Appendix D

Geotechnical Analysis

GEOTECHNICAL INVESTIGATION PROPOSED COMMERCIAL/INDUSTRIAL DEVELOPMENT

SWC Beverly Boulevard and San Gabriel River Freeway (Interstate 605)
Pico Rivera, California
for
InSite Property Group

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

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A California Corporation

June 4, 2020

InSite Property Group
811 N. Catalina Avenue
Redondo Beach, California 90277

Attention: Mr. Brian Sorensen

Project No.: **20G147-1**

Subject: **Geotechnical Investigation**
Proposed Commercial/Industrial Development
SWC Beverly Boulevard and San Gabriel River Freeway (Interstate 605)
Pico Rivera, 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.

Daniel W. Nielsen, RCE 77915
Senior Engineer

Robert G. Trazo, M.Sc., GE 2655
Principal Engineer

Distribution: (1) Addressee

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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 subject site is located an area designated by the California Geological Survey (CGS) as a liquefaction hazard zone.
- Our site-specific liquefaction evaluation included two (2) borings advanced to depths of 50± feet. Liquefiable soils were encountered at one of these borings.
- The total potential liquefaction-induced settlements at these borings range from 0± to 0.7± inches.
- Based on the estimated magnitude of the differential settlements, proposed Building A may be supported on shallow foundations. Additional design considerations related to the potentially liquefiable soils are presented in this report.
- At this time, structural information regarding the proposed construction type, proposed foundation system, and foundation loads for Building B is not available. Detailed foundation design and construction recommendations should be prepared for Building B after this information becomes available.
- The subject site is underlain by undocumented fill soils and variable strength 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 within the area of Proposed Building A to remove the undocumented artificial fill soils and a portion of the near-surface alluvial soils from the proposed building pad area 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.
- Remedial grading is also expected to be necessary in the area of proposed Building B. However, grading recommendations for Building B should be prepared after preliminary structural information, including the type of construction and foundation loads, becomes available.
- Additional subsurface exploration and laboratory testing may be necessary for Building B based on the foundation loads and the type of foundation system.

Site Preparation

- Initial site preparation should include stripping of any surficial vegetation. Based on the conditions observed at the time of subsurface exploration, stripping should include all vegetation including grass and weed growth, shrubs, trees and any organic soils. These materials should be properly disposed of off-site.
- Demolition of the existing concrete swale will be necessary to facilitate construction of the proposed development. Debris resultant from demolition activities should be disposed of off-site. Alternatively, concrete debris may be re-used within compacted fills, provided they are pulverized to a maximum particle size of less than 2 inches, and thoroughly mixed with the on-site soils.

- The existing soils within the area of proposed Building A should be overexcavated to a depth of 4 feet below existing grade and to a depth of at least 4 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 which extend to depths of 1½ to 7± feet at the boring and trench locations.
- The proposed foundation influence zones should be overexcavated to a depth of at least 3 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 0 to 4 percent above optimum, and recompacted. The resulting soils may be replaced as compacted structural fill.

Building Foundations – Building A
- 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.
- The construction type, type of foundation system, and foundation loads for Building B are not available at the time of this report. Detailed foundation design and construction recommendations should be prepared after preliminary structural information, including preliminary foundation loads becomes available.

Building Floor Slabs
- Conventional Slab-on-Grade, 6 inches thick.
- Minimum reinforcement of the floor slab should consist of No. 3 bars at 18-inches on center in both directions, due to the presence of potentially liquefiable soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed slab loading.
- Modulus of Subgrade Reaction: 150 psi/in.

Pavements

ASPHALT PAVEMENTS (R = 40)					
Materials	**Thickness (inches)**				
	Parking Stalls (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	3	4	5	7	8
Compacted Subgrade	12	12	12	12	12

<table>
<tr><th colspan="5">PORTLAND CEMENT CONCRETE PAVEMENTS (R = 40)</th></tr>
<tr><th rowspan="3">Materials</th><th colspan="4">Thickness (inches)</th></tr>
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<tr><td>PCC</td><td>5</td><td>5</td><td>5½</td><td>6½</td></tr>
<tr><td>Compacted Subgrade (95% minimum compaction)</td><td>12</td><td>12</td><td>12</td><td>12</td></tr>
</table>

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 20P222, dated May 4, 2020. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements for Building A along with site preparation recommendations and construction considerations for the proposed development. Building B was added to the proposed development following the completion of the referenced proposal. Additional subsurface exploration was performed in the area of Building B. However, we understand that Building B will consist of a 7-story self-storage building. No information is presently available regarding the proposed construction type, the type of foundation system, nor the foundation loads. Additional structural information will be required before SCG can provide detailed grading and foundation design recommendations for the proposed Building B.

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 on the south side of Beverly Boulevard at the intersection of Beverly Boulevard and the south-bound on-ramp to the San Gabriel Freeway (Interstate 605) in Pico Rivera, California. The site is bounded to the north by Beverly Boulevard, to the west by a railroad easement, to the south by existing single-family residences and the terminus of Eduardo Avenue, and to the east by the San Gabriel River Freeway. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The subject site consists of several contiguous irregularly-shaped parcels which total 19.06± acres in size. The site is vacant and undeveloped except for a concrete lined drainage swale located in the northeastern area of the site. The swale is about 350± feet long, 15± feet wide and 3 to 5± feet deep. The ground surface cover consists of exposed soil with sparse to dense native grass and weed growth. Additionally, the northwestern portion of the site possess scattered debris and trash.

Our review of readily available historical aerial photographs from NETRonline, indicates that the northern portion of the site was previously a citrus orchard in 1948. Sometime between the time of the 1948 photograph and the 1953 photograph, the orchard was removed. The San Gabriel Freeway was constructed adjacent to the site sometime between photographs taken during 1963 and 1964. During this time the existing, concrete lined drainage swale was constructed on site.

Based on a topographic plan prepared by Michael Baker International, overall site topography ranges from 220± feet mean sea level (msl) in the north corner of the site to 192± feet msl in the southwest corner of site. The northwestern portion of the site slopes towards the concrete lined drainage swale at a gradient of 6 to 10± percent. A slope is present in the central and northern portions of the site. This slope possesses an inclination of approximately 2h:1v and descends downward toward the western and northern property lines. The slope ranges in height between 3 and 16± feet, increasing in the northern portion of the site. The remaining areas of the site generally slope downward to the southeast at a gradient of 6± percent.

3.2 Proposed Development

Based on the conceptual site plan provided by the client, the site will be developed with two (2) commercial/industrial buildings, identified as Building A and Building B. Building A, will possess a footprint area of 386,530± ft^2 with limited areas of 2nd story mezzanine. Dock-high doors will be constructed along the northwestern side of the building. Detailed structural information has not been provided for Building A. It is assumed that Building A will be of tilt-up concrete construction, typically supported on a conventional shallow foundation system with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 4 to 7 kips per linear foot, respectively.

Building B, will be located in the northern area of the site and, based on very recent discussions with the project civil engineer, may be utilized as a self-storage facility. Building B will possess a footprint area of about 15,000± ft^2. The building is expected to be seven (7) stories in height, with a gross building area of about 105,000± ft^2. Detailed structural information for Building B not been provided, and it is our understanding that no structural engineer has currently been retained for the design of this building. At this time, the type of structure and the structural loads are unknown to SCG. No information about the proposed foundation system has been provided to our office. Based on the relatively high structural loads anticipated for a 7-story storage building, conventional shallow foundations may not be suitable for the support of this 7-story structure. More detailed structural information will be necessary in order for SCG to provide detailed foundation design recommendations for Building B.

We expect that the buildings will be surrounded by Portland cement concrete pavements in truck loading areas, asphaltic concrete pavements in the automobile parking and drive areas, and some concrete flatwork and landscape planters throughout the site.

Grading plans for the proposed development were not available at the time of this report. The proposed development is not expected to include any significant amounts of below-grade construction such as basements or crawl spaces. Based on the existing topography, and assuming a relatively balanced site, cuts and fills of 10 to 12± feet are expected to be necessary to achieve the proposed site grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of eight (8) borings advanced to depths of 20 to 50± feet below the existing site grades. Two (2) of these borings were advanced to a depth of 50± feet as a part of the liquefaction evaluation. Additionally, four (4) trenches were excavated to depths of 15 to 17± feet below existing site grades. All of the borings and trenches were logged during drilling and excavation by a member of our staff.

All of the borings were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig. The trenches were excavated using a rubber tire backhoe with a 36-inch-wide bucket. Representative bulk and relatively undisturbed soil samples were taken during drilling. Relatively undisturbed soil samples were taken with a split barrel "California Sampler" containing a series of one-inch-long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. Samples were also taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

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

4.2 Geotechnical Conditions

Artificial Fill

Artificial fill soils were encountered at the ground surface at all of the boring and trench locations, with the exception of Boring Nos. B-2 and B-7. The fill soils extend to depths of 1½ to 7± feet below the existing site grades at the boring and trench locations. The fill soils generally consist of loose to medium dense silty fine sands and fine sands with varying amounts of medium to coarse sand and fine gravel. The fill soils possess a disturbed and mottled appearance, and occasional samples possess artificial debris including glass and/or asphaltic concrete fragments, resulting in their classification as artificial fill.

Soils classified as possible fill were encountered at the ground surface at Boring No. B-7, extending to a depth of 7± feet below the existing ground surface. The possible fill materials generally consist of fine sands with little medium to coarse sand and fine to coarse gravel content.

These soils possess a slightly disturbed appearance and resemble the some of the near-surface alluvium, resulting in their classification as possible fill. The possible fill soils lack obvious indicators of fill, such as artificial debris content or an extensively disturbed appearance.

Alluvium

Native alluvial soils were encountered at the ground surface at Boring No. B-2, and beneath the artificial fill soils or possible fill soils at all of the remaining boring and trench locations. Some of the alluvial soils encountered in the upper 3 to 12± feet consist of loose to medium dense fine sand, silty fine sands, and occasional fine sandy silts. However, all of the borings and trenches encountered medium dense to very dense well-graded sands, gravelly sands, and/or sandy gravels at depths as shallow as 3 to 12± feet. These sandy soils possess occasional to some cobble content, as noted on the trench logs. The thicknesses of these well-graded sandy strata vary at the boring and trench locations, but in general, the well graded sandy soils extend to maximum depths of 12 to 22± feet below the ground surface.

The medium dense to very dense well-graded sand strata are generally underlain by medium dense fine sandy silts and stiff to very stiff fine sandy clays and silty clays, extending to the maximum depth explored of 50± feet at the boring locations. A low-plasticity silt layer was encountered at Boring No. B-1 between depths of 47 and 50± feet. Some of these soils possess iron oxide staining and calcareous nodules/veining.

Groundwater

Free water was encountered during drilling at Boring No. B-4 at a depth of 49½± feet below ground surface. Due to caving within the open borehole, a delayed water level measurement was not possible. Based on the moisture contents of the recovered soil samples, and the water measurement taken within the open borehole, the static groundwater table is considered to have existed at a depths of 49½± feet and greater at the time of the subsurface exploration.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the historic groundwater depths in this area is the CGS Seismic Hazard Zone Report 024, the Seismic Hazard Zone Report for the El Monte 7.5-Minute Quadrangle, which indicates that the historic high groundwater level for the site is 20± feet below the ground surface.

More 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 3,200± feet east from the subject site. Water level readings within these monitoring wells indicate a high groundwater level of 76± feet below the ground surface in August 2019.

5.0 LABORATORY TESTING

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

Classification

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

Dry Density and Moisture Content

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

Consolidation

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

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-4 @ 0 to 5 feet	0.002	Not Applicable (S0)
B-8 @ 0 to 5 feet	0.001	Not Applicable (S0)

Maximum Dry Density and Optimum Moisture Content

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

Organic Content Testing

Selected soil samples have been tested to determine their organic content, in accordance with ASTM Test Method 2974. The results of the testing are as follows:

Sample Identification	**Organic Content (%)**
T-1 @ 0 to 6 inches	2.7
T-1 @ 6 to 12 inches	2.5
T-1 @ 12 to 18 inches	2.5
T-1 @ 18 to 24 inches	2.8
T-4 @ 0 to 6 inches	3.6
T-4 @ 6 to 12 inches	2.1
T-4 @ 12 to 18 inches	2.3
T-4 @ 18 to 24 inches	2.2

Corrosivity Testing

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

Sample Identification	**Resistivity** (ohm-cm)	**pH**	**Chlorides** (mg/kg)
B-4 @ 0 to 5 feet	2,840	7.9	7.2
B-8 @ 0 to 5 feet	3,600	7.6	6.0

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

6.1 Seismic Design Considerations

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

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

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

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

The 2019 CBC Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool, a web-based software application available at the website www.seismicmaps.org. This software application calculates seismic design parameters in accordance with several building code reference documents, including ASCE 7-16, upon which the 2019 CBC is based. The application utilizes a database of risk-targeted maximum considered earthquake (MCE_R) site accelerations at 0.01-degree intervals for each of the code documents. The tables below were created using data obtained from the application. The output generated from this program is included as Plate E-1 in Appendix E of this report.

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

We do not expect that the exception in Section 11.4.8 of ASCE 7-16 will apply to the proposed 7-story self-storage building (Building B). The structural engineer should verify if the exception in ASCE 7-16 Section 11.4.8 applies to this structure. However, it is likely that a site response analysis (ASCE 7-16 Section 21.1) or ground motions hazard analysis (ASCE 7-16 Section 21.2) will be required for this structure. In the event that this exception does not apply this structure, the parameters provided below will not be applicable and the site-specific analysis will supersede the values provided in the table below.

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.900
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.679
Site Class	---	D*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.900
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.154
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.267
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.770

*The 2019 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site *coefficients* are to be determined in accordance with Section 11.4.7 of ASCE 7-16. However, Section 20.3.1 of ASCE 7-16 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. <u>The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of the structures is less than 0.5 seconds.</u> 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 will be required and additional subsurface exploration will be necessary.

It should be noted that the site coefficient F_v and the parameters S_{M1} and S_{D1} were not included in the <u>SEAOC/OSHPD Seismic Design Maps Tool</u> 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 <u>Seismic Design Maps Tool</u>, assuming that a site-specific ground motion hazards analysis is not required for the proposed buildings at this site.

<u>Ground Motion Parameters</u>

For the purposes of the liquefaction analysis performed for this study, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2019 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-16. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-16. The web-based software application <u>SEAOC/OSHPD Seismic Design Maps Tool</u> (described in the previous section) was used to determine PGA_M, which is 0.905g. A portion of the program output is included as Plate E-1 of this report. An associated earthquake magnitude was obtained from the USGS Unified Hazard Tool, Interactive Deaggregation application available on the USGS website. The deaggregated mean magnitude is 6.83, based on the peak ground acceleration and soil classification D.

<u>Liquefaction</u>

Research of the <u>Seismic Hazard Zone Report of the El Monte, California 7.5-Minute Quadrangle,</u> Seismic Hazard Report 024, published by the California Geological Survey, indicates that the site is located in a designated liquefaction hazard zone. Therefore, the scope of this investigation liquefaction evaluation in order to study 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. 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% 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.

As part of the liquefaction evaluation, Boring Nos. B-1, and B-4 were extended to a depth of 50± feet. The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report, using the data obtained from these borings. The liquefaction potential of the site was analyzed utilizing a PGA_M of 0.905g for a magnitude 6.83 seismic event.

The historic high groundwater depth used in the liquefaction analysis was obtained from CGS Open File Report 024, the Seismic Hazard Zone Report for the El Monte, California, 7.5-Minute Quadrangle, which indicates a historic high groundwater depth of about 20± 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 in Appendix F.

Conclusions and Recommendations

A potentially liquefiable soil stratum was encountered at one of the 50-foot deep boring locations. The potentially liquefiable stratum identified at Boring No. B-1 is present between depths of 47 to 50± feet and consists of a medium dense low-plasticity silt stratum. The remaining soil strata encountered below the historic high groundwater table either possess adequate factors of safety, or are considered non-liquefiable due to their cohesive characteristics and the results of the Atterberg limits testing with respect to the requirements of Special Publication 117A. Settlement analyses were performed for the potentially liquefiable stratum. No liquefiable soils were identified in our analysis of Boring No. B-4. The results of the settlement analyses indicate a potential total deformation of 0.72 inches at Boring No. B-1.

Based on the results of the settlement analyses, total dynamic settlements due to liquefaction are expected to range from be 0 to 0.7± inches. The resulting differential settlements are expected to be on the order of ½± inch. The estimated differential settlement can be assumed to occur across a distance of 50 feet, indicating a maximum angular distortion of approximately 0.001 inches per inch.

Based on our understanding of the proposed development, it is considered feasible to support the proposed concrete tilt-up warehouse structure, Building A, on conventional shallow foundations. 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 structure to remain completely undamaged during a seismic event that could occur once every 2475 years (the code-specified return period used in the liquefaction analysis) is not considered to be economically feasible. Based on this understanding, the use of shallow foundation systems is considered to be the most economical means of supporting the proposed structures.

Not enough information has been provided to determine the appropriate foundation system for Building B. The potential total and dynamic liquefaction-induced settlements should be considered when designing the foundation system for Building B.

In order to support proposed Building A on shallow foundations (such as spread footings) the structural engineer should verify that the structure would not catastrophically fail due to the predicted dynamic differential settlements. Any utility connections to the structure should be designed to withstand the estimated differential settlement. 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 for proposed Building A, as described in this report, 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 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 techniques or mat foundations.

6.2 Geotechnical Design Considerations

General

We understand that the proposed development will consist of two buildings one of these, Building A, will be new warehouse building of concrete tilt-up construction. The second building, Building B, may be a 7-story self-storage structure. The type of construction and preliminary foundation loads for Building B have not been provided to us at this time.

The soils encountered at the ground surface at this site consist of low to moderate strength fill and native alluvium. The fill soils encountered during subsurface exploration, extending to depths of 1½ to 7± feet throughout the site, are loose to medium dense and are considered to represent undocumented fill materials due to their disturbed appearance, debris content, and the lack of any documentation regarding the placement and compaction of these materials. In addition, the near-surface alluvium possesses variable strengths and composition, and the results of laboratory testing indicate these near-surface alluvial soils possess a minor potential for consolidation settlement. Based on these conditions, remedial grading is considered warranted within the area of proposed Building A in order to remove the upper portion of the alluvium and all of the artificial fill soils and to replace these soils as compacted structural fill. Some remedial grading may be required in the area of Building B depending upon the proposed foundation system. Not enough information is available regarding the structure type and foundation loads to determine the appropriate foundation system for Building B at the time of this report.

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

The recommended remedial grading in the area of Building A will remove the artificial fill soils and a portion of the variable strength near-surface native alluvium. The excavated soils will be replaced as compacted structural fill. The native alluvial soils that will remain in place below the recommended depth of overexcavation will not be subject to significant load increases from the foundations of the new structure. Provided that the recommended remedial grading is completed, the post-construction static settlements of the proposed structures are expected to be within tolerable limits.

Detailed Structural information has not been provided for Building B. When preliminary information regarding the structure type and foundation loads becomes available, a detailed settlement analysis should be performed as a part of the foundation design for proposed Building B.

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 areas.

Corrosion Potential

The results of laboratory testing indicate that the tested samples of the near-surface soils possess resistivity values ranging between 2,840 to 3,600 ohm-cm, and pH values ranging between 7.6 to 7.9. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity, pH, Sulfides, and redox potential are factors that enter into the evaluation procedure. Relative soil moisture content is also considered. **Based on these factors, and utilizing the DIPRA procedure, some of the on-site soils are considered to be corrosive to ductile iron pipe. Therefore, protection for embedded metal improvements is expected to be required.** Since SCG does not practice in the area of corrosion engineering, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.

The results of chloride content testing indicate that the on-site soils possess chloride concentrations ranging between 6 and 7.2 parts per million (ppm). The Caltrans Memo to Designers 10-5, Protection of Reinforcement Against Corrosion Due to Chlorides, Acids and Sulfates, dated June 2010, indicates that soils possessing chloride concentrations greater than 500 ppm are considered to be corrosive. The chloride concentrations present in the soils tested are not considered to constitute a corrosive exposure to steel reinforcement within reinforced concrete.

Organic Content

The results of laboratory testing indicate that some of the soils present within the upper 2± feet possess organic contents of 2.1 to 3.6 percent.

As previously discussed, it is recommended that any vegetation and organic materials be removed from the site in their entirety. If any additional organic materials are encountered in buried fills they should also be segregated and removed from the site during grading.

Based on the results of laboratory testing, it is considered feasible to reuse the near surface soils in structural fills, provided that these soils are cleaned of all visually apparent vegetation and any highly organic material, if present. Based on the grading and site preparation recommendations provided in this report, the near surface soils are expected to possess organic contents of less than 3 percent at the completion of rough grading.

Expansion

The near-surface soils generally consist of sands, and silty sands, and sandy silts. Based on their composition and lack of any appreciable plasticity, these soils are considered to be non-expansive. Therefore, no design considerations related to expansive soils are considered warranted for this site. All imported fill soils should also possess very low expansive characteristics.

Shrinkage/Subsidence

Removal and recompaction of the near-surface alluvium and fill soils is estimated to result in an average shrinkage of 8 to 15 percent. It should be noted that the potential shrinkage estimate is based on dry density testing performed on small-diameter samples taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

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

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

Grading and Foundation Plan Review

Grading and foundation plans 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 stripping should include removal of any surficial vegetation from the site. This should include any weeds, grasses, shrubs, and trees. Root masses associated with the trees should be removed in their entirety, and the resultant excavations should be backfilled with compacted structural fill soils. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

If any tree root masses are encountered during site grading, they should be removed in their entirety and the resulting excavations should be backfilled with compacted structural fill soils.

Demolition of the existing concrete swale will be necessary in order to facilitate the construction of the proposed development. No former structures associated with the orchard are known to SCG. However, any remnants of former development that will not be reused with the proposed development should be demolished, including all foundations, utilities, septic systems, and any other subsurface improvements that will not remain in place with the new development. Demolition debris should be disposed of offsite in accordance with local regulations. Alternatively, concrete debris may be re-used within compacted fills, provided they are pulverized to a maximum particle size of less than 2 inches, and thoroughly mixed with the on-site soils.

Treatment of Existing Soils: Building Pad – Building A

Remedial grading should be performed within the area of proposed Building A 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. Existing undocumented fill soils extend to depths of 1½ to 7 feet at the boring and trench locations. Based on the conditions encountered at the boring and trench locations, the existing soils within the proposed building area are recommended to be overexcavated to a depth of at least 4 feet below the proposed building pad subgrade elevation and to a depth of at least 4 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 3 feet below proposed foundation bearing grade.

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

Following completion of the overexcavation, the subgrade soils within the building area should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or loose, porous, or low-density native soils are encountered at the base of the overexcavation. Any soils classified as possible fill soils on the boring logs, such as those encountered within the upper 7± feet at Boring No. B-7, 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.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, thoroughly moisture conditioned, and recompacted. Overexcavation bottoms should be thoroughly moisture conditioned to achieve a moisture content of 0 to 4 percent above the optimum moisture content, extending to a depth of 18 to 24 inches below the overexcavation subgrade. The previously excavated soils may then be replaced as compacted structural fill.

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

Some remedial grading similar to that described above for Building A is expected to be necessary within the building pad area for Building B. At this time, the type of construction, type of foundation system, and foundation loads for Building B have not yet been determined. Remedial grading recommendations should be prepared for Building B after this information becomes available.

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

The existing soils within the areas of proposed retaining and non-retaining site walls should be overexcavated to a depth of at least 3 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pad for Building A. 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 proposed Building A. 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.

<u>Treatment of Existing Soils: Parking and Drive Areas</u>

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 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength alluvial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed 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.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 0 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2019 CBC and the grading code of the city of Pico Rivera.
- 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 Pico Rivera. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

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

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of sands, silty sands, and sandy silts. These materials will likely be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 2h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

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

Groundwater

Ground water was encountered at a depth of 49½± feet at one of the boring locations but not at any of the other boring and trench locations. Groundwater is therefore not expected to impact the grading or foundation construction activities for Building A.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad for Building A will be underlain by structural fill soils used to replace the undocumented fill soils and near-surface alluvial soils. These new structural fill soils in the area of Building A are expected to extend to depths of at least 3 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 Building A may be supported on conventional shallow foundations.

As previously discussed, the construction type, type of foundation system, and foundation loads are not available at the time of this report. We do not expect that the foundation design and construction recommendations provided in this section will be applicable to Building B, as currently proposed. Foundation design and construction recommendations for Building B should be prepared after preliminary structural information, including preliminary foundation loads, becomes available.

Foundation Design Parameters - Building A

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

Estimated Foundation Settlements

Post-construction total and differential 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: 0.30

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

6.6 Floor Slab Design and Construction

Subgrades which will support the new floor 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 = 150 psi/in.

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

- Slab underlayment: If moisture sensitive floor coverings will be used then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire slab area where such moisture sensitive floor coverings are expected. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

- Moisture condition the floor slab subgrade soils to 0 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

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

- These recommendations should be reviewed and revised, as necessary, for Building B after preliminary structural information becomes available.

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 truck court and in the dock-high areas of Building A. 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 of sands silty sands with occasional sandy silts. Based on their classifications, the sand and silty sand materials are expected to possess a friction angle of at least 30 degrees when compacted to at least 90 percent of the ASTM-1557 maximum dry density.

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

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type: On-Site Sands and Silty Sands
Internal Friction Angle (ϕ)		30°
Unit Weight		125 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	42 lbs/ft^3
	Active Condition (2h:1v backfill)	67 lbs/ft^3
	At-Rest Condition (level backfill)	63 lbs/ft^3

Regardless of the backfill type, the walls should be designed using a soil-footing coefficient of friction of 0.30 and an equivalent passive pressure of 300 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

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

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

Seismic Lateral Earth Pressures

In accordance with the 2019 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, extending to a depth of at least 3 feet below the proposed bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

On-site 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 primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface soils generally consist of sands, silty sands, and fine sandy silts. These soils are considered to possess favorable pavement support characteristics with estimated R-values of 40 to 50. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon an assumed R-value of 40. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading.

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 = 40)					
Materials	**Thickness (inches)**				
	Parking Stalls (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	3	4	5	7	8
Compacted Subgrade	12	12	12	12	12

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

Portland Cement Concrete

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

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 40)				
Materials	**Thickness (inches)**			
	Automobile Parking and Drive Areas (TI=4.0 & 5.0)	Truck Traffic		
		(TI =6.0)	(TI =7.0)	(TI =8.0)
PCC	5	5	5½	6½
Compacted Subgrade (95% minimum compaction)	12	12	12	12

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

7.0 GENERAL COMMENTS

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

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

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

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

8.0 REFERENCES

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

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

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

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

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

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

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

APPENDIX A

SOURCE: USGS TOPOGRAPHIC MAP OF THE EL MONTE QUADRANGLE, LOS ANGELES, CALIFORNIA, 2012.

SITE LOCATION MAP	
PROPOSED COMMERCIAL/INDUSTRIAL DEVELOPMENT	
PICO RIVERA, CALIFORNIA	
SCALE: 1" = 2000'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAH CHKD: DN	
SCG PROJECT 20G147-1	
PLATE 1	

GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

APPROXIMATE TRENCH LOCATION

NOTE: CONCEPTUAL SITE PLAN PREPARED BY MICHAEL BAKER INTERNATIONAL.

BORING AND TRENCH LOCATION PLAN

PROPOSED COMMERCIAL/INDUSTRIAL DEVELOPMENT

PICO RIVERA, CALIFORNIA

SCALE: 1" = 80'

DRAWN: JAH
CHKD: RGT

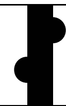
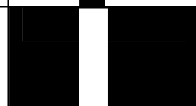
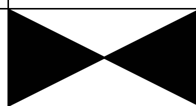
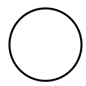
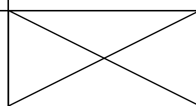
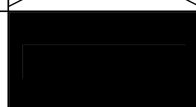
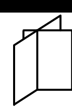
SCG PROJECT
20G147-1

PLATE 2

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

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

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

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

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

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

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

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

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

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

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS	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
SoCalGeo
A California Corporation

BORING NO. B-1

JOB NO.: 20G147-1
PROJECT: Commercial/Industrial Development
LOCATION: Pico Rivera, California

DRILLING DATE: 5/15/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 38 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
		17			FILL: Gray Brown fine Sand, trace medium to coarse Sand, medium dense-dry	98	2					
		23			ALLUVIUM: Gray Brown fine to coarse Sand, trace fine to coarse Gravel, medium dense to dense-dry to damp		2					Disturbed Sample
5		30				141	2					
		33				105	2					
10		48				117	3					
		57			Gray Brown Gravelly fine to coarse Sand, very dense-dry		2					
15												
		18			Light Brown fine Sandy Silt, trace Clay, some Iron oxide staining, medium dense-very moist		28			92		
20												
		20	4.0		Light Brown to Red Brown fine Sandy Clay, trace medium to coarse Sand, trace calcareous veining, trace Iron oxide staining, very stiff-moist to very moist		27 16			86 70		
25												
		16	4.0				22			79		
30												
		13	3.5		Light Brown Silty Clay, little calcareous nodules, little calcareous veining, trace Iron oxide staining, very stiff-very moist		29	46	21	95		

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 20G147-1
PROJECT: Commercial/Industrial Development
LOCATION: Pico Rivera, California

DRILLING DATE: 5/15/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 38 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Light Brown Silty Clay, little calcareous nodules, little calcareous veining, trace Iron oxide staining, very stiff-very moist							
					Light Brown Silty Clay, little fine Sand, little Iron oxide staining, stiff to very stiff-very moist							
	X	17	3.0				29			87		
40												
					Light Brown fine Sandy Silt, little Clay, trace Iron oxide staining, medium dense-very moist							
	X	22					24			84		
45												
					Light Brown Silt, trace fine Sand, little calcareous veining, medium dense-very moist							
	X	17					29			94		
50												
					Boring Terminated at 50'							

TBL 20G147-1.GPJ SOCALGEO.GDT 6/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 20G147-1
PROJECT: Commercial/Industrial Development
LOCATION: Pico Rivera, California

DRILLING DATE: 5/15/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 18 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: Brown Silty fine Sand, trace fine to coarse Gravel, loose to medium dense-damp to moist							
	X	9					11					
	X	12			@ 3½ feet, trace medium to coarse Sand		9					
5												
	X	16					7					
					Gray Brown Gravelly fine to coarse Sand, dense-dry to damp							
	X	48					1					
10												
	X	50					2					
15												
	X	59					3					
20												
					Brown to Red Brown fine Sandy, Clay, trace medium to coarse Sand, very stiff-very moist							
	X	18	4.0				19					
25												
					Boring Terminated at 25'							

TBL 20G147-1.GPJ SOCALGEO.GDT 6/4/20

JOB NO.: 20G147-1
PROJECT: Commercial/Industrial Development
LOCATION: Pico Rivera, California

DRILLING DATE: 5/16/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 14 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
					FILL: Brown fine Sand, trace Silt, trace fine root fibers, loose-damp to moist							
		6				103	9					
		13			@ 3 feet, trace glass fragments	101	7					
5		20			ALLUVIUM: Gray Brown Silty fine Sand, trace medium to coarse Sand, trace calcareous nodules, trace Iron oxide staining, medium dense-damp	113	16					
		44			Light Gray Gravelly fine to coarse Sand, medium dense to very dense-dry	121	2					
10		77			@ 9 feet, occasional cobbles	107	2					Disturbed Sample
		63					2					
15												
					Light Brown fine Sandy Clay, trace to little Iron oxide staining, stiff to very stiff, dense-very moist							
20		19	3.0				33					
					Boring Terminated at 20'							

TBL 20G147-1.GPJ SOCALGEO.GDT 6/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

BORING NO. B-4

JOB NO.: 20G147-1
PROJECT: Commercial/Industrial Development
LOCATION: Pico Rivera, California

DRILLING DATE: 5/16/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: 49.5 feet
CAVE DEPTH: 31 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	7			FILL: Brown Silty fine Sandy to fine Sandy Silt, trace medium to coarse Sand, trace fine Gravel, trace Clay, trace fine root fibers, trace calcareous veining, trace Iron oxide staining, loose-moist to very moist		14					
	X	12			ALLUVIUM: Brown fine to coarse Sand, some fine to coarse Gravel, medium dense-dry to damp		3					
5												
	X	30			Gray Brown Gravelly fine to coarse Sand, dense-dry to damp		3					
	X	38					3					
10												
					Light Brown Silty Clay with interbedded Silty fine Sand lenses, some Iron oxide staining, very stiff-very moist							
	X	21	3.5				19			89		
15												
	X	20	3.5				26			95		
20												
					Light Brown to Brown Silty Clay, some Iron oxide staining, micaceous, stiff to very stiff-very moist							
	X	18	3.0				25			98		
25												
	X	14	1.5				40			98		
30												
	X	22	1.5				27			90		

TBL 20G147-1.GPJ SOCALGEO.GDT 6/4/20

TEST BORING LOG

PLATE B-4a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 20G147-1
PROJECT: Commercial/Industrial Development
LOCATION: Pico Rivera, California
DRILLING DATE: 5/16/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward
WATER DEPTH: 49.5 feet
CAVE DEPTH: 31 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Light Brown to Brown Silty Clay, some Iron oxide staining, micaceous, stiff to very stiff-very moist							
					Light Brown to Brown Silty Clay, trace to little fine Sand, trace Iron oxide staining, very stiff-very moist to wet							
	X	21	4.0				26			84		
40												
	X	24	3.0				33			94		
45												
	X	14	3.0		@ 48½ to 50 feet, occasional thinly interbedded fine Sandy Silt lenses @ 49½ feet, Groundwater encountered during drilling		29			84		
50												

Boring Terminated at 50'

TBL 20G147-1.GPJ SOCALGEO.GDT 6/4/20

BORING NO.
B-5

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 20G147-1
PROJECT: Commercial/Industrial Development
LOCATION: Pico Rivera, California

DRILLING DATE: 5/16/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 17 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
					FILL: Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, mottled, loose-very moist							
		12				112	13					
		16			@ 3 feet, trace fine root fibers	112	15					
5		16				107	18					
		24			ALLUVIUM: Brown Silty fine Sand, trace medium to coarse Sand, trace fine to coarse Gravel, medium dense-moist	115	11					
		31			Brown fine to medium Sand, little coarse Sand, trace to little fine Gravel, trace Silt, micaceous, medium dense-damp	111	5					
10												
					Gray Brown Gravelly fine to coarse Sand, very dense-dry							
		58					2					
15												
		50/6"					1					
20												
					Light Brown fine Sandy Clay, some Iron oxide staining, very stiff-very moist							
		23	4.5				19					
25												
					Boring Terminated at 25'							

TBL 20G147-1.GPJ SOCALGEO.GDT 6/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 20G147-1
PROJECT: Commercial/Industrial Development
LOCATION: Pico Rivera, California
DRILLING DATE: 5/16/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward
WATER DEPTH: Dry
CAVE DEPTH: 16 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					FILL: Brown to Dark Brown Silty fine Sand, trace medium to coarse Sand, trace fine to coarse Gravel, trace glass fragments, medium dense-moist							
5	X	11					11					
					ALLUVIUM: Brown Silty fine Sand, trace calcareous veining, loose to medium dense-moist							
10	X	10					10					
					Gray Brown Gravelly fine to coarse Sand, dense-dry		2					
15	X	30	3.5		Light Brown fine Sandy Clay, some Iron oxide staining, very stiff to hard-very moist		18					
20	X	21	3.0				21					
					Boring Terminated at 20'							

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-7

JOB NO.: 20G147-1
PROJECT: Commercial/Industrial Development
LOCATION: Pico Rivera, California
DRILLING DATE: 5/16/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward
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
					POSSIBLE FILL: Gray Brown fine Sand, little to some medium to coarse Sand, little fine to coarse Gravel, medium dense-dry to damp							
	X	18					3					
5												
					ALLUVIUM: Gray Brown Gravelly fine to coarse Sand, very dense-dry to damp							
	X	57					3					
10												
	X	66					3					
15												
	X	51			@ 18½ feet, some Iron oxide staining		4					
20												
					Boring Terminated at 20'							

TBL 20G147-1.GPJ SOCALGEO.GDT 6/4/20

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 20G147-1
PROJECT: Commercial/Industrial Development
LOCATION: Pico Rivera, California

DRILLING DATE: 5/15/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
		8			FILL: Brown Silty fine Sand, trace medium to coarse Sand, trace fine root fibers, loose-moist to very moist	106	11					
		8				101	15					
5		10			FILL: Dark Gray Silty fine Sand, trace medium to coarse Sand, trace Asphaltic concrete fragments, loose-moist	104	10					
		9			ALLUVIUM: Gray Brown fine to coarse Sand, little to some fine to coarse Gravel, loose to medium dense-dry	110	2					
10		28			Brown to Gray Brown fine Sandy Silt, trace coarse Gravel, micaceous, medium dense-damp	114	2 5					
		41			Gray Brown Gravelly fine to coarse Sand, 2 inch lens of fine Sandy Silt, dense-dry to damp		3					
15												
		16	3.5		Light Brown Silty Clay, trace fine to medium Sand, little to some Iron oxide staining, very stiff-very moist		19					
20												
		21			Light Brown fine Sandy Silt, trace to little Clay, little to some calcareous veining, medium dense-very moist		26					
25					Boring Terminated at 25'							

TBL 20G147-1.GPJ SOCALGEO.GDT 6/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-1

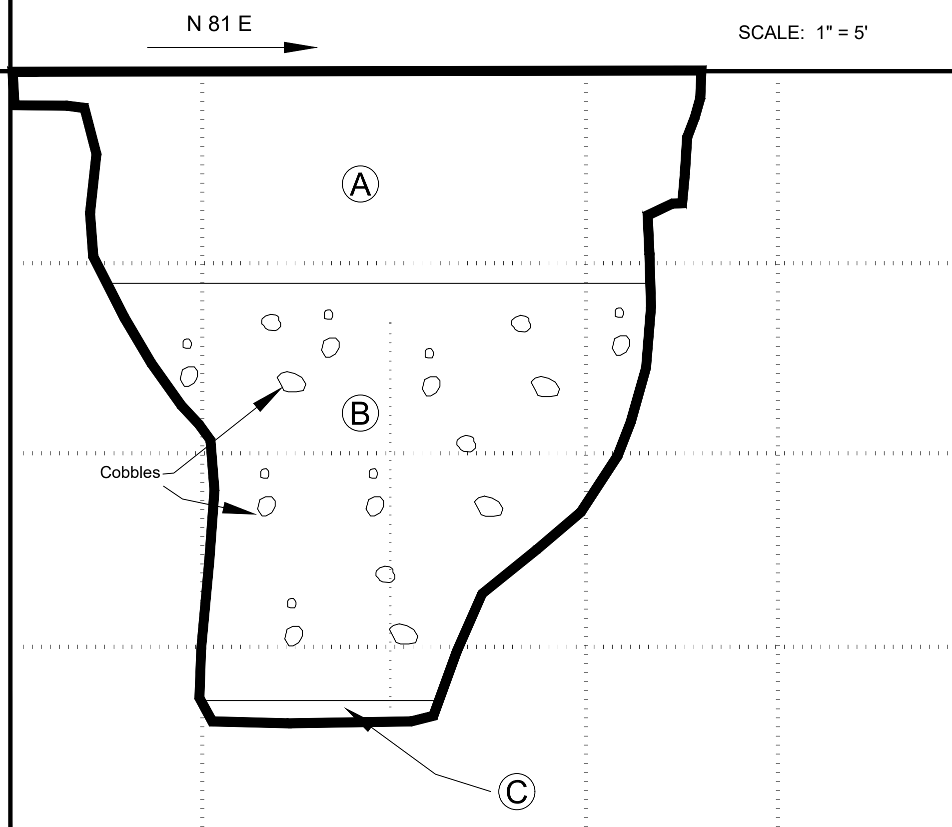
JOB NO.: 20G147-1
PROJECT: Prop Commercial/Industrial Development
LOCATION: Pico Riveria, CA
DATE: 5-14-2020

EQUIPMENT USED: Backhoe
LOGGED BY: Jamie Hayward
ORIENTATION: N 81 E

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	ORGANIC CONTENT (%)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b	2.7	6	A: FILL: Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, little to some Fine root fibers, mottled, loose to medium dense-damp to moist
	b	2.5	8	
	b	2.5	10	
	b	2.8	10	
	b		8	
5	b		8	
	b		1	B: ALLUVIUM: Gray Brown Gravelley fine to coarse Sand, some Cobbles, medium dense to dense-dry to damp
10				
	b		2	
15				C: Light Brown interbedded lenses of fine Sandy Clay and fine Sandy Silt, little Iron oxide staining, stiff to medium dense-very moist
	b		28	
				Trench Terminated @ 17 feet

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-9

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-2

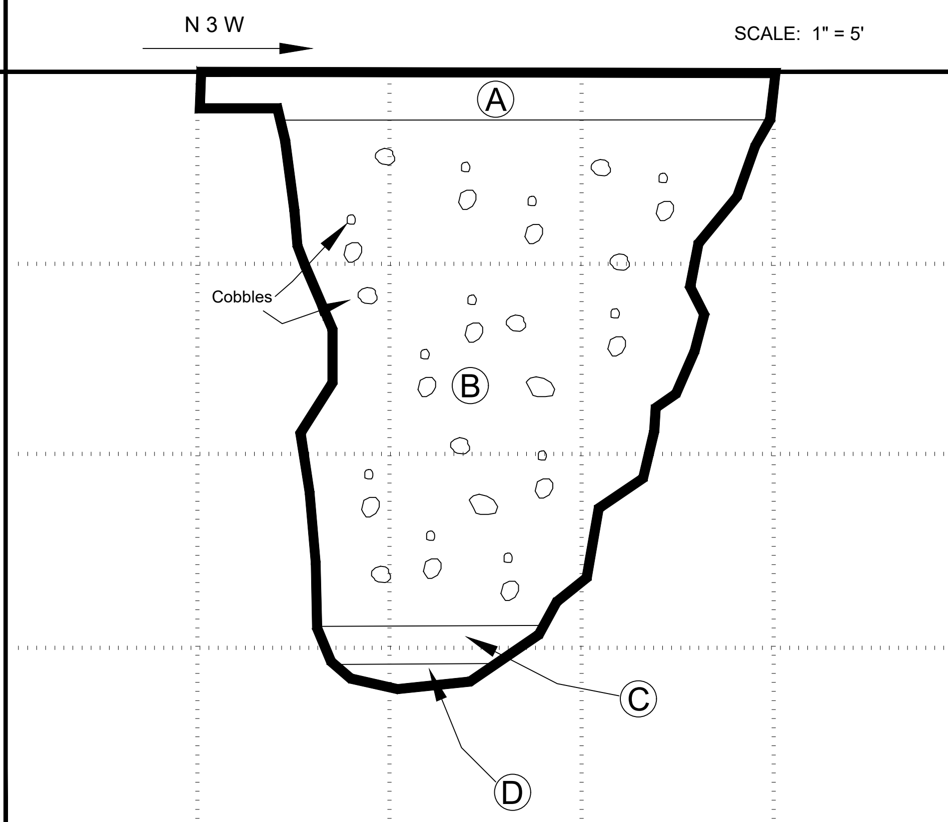
JOB NO.: 20G147-1
PROJECT: Prop Commercial/Industrial Development
LOCATION: Pico Rivera, CA
DATE: 5-14-2020

EQUIPMENT USED: Backhoe
LOGGED BY: Jamie Hayward
ORIENTATION: N 3 W

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	ORGANIC CONTENT (%)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		4	A: FILL: Brown fine Sand, trace medium to coarse Sand, trace to little Silt, little Fine root fibers, medium dense-damp
	b		2	B: ALLUVIUM: Gray Brown Gravelly fine to coarse Sand, some Cobbles, loose to medium dense-dry to damp
5				
	b		2	
10	b		2	
	b		2	
				C: Light Brown fine Sandy Silt to Silty fine Sand, little Iron oxide staining, medium dense-very moist
15	b		23	D: Light Brown Silty fine to medium Sand, trace coarse Sand, little fine Gravel, some Iron oxide staining, medium dense-moist
	b		11	
				Trench Terminated @ 16 feet

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-10

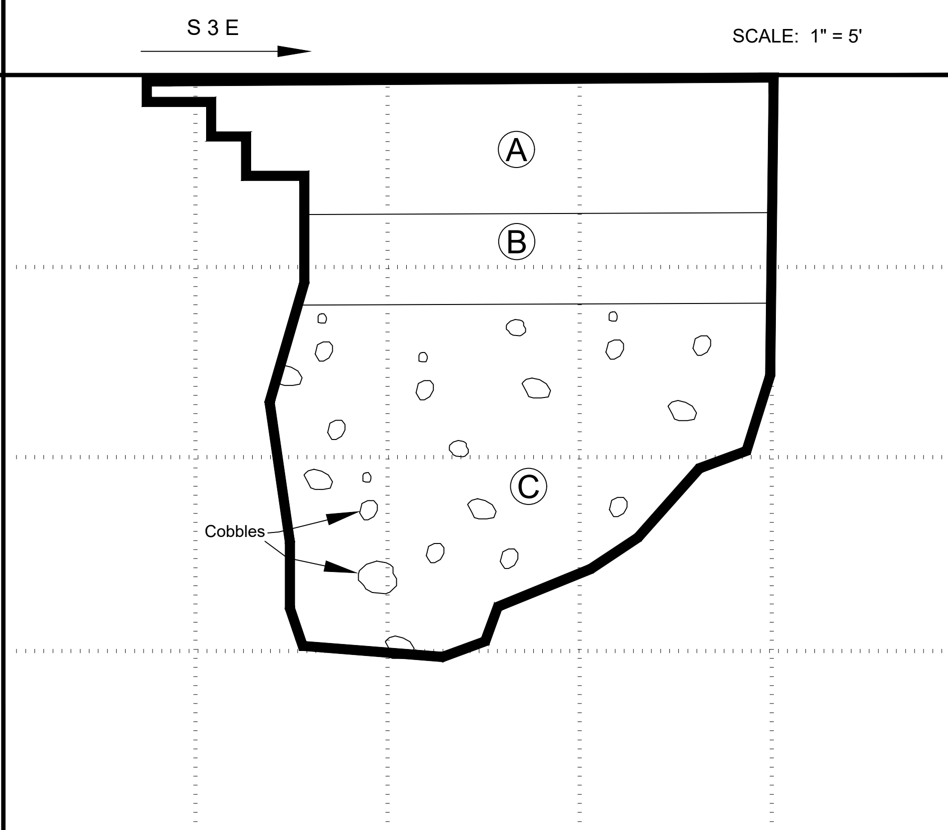
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-3

JOB NO.: 20G147-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Prop Commercial/Industrial Development	LOGGED BY: Jamie Hayward	SEEPAGE DEPTH: Dry
LOCATION: Pico Riveria, CA	ORIENTATION: S 3 E	READINGS TAKEN: At Completion
DATE: 5-14-2020		

DEPTH	SAMPLE	ORGANIC CONTENT (%)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		8	A: FILL: Brown fine Sand, trace medium to coarse Sand, trace fine Gravel, little Fine root fibers, micaceous, porous, slightly mottled, loose to medium dense-damp to moist
	b		7	B: ALLUVIUM: Gray Brown fine Sand, trace medium to coarse Sand, micaceous, loose-moist
	b		8	
	b		6	
5	b		4	@ 5 to 5.5 feet, intermixed with Silty fine Sand
	b		3	C: ALLUVIUM: Gray Brown Gravelly fine to coarse Sand, some Cobbles, medium dense-damp
10	b		4	
				@ 14 feet, some Iron oxide staining
15	b		2	Trench Terminated @ 15 feet

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-4

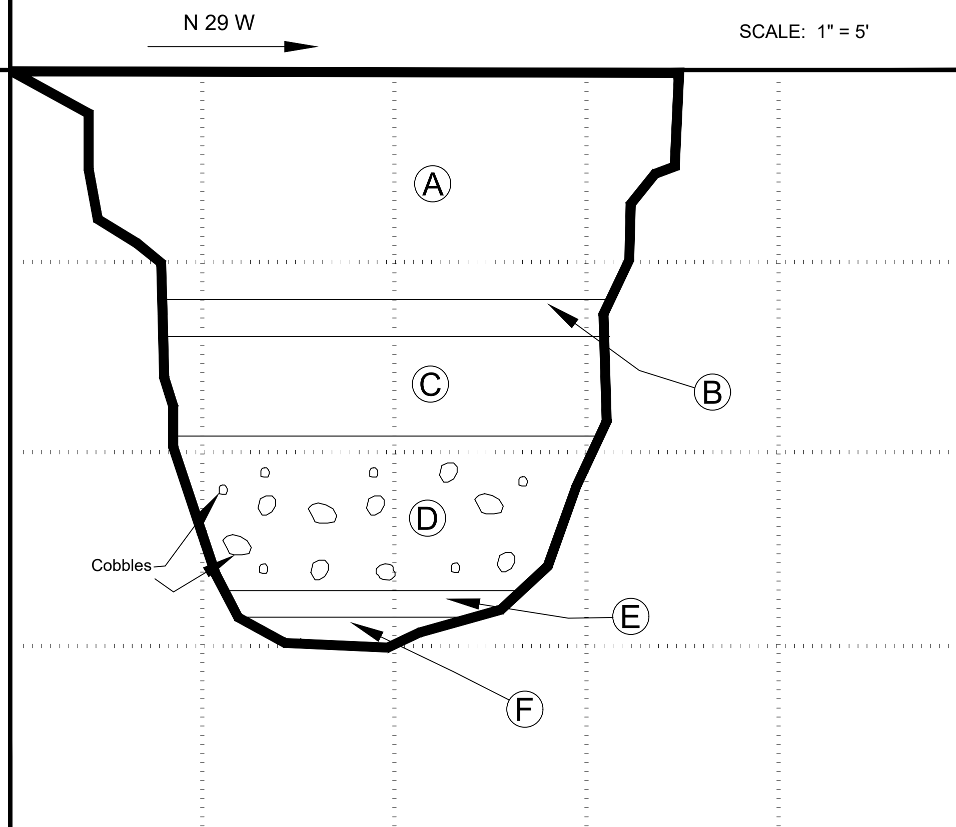
JOB NO.: 20G147-1
PROJECT: Prop Commercial/Industrial Development
LOCATION: Pico Rivera, CA
DATE: 5-14-2020

EQUIPMENT USED: Backhoe
LOGGED BY: Jamie Hayward
ORIENTATION: N 29 W

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	ORGANIC CONTENT (%)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b	3.6	11	A: FILL: Brown Silty fine Sand to fine Sandy Silt, little fine root fibers, porous, loose to medium dense-moist to very moist
	b	2.1	13	
	b	2.3	15	
	b	2.2	15	
	b		12	
5				
	b		6	B: ALLUVIUM: Brown fine Sand, medium dense-damp C: ALLUVIUM: Gray Brown fine to coarse Sand, occasional Cobbles, dense-damp
				D: ALLUVIUM: Gray Brown Gravelly fine to coarse Sand, some Cobbles, dense-dry
10	b		1	
				E: Gray to Dark Gray Silty fine Sand, trace Fine root fibers, trace calcareous staining/veining, porous, dense-very moist
	b		19	F: Gray Brown Gravelly fine to coarse Sand, occasional Cobbles, dense-dry to damp
	b		2	
15				Trench Terminated @ 15 feet

GRAPHIC REPRESENTATION



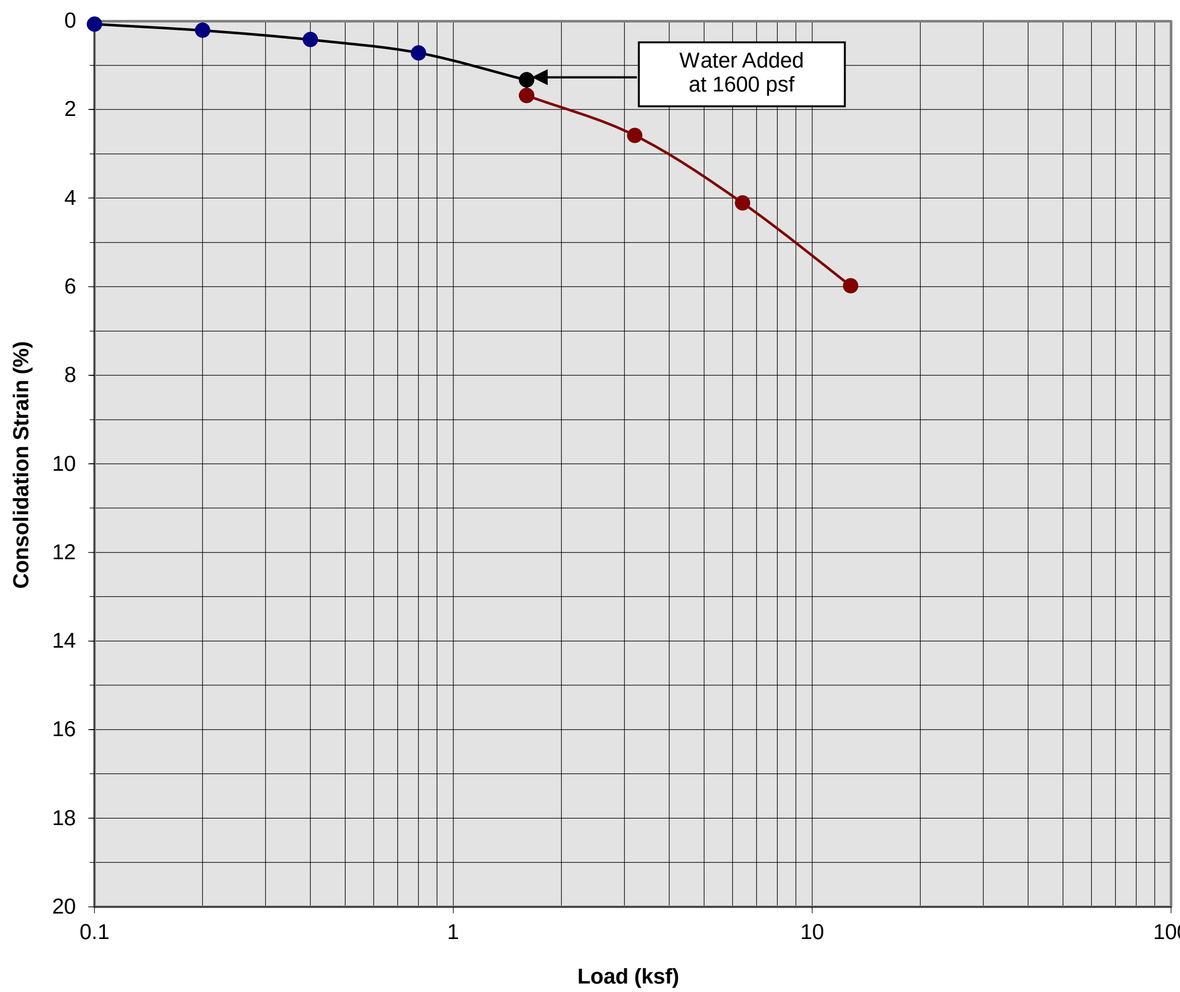
KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-12

APPENDIX C

Consolidation/Collapse Test Results

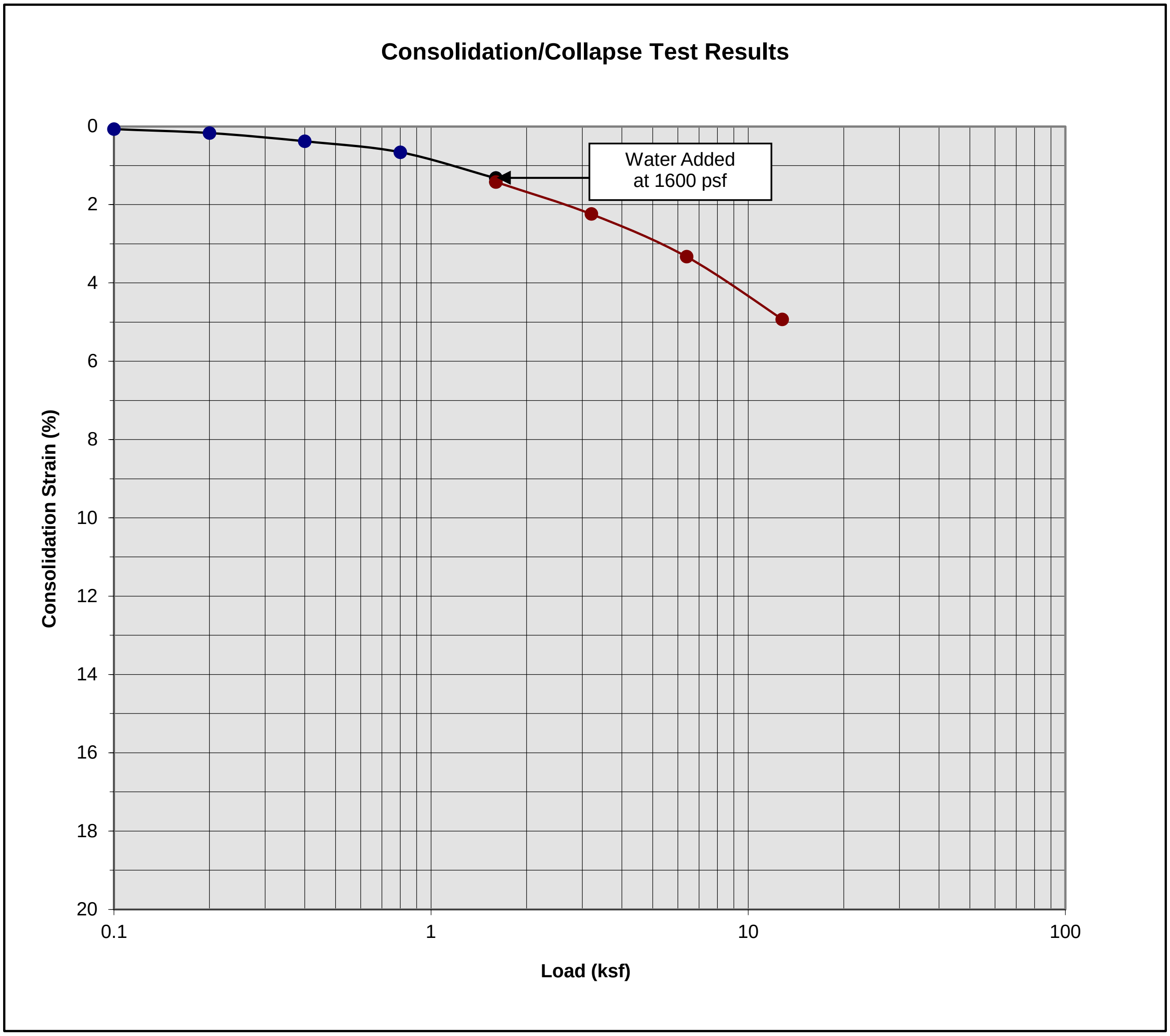


Classification: FILL: Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel

Boring Number:	B-5	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	3 to 4	Initial Dry Density (pcf)	111.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	118.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.35

Proposed Commercial/Industrial Building
Pico Riveria, California
Project No. 20G147-1
PLATE C- 1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



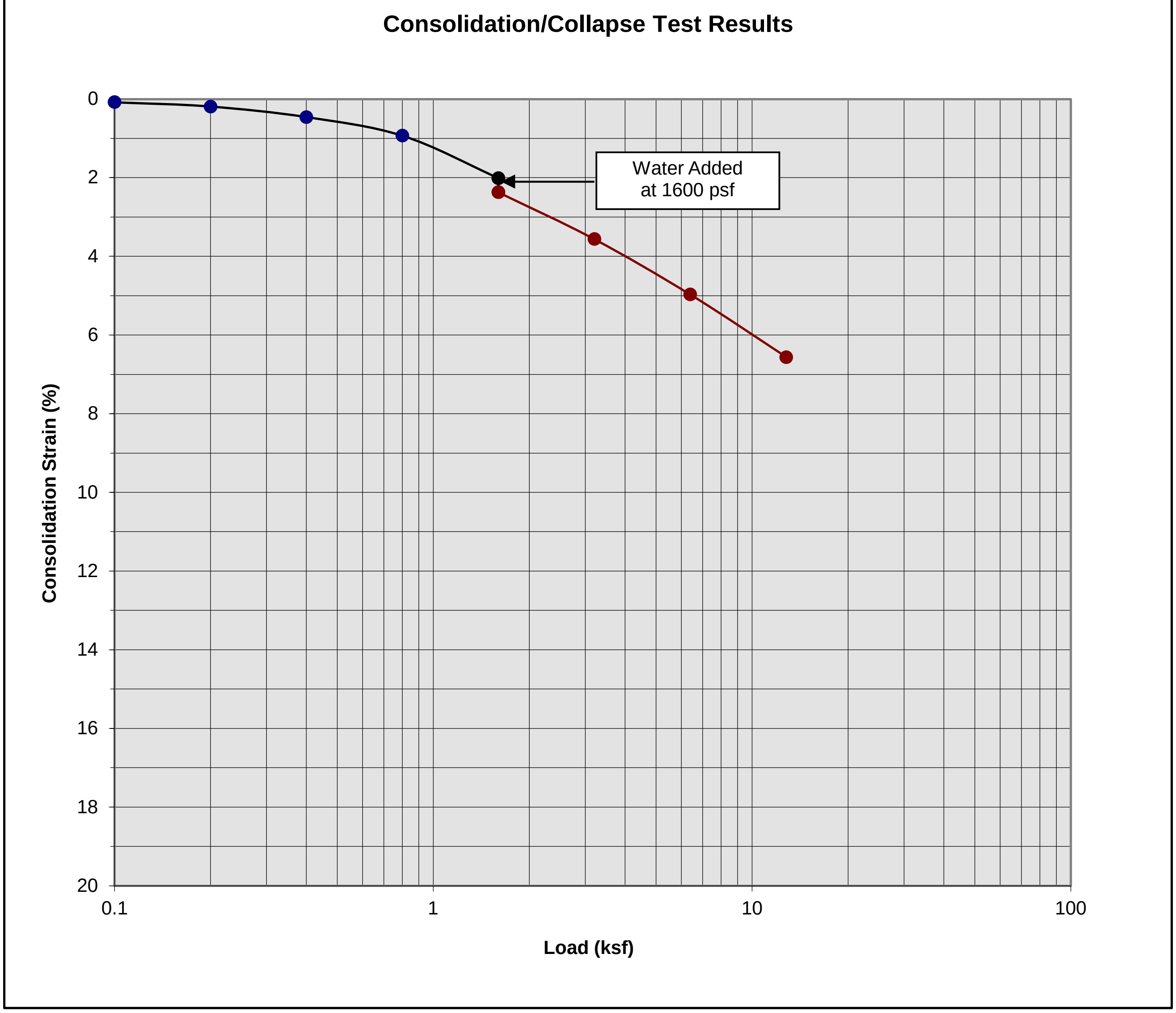
Classification: FILL: Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel

Boring Number:	B-5	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	5 to 6	Initial Dry Density (pcf)	108.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.10

Proposed Commercial/Industrial Building
Pico Riveria, California
Project No. 20G147-1

PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

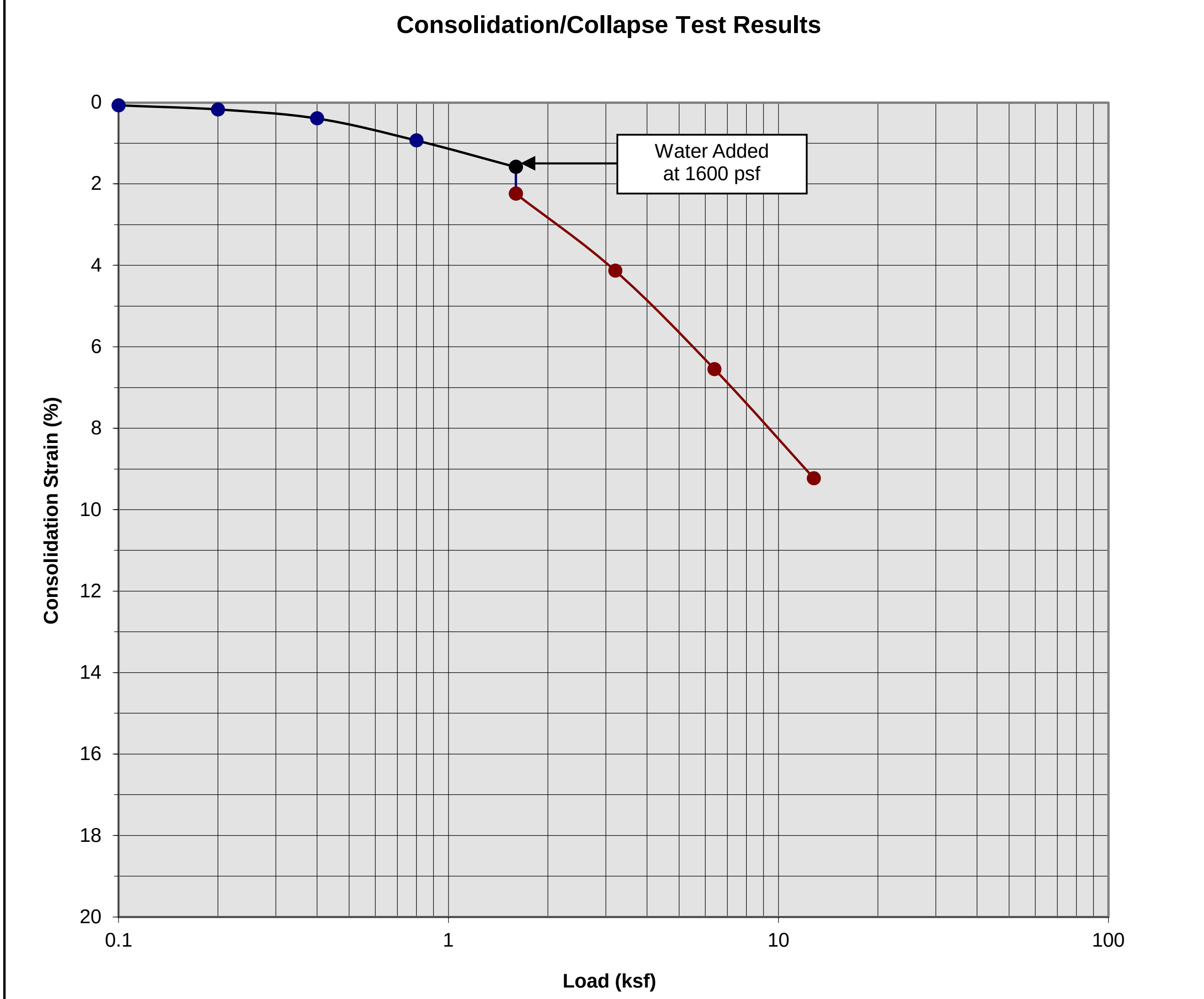


Classification: Brown Silty fine Sand, trace medium to coarse Sand, trace fine to coarse Gravel

Boring Number:	B-5	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	7 to 8	Initial Dry Density (pcf)	114.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	123.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.36

Proposed Commercial/Industrial Building
Pico Riveria, California
Project No. 20G147-1
PLATE C- 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

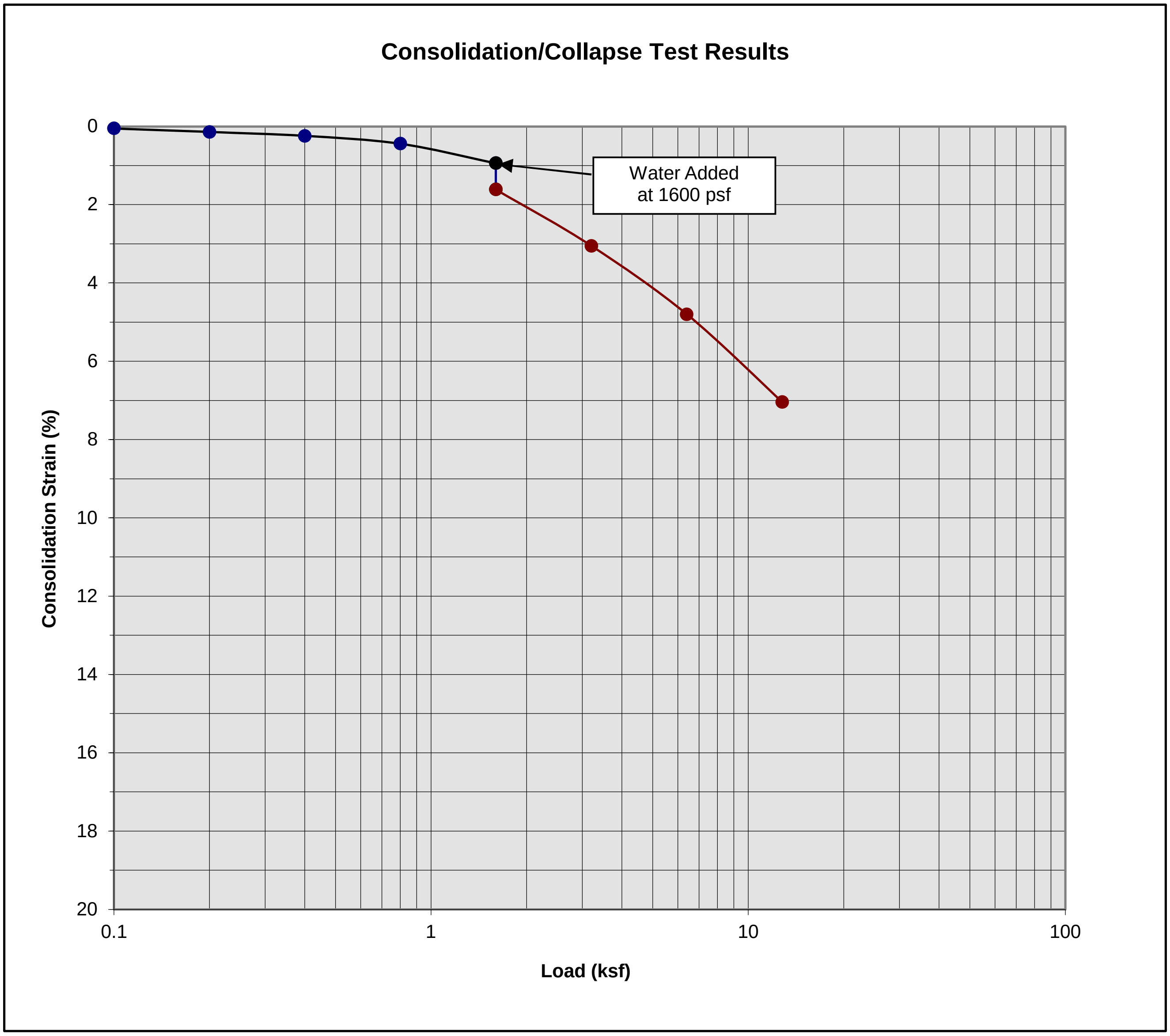


Classification: FILL: Brown Silty fine Sand, trace medium to coarse Sand

Boring Number:	B-8	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	3 to 4	Initial Dry Density (pcf)	101.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	110.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.66

Proposed Commercial/Industrial Building
Pico Riveria, California
Project No. 20G147-1
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

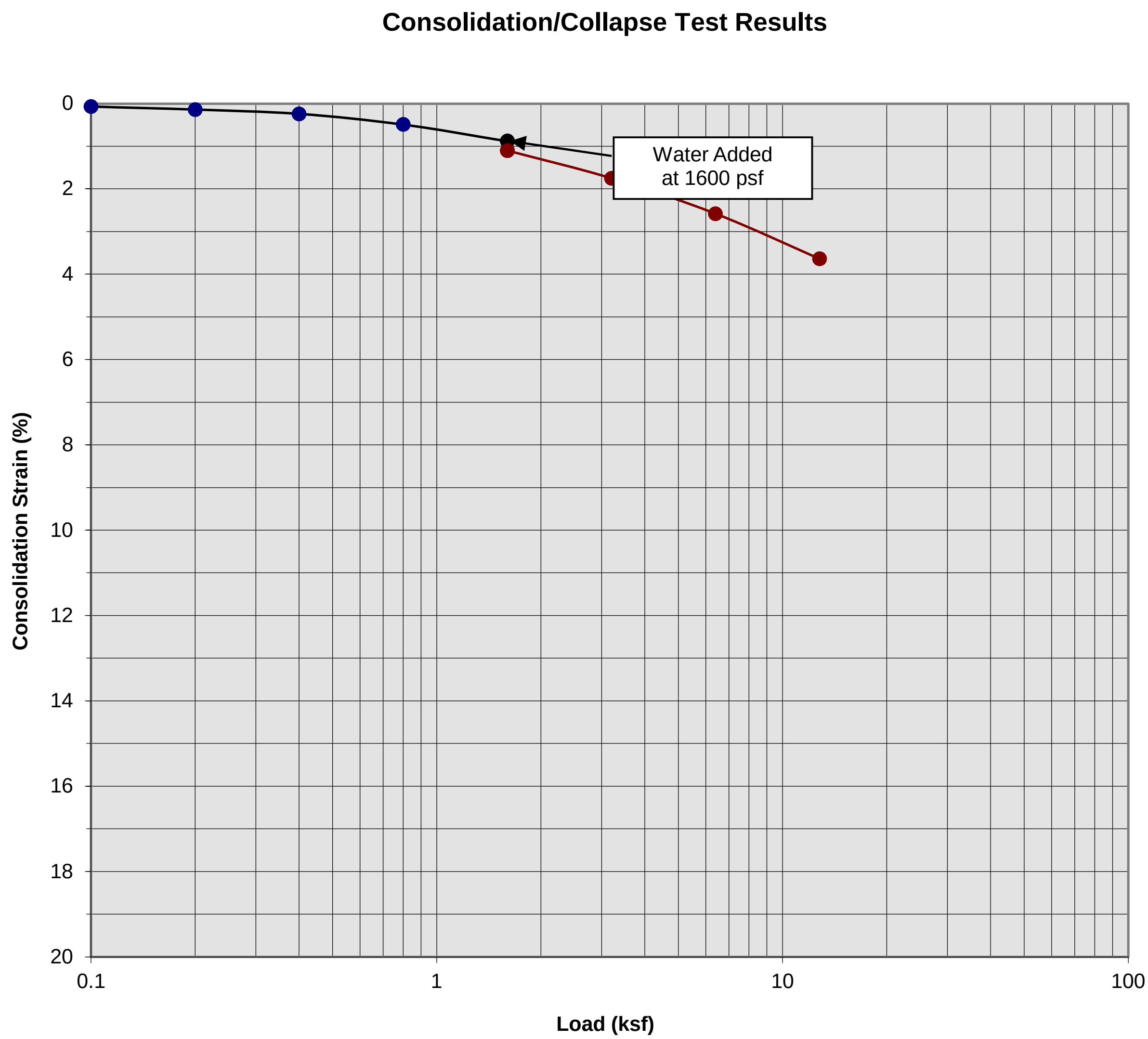


Classification: FILL: Dark Gray Silty fine Sand, trace medium to coarse Sand

Boring Number:	B-8	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	5 to 6	Initial Dry Density (pcf)	103.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.67

Proposed Commercial/Industrial Building
Pico Riveria, California
Project No. 20G147-1
PLATE C- 5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

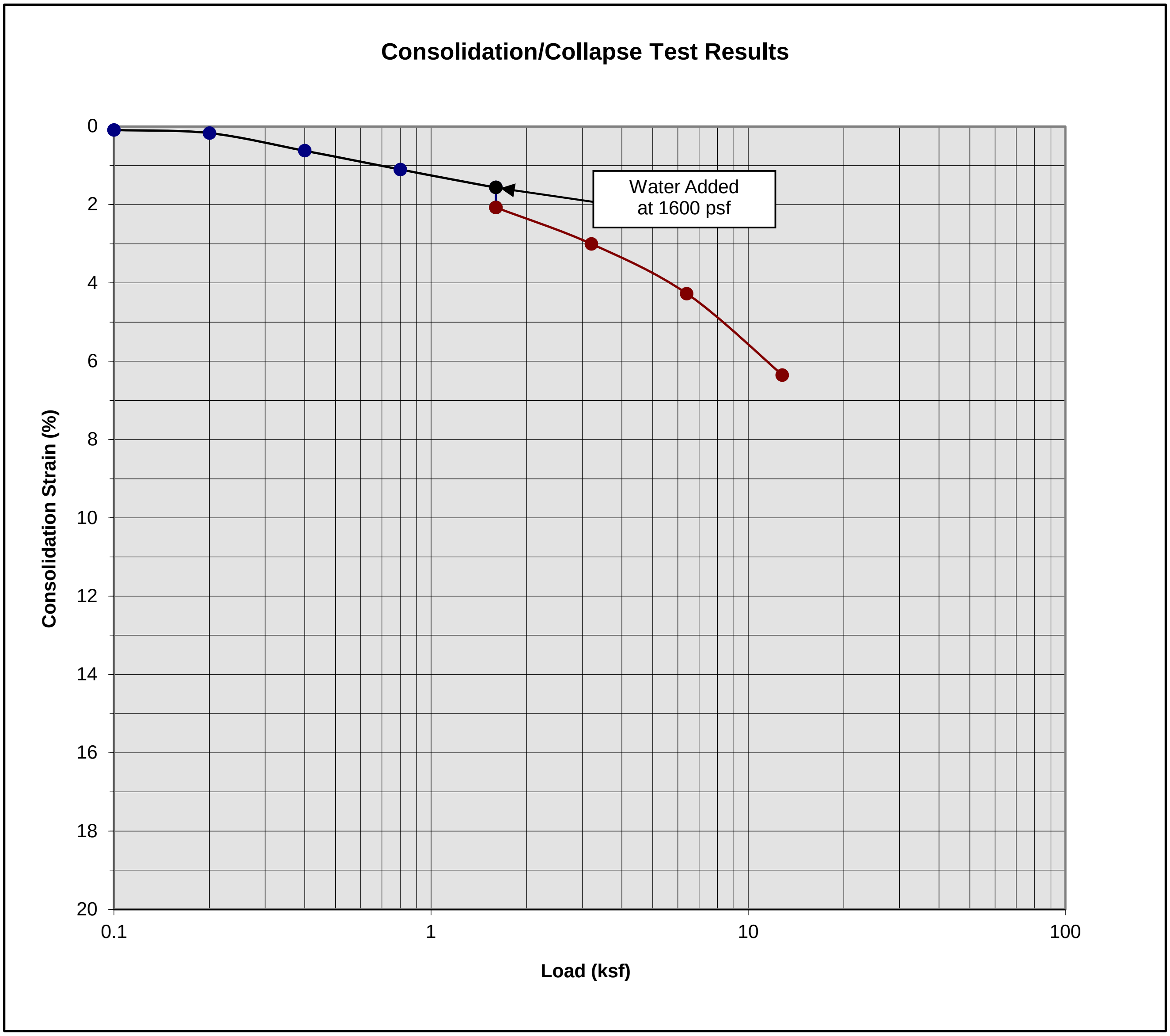


Classification: Gray Brown fine to coarse Sand, little to some fine to coarse Gravel

Boring Number:	B-8	Initial Moisture Content (%)	1
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	7 to 8	Initial Dry Density (pcf)	109.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	113.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.22

Proposed Commercial/Industrial Building
Pico Riveria, California
Project No. 20G147-1
PLATE C- 6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

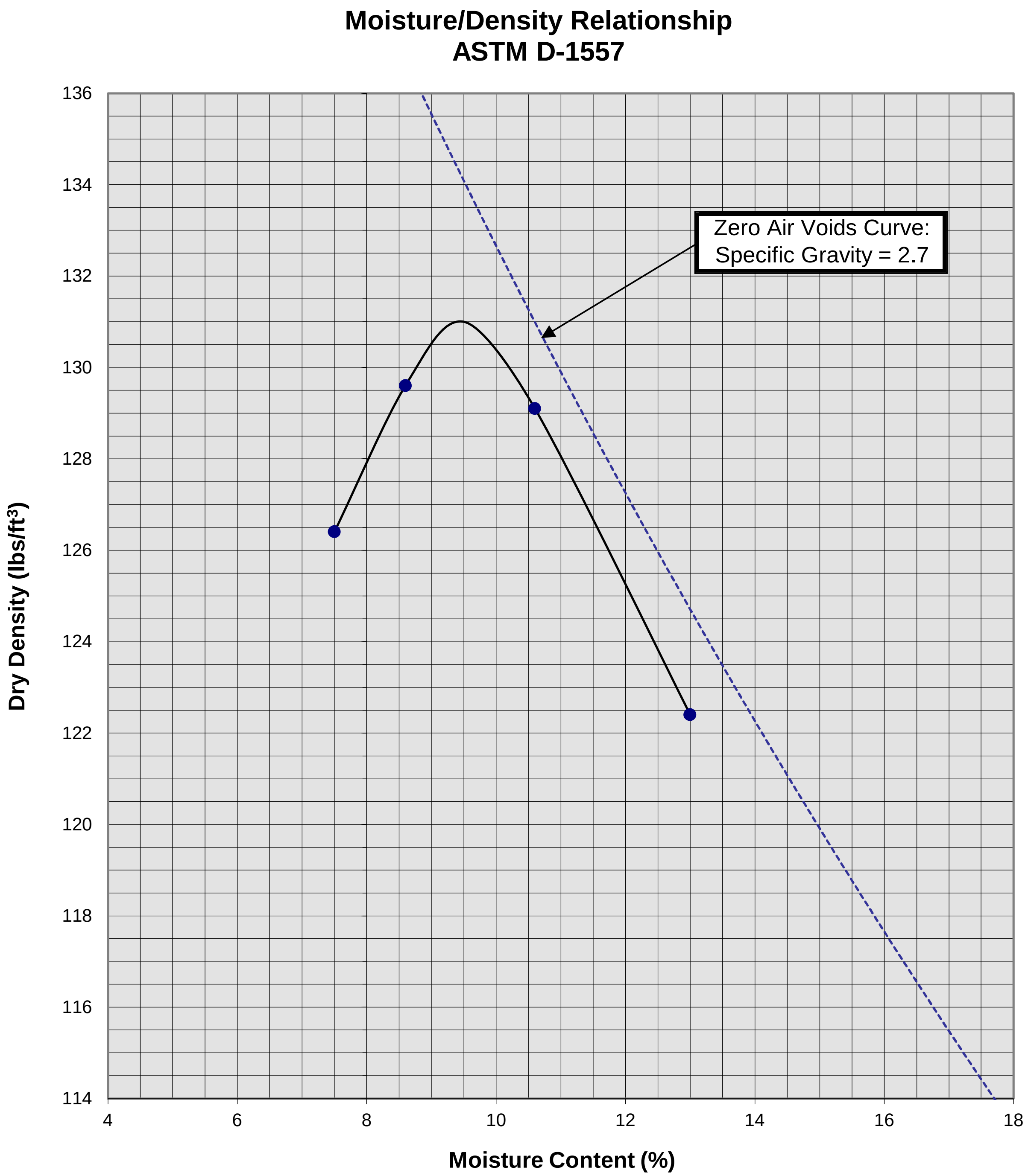


Classification: Gray Brown to Brown fine Sandy Silt, trace coarse Gravel

Boring Number:	B-8	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	9 to 10	Initial Dry Density (pcf)	114.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	121.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.51

Proposed Commercial/Industrial Building
Pico Riveria, California
Project No. 20G147-1
PLATE C- 7

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number		B-8 @ 0-5'
Optimum Moisture (%)		9.5
Maximum Dry Density (pcf)		131
Soil Classification	Brown Silty fine Sand, trace medium to coarse Sand	

Proposed Commercial/Industrial Building
Pico Riveria, California
Project No. 20G147-1
PLATE C-8

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.

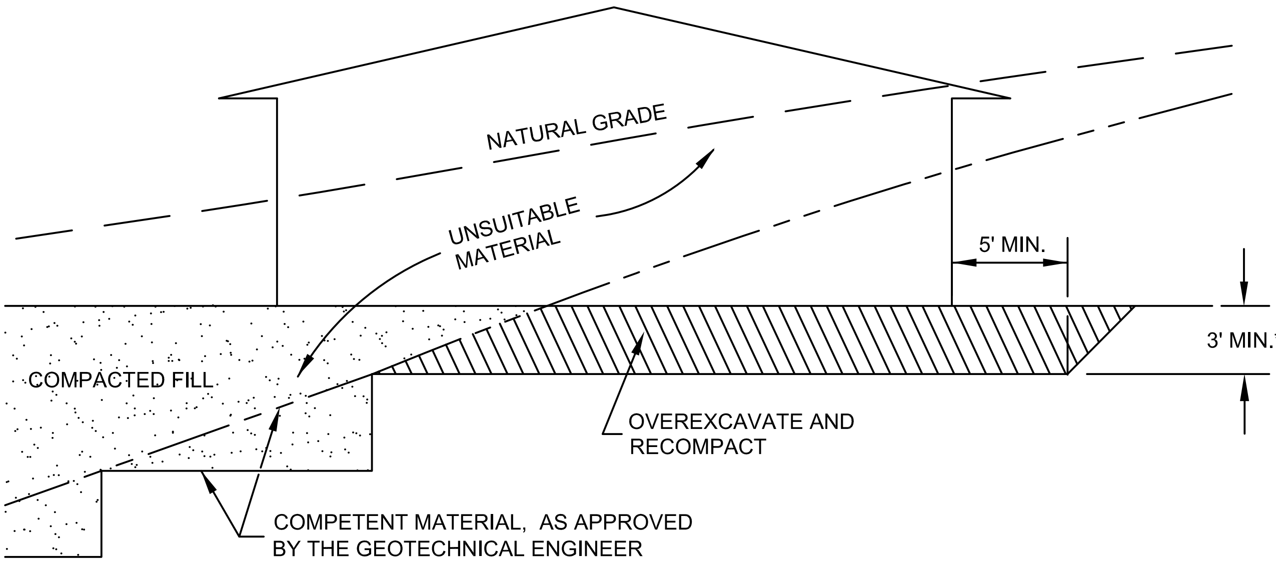
Subdrains

- 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.

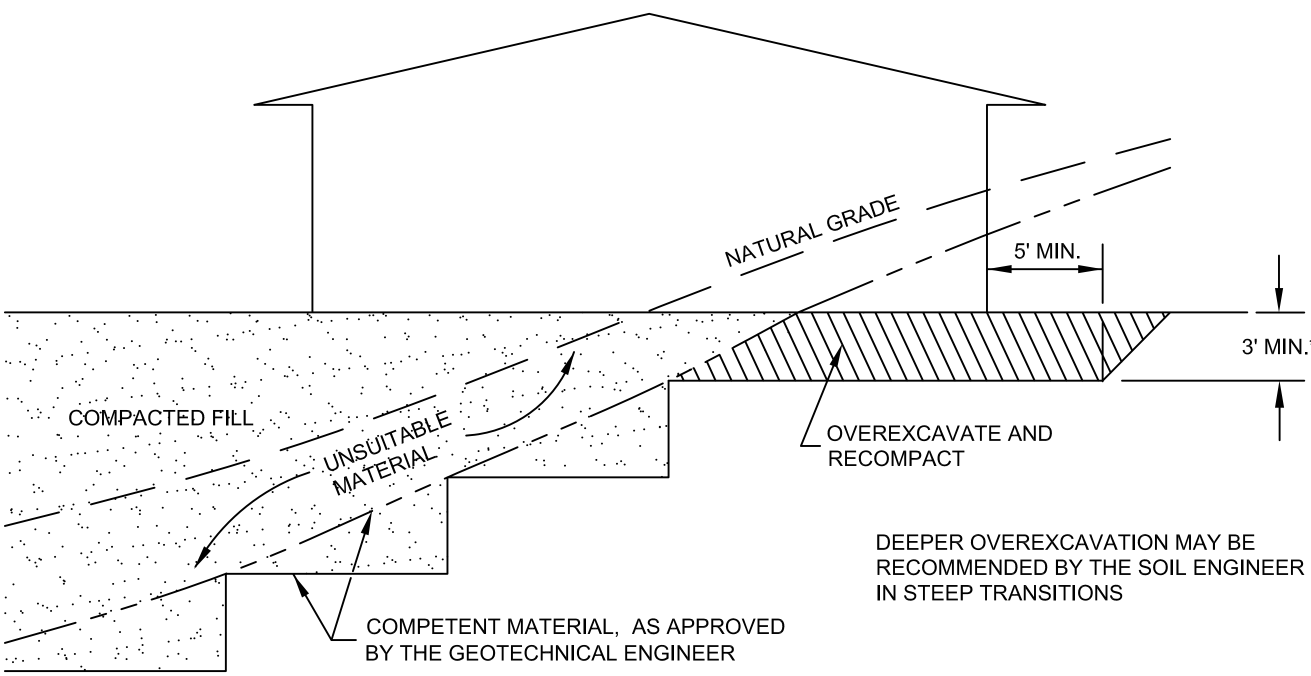
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- 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)

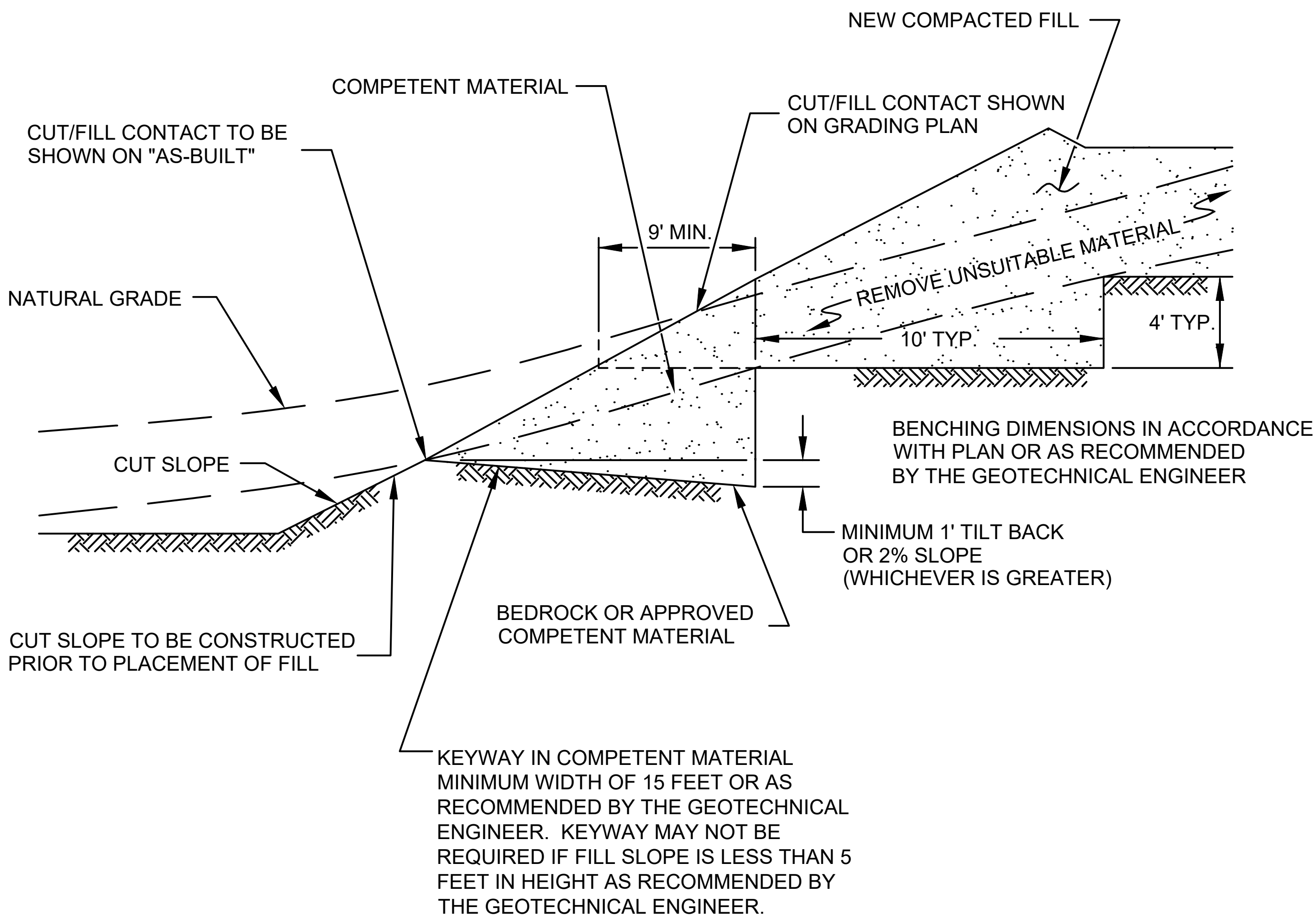


*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION. ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

TRANSITION LOT DETAIL

GRADING GUIDE SPECIFICATIONS

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



FILL ABOVE CUT SLOPE DETAIL

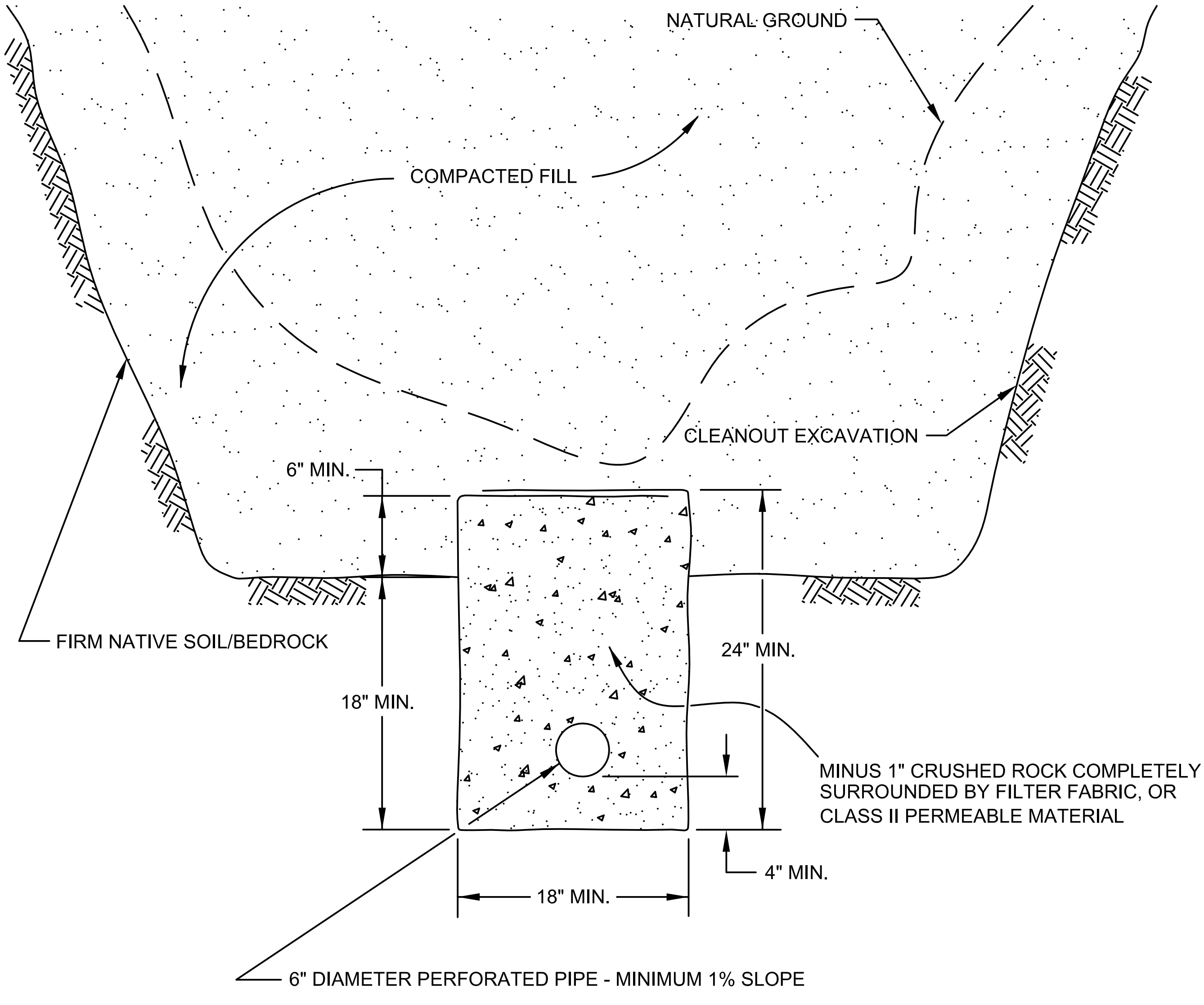
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

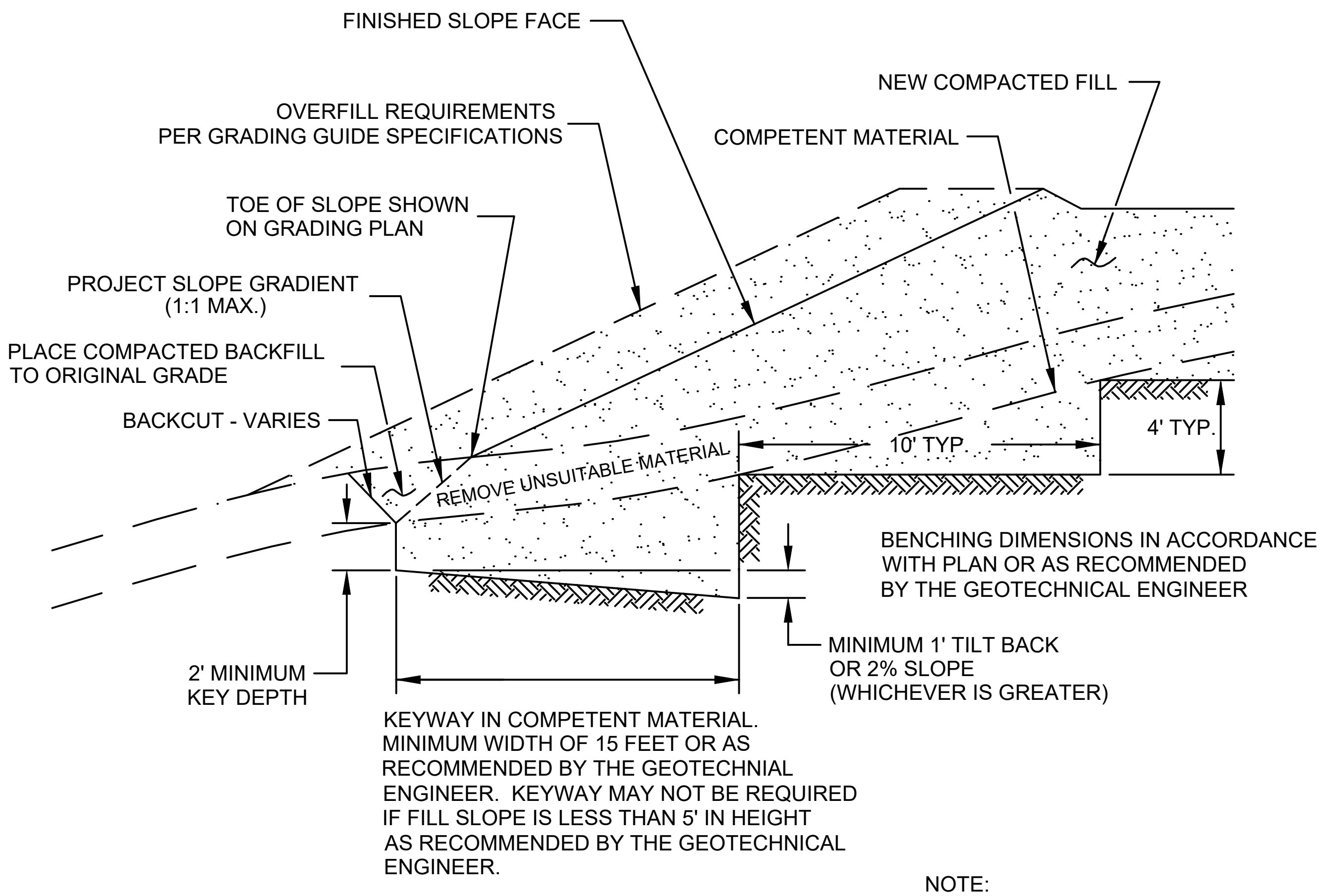
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

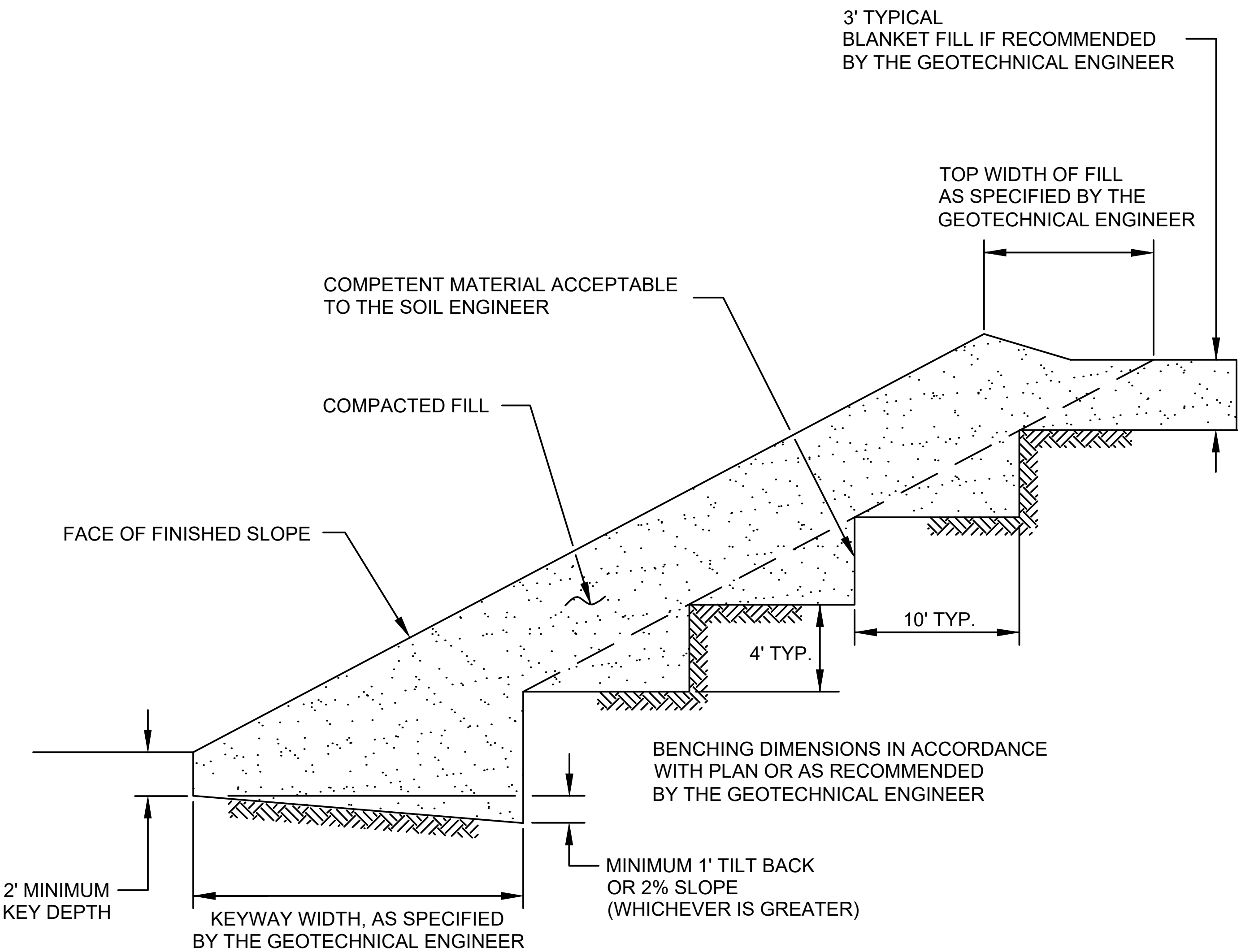
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

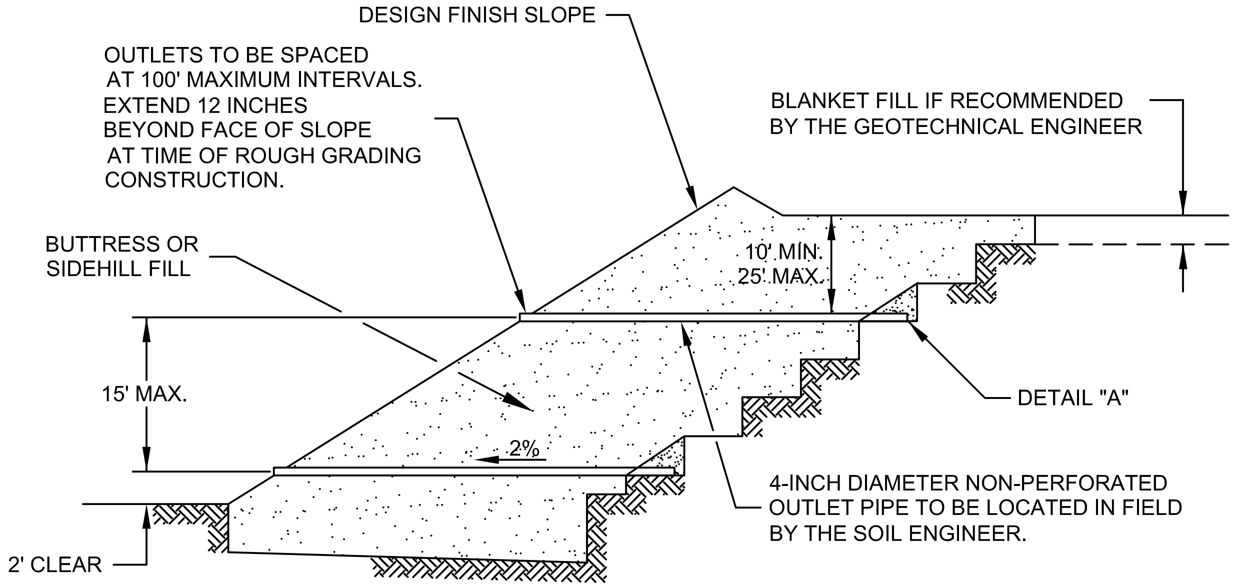
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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

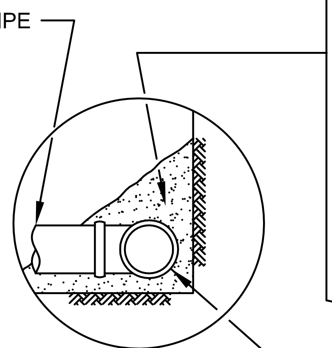
SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW



FILTER MATERIAL - MINIMUM OF FIVE CUBIC FEET PER FOOT OF PIPE. SEE ABOVE FOR FILTER MATERIAL SPECIFICATION.

ALTERNATIVE: IN LIEU OF FILTER MATERIAL FIVE CUBIC FEET OF GRAVEL PER FOOT OF PIPE MAY BE ENCASED IN FILTER FABRIC. SEE ABOVE FOR GRAVEL SPECIFICATION.

FILTER FABRIC SHALL BE MIRAFI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.

MINIMUM 4-INCH DIAMETER PVC SCH 40 OR ABS CLASS SDR 35 WITH A CRUSHING STRENGTH OF AT LEAST 1,000 POUNDS, WITH A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

DETAIL "A"

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

SLOPE FILL SUBDRAINS

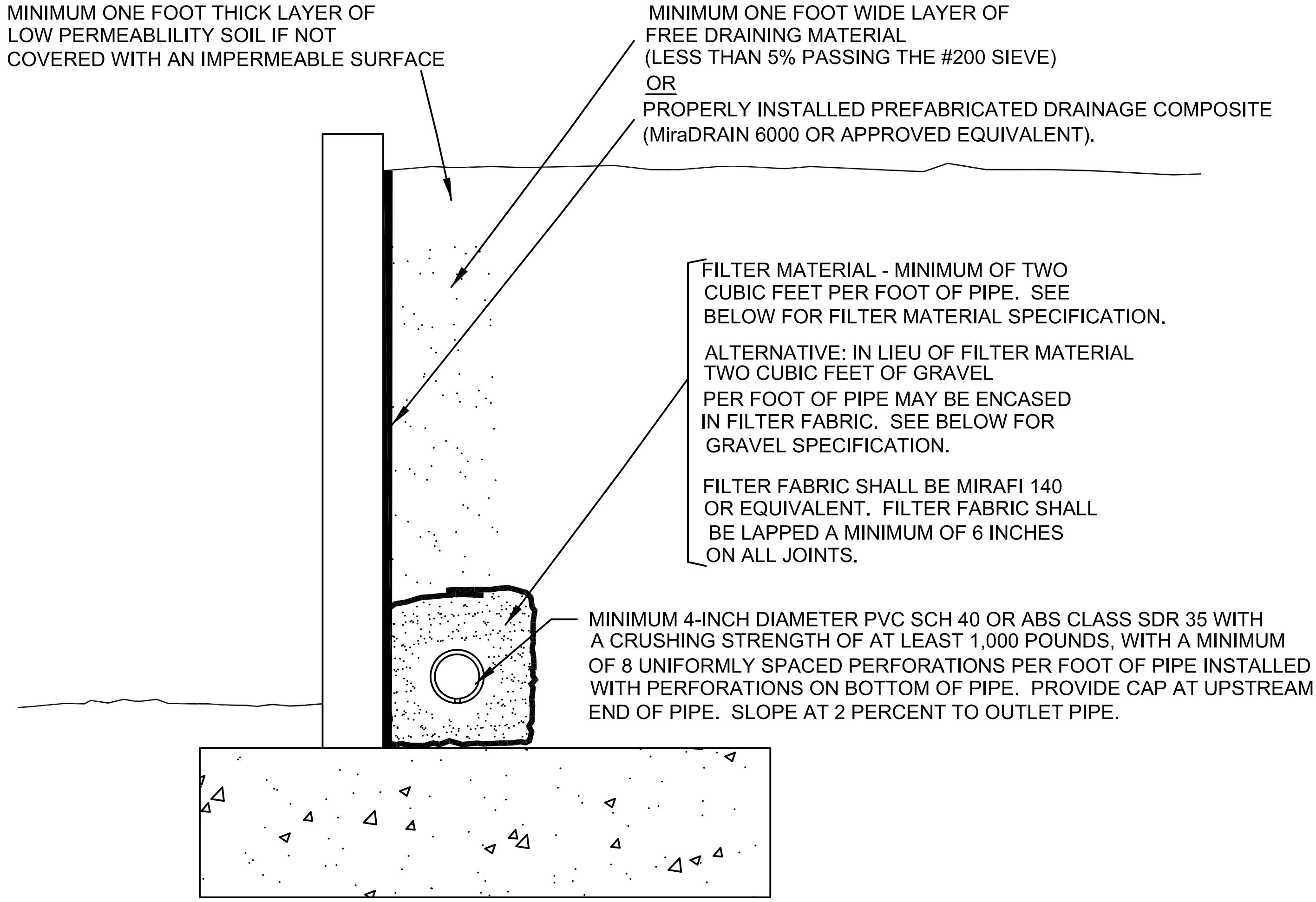
GRADING GUIDE SPECIFICATIONS

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

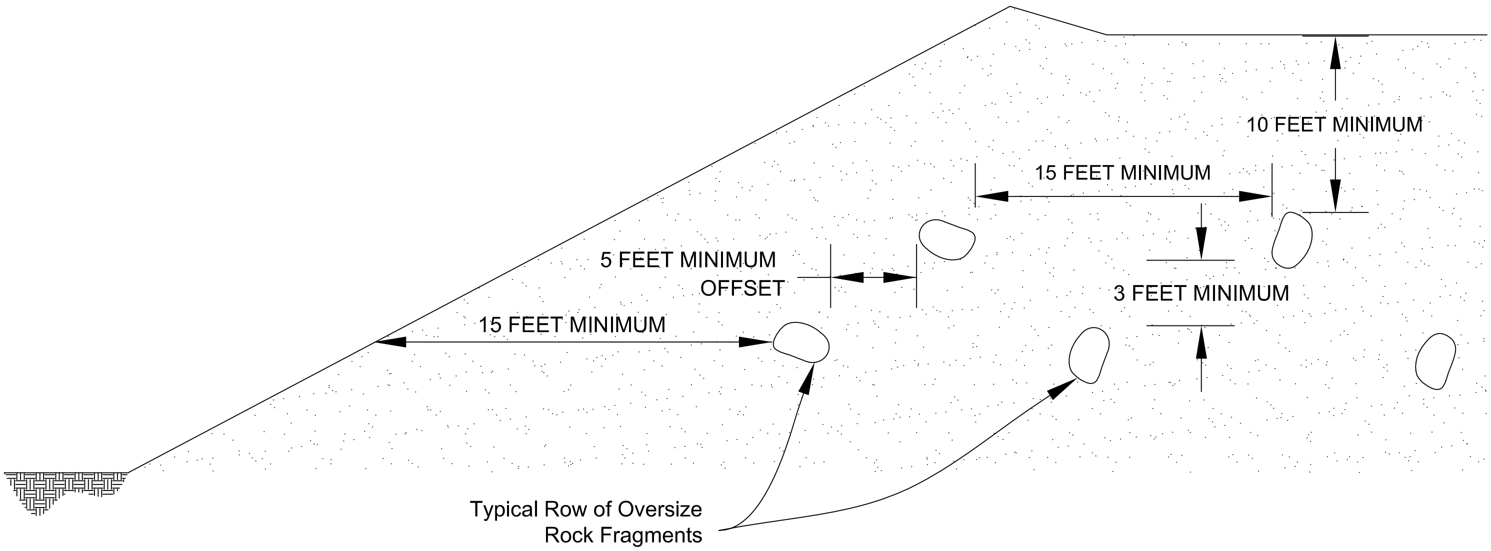
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

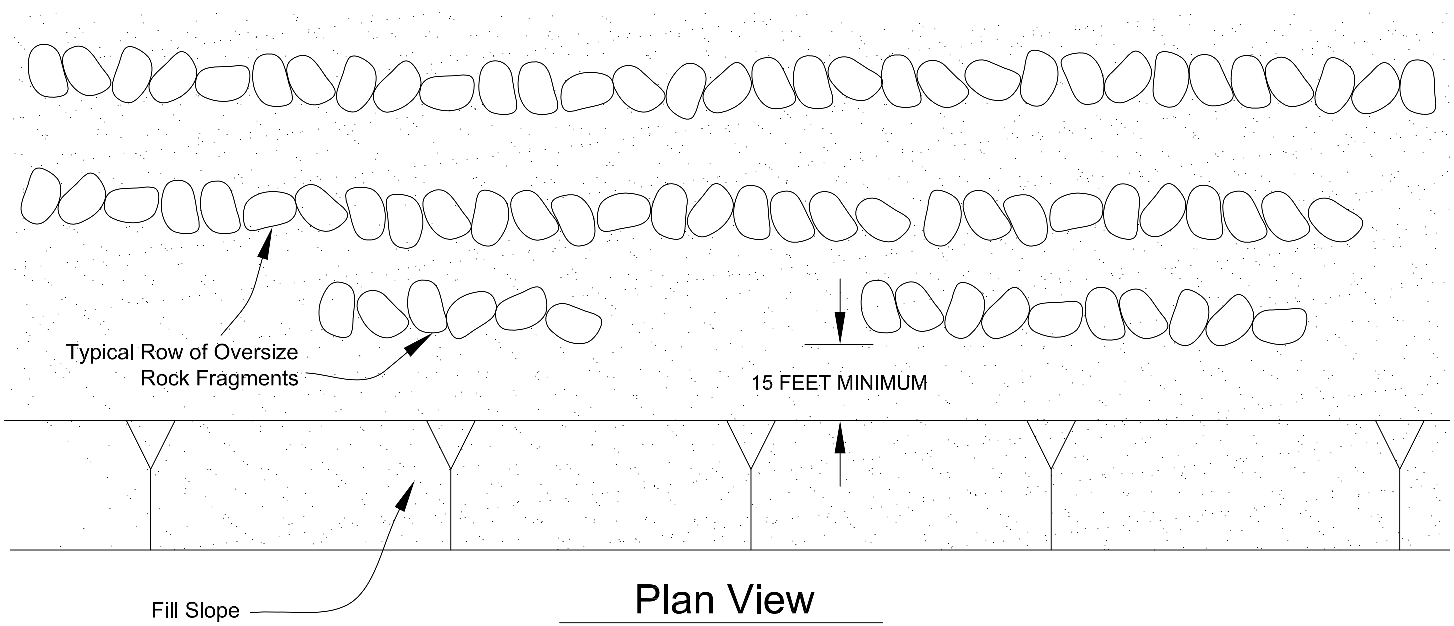
DRAWN: JAS
CHKD: GKM

PLATE D-7

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

OSHPD

Latitude, Longitude: 34.002530, -118.067242

Date	5/21/2020, 10:44:22 AM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.9	MCE_R ground motion. (for 0.2 second period)
S_1	0.679	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.9	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.267	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	0.822	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.905	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.9	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.122	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.337	Factored deterministic acceleration value. (0.2 second)
S1RT	0.679	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.756	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.917	Factored deterministic acceleration value. (1.0 second)
PGAd	0.971	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.895	Mapped value of the risk coefficient at short periods
C_{R1}	0.897	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 COMMERCIAL/INDUSTRIAL DEVELOPMENT

PICO RIVERIA, CALIFORNIA

DRAWN: JAH
CHKD: DN

SCG PROJECT
20G147-1

PLATE E-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Proposed Comm/Ind Development	MCE_G Design Acceleration	0.905 (g)
Project Location	Pico Rivera	Design Magnitude	6.83
Project Number	20G147-1	Historic High Depth to Groundwater	20 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-1		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	KS	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.83)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	8	4		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	480	480	480	0.99	1.02	1.08	0.06	0.07	N/A	N/A	Above Water Table
9.5	8	12	10	31	120		1.3	1.05	1.3	1.15	0.75	47.6	47.6	1200	1200	1200	0.97	1.29	1.1	2.00	2.00	0.57	3.50	Above Water Table
14.5	12	17	14.5	57	120		1.3	1.05	1.3	1.01	0.85	87.1	87.1	1740	1740	1740	0.95	1.29	1.06	2.00	2.00	0.56	3.58	Above Water Table
19.5	17	22	19.5	18	120	92	1.3	1.05	1.292	0.97	0.95	29.2	34.7	2340	2340	2340	0.93	1.29	0.97	1.04	1.30	0.54	2.38	Above Water Table
24.5	22	24	23	20	120	86	1.3	1.05	1.3	0.92	0.95	31.0	36.5	2760	2573	2760	0.91	1.29	0.94	1.55	1.88	0.57	3.28	Nonliquefiable
24.5	24	27	25.5	20	120	70	1.3	1.05	1.299	0.89	0.95	29.9	35.4	3060	2717	3060	0.89	1.29	0.93	1.21	1.46	0.59	2.46	Nonliquefiable
29.5	27	32	29.5	16	120	79	1.3	1.05	1.204	0.82	0.95	20.4	26.0	3540	2947	3540	0.87	1.19	0.94	N/A	N/A	N/A	N/A	Non-liq: PI>18 w<.8*LL
34.5	32	37	34.5	13	120	95	1.3	1.05	1.153	0.75	1	15.3	20.8	4140	3235	4140	0.84	1.13	0.94	N/A	N/A	N/A	N/A	Non-liq: PI>18 w<.8*LL
39.5	37	42	39.5	17	120	87	1.3	1.05	1.203	0.73	1	20.3	25.9	4740	3523	4740	0.81	1.18	0.91	N/A	N/A	N/A	N/A	Non-liq: PI>18 w<.8*LL
44.5	42	47	44.5	22	120	84	1.3	1.05	1.281	0.73	1	28.1	33.7	5340	3811	5340	0.78	1.29	0.85	0.85	0.94	0.65	1.46	Nonliquefiable
49.5	47	50	48.5	17	120	94	1.3	1.05	1.181	0.66	1	18.1	23.6	5820	4042	5820	0.76	1.16	0.9	0.26	0.27	0.64	0.42	**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 Comm/Ind Development
Project Location	Pico Rivera
Project Number	20G147-1
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 cont	$(N_1)_{60\text{-CS}}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	8	4	0.0	0.0	0.0	N/A	0.50	0.95	0.00	8.00		0.000		0.00	Above Water Table
9.5	8	12	10	47.6	0.0	47.6	3.50	0.00	-1.40	0.00	4.00		0.000		0.00	Above Water Table
14.5	12	17	14.5	87.1	0.0	87.1	3.58	0.00	-4.85	0.00	5.00		0.000		0.00	Above Water Table
19.5	17	22	19.5	29.2	5.5	34.7	2.38	0.02	-0.41	0.00	5.00		0.000		0.00	Above Water Table
24.5	22	24	23	31.0	5.5	36.5	3.28	0.02	-0.54	0.00	2.00		0.000		0.00	Nonliquefiable
24.5	24	27	25.5	29.9	5.6	35.4	2.46	0.02	-0.47	0.00	3.00		0.000		0.00	Nonliquefiable
29.5	27	32	29.5	20.4	5.5	26.0	N/A	0.08	0.17	0.00	5.00		0.000		0.00	Non-liq: PI>18 w<.8*LL
34.5	32	37	34.5	15.3	5.5	20.8	N/A	0.15	0.48	0.00	5.00		0.000		0.00	Non-liq: PI>18 w<.8*LL
39.5	37	42	39.5	20.3	5.5	25.9	N/A	0.08	0.18	0.00	5.00		0.000		0.00	Non-liq: PI>18 w<.8*LL
44.5	42	47	44.5	28.1	5.5	33.7	1.46	0.03	-0.34	0.01	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	18.1	5.5	23.6	0.42	0.11	0.32	0.11	3.00		0.020		0.72	**Liquefiable**
											Total Deformation (in)				0.72	

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 Comm/Ind Development	MCE_G Design Acceleration	0.905 (g)
Project Location	Pico Rivera	Design Magnitude	6.83
Project Number	20G147-1	Historic High Depth to Groundwater	20 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	49.5 (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=6.83)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	8	4		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	480	480	480	0.99	1.02	1.08	0.06	0.07	N/A	N/A	Above Water Table
9.5	8	12	10	38	120		1.3	1.05	1.3	1.12	0.75	56.8	56.8	1200	1200	1200	0.97	1.29	1.1	2.00	2.00	0.57	3.50	Above Water Table
14.5	12	17	14.5	21	120	89	1.3	1.05	1.3	1.06	0.85	33.6	39.1	1740	1740	1740	0.95	1.29	1.06	2.00	2.00	0.56	3.58	Above Water Table
19.5	17	22	19.5	20	120	95	1.3	1.05	1.3	0.97	0.95	32.7	38.2	2340	2340	2340	0.93	1.29	0.97	2.00	2.00	0.54	3.67	Above Water Table
24.5	22	27	24.5	18	120	98	1.3	1.05	1.263	0.89	0.95	26.3	31.7	2940	2659	2940	0.90	1.27	0.95	N/A	N/A	N/A	N/A	Non-liq: PI>18 w<.8*LL
29.5	27	32	29.5	14	120	98	1.3	1.05	1.171	0.81	0.95	17.1	22.6	3540	2947	3540	0.87	1.15	0.95	N/A	N/A	N/A	N/A	Non-liq: PI>18 w<.8*LL
34.5	32	37	34.5	22	120	90	1.3	1.05	1.3	0.81	1	31.6	37.1	4140	3235	4140	0.84	1.29	0.87	1.79	2.00	0.63	3.16	Nonliquefiable
39.5	37	42	39.5	21	120	84	1.3	1.05	1.279	0.76	1	27.9	33.4	4740	3523	4740	0.81	1.29	0.88	0.81	0.92	0.64	1.43	Nonliquefiable
44.5	42	47	44.5	24	120	94	1.3	1.05	1.3	0.75	1	31.8	37.3	5340	3811	5340	0.78	1.29	0.82	1.90	2.00	0.65	3.10	Nonliquefiable
49.5	47	50	48.5	14	120	84	1.3	1.05	1.139	0.64	1	13.9	19.4	5820	4042	5820	0.76	1.11	0.91	N/A	N/A	N/A	N/A	Non-liq: 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 Comm/Ind Development
Project Location	Pico Rivera
Project Number	20G147-1
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 cont	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	8	4	0.0	0.0	0.0	N/A	0.50	0.95	0.00	8.00		0.000		0.00	Above Water Table
9.5	8	12	10	56.8	0.0	56.8	3.50	0.00	-2.15	0.00	4.00		0.000		0.00	Above Water Table
14.5	12	17	14.5	33.6	5.5	39.1	3.58	0.01	-0.74	0.00	5.00		0.000		0.00	Above Water Table
19.5	17	22	19.5	32.7	5.5	38.2	3.67	0.01	-0.67	0.00	5.00		0.000		0.00	Above Water Table
24.5	22	27	24.5	26.3	5.5	31.7	N/A	0.04	-0.21	0.00	5.00		0.000		0.00	Non-liq: PI>18 w<.8*LL
29.5	27	32	29.5	17.1	5.5	22.6	N/A	0.12	0.37	0.00	5.00		0.000		0.00	Non-liq: PI>18 w<.8*LL
34.5	32	37	34.5	31.6	5.5	37.1	3.16	0.02	-0.59	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	27.9	5.5	33.4	1.43	0.03	-0.32	0.02	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	31.8	5.5	37.3	3.10	0.01	-0.60	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	13.9	5.5	19.4	N/A	0.17	0.55	0.00	3.00		0.000		0.00	Non-liq: PI>18 w<.8*LL
											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)