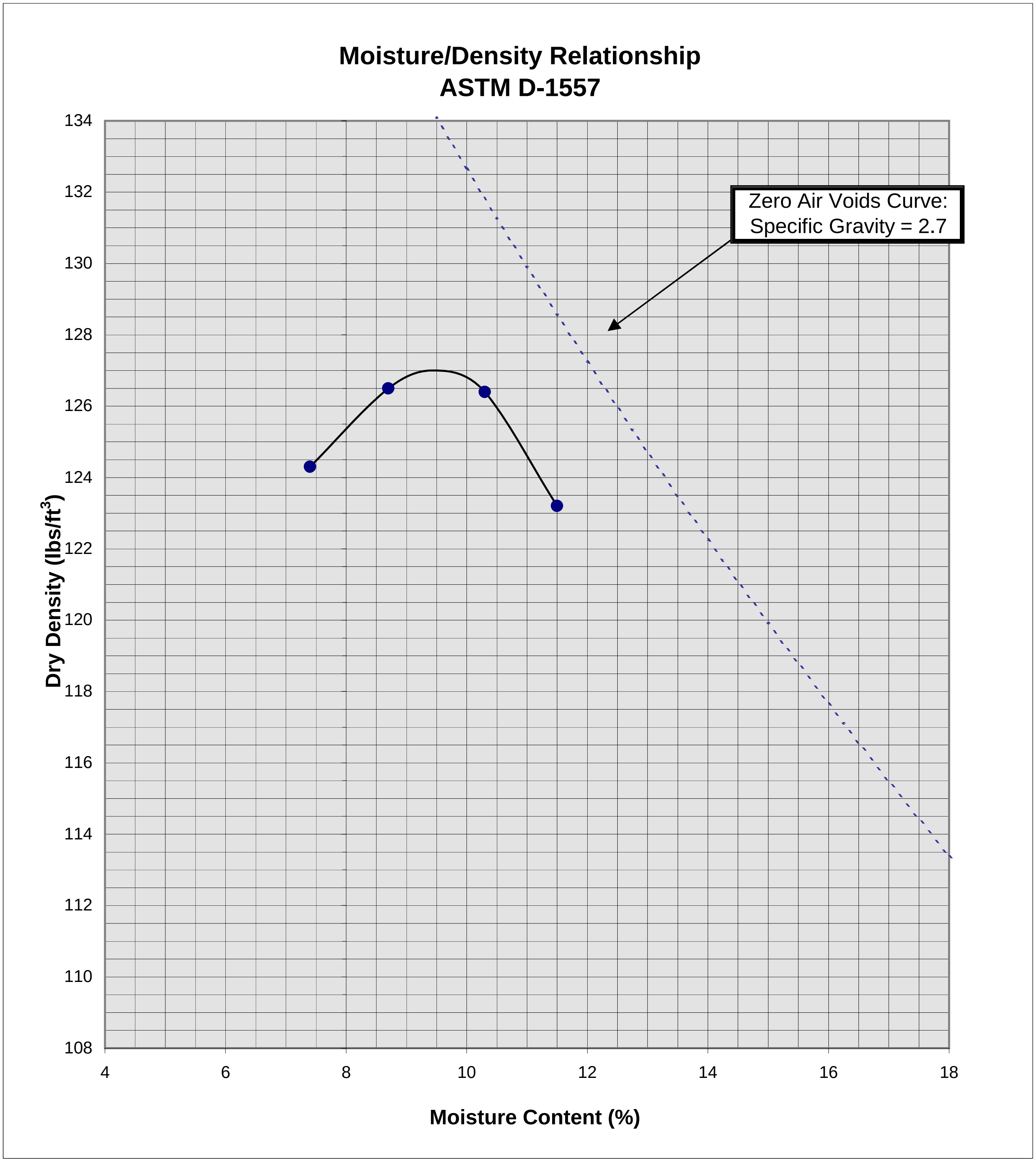


Soil ID Number	B-9 @ 0 to 5'
Optimum Moisture (%)	11
Maximum Dry Density (pcf)	127.5
Soil Classification	Red Brown Clayey fine Sand

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C-18

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-11 @ 0 to 5'
Optimum Moisture (%)	9.5
Maximum Dry Density (pcf)	127
Soil Classification	Brown Silty fine Sand

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C-19

SOUTHERN CALIFORNIA GEOTECHNICAL
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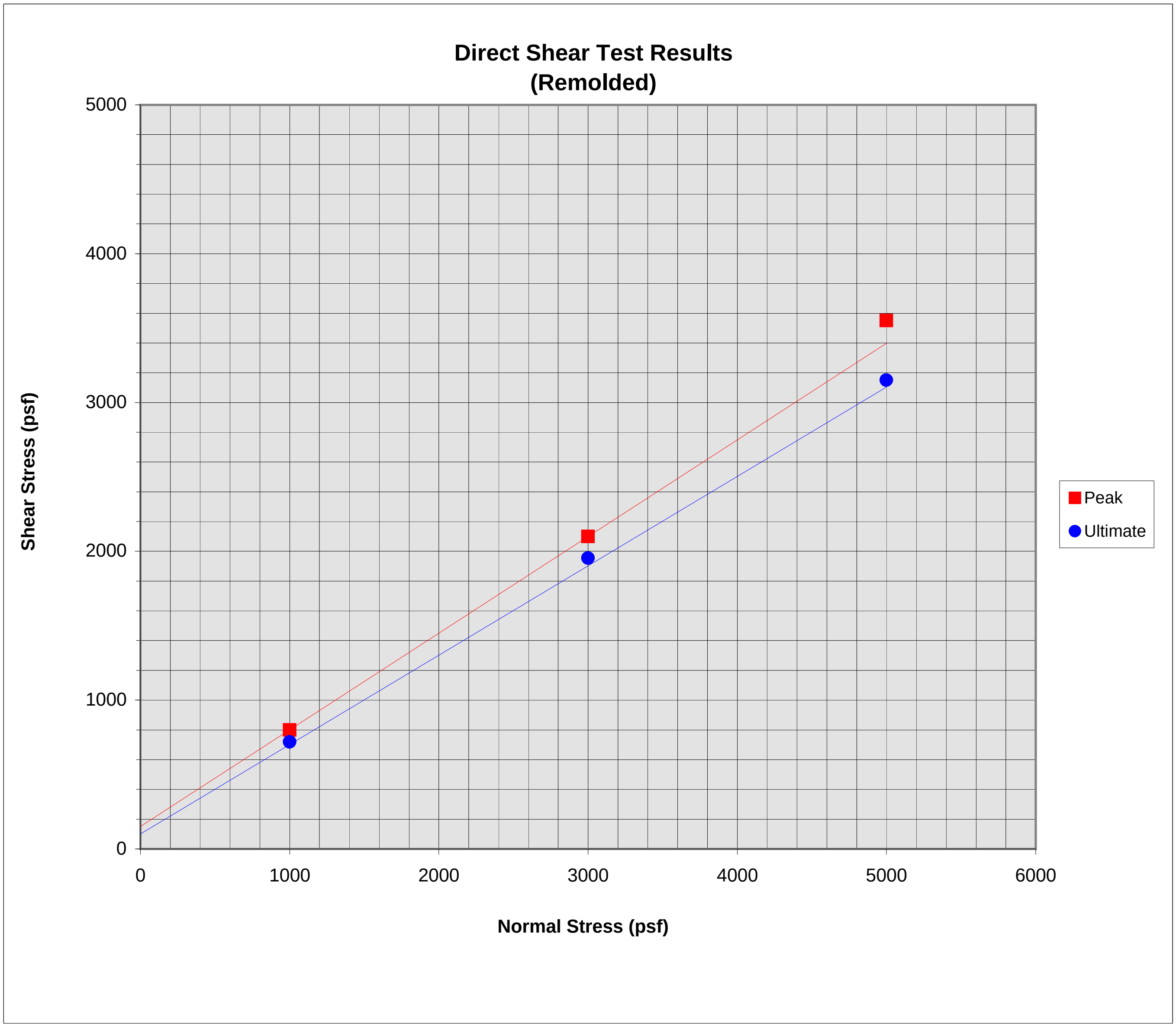

Sample Description: B-1 @ 0 to 5 feet

Classification: Brown Silty fine Sand

Sample Data	
Remolded Moisture Content	9.0
Final Moisture Content	---
Remolded Dry Density	117.9
Percent Compaction	90.0
Final Dry Density	---
Specimen Diameter (in)	2.4
Specimen Thickness (in)	1.0

Test Results	Peak	Ultimate
ϕ (°)	33.0	31.0
C (psf)	150	100

Norco Ranch Commerce Park
Norco, California
Project No. 11G114

PLATE C-20

SOUTHERN CALIFORNIA GEOTECHNICAL
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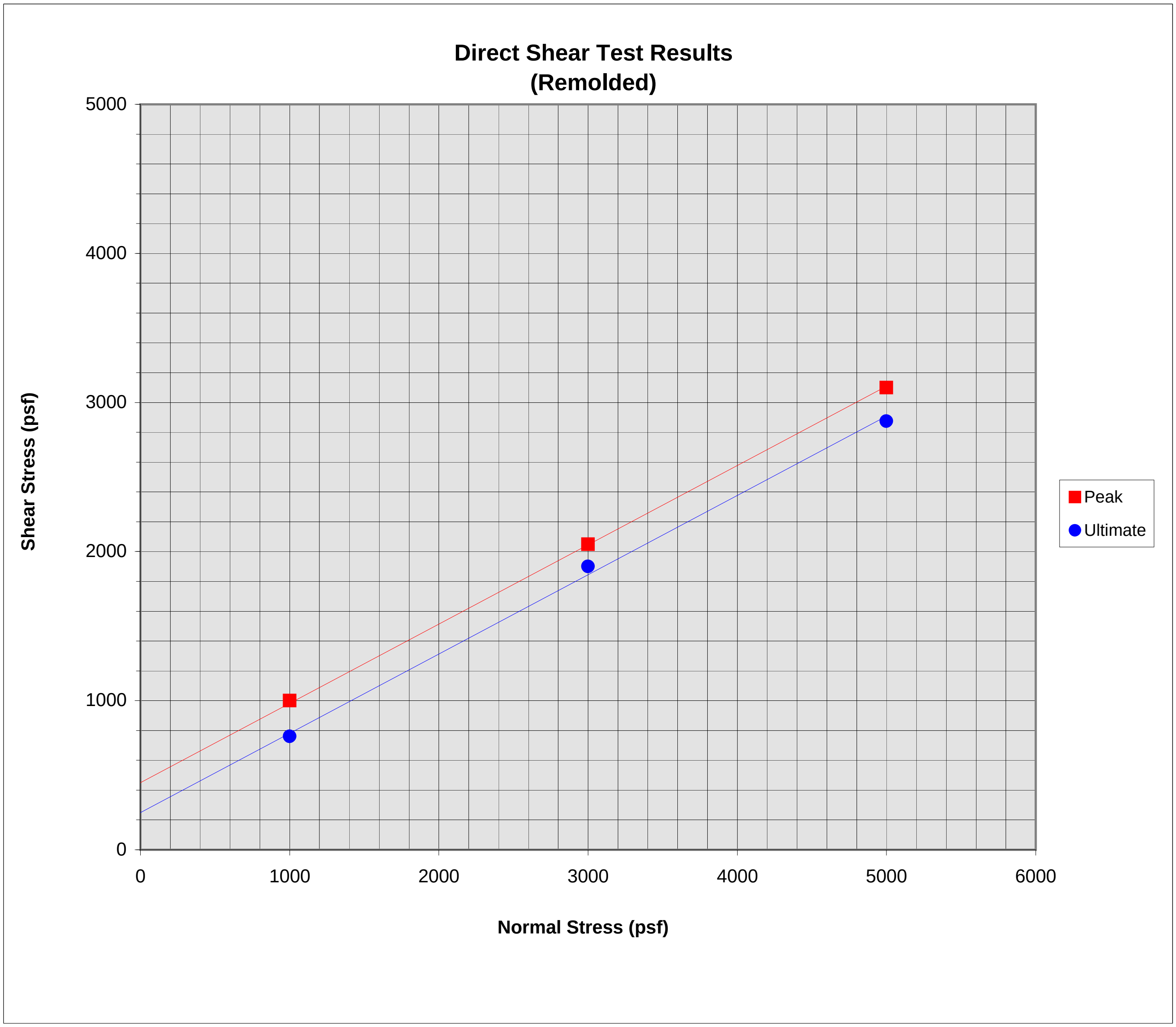


Sample Description: B-9 @ 0 to 5 feet

Classification: Red Brown Clayey fine Sand

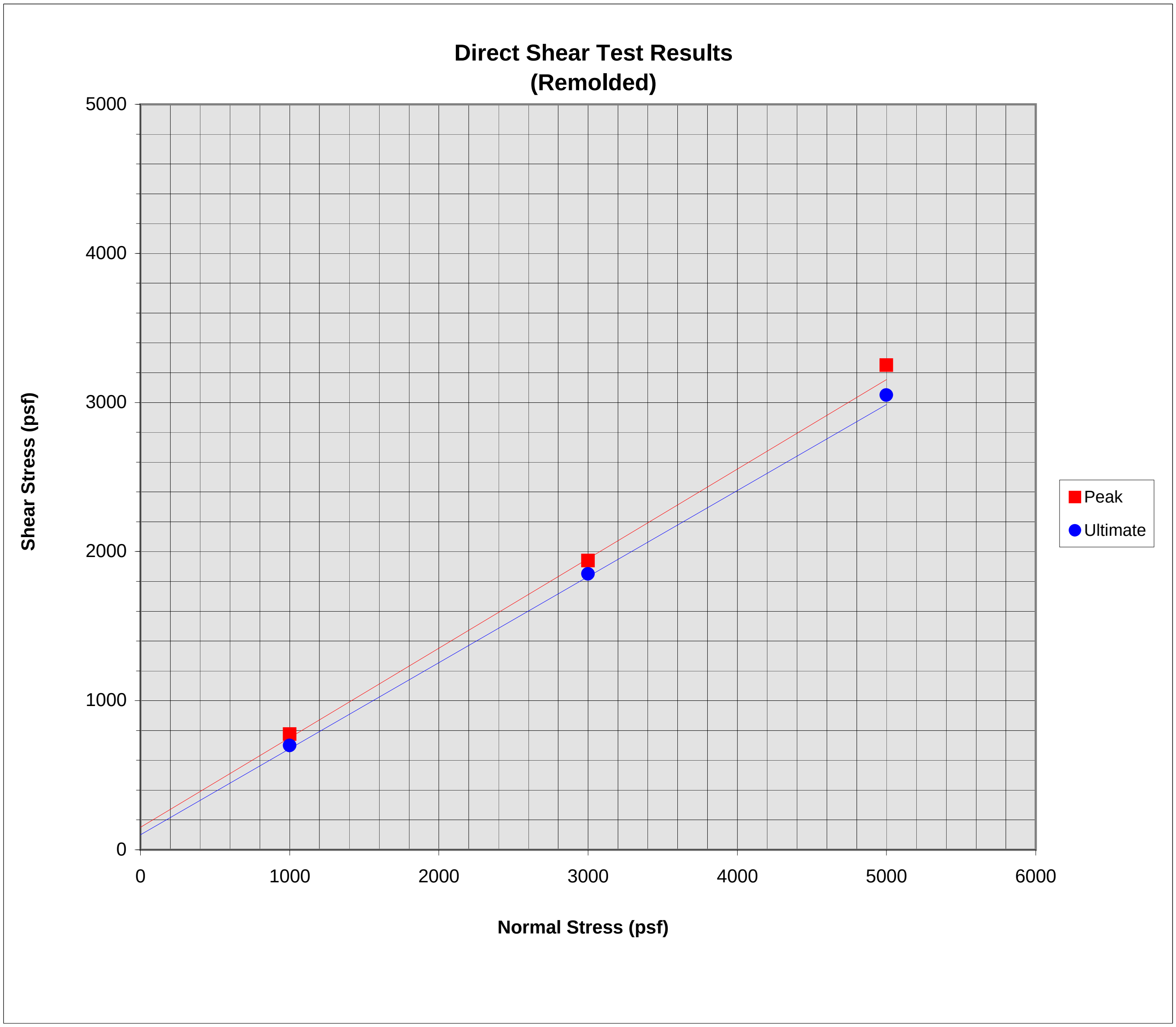
Sample Data	
Remolded Moisture Content	9.0
Final Moisture Content	---
Remolded Dry Density	114.7
Percent Compaction	90.0
Final Dry Density	---
Specimen Diameter (in)	2.4
Specimen Thickness (in)	1.0

Test Results	Peak	Ultimate
ϕ (°)	28.0	28.0
C (psf)	450	250

Norco Ranch Commerce Park
Norco, California
Project No. 11G114

PLATE C-21

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL A California Corporation



Sample Description: B-11 @ 0 to 5 feet

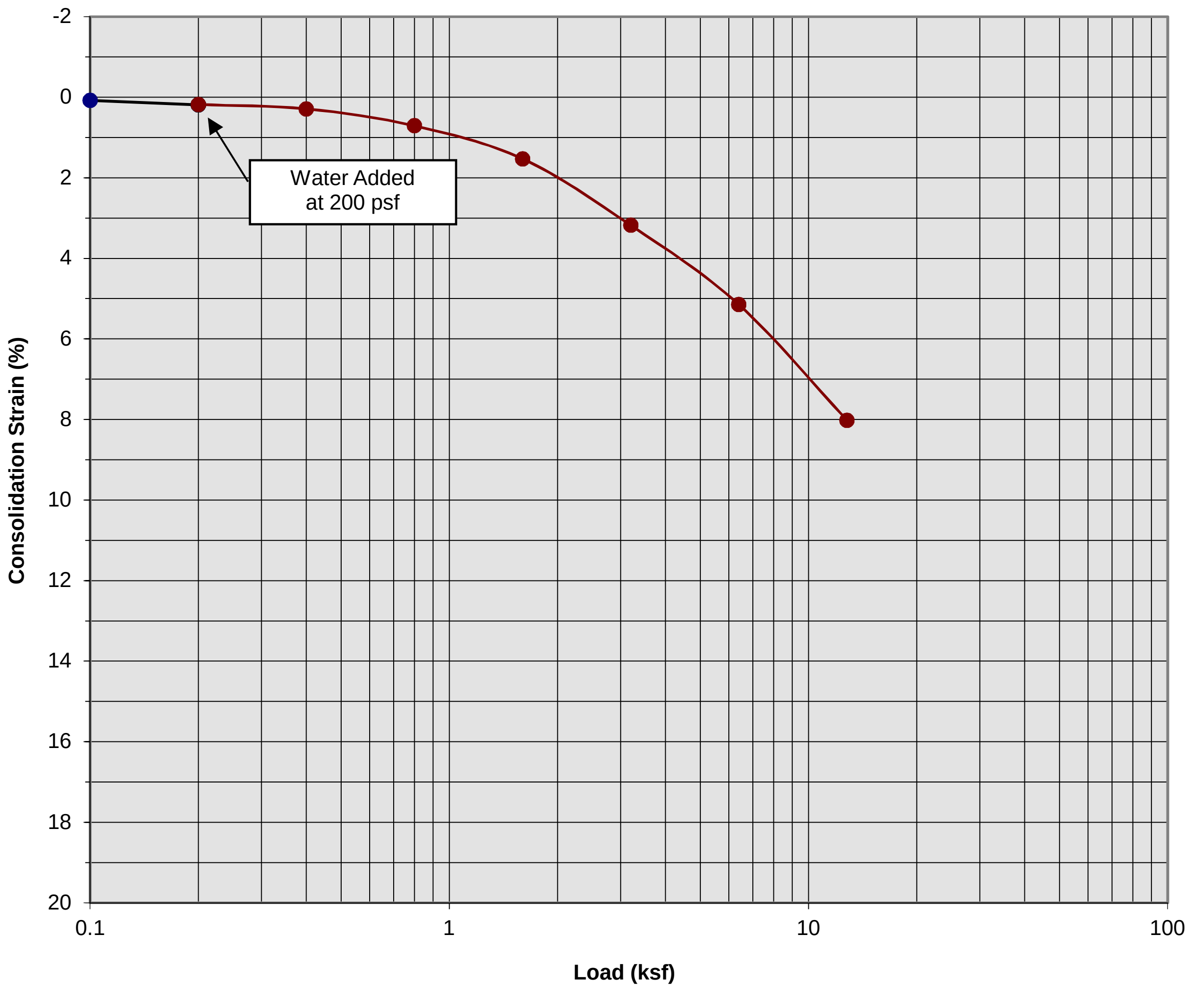
Classification: Brown Silty fine Sand

Sample Data			Test Results	
Remolded Moisture Content	9.0			
Final Moisture Content	---		Peak	Ultimate
Remolded Dry Density	114.3	ϕ (°)	31.0	30.0
Percent Compaction	90.0	C (psf)	150	100
Final Dry Density	---			
Specimen Diameter (in)	2.4			
Specimen Thickness (in)	1.0			

Norco Ranch Commerce Park
Norco, California
Project No. 11G114
PLATE C-22

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



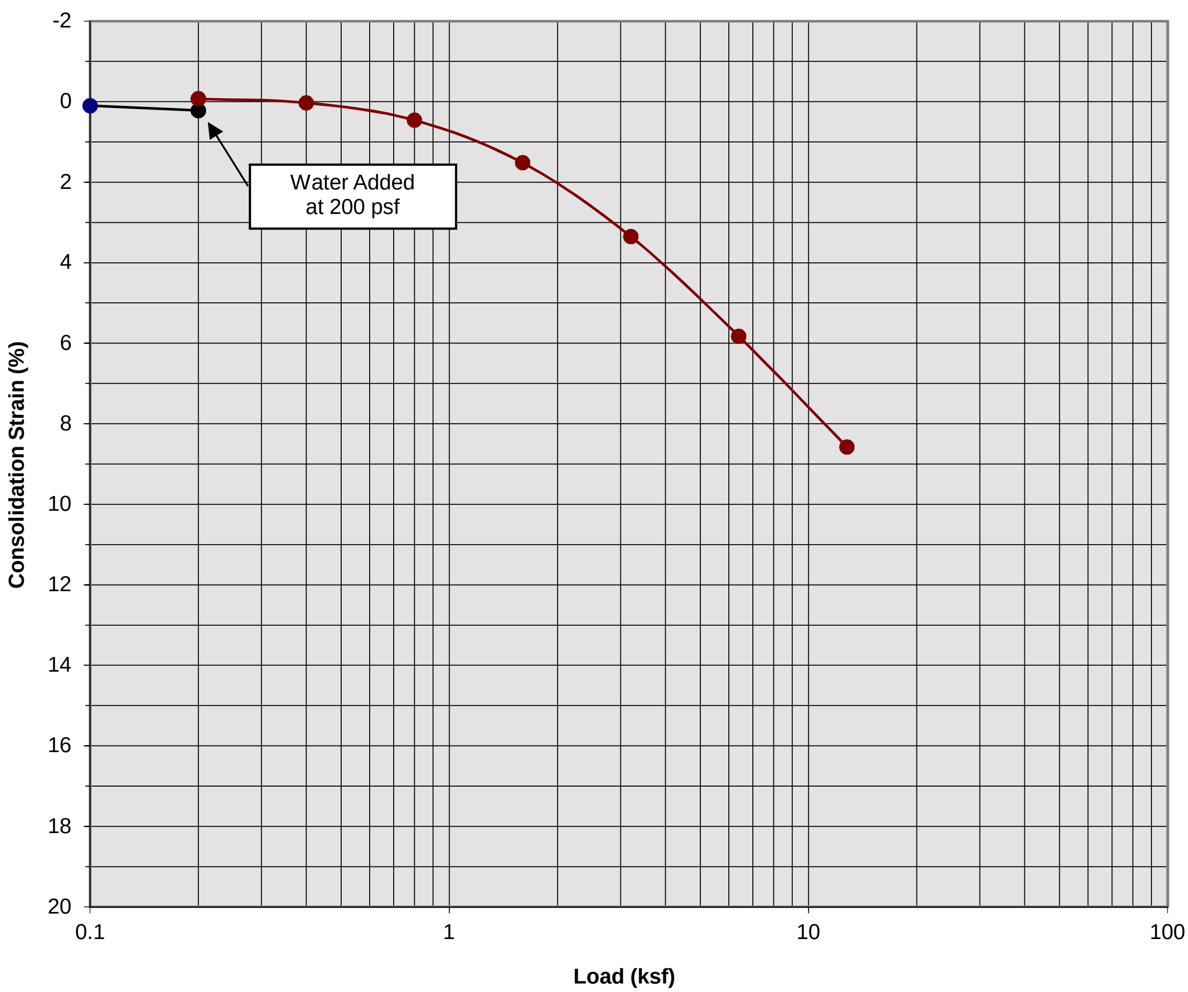
Classification: Dark Brown Silty Clay

Boring Number:	B-19	Initial Moisture Content (%)	20
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	1 to 2	Initial Dry Density (pcf)	103.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	112.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.01

Proposed Commercial/Industrial Development
Norco, California
Project No. 17G105
PLATE C- 1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

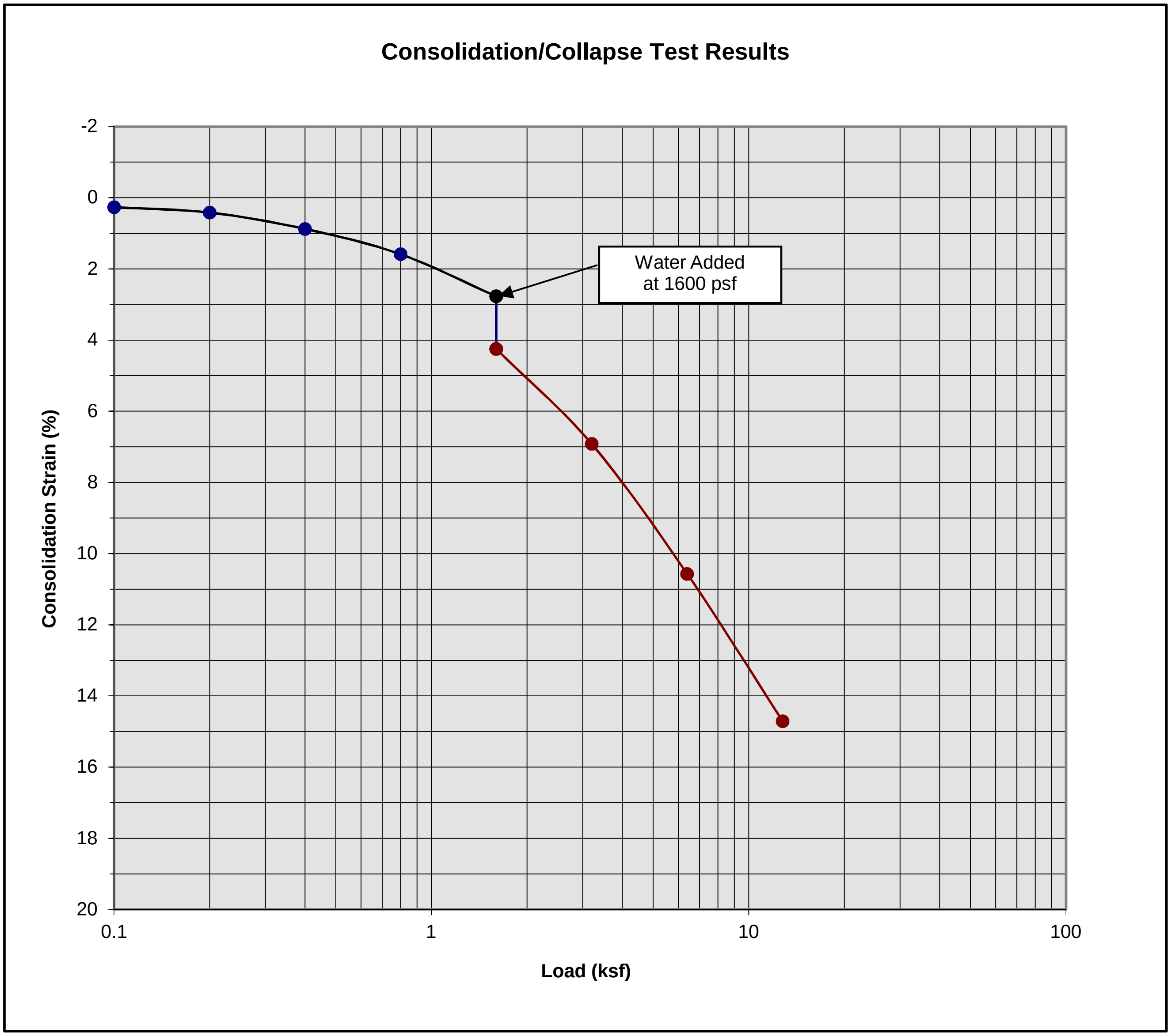


Classification: Gray Brown Silty Clay

Boring Number:	B-19	Initial Moisture Content (%)	20
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	3 to 4	Initial Dry Density (pcf)	104.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	114.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.29

Proposed Commercial/Industrial Development
Norco, California
Project No. 17G105
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

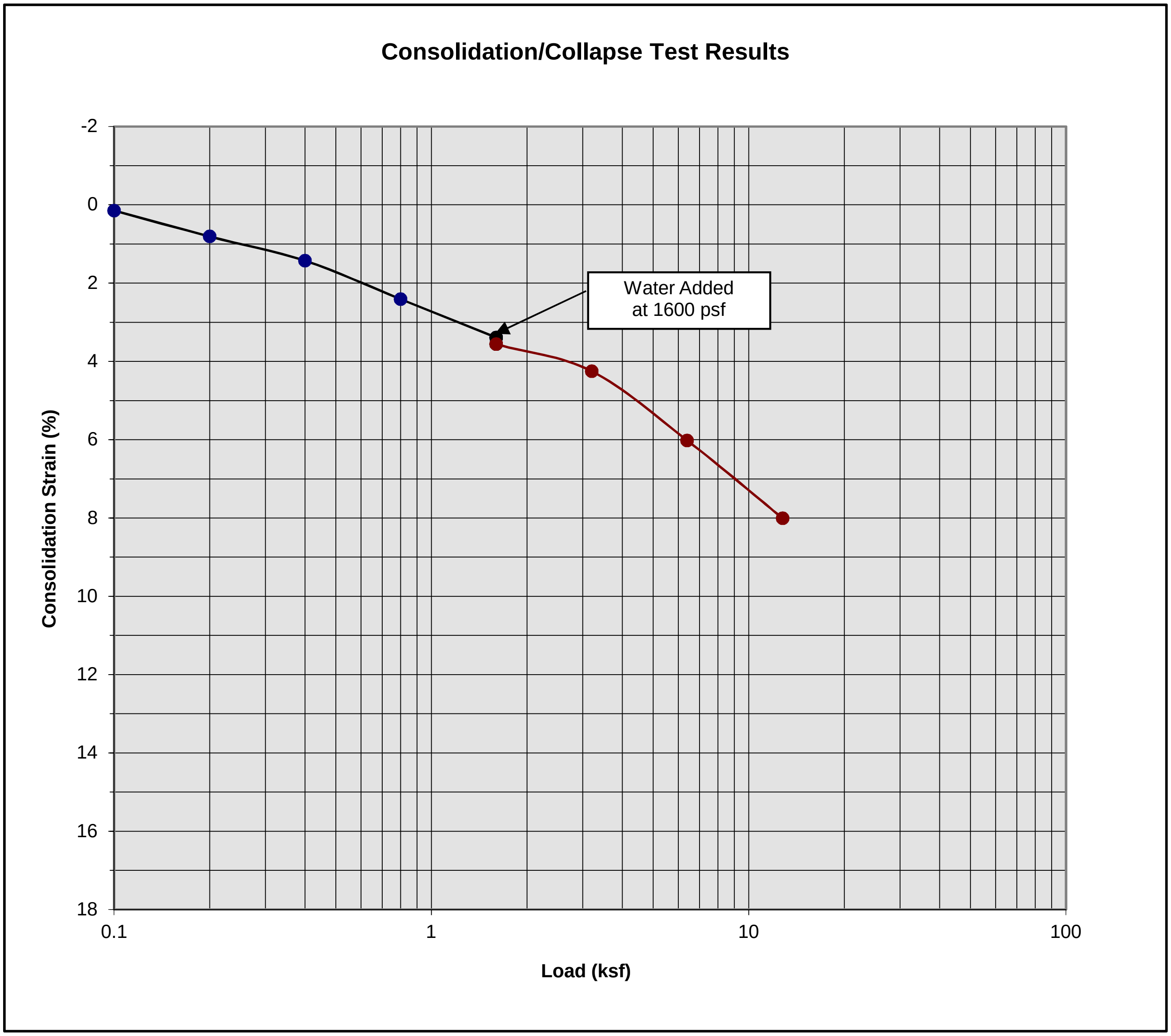


Classification: Light Gray to White Silt, little Clay

Boring Number:	B-19	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	5 to 6	Initial Dry Density (pcf)	105.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	123.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.48

Proposed Commercial/Industrial Development
Norco, California
Project No. 17G105
PLATE C- 3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

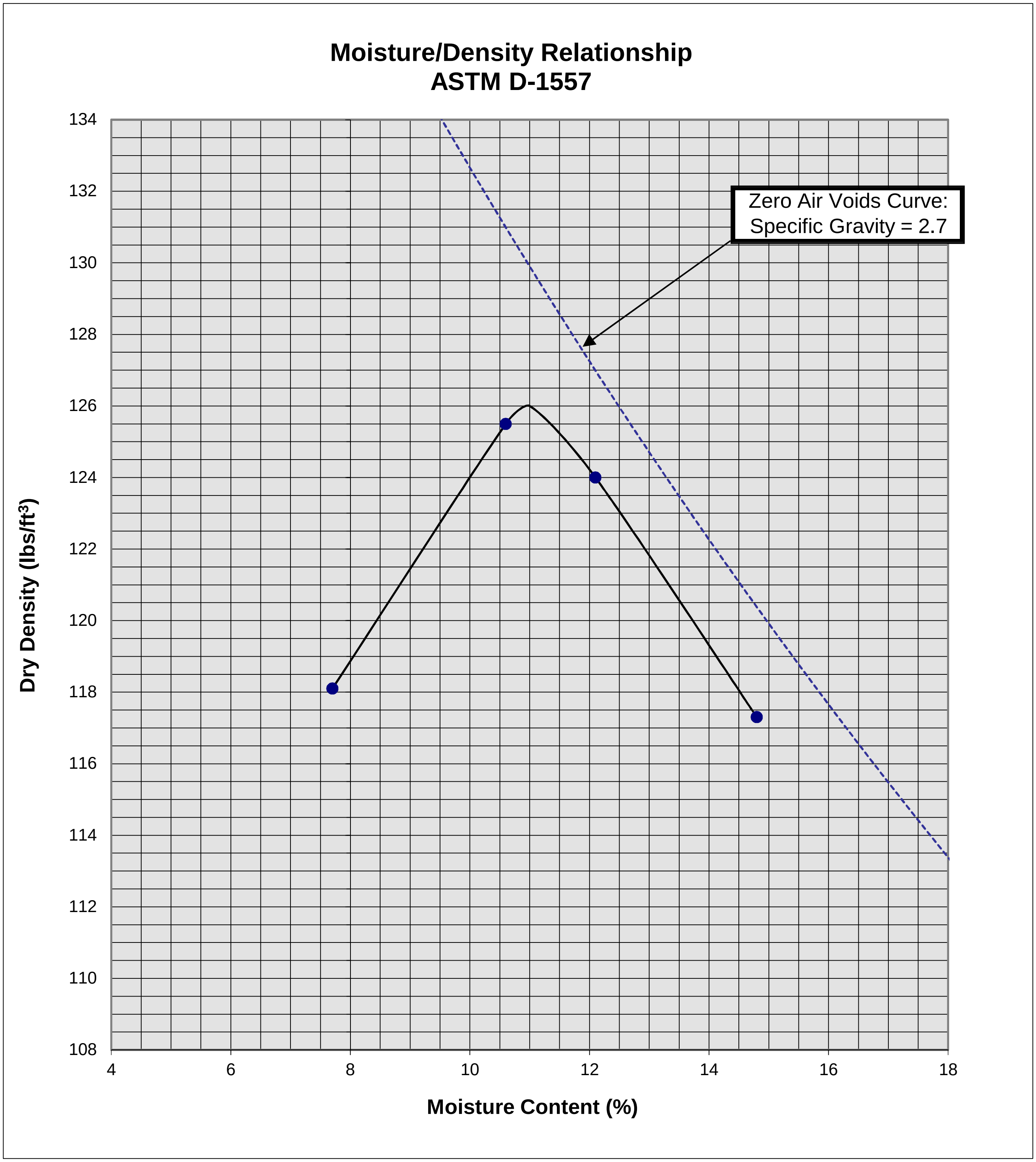


Classification: Light Gray fine to medium Sand, trace coarse Sand, trace fine Gravel

Boring Number:	B-19	Initial Moisture Content (%)	1
Sample Number:	---	Final Moisture Content (%)	8
Depth (ft)	9 to 10	Initial Dry Density (pcf)	126.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	137.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.17

Proposed Commercial/Industrial Development
Norco, California
Project No. 17G105
PLATE C- 4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number		B-19 @ 0 to 5'
Optimum Moisture (%)		11
Maximum Dry Density (pcf)		126
Soil Classification	Light Brown Clayey fine Sand to fine Sandy Clay	

Proposed C/I Development
Norco, California
Project No. 17G105
PLATE C-5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

November 13, 2019

Caprock Partners
1300 Dove Street, Suite 200
Newport Beach, California 92660

Attention: Mr. Clark Cashion

Project No.: **17G105-4**

Subject: **Response Report**
Proposed Palomino Business Park
NEC First Street and Pacific Avenue
Norco, California

References: 1) Geotechnical Investigation, Proposed Palomino Business Park, NEC First Street and Pacific Avenue, Norco, California, prepared by Southern California Geotechnical, Inc. (SCG) for Caprock Acquisitions, LLC, SCG Project No. 17G105-3, dated July 9, 2019.

2) Results of Infiltration Testing, Proposed Palomino Business Park, NEC First Street and Pacific Avenue, Norco, California, prepared by SCG for Caprock Partners, SCG Project No. 17G105-2, dated May 31, 2019.

3) Memorandum, City of Norco, City Engineer's Office, Comments on EIR (Administrative Draft), prepared by Dominic Milano, City Engineer, dated October 9, 2019.

Gentlemen:

In accordance with the request of Mr. Brent Caldwell of Caldwell Land Solution, we have prepared this report to address the review comments generated by the city of Norco, following their review of the submitted geotechnical investigation. Each of the comments issued by the reviewer (Norco) are presented below, followed by SCG response. A copy of the review sheet is enclosed with this correspondence for reference purposes.

City Comments on EIR (Administrative Draft)

Norco: *5.9-11 In the Operation Section – "Infiltration Basins" are listed as a Storm Water Improvement. There wasn't enough soils testing done and discussion in other sections to verify that the infiltration is a viable alternative. Also some exhibits list the infiltration basins as detention basins. Infiltration basins are also discussed in other mitigation measures of this chapter*.

SCG: SCG performed infiltration testing at the subject site. The results of the infiltration testing were presented in Reference 2. It appears that the city of Norco has only reviewed the geotechnical investigation report (Reference 1) for this project. The referenced infiltration report (Reference 2) should also be submitted to the city with this response letter.

Geotechnical Investigation City Comments

Norco: *Page 12 Section 6.1 Seismic design considerations use the 2016 CBC. The 2019 CBC will be effect for the project.*

SCG: The seismic design paramters presented in the referenced geotechnical report (Reference 1) are herein updated the 2019 California Building Code (CBC) parameters. The updated seismic design parameters are presented below:

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 the 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. The table below was created using data obtained from the application. The output generated from this program is enclosed as Plate E of this report. Based on this output, the following parameters may be utilized for the subject site:

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.716
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.667
Site Class	---	D*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.716
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.134
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.144
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.756

*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 11.4.8 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 (F_a and F_v) 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 structures have fundamental periods greater than 0.5 seconds, a site-specific seismic hazards analysis would be required and additional subsurface exploration would be necessary.

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

Updated Ground Motion Parameters

The liquefaction evaluation performed in Reference 1 was updated for the purposes of this current report to include 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, based on ASCE 7-16 as the building code reference document. A portion of the program output is included in Plate E of this report. As indicated on Plate E, the PGA_M for this site is 0.723. An associated earthquake magnitude was obtained from the USGS Unified Hazard Tool, Interactive Deaggregation application available on the USGS website. The deaggregated modal magnitude is 8.8, based on the peak ground acceleration and soil classification D for a return period of 2,950 years.

Updated Liquefaction Evaluation

The Riverside County GIS website indicates that the subject site is located within a zone of high liquefaction susceptibility. Therefore, this report includes an updated liquefaction evaluation in order to determine the site-specific liquefaction potential.

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and plasticity characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Non-sensitive clayey (cohesive) soils which possess a plasticity index of at least 18 (Bray and Sancio, 2006) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The liquefaction analysis was conducted in accordance with the requirements of Special Publication 117A (CDMG, 2008), and currently accepted practice (SCEC, 1997). The liquefaction potential of the subject site was evaluated using the empirical method developed by Boulanger and Idriss (Boulanger and Idriss, 2008, 2014). This method predicts the earthquake-induced liquefaction potential of the site based on a given design earthquake magnitude and peak ground acceleration at the subject site. This procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude). CRR is determined as a function of the corrected SPT N-value $(N_1)_{60\text{-}cs}$, adjusted for fines content or the corrected CPT tip stress, q_{c1Ncs}. 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 not considered to be susceptible to liquefaction. Non-sensitive soils with a PI greater than 18 are also considered non-liquefiable. Soils possessing a PI between 12 and 18, may also be moderately susceptible to liquefaction if the moisture content is greater than 85 percent of the liquid limit.

The updated liquefaction analyses for the seven (7) borings are tabulated on the spreadsheet forms included in Appendix F of this report. The liquefaction potential for the on-site soils was also determined using data obtained at the four CPT locations. This data was analyzed using the computer program Cliq V3.0.2.4, which was developed by Geologismiki, copyright 2006. The analysis method for both the boring and CPT data is based on Boulanger and Idriss, 2014. The liquefaction potential was analyzed at the boring locations utilizing a PGA_M of 0.723g related to a 8.8 magnitude seismic event. The liquefaction evaluation was performed using the reported historic high groundwater depth of 21 feet.

If liquefiable soils are identified, the potential settlements that could occur as a result of liquefaction are determined using the equation for volumetric strain due to post-cyclic reconsolidation (Yoshimine et. al, 2006). This procedure uses an empirical relationship between the induced cyclic shear strain and the corrected N-value to determine the expected volumetric strain of saturated sands subjected to earthquake shaking. This analysis is also documented on the spreadsheets included with this report.

Conclusions and Recommendations

The results of the updated liquefaction analysis have identified potentially liquefiable soil strata at most of the boring locations, with the exception of Boring Nos. B-1 and B-17. Soils which are located above the historic groundwater table (21 feet), or possessing factors of safety in excess of 1.3 are considered non-liquefiable. Settlement analysis was conducted for the potentially liquefiable strata.

Based on the settlement analyses (also documented on the spreadsheets included with this report) the following total dynamic (liquefaction-induced) settlements could be expected at each of the boring locations:

- Boring No. B-1: 0.00 inches
- Boring No. B-4: 1.56 inches
- Boring No. B-11: 1.85 inches
- Boring No. B-13: 1.45 inches
- Boring No. B-16: 1.75 inches
- Boring No. B-17: 0.00 inches
- Boring No. B-18: 1.10 inches

Liquefiable soils were generally encountered at similar depths at the four CPT locations. Settlement analyses were conducted for each of the potentially liquefiable strata. It should be noted that CPT-1 and CPT-2 encountered refusal conditions between depths of 34 and 35 feet. The total liquefaction settlement for each CPT is presented below:

- CPT-1: 0.33± inches
- CPT-2: 0.93± inches
- CPT-3: 2.55± inches
- CPT-4: 1.76± inches

Based on the estimated total settlements, differential settlements are expected to be less than 1¼ inches. The estimated differential settlement can be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of less than 0.002± inches per inch.

These differential settlements are considered to be within the structural tolerances of a typical building supported on a shallow foundation system provided that structural mitigation measures are implemented. However, it should be noted that minor to moderate repairs, including repair of damaged drywall and stucco, etc., could be required after the occurrence of liquefaction-induced settlements.

The use of a shallow foundation system, as described in Reference 1, is typical for buildings of this type, where they are underlain by the extent of liquefiable soils encountered at this site. The post-liquefaction damage that could occur within the building 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.

Norco: *Page 22 Treatment of Existing Soils: Parking and Drive Areas. Should optional over excavating in truck drive area even be considered given the truck loading?*

SCG: As indicated in Reference 1, overexcavation of the existing fill and near-surface alluvium in the new parking and drive areas is not considered warranted, **with the exception of areas where lower strength or unstable soils are identified by the geotechnical engineer during grading**. 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 for the proposed parking and drive areas, presented in Reference 1, do not completely mitigate the extent of existing undocumented fill soils and variable strength alluvium in the parking and drive areas. As such, settlement and associated pavement distress could occur. Typically, **repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction.** If the owner cannot tolerate the risk of such settlements, the parking and drive areas should be overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill. Regardless of whether or not the client elects to perform the optional remedial grading throughout the entire pavement area, some overexcavation will be performed in areas where lower strength or otherwise unsuitable soils are identified during grading, as indicated above.

Norco: *Page 31 Section 6.8. R-Values must be taken after rough grading. The option not to test should not be given*.

SCG: As requested by the city of Norco, SCG recommends that R-value testing be performed after the completion of grading to verify the R-value of the as-graded parking subgrade.

Norco: *Page 32. An option of a TI of 4.0 for auto parking should not be given.*

SCG: As requested by the city of Norco, SCG has updated the pavement designs based on the traffic indices (TI's) indicated on the tables below. The city has stated that a traffic index of 4.0 should not be used for this project, even in areas only intended for use by automobiles.

ASPHALT PAVEMENTS (R = 25)				
Materials	**Thickness (inches)**			
	Automobile Parking and Drives (TI = 5.0)	Truck Traffic		
		(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3½	4	5
Aggregate Base	7	9	11	12
Compacted Subgrade	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R=25)				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI =6.0)	(TI = 7.0)	(TI = 8.0)
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

Norco: *Exhibits in the Appendix A do not have infiltration basins only detention basins. See comments on EIR*.

SCG: The exhibit, Plate 2, provided in Reference 1, Appendix A, was based on the information provided by the project civil engineer during the preparation of the geotechnical report. However, at the time of Reference No. 2, SCG was provided with potential infiltration basin locations, and infiltration testing was performed in these areas. Additionally, SCG has recommended that grading plans be provided to our office during the design phase of the project in order to confirm that our recommendations have been implemented and that the assumptions we made during the development of our reports are accurate. We will confirm that appropriated testing has been performed in the proposed infiltration basin areas at the time of our plan review.

Norco: *Percolation Is not discussed in this report*.

SCG: SCG performed infiltration testing at the subject site. The results of the infiltration testing were presented in Reference 2. We recommend that the infiltration testing report be submitted to the city with this response letter.

Geotechnical Report Update

This report may serve as an update to the referenced geotechnical report (Reference 1). Provided that the updated considerations and recommendations contained within this report are implemented, the previous geotechnical report is considered valid for the currently proposed development.

Closure

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Joseph Lozano Leon
Staff Engineer

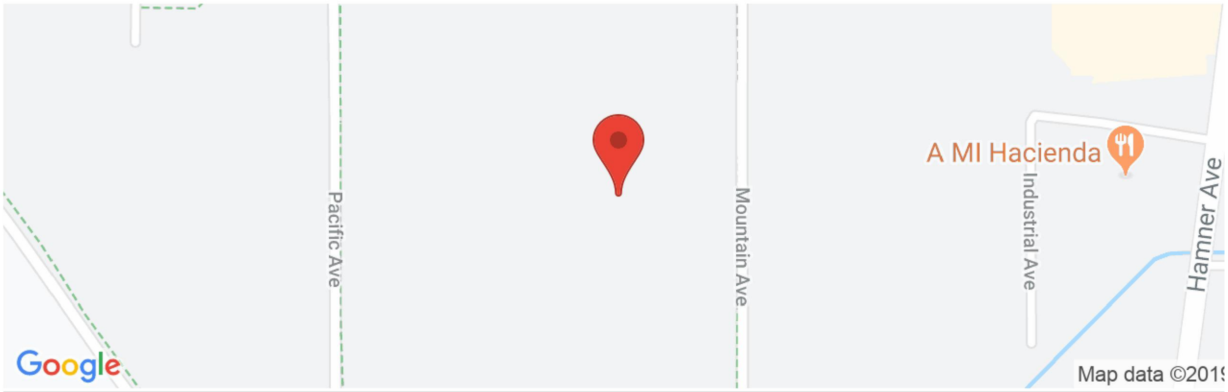
Daniel W. Nielsen, RCE 77915
Senior Engineer

Enclosures: Plate E – Seismic Design Parameters – 2019 CBC
Liquefaction Evaluation Spreadsheets (14 pages)
Memorandum, City of Norco, City Engineer's Office, Comments on EIR (Administrative Draft), dated October 9, 2019

Distribution: (1) Addressee

OSHPD

Latitude, Longitude: 33.906213, -117.568225



Date	11/11/2019, 10:04:24 AM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.716	MCE_R ground motion. (for 0.2 second period)
S_1	0.667	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.716	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.144	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	0.723	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.796	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.891	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.039	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.716	Factored deterministic acceleration value. (0.2 second)
S1RT	0.671	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.732	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.667	Factored deterministic acceleration value. (1.0 second)
PGAd	0.723	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.927	Mapped value of the risk coefficient at short periods
C_{R1}	0.917	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 PALOMINO BUSINESS PARK

NORCO, CALIFORNIA

DRAWN: JLL
CHKD: DWN

SCG PROJECT
17G105-4

PLATE E

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

LIQUEFACTION EVALUATION

Project Name	Proposed Palomino Business Park	MCE_G Design Acceleration	0.723 (g)
Project Location	Norco, CA	Design Magnitude	8.8
Project Number	17G105-4	Historic High Depth to Groundwater	21 (ft)
Engineer	JLL	Depth to Groundwater at Time of Drilling	29.5 (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=8.8)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	1.00	0.97	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	27	120		1.3	1.05	1.3	0.95	0.95	43.1	43.1	2580	2549	2580	1.00	0.56	0.94	2.00	1.05	0.47	2.22	Non-Liquefiable
24.5	22	27	24.5	38	120		1.3	1.05	1.3	0.94	0.95	60.2	60.2	2940	2722	2940	1.00	0.56	0.92	2.00	1.03	0.51	2.04	Non-Liquefiable
29.5	27	29.5	28.3	35	120		1.3	1.05	1.3	0.90	0.95	53.1	53.1	3390	2938	3390	0.99	0.56	0.9	2.00	1.01	0.54	1.87	Non-Liquefiable
29.5	29.5	32	30.8	35	120		1.3	1.05	1.3	0.88	0.95	52.2	52.2	3690	3082	3612	0.99	0.56	0.89	2.00	0.99	0.56	1.78	Non-Liquefiable
34.5	32	37	34.5	38	120		1.3	1.05	1.3	0.89	1	60.3	60.3	4140	3298	3828	0.99	0.56	0.87	2.00	0.97	0.58	1.66	Non-Liquefiable
39.5	37	42	39.5	57	120		1.3	1.05	1.3	0.99	1	100.1	100.1	4740	3586	4116	0.98	0.56	0.84	2.00	0.94	0.61	1.54	Non-Liquefiable
44.5	42	47	44.5	71	120		1.3	1.05	1.3	1.09	1	137.1	137.1	5340	3874	4404	0.98	0.56	0.82	2.00	0.92	0.63	1.44	Non-Liquefiable
49.5	47	50	48.5	82	120		1.3	1.05	1.3	1.20	1	174.2	174.2	5820	4104	4634	0.97	0.56	0.8	2.00	0.90	0.65	1.38	Non-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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-4
Engineer	JLL

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 $F\alpha$	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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	43.1	0.0	43.1	2.22	0.00	-1.04	0.00	1.00		0.000		0.00	Non-Liquefiable
24.5	22	27	24.5	60.2	0.0	60.2	2.04	0.00	-2.44	0.00	5.00		0.000		0.00	Non-Liquefiable
29.5	27	29.5	28.3	53.1	0.0	53.1	1.87	0.00	-1.84	0.00	2.50		0.000		0.00	Non-Liquefiable
29.5	29.5	32	30.8	52.2	0.0	52.2	1.78	0.00	-1.77	0.00	2.50		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	60.3	0.0	60.3	1.66	0.00	-2.45	0.00	5.00		0.000		0.00	Non-Liquefiable
39.5	37	42	39.5	100.1	0.0	100.1	1.54	0.00	-6.08	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	137.1	0.0	137.1	1.44	0.00	-9.71	0.00	5.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				0.00	

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 Palomino Business Park	MCE_G Design Acceleration	0.723 (g)
Project Location	Norco, CA	Design Magnitude	8.8
Project Number	17G105-4	Historic High Depth to Groundwater	21 (ft)
Engineer	JLL	Depth to Groundwater at Time of Drilling	29.5 (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=8.8)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	1.00	0.97	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	24	120	5	1.3	1.05	1.3	0.94	0.95	38.0	38.0	2580	2549	2580	1.00	0.56	0.94	2.00	1.05	0.47	2.22	Non-Liquefiable
24.5	22	27	24.5	33	120	5	1.3	1.05	1.3	0.93	0.95	51.5	51.5	2940	2722	2940	1.00	0.56	0.92	2.00	1.03	0.51	2.04	Non-Liquefiable
29.5	27	32	29.5	43	120	5	1.3	1.05	1.3	0.92	0.95	66.9	66.9	3540	3010	3540	0.99	0.56	0.89	2.00	1.00	0.55	1.82	Non-Liquefiable
34.5	32	37	34.5	33	120	5	1.3	1.05	1.3	0.87	1	50.9	50.9	4140	3298	3828	0.99	0.56	0.87	2.00	0.97	0.58	1.66	Non-Liquefiable
39.5	37	42	39.5	18	120	39	1.3	1.05	1.24	0.78	1	23.8	29.4	4740	3586	4116	0.98	0.65	0.89	0.45	0.26	0.61	0.43	**Liquefiable**
44.5	42	44.5	43.3	13	120	6	1.3	1.05	1.14	0.70	1	14.2	14.3	5190	3802	4332	0.98	0.89	0.94	0.15	0.13	0.63	0.20	**Liquefiable**
44.5	44.5	47	45.8	13	120	74	1.3	1.05	1.15	0.72	1	14.6	20.2	5490	3946	4476	0.98	0.82	0.92	N/A	N/A	N/A	N/A	Non-Liq: PI>18
49.5	47	49	48	22	120		1.3	1.05	1.29	0.75	1	29.1	29.1	5760	4075	4606	0.97	0.65	0.87	N/A	N/A	N/A	N/A	Non-Liq: PI>18
49.5	49	50	49.5	22	120	19	1.3	1.05	1.3	0.77	1	29.9	34.2	5940	4162	4692	0.97	0.56	0.83	0.94	0.43	0.65	0.67	**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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-4
Engineer	JLL

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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	38.0	0.0	38.0	2.22	0.01	-0.66	0.00	1.00		0.000		0.00	Non-Liquefiable
24.5	22	27	24.5	51.5	0.0	51.5	2.04	0.00	-1.71	0.00	5.00		0.000		0.00	Non-Liquefiable
29.5	27	32	29.5	66.9	0.0	66.9	1.82	0.00	-3.02	0.00	5.00		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	50.9	0.0	50.9	1.66	0.00	-1.66	0.00	5.00		0.000		0.00	Non-Liquefiable
39.5	37	42	39.5	23.8	5.6	29.4	0.43	0.05	-0.05	0.05	5.00		0.010		0.62	**Liquefiable**
44.5	42	44.5	43.3	14.2	0.0	14.3	0.20	0.30	0.78	0.30	2.50		0.030		0.89	**Liquefiable**
44.5	44.5	47	45.8	14.6	5.6	20.2	N/A	0.16	0.51	0.00	2.50		0.000		0.00	Non-Liq: PI>18
49.5	47	49	48	29.1	0.0	29.1	N/A	0.05	-0.03	0.00	2.00		0.000		0.00	Non-Liq: PI>18
49.5	49	50	49.5	29.9	4.3	34.2	0.67	0.03	-0.38	0.03	1.00		0.004		0.05	**Liquefiable**
											Total Deformation (in)				1.56	

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 Palomino Business Park	MCE$_G$ Design Acceleration	0.723 (g)
Project Location	Norco, CA	Design Magnitude	8.8
Project Number	17G105-4	Historic High Depth to Groundwater	21 (ft)
Engineer	JLL	Depth to Groundwater at Time of Drilling	55 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-11		

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=8.8)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	1.00	0.97	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	20	120	6	1.3	1.05	1.3	0.93	0.95	31.4	31.5	2580	2549	2580	1.00	0.60	0.96	0.59	0.34	0.47	0.72	**Liquefiable**
24.5	22	27	24.5	10	120	43	1.3	1.05	1.13	0.86	0.95	12.6	18.2	2940	2722	2940	1.00	0.84	0.97	0.19	0.15	0.51	0.30	**Liquefiable**
29.5	27	32	29.5	24	120	4	1.3	1.05	1.3	0.84	0.95	34.0	34.0	3540	3010	3540	0.99	0.56	0.91	0.92	0.47	0.55	0.85	**Liquefiable**
34.5	32	37	34.5	43	120		1.3	1.05	1.3	0.91	1	69.2	69.2	4140	3298	4140	0.99	0.56	0.87	2.00	0.97	0.58	1.66	Non-Liquefiable
39.5	37	42	39.5	34	120		1.3	1.05	1.3	0.82	1	49.6	49.6	4740	3586	4740	0.98	0.56	0.84	2.00	0.94	0.61	1.54	Non-Liquefiable
44.5	42	47	44.5	44	120		1.3	1.05	1.3	0.87	1	67.9	67.9	5340	3874	5340	0.98	0.56	0.82	2.00	0.92	0.63	1.44	Non-Liquefiable
49.5	47	50	48.5	27	120	45	1.3	1.05	1.3	0.75	1	35.7	41.3	5820	4104	5820	0.97	0.56	0.8	2.00	0.90	0.65	1.38	Non-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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-4
Engineer	JLL

Boring No. B-11

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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	31.4	0.0	31.5	0.72	0.04	-0.19	0.04	1.00		0.007		0.09	**Liquefiable**
24.5	22	27	24.5	12.6	5.6	18.2	0.30	0.20	0.61	0.20	5.00		0.025		1.49	**Liquefiable**
29.5	27	32	29.5	34.0	0.0	34.0	0.85	0.03	-0.37	0.03	5.00		0.004		0.27	**Liquefiable**
34.5	32	37	34.5	69.2	0.0	69.2	1.66	0.00	-3.23	0.00	5.00		0.000		0.00	Non-Liquefiable
39.5	37	42	39.5	49.6	0.0	49.6	1.54	0.00	-1.56	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	67.9	0.0	67.9	1.44	0.00	-3.11	0.00	5.00		0.000		0.00	Non-Liquefiable
49.5	47	50	48.5	35.7	5.6	41.3	1.38	0.01	-0.90	0.00	3.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				1.85	

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 Palomino Business Park	MCE$_G$ Design Acceleration	0.723	(g)
Project Location	Norco, CA	Design Magnitude	8.8	
Project Number	17G105-4	Historic High Depth to Groundwater	21	(ft)
Engineer	JLL	Depth to Groundwater at Time of Drilling	22	(ft)
		Borehole Diameter	6	(in)
Boring No.	B-13			

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=8.8)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	1.00	0.97	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	20	120	4	1.3	1.05	1.3	0.93	0.95	31.4	31.4	2580	2549	2580	1.00	0.60	0.96	0.59	0.34	0.47	0.72	**Liquefiable**
24.5	22	24	23	15	120	5	1.3	1.05	1.21	0.90	0.95	21.2	21.2	2760	2635	2698	1.00	0.80	0.97	0.22	0.17	0.49	0.35	**Liquefiable**
24.5	24	27	25.5	15	120	44	1.3	1.05	1.21	0.89	0.95	21.0	26.6	3060	2779	2842	1.00	0.70	0.95	0.33	0.22	0.51	0.43	**Liquefiable**
29.5	27	32	29.5	28	120	9	1.3	1.05	1.3	0.90	0.95	42.5	43.3	3540	3010	3072	0.99	0.56	0.89	2.00	1.00	0.55	1.82	Non-Liquefiable
34.5	32	37	34.5	8	120	59	1.3	1.05	1.1	0.80	1	9.6	15.2	4140	3298	3360	0.99	0.88	0.95	N/A	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL
39.5	37	39	38	17	120	46	1.3	1.05	1.24	0.83	1	23.7	29.3	4560	3499	3562	0.99	0.65	0.9	0.44	0.26	0.60	0.43	**Liquefiable**
39.5	39	42	40.5	17	120	76	1.3	1.05	1.23	0.81	1	23.2	28.8	4860	3643	3706	0.98	0.66	0.89	N/A	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL
44.5	42	47	44.5	10	120	74	1.3	1.05	1.11	0.75	1	11.4	16.9	5340	3874	3936	0.98	0.86	0.93	N/A	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL
49.5	47	50	48.5	13	120	61	1.3	1.05	1.15	0.75	1	15.2	20.8	5820	4104	4166	0.97	0.81	0.91	N/A	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL

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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-4
Engineer	JLL

Boring No. B-13

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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	31.4	0.0	31.4	0.72	0.04	-0.19	0.04	1.00		0.007		0.09	**Liquefiable**
24.5	22	24	23	21.2	0.0	21.2	0.35	0.14	0.45	0.14	2.00		0.022		0.53	**Liquefiable**
24.5	24	27	25.5	21.0	5.6	26.6	0.43	0.07	0.13	0.07	3.00		0.016		0.59	**Liquefiable**
29.5	27	32	29.5	42.5	0.7	43.3	1.82	0.00	-1.05	0.00	5.00		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	9.6	5.6	15.2	N/A	0.27	0.75	0.00	5.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL
39.5	37	39	38	23.7	5.6	29.3	0.43	0.05	-0.04	0.05	2.00		0.010		0.25	**Liquefiable**
39.5	39	42	40.5	23.2	5.6	28.8	N/A	0.05	-0.01	0.00	3.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL
44.5	42	47	44.5	11.4	5.6	16.9	N/A	0.22	0.67	0.00	5.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL
49.5	47	50	48.5	15.2	5.6	20.8	N/A	0.14	0.47	0.00	3.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL
											Total Deformation (in)				1.45	

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 Palomino Business Park	MCE_G Design Acceleration	0.723 (g)
Project Location	Norco, CA	Design Magnitude	8.8
Project Number	17G105-4	Historic High Depth to Groundwater	21 (ft)
Engineer	JLL	Depth to Groundwater at Time of Drilling	41 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-16		

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=8.8)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	1.00	0.97	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	36	120		1.3	1.05	1.3	0.96	0.95	58.4	58.4	2580	2549	2580	1.00	0.56	0.94	2.00	1.05	0.47	2.22	Non-Liquefiable
24.5	22	26	24	13	120	30	1.3	1.05	1.17	0.88	0.95	17.4	22.8	2880	2693	2880	1.00	0.77	0.96	0.25	0.18	0.50	0.37	**Liquefiable**
29.5	26	31	28.5	17	120	46	1.3	1.05	1.23	0.83	0.95	22.5	28.2	3420	2952	3420	0.99	0.67	0.94	0.39	0.25	0.54	0.45	**Liquefiable**
34.5	31	37	34	60	120		1.3	1.05	1.3	1.01	1	107.3	107.3	4080	3269	4080	0.99	0.56	0.87	2.00	0.97	0.58	1.67	Non-Liquefiable
39.5	37	50	43.5	50	120		1.3	1.05	1.3	0.93	1	82.2	82.2	5220	3816	5064	0.98	0.56	0.82	2.00	0.92	0.63	1.46	Non-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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-4
Engineer	JLL

Boring No. B-16

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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	58.4	0.0	58.4	2.22	0.00	-2.28	0.00	1.00		0.000		0.00	Non-Liquefiable
24.5	22	26	24	17.4	5.4	22.8	0.37	0.12	0.36	0.12	4.00		0.021		0.99	**Liquefiable**
29.5	26	31	28.5	22.5	5.6	28.2	0.45	0.06	0.03	0.06	5.00		0.013		0.76	**Liquefiable**
34.5	31	37	34	107.3	0.0	107.3	1.67	0.00	-6.77	0.00	6.00		0.000		0.00	Non-Liquefiable
39.5	37	50	43.5	82.2	0.0	82.2	1.46	0.00	-4.40	0.00	13.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				1.75	

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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-4
Engineer	JLL
Boring No.	B-17

MCE_G Design Acceleration	0.723	(g)
Design Magnitude	8.8	
Historic High Depth to Groundwater	21	(ft)
Depth to Groundwater at Time of Drilling	30	(ft)
Borehole Diameter	6	(in)

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=8.8)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	1.00	0.97	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	27	120		1.3	1.05	1.3	0.95	0.95	43.1	43.1	2580	2549	2580	1.00	0.56	0.94	2.00	1.05	0.47	2.22	Non-Liquefiable
24.5	22	27	24.5	46	120		1.3	1.05	1.3	0.96	0.95	74.5	74.5	2940	2722	2940	1.00	0.56	0.92	2.00	1.03	0.51	2.04	Non-Liquefiable
29.5	27	31	29	34	120		1.3	1.05	1.3	0.89	0.95	51.0	51.0	3480	2981	3480	0.99	0.56	0.9	2.00	1.00	0.54	1.84	Non-Liquefiable
34.5	31	37	34	10	120	59	1.3	1.05	1.12	0.76	1	11.5	17.2	4080	3269	3830	0.99	0.86	0.95	0.18	N/A	N/A	N/A	Non-liq: PI>18
39.5	37	42	39.5	15	120	51	1.3	1.05	1.18	0.76	1	18.4	24.1	4740	3586	4147	0.98	0.75	0.92	0.27	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL
44.5	42	47	44.5	12	120	45	1.3	1.05	1.13	0.72	1	13.3	18.9	5340	3874	4435	0.98	0.83	0.92	0.19	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL
49.5	47	50	48.5	23	120	39	1.3	1.05	1.3	0.78	1	31.8	37.4	5820	4104	4666	0.97	0.56	0.8	1.94	0.87	0.65	1.34	Non-Liq: PI>18

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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-4
Engineer	JLL

Boring No. B-17

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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	43.1	0.0	43.1	2.22	0.00	-1.04	0.00	1.00		0.000		0.00	Non-Liquefiable
24.5	22	27	24.5	74.5	0.0	74.5	2.04	0.00	-3.70	0.00	5.00		0.000		0.00	Non-Liquefiable
29.5	27	31	29	51.0	0.0	51.0	1.84	0.00	-1.67	0.00	4.00		0.000		0.00	Non-Liquefiable
34.5	31	37	34	11.5	5.6	17.2	N/A	0.22	0.66	0.00	6.00		0.000		0.00	Non-liq: PI>18
39.5	37	42	39.5	18.4	5.6	24.1	N/A	0.10	0.29	0.00	5.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL
44.5	42	47	44.5	13.3	5.6	18.9	N/A	0.18	0.57	0.00	5.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL
49.5	47	50	48.5	31.8	5.6	37.4	1.34	0.01	-0.61	0.00	3.00		0.000		0.00	Non-Liq: PI>18
											Total Deformation (in)				0.00	

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 Palomino Business Park	MCE$_G$ Design Acceleration	0.723	(g)
Project Location	Norco, CA	Design Magnitude	8.8	
Project Number	17G105-4	Historic High Depth to Groundwater	21	(ft)
Engineer	JLL	Depth to Groundwater at Time of Drilling	29	(ft)
		Borehole Diameter	6	(in)
Boring No.	B-18			

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=8.8)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7.5	0	21	10.5		120		1.3	1.05	1.1	1.50	0.75	0.0	0.0	1260	1260	1260	1.00	0.97	1.03	0.06	0.06	N/A	N/A	Above Ground Water
19.5	21	22	21.5	50	120		1.3	1.05	1.3	0.98	0.95	82.9	82.9	2580	2549	2580	1.00	0.56	0.94	2.00	1.05	0.47	2.22	Non-Liquefiable
24.5	22	27	24.5	34	120		1.3	1.05	1.3	0.93	0.95	53.3	53.3	2940	2722	2940	1.00	0.56	0.92	2.00	1.03	0.51	2.04	Non-Liquefiable
29.5	27	32	29.5	42	120		1.3	1.05	1.3	0.92	0.95	65.2	65.2	3540	3010	3509	0.99	0.56	0.89	2.00	1.00	0.55	1.82	Non-Liquefiable
34.5	32	37	34.5	12	120	55	1.3	1.05	1.15	0.77	1	14.5	20.1	4140	3298	3797	0.99	0.82	0.94	0.21	N/A	N/A	N/A	Non-liq: 12<PI<18, w<.8*LL
39.5	37	42	39.5	23	120	37	1.3	1.05	1.3	0.82	1	33.4	38.9	4740	3586	4085	0.98	0.56	0.84	2.00	0.94	0.61	1.54	Non-Liquefiable
44.5	42	47	44.5	17	120	59	1.3	1.05	1.21	0.76	1	21.3	26.9	5340	3874	4373	0.98	0.70	0.89	0.34	N/A	N/A	N/A	Non-Liq: PI>18
49.5	47	50	48.5	8	120	78	1.3	1.05	1.1	0.68	1	8.1	13.7	5820	4104	4603	0.97	0.90	0.93	0.15	0.12	0.65	0.19	**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 Palomino Business Park
Project Location	Norco, CA
Project Number	17G105-4
Engineer	JLL

Boring No. B-18

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)			
7.5	0	21	10.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	21.00		0.000		0.00	Above Ground Water
19.5	21	22	21.5	82.9	0.0	82.9	2.22	0.00	-4.46	0.00	1.00		0.000		0.00	Non-Liquefiable
24.5	22	27	24.5	53.3	0.0	53.3	2.04	0.00	-1.86	0.00	5.00		0.000		0.00	Non-Liquefiable
29.5	27	32	29.5	65.2	0.0	65.2	1.82	0.00	-2.87	0.00	5.00		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	14.5	5.6	20.1	N/A	0.16	0.51	0.00	5.00		0.000		0.00	Non-liq: 12<PI<18, w<.8*LL
39.5	37	42	39.5	33.4	5.5	38.9	1.54	0.01	-0.72	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	21.3	5.6	26.9	N/A	0.07	0.12	0.00	5.00		0.000		0.00	Non-Liq: PI>18
49.5	47	50	48.5	8.1	5.6	13.7	0.19	0.32	0.81	0.32	3.00		0.031		1.10	**Liquefiable**
											Total Deformation (in)				1.10	

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)

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(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)

Memorandum

CITY OF NORCO
City Engineer's Office

DATE: **October 9, 2019**

TO: **STEVE KING, DIRECTOR OF PLANNING**

FROM: **DOMINIC MILANO, CITY ENGINEER**

SUBJECT: **COMMENTS ON EIR (ADMINISTRATIVE DRAFT)**

Comments on EIR (Administrative Draft)

- Figure 3-4 Calls out two "Infiltration Basins". Geotechnical Report calls them out as a detention basin. There is a difference. Boring number B-14 shows some clay but no analysis of percolation.
- Figure 3-9 Mission Avenue is Mountain Avenue.
- Page 3-24 Traffic analysis also calls out a signal at Parkridge Avenue and Second Street at City's discretion.
- Page 3-27 Infiltration Basins not discussed in Geotechnical Report.
- Figure 3-13 Infiltration Basins called out. But is it?
- Page 5.4-9 What is being proposed by the Historic American Building survey for the EGG Ranch beside photos (page 5.4-11)? Shouldn't there be more?
- 5.9-11 In the Operation Section - "Infiltration Basins" are listed as a Storm Water Improvement. There wasn't enough soils testing done and discussion in other sections to verify that infiltration is a viable alternative. Also some exhibits list the infiltration basins as detention basins. Infiltration basins are also discussed in other mitigation measures of this chapter.
- Section 5.13.3 See traffic comments.

Water Quality Management Plan

- Page 9 References Appendix 3, which is not included. But concludes information from a Geotechnical Report that there is favorable infiltration rates. I did not see Geotechnical Report that included "favorable" rates. Comments on infiltration included throughout the plan.
- Appendix 1 through 5 and 7 through 10 were not included. Review incomplete.

Jurisdictional Delineation Report

- No Comment

Biological Technical Report

- No Comment but we need to be aware of the requirement in Section 6.2 nesting birds for an onsite biologist depending on the time of year for grading.

Geotechnical Investigation

- Page 12 Section 6.1 Seismic design considerations use the 2016 CBC. The 2019 CBC will be effect for the project.
- Page 22 Treatment of Existing Soils: Parking and Drive Areas. Should optional over excavating in truck drive area even be considered given the truck loading?
- Page 31 Section 6.8. R-Values must be taken after rough grading. The option not to test should not be given.
- Page 32. An option of a TI of 4.0 for auto parking should not be given.
- Exhibits in the Appendix A do not have infiltration basins only detention basins. See comments on EIR.
- Percolation is not discussed in this report.

Traffic Impact Analysis

- Page 4 Mitigation Measure 3.1 references Table 7-5 for improvements. Table 7-5 shows the improvement at an intersection were made but does not list what the improvements were. Should there be a different reference table?
- Page 6 Agreement needs to be included in report.
- Page 10 Off ramp analysis. Report states there are no movements experiencing queuing issues but Caltrans is widening the ramps because of queuing issues.
- Page 11 Is "Opening Year (2022) with project" after mitigation?
- Page 12 Second bullet point at top of page. Isn't southbound on ramp at Second Street also at an E in PM?
- Page 15 First paragraph. Statements don't make sense.
- Page 16 Hamner Avenue and Second Street, 3rd bullet point. Verify that there is enough room to accomplish the proposed striping.
- Page 17 Section 1.5.4 How can you conclude that no intersection is anticipated to result in an unacceptable LOS?
- Page 32 Second bullet point. Use 25' radius instead of 20'. (Also Exhibit 1-5)
- Page 33 Missing 2 radii on driveway 6.
- Page 42 Section 2.8 Change County Engineer to City Engineer.
- Page 63 Number 32. Please verify acceptable level. Caltrans believes there is an issue. See page 67 PM is an "F".
- Page 65 Section 3.10 Are the LOS's consistent throughout report?
- Page 95 Table 5.1 Isn't Hamner Avenue at Second Street at an "F".
- Page 97 How do these number relate to table 5-4?

- Page 117 Exhibit 6-7 Why did the volume at this off ramp southbound Second Street AM peak decrease from 2018?
- Page 220 Table 6-4 Note again Second Street on ramp at an “F”.