GEOTECHNICAL INVESTIGATION
PROPOSED WAREHOUSE DEVELOPMENT

SEC Simpson Road and Fuego Road
Riverside County (Winchester Area), California
for
Newland Capital Group

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May 2, 2022
(Revised April 10, 2024)

Newland Capital Group
200 Spectrum Center Drive, Suite 300
Irvine, California 92618

Attention: Ms. Rocio Budetta
              Managing Director

Project No.:   **22G127-1R**

Subject: **Geotechnical Investigation**
              Proposed Warehouse Development
              SEC Simpson Road and Fuego Road
              Riverside County (Winchester Area), California

Ms. Budetta:

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.

Robert G. Trazo, GE 2655
Principal Engineer

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Senior 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 Riverside County GIS website indicates that the subject site is located within a zone of moderate to high liquefaction susceptibility.
- Our site-specific liquefaction evaluation included six (6) borings extended to a depth of 50± feet. Potentially liquefiable soil strata were encountered at Boring No. B-1 between depths of 30 and 32± feet, 37 to 42± feet, and 47 and 50± feet. Potentially liquefiable soil strata were also encountered at Boring No. B-3 between depths of 30 and 47± feet. Potentially liquefiable soil strata were encountered at Boring No. B-9 between depths of 42 and 47± feet. Potentially liquefiable soil strata were also encountered at Boring No. B-13 between depths of 30 and 37± feet.
- The potential total dynamic settlements at these boring locations are estimated to be between 0.53 and 3.00± inches.
- Based on the estimated magnitude of the differential settlements, the proposed structures may be supported on shallow foundations. Additional design considerations related to the potentially liquefiable soils are presented within this report.
- Most of the borings encountered artificial fill materials, extending to depths of 3 to 8± feet below the existing site grades. The fill soils possess varying strengths and densities, and are considered to represent undocumented fill. These soils, in their present condition, are not considered suitable for support of the foundation loads of the new structures.
- These fill soils are underlain by native alluvium which possesses varying strengths and densities. Furthermore, the results of laboratory testing indicate that the near-surface soils within the upper 4 to 5± feet possess a severe potential for consolidation when exposed to load increases in the range of those that will be exerted by the new foundations. It should be noted that Boring Nos. B-1, B-4, B-5, B-7, B-12, B-17 and B-18 encountered loose native soils, extending to a depth of up to 10± feet.
- Based on the results of corrosivity testing, the on-site soils are considered to be corrosive to ductile iron pipe and to copper pipe.

Site Preparation
- Initial site preparation should include stripping of any surficial vegetation and surficial topsoil associated with the farming activities. The surficial vegetation, and any organic soils should be properly disposed of off-site.
- Remedial grading is recommended to be performed within the proposed building areas in order to remove all of the undocumented fill soils in their entirety, the upper portion of the near-surface native alluvial soils, and any soils disturbed during the demolition process. The proposed building area should be overexcavated to a depth of at least 4 feet below existing grade and to a depth of 3 feet below proposed building pad subgrade elevation, whichever is greater. Within the foundation influence zones, the overexcavation should extend to a depth

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of at least 3 feet below proposed foundation bearing grade. The overexcavation should extend horizontally at least 5 feet beyond the building and foundation perimeters.
- After overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated. The resulting soils should be scarified and moisture conditioned to achieve a moisture content of 0 to 4 percent above optimum moisture, to a depth of at least 12 inches. The overexcavation subgrade soils should then be recompacted under the observation of the geotechnical engineer. The previously excavated soils may then be replaced as compacted structural fill.
- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

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

Building Floor Slabs
- Conventional Slab-on-Grade, 6 inches thick.
- Modulus of Subgrade Reaction: k = 150 psi/in.
- Minimum slab reinforcement: Reinforcement of the floor slab should consist of No. 3 bars at 16-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 upon the imposed loading.

Pavements

Materials	ASPHALT PAVEMENTS (R = 40)				
	Thickness (inches)				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	4	6	7	8	10
Compacted Subgrade	12	12	12	12	12

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PORTLAND CEMENT CONCRETE PAVEMENTS (R = 40)				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

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2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 22P137, dated January 28, 2022. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slabs, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. Based on the location of this site, the geotechnical 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 southwest and southeast corner of Warren Road and Simpson Road in the unincorporated Winchester area of Riverside County, California. The site is also referenced by the street address 35655 Simpson Road. The site is bounded to the north by Simpson Road, to the west by the El Fuego Road easement and to the southeast by the Olive Avenue easement. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The site consists of two irregularly shaped parcels, which total 74.88± acres in size. The site is presently being utilized for agricultural purposes. The majority of the site is planted with row crops. Several unpaved access roads that trend east-west and north-south are located throughout the site. Farming equipment, mobile trailers and cars are present in the northeast area of the site. Warren Road transects the eastern one-third of the site and separates the eastern 8.28± acres of the site from the western 66.6± acres of the site.

Elevation data was obtained from the conceptual site plan prepared by Ware Malcomb, the project civil engineer. Based on the site plan, the site topography ranges from a topographic high of 1504± feet mean sea level (MSL) in the northeastern corner of the site to a topographic low of 1490± feet MSL in the southwestern corner of the site. The maximum site elevation differential is 14± feet. Please note that these elevations exclude Warren Road which ranges from 1501± feet MSL at Simpson Road to 1515± feet MSL along the south boundary of the subject site. The site topography slopes down on both the east and west sides of Warren Road at inclinations of 2h:1v to the subject site.

3.2 Proposed Development

Based on the preliminary conceptual site plan, prepared by Ware Malcomb, the site will be developed with two warehouses (identified as Building 1 and Building 2). Building 1 will be located in the western area of the site and will have a footprint of 882,700± ft². Building 2 will be located in the central area of the site and will have a footprint of 309,077± ft². Dock-high doors will be constructed along portions of the east and west building walls of Building 1 and along a portion of the west building wall for Building 2. The buildings will be surrounded by asphaltic concrete pavements in the parking and drive lanes, Portland cement concrete pavements in the loading dock areas, and concrete flatwork with limited areas of landscape planters throughout.

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

No significant amounts of below grade construction, such as crawl spaces or new basements, are expected to be included in the proposed development. Based on the assumed topography, cuts and fills of up to 4 to 5± feet are expected to be necessary to achieve the proposed site grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration for this project consisted of eighteen (18) borings (identified as Boring Nos. B-1 through B-18) advanced to depths of 10 to 50± feet below the existing site grades. Six (6) of the borings were advanced to a depth of 50± feet as a part of the liquefaction analysis. All of the borings were logged during drilling by a member of our staff.

Boring Nos. B-3, B-6, B-9, B-11, B-12, B-13 and B-18 were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig. The remaining borings were advanced with hollow-stem augers, by a limited-access, track-mounted drilling rig. Representative bulk and relatively undisturbed soil samples were taken during drilling. Relatively undisturbed soil samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. Samples were also taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

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

4.2 Geotechnical Conditions

Artificial Fill

Artificial fill soils were encountered at the ground surface at all of the borings, extending to depths of 3 to 8± feet below the existing site grades. The fill soils generally consist of very loose to medium dense silty sands and sandy silts as well as stiff to very stiff silty clays. The fill soils possess a disturbed and mottled appearance, resulting in their classification as artificial fill.

Alluvium

Native alluvial soils were encountered beneath the fill soils at all of the boring locations, extending to at least the maximum depth explored of 50± feet below the existing site grades. The alluvial soils generally consist of loose to medium dense silty sands, sandy silts, sands with varying

amounts of silt with varying clay content. Boring No. B-13 encountered a stratum of very stiff clayey silts at a depth of 32 to 37± feet.

Groundwater

Free water was encountered during drilling at Boring Nos. B-1, B-3, B-6, B-8, B-9, and B-13 at depths ranging from 34 to 41± feet below the ground surface, respectively. Based on these observations, the static groundwater table is considered to have been present at a depth of 34 to 41± feet below the existing site grades 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 groundwater depths in this area is the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. The nearest monitoring well is located near the central portion of the site. Water level readings within this monitoring well indicate a high groundwater level of 31± feet below the ground surface in March 2012.

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5.0 LABORATORY TESTING

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

Classification

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

Dry Density and Moisture Content

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

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-8 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

Three (3) representative bulk samples have been tested for their maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plates C-9 through C-11 in Appendix C of this report. This test is generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Soluble Sulfates

Representative samples of the near-surface soil 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	0.0238	Not Applicable (S0)
B-8 @ 0 to 5 feet	0.0189	Not Applicable (S0)
B-9 @ 0 to 5 feet	0.0178	Not Applicable (S0)

Corrosivity Testing

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

Sample Identification	Saturated Resistivity (ohm-cm)	pH	Chlorides (mg/kg)	Nitrates (mg/kg)
B-4 @ 0 to 5 feet	1,072	7.8	197.7	200.3
B-8 @ 0 to 5 feet	1,072	8.1	56.7	69.8
B-9 @ 0 to 5 feet	2,814	8.7	67.1	67.9

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 enclosed boring logs.

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on selected samples of various soil strata encountered at the site. This test is used to determine the Liquid Limit and Plastic Limit of the soil. The Plasticity Index (PI) 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 plasticity, and a high expansion potential. Soils with a PI greater than 18 are not considered to be 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.

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 (%)
B-4 @ 1 to 1½ feet	3.1
B-4 @ 3 to 3½ feet	1.1
B-10 @ 1 to 1½ feet	15.3
B-10 @ 3 to 3½ feet	1.4
B-12 @ 1 to 1½ feet	13.8
B-12 @ 3 to 3½ feet	3.8
B-12 @ 5 to 5½ feet	0.6
B-12 @ 7 to 7½ feet	10.1

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

6.1 Seismic Design Considerations

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

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. In addition, our review of the Riverside County RCIT GIS website indicates that the site is not located within a Riverside County fault zone. Furthermore, SCG did not identify any evidence of faulting during the geotechnical investigation. Therefore, the possibility of significant fault rupture on the site is considered to be low Therefore, the possibility of significant fault rupture on the site is considered to be low.

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

Seismic Design Parameters

The 2022 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site. Based on the adoption of the 2022 CBC on January 1, 2023, we expect that the proposed development will be designed in accordance with the 2022 CBC.

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

The 2022 CBC states that for Site Class D sites with a mapped S1 value greater than 0.2, a site-specific ground motion analysis may be required in accordance with Section 11.4.8 of ASCE 7-16. Supplement 3 to ASCE 7-16 modifies Section 11.4.8 of ASCE 7-16 and states that "a ground motion hazard analysis is not required where the value of the parameter SM1 determined by Eq. (11.4-2) is increased by 50% for all applications of SM1 in this Standard. The resulting value of the parameter SD1 determined by Eq. (11.4-4) shall be used for all applications of SD1 in this Standard."

The seismic design parameters presented in the table below were calculated using the site coefficients (Fa and Fv) from Tables 1613.2.3(1) and 1613.2.3(2) presented in Section 16.4.4 of the 2022 CBC. It should be noted that the site coefficient Fv and the parameters SM1 and SD1 were not included in the SEAOC/OSHPD Seismic Design Maps Tool output for the ASCE 7-16 standard. We calculated these parameters-based on Table 1613.2.3(2) in Section 16.4.4 of the 2022 CBC using the value of S1 obtained from the Seismic Design Maps Tool. **The values of SM1 and SD1 tabulated below** were evaluated using equations 11.4-2 and 11.4-4 of ASCE 7-16 (Equations 16-20 and 16-23, respectively, of the 2022 CBC) and **do not include a 50 percent increase.** As discussed above, if a ground motion hazard analysis has not been performed, SM1 and SD1 must be increased by 50 percent for all applications with respect to ASCE 7-16.

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.500
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.600
Site Class	---	D[1]
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.500
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.020[2]
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.000
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.680[2]

[1]Note: The 2022 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site *coefficients* are to be determined in accordance with Section 11.4.7 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. Based on the proposed construction, we expect that the proposed building will possess a fundamental period of vibration less than 0.5 seconds. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of the structure is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. Therefore, if the proposed structure has a fundamental period greater than 0.5 seconds, a site-specific seismic hazards analysis will be required and additional subsurface exploration will be necessary.

[2]Note: These values must be increased by 50 percent if a site-specific ground motion hazard analysis has not been performed. However, this increase is not expected to affect the design of the structure type proposed for this site. This assumption should be verified by the project structural engineer. The values tabulated above do not include a 50-percent increase.

Liquefaction

The Riverside County GIS website indicates that the subject site is located within a zone of moderate to high liquefaction susceptibility. Based on this mapping, the scope of this investigation included additional subsurface exploration, laboratory testing, and engineering analysis in order to determine the site-specific liquefaction potential.

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

The liquefaction analysis was conducted in accordance with the requirements of Special Publication 117A (CDMG, 2008), and currently accepted practice (SCEC, 1997). The liquefaction potential of the subject site was evaluated using the empirical method developed by Boulanger and Idriss (Boulanger and Idriss, 2008, 2014). This method predicts the earthquake-induced liquefaction potential of the site based on a given design earthquake magnitude and peak ground acceleration at the subject site. This procedure essentially compares the cyclic resistance ratio

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(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 considered to be insusceptible to liquefaction. Non-sensitive soils with a PI greater than 18 are also considered non-liquefiable.

As part of the liquefaction evaluation, Boring Nos. B-1, B-3, B-6, B-8, B-9 and B-13 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 0.571g for a magnitude 7.06 seismic event. The liquefaction evaluation was performed using the reported historic high groundwater depth of 30 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

Potentially liquefiable soils were encountered at most of the 50±-foot deep boring locations. Potentially liquefiable soil strata were encountered at Boring No. B-1 between depths of 30 and 32± feet, 37 to 42± feet, and 47 and 50± feet. Potentially liquefiable soil strata were also encountered at Boring No. B-3 between depths of 30 and 47± feet. Potentially liquefiable soil strata were also encountered at Boring No. B-9 between depths of 42 and 47± feet. Potentially liquefiable soil strata were also encountered at Boring No. B-13 between depths of 30 and 37± feet. The remaining soil strata encountered below the historic high groundwater table either possess factors of safety in excess of 1.3 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 strata. The results of the settlement analyses indicate the following total deformations:

- Boring No. B-1:   1.58 inches
- Boring No. B-3:   3.00 inches
- Boring No. B-6:   0.00 inches
- Boring No. B-8:   0.00 inches
- Boring No. B-9:   0.53 inches
- Boring No. B-13:   0.73 inches

Based on the results of the settlement analyses, differential settlements are expected to be on the order of 2± inches or less. The estimated differential settlement can be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of less than 0.002 inches per inch.

Based on our understanding of the proposed development, it is considered feasible to support the proposed structures on 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 structures to remain completely undamaged during a major seismic event is not considered to be economically feasible. Based on this understanding, the use of shallow foundation systems is considered to be the most economical means of supporting the proposed structures.

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

The use of a shallow foundation system, as described in this report, is typical for buildings of these types, 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

Most of the borings encountered artificial fill materials, extending to depths of 3 to 8± feet below the existing site grades. Based on a lack of documentation regarding the placement and compaction of the existing fill materials, these soils are considered to consist of undocumented fill, and are not suitable for the support of the foundation loads of the proposed buildings. These fill soils are underlain by native alluvium which possesses varying strengths and densities. Furthermore, the results of laboratory testing indicate that the near-surface soils within the upper 4 to 5± feet possess a minor to moderate potential for consolidation when exposed to load increases in the range of those that will be exerted by the new foundations. It should be noted that Boring Nos. B-1, B-4, B-5, B-7, B-12, B-17 and B-18 encountered loose native soils, extending to a depth of up to 10± feet. Based on these conditions, remedial grading will be necessary within the proposed building areas to provide a subgrade suitable for support of the new foundations and floor slabs of the proposed buildings. The remedial grading will also serve to create more uniform support characteristics across the proposed building pad areas.

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

<u>Settlement</u>

The recommended remedial grading will remove the existing undocumented fill soils and a portion of the near-surface native alluvial soils and replace these materials as compacted structural fill. The native soils that will remain in place below the recommended depth of overexcavation will not be subject to significant stress increases from the foundations of the new structures. Therefore, following completion of the recommended grading, post-construction settlements are expected to be within tolerable limits.

<u>Expansion</u>

The near surface soils at this site generally consist of silty sands, sands and sandy silts. These materials have been visually classified as non-expansive. Therefore, no design considerations related to expansive soils are considered warranted for this site.

<u>Organic Content</u>

Laboratory testing indicates that the tested samples of near-surface soils possess organic contents ranging from 0.6 to 15.3 percent by weight. Based on the results of the organic content testing, the near-surface soils are considered to possess low to moderate organic contents. Soils possessing the highest organic contents were encountered within the upper 1½ feet at some of the borings and at 7 to 7½± feet at Boring No. B-12. Soils possessing low to moderate organic content (less than approximately 5 percent organics), with no appreciable organic fibers, may be blended with the underlying relatively non-organic soils, such that the final mixture contains less than 3 percent organics, by dry weight. However, any remaining soils containing appreciable organic fibers that cannot practically be segregated or any other organic materials, including topsoils, if present) or any vegetation that may develop before construction be removed from the site in its entirety or disposed of in nonstructural areas, such as landscape planters. We would highly recommend that all soils possessing appreciable organic fibers or soils containing greater than 5 percent organics be stripped and removed from the site. Any additional organic materials encountered during site grading should also be segregated and removed from the site during grading.

<u>Soluble Sulfates</u>

The results of the soluble sulfate testing indicate that the tested soil samples possess levels of soluble sulfates that are considered to be "not applicable" (S0) with respect to the American Concrete Institute (ACI) Publication 318-14 <u>Building Code Requirements for Structural Concrete and Commentary</u>, 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

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additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building area.

Corrosion Potential

The results of laboratory testing indicate that the tested sample of the on-site soils possesses a saturated resistivity values ranging from 1,072 to 2,814 ohm-cm, and pH values ranging from 7.8 to 8.7. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity and pH are two of the five factors that enter into the evaluation procedure. Redox potential, relative soil moisture content and sulfides are also included. Although sulfide testing was not part of the scope of services for this project, we have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH and moisture content. **Based on these factors, and utilizing the DIPRA procedure, the on-site soils are considered to be moderately to severely corrosive to ductile iron pipe. Therefore, polyethylene protection is expected to be required for cast iron or ductile iron pipes.** It should be noted that SCG does not practice in the field of corrosion engineering. **Therefore, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.**

Based on American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary, reinforced concrete that is exposed to external sources of chlorides requires corrosion protection for the steel reinforcement contained within the concrete. ACI 318 defines concrete exposed to moisture and an external source of chlorides as "severe" or exposure category C2. ACI 318 does not clearly define a specific chloride concentration at which contact with the adjacent soil will constitute a "C2" or severe exposure. However, the Caltrans Memo to Designers 10-5, Protection of Reinforcement Against Corrosion Due to Chlorides, Acids and Sulfates, dated June 2010, indicates that soils possessing chloride concentrations greater than 500 mg/kg are considered to be corrosive to reinforced concrete. The results of the laboratory testing indicate chloride concentrations ranging from 56.7 to 197.7 mg/kg. Although the soils contain some chlorides, we do not expect that the chloride concentrations of the tested soils are high enough to constitute a "severe" or C2 chloride exposure. Therefore, a chloride exposure category of C1 is considered appropriate for this site.

Nitrates

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested sample possesses nitrate concentrations ranging from 67.9 to 200.3 mg/kg. **Based on this test result, the on-site soils are considered to be corrosive to copper pipe. Since SCG does not practice in the area of corrosion engineering, we recommend that the client contact a corrosion engineer to provide recommendations for the protection of copper tubing/pipe in contact with the on-site soils.**

Shrinkage/Subsidence

Removal and recompaction of the existing fill soils and near-surface alluvium is estimated to result in an average shrinkage of 3 to 13 percent. However, potential shrinkage for individual samples ranged locally between 1 and 16 percent. 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

Demolition of the existing utilities and any associated improvements will be necessary to facilitate the construction of the proposed development. Debris resultant from demolition should be disposed of off-site. All applicable federal, state and local specifications and regulations should be followed in demolition, abandonment, and disposal of the existing utilities and resulting debris.

Initial site stripping should include removal of the surficial vegetation from the site. Stripping should include existing crops. These materials should be properly disposed of off-site. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pads

Remedial grading should be performed within the proposed building areas in order to remove the existing undocumented fill soils, any soils disturbed during demolition, and the upper portion of the near-surface native alluvium. Based on conditions encountered at the boring locations, the existing soils within the proposed building areas are recommended to be overexcavated to a

depth of at least 4 feet below existing grades and to a depth of at least 3 feet below proposed building pad subgrade elevation, 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 and foundation perimeters, and to an extent equal to the depth of fill placed below the foundation bearing grade, whichever is greater. If the proposed structure incorporates any exterior columns (such as for a canopy or overhang) the area of overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the overexcavation areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling and probing to identify any soft, loose, or otherwise unstable soils that must be removed. **Some localized areas of deeper excavation may be required if any artificial fill or loose, porous, or low-density native soils are encountered at the base of the overexcavation.** It should be noted that Boring Nos. B-1, B-4, B-5, B-7, B-12, B-17, and B-18 encountered loose native soils, extending to a depth of up to 10± feet.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches and moisture conditioned or air dried to achieve a moisture content of 0 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

The building pad areas may then be raised to grade with previously excavated soils or imported, very low expansive structural fill. All structural fill soils present within the proposed building areas should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining walls and site walls should be overexcavated to a depth of 3 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pads. Any undocumented fill soils or disturbed native alluvium within any of these foundation areas should be removed in their entirety. The overexcavation areas should extend at least 3 feet beyond the foundation perimeters, and to an extent equal to the depth of fill below the new foundations. Any erection pads for tilt-up concrete walls are considered to be part of the foundation system. Therefore, these overexcavation recommendations are applicable to erection pads. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning to within 0 to 4 percent above the optimum moisture content, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

If the full lateral recommended remedial grading cannot be completed for the proposed retaining walls and site walls located along property lines, the foundations for those walls should be designed using a reduced allowable bearing pressure. Furthermore, the contractor should take

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necessary precautions to protect the adjacent improvements during rough grading. Specialized grading techniques, such as A-B-C slot cuts, will likely be required during remedial grading. The geotechnical engineer of record should be contacted if additional recommendations, such as shoring design recommendations, are required during grading.

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

Based on economic considerations, overexcavation of the existing near-surface existing soils in the new flatwork, parking and drive areas is not considered warranted, with the exception of areas where lower strength or unstable soils are identified by the geotechnical engineer during grading. Subgrade preparation in the new flatwork, parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations.

The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. Any such materials should be removed to a level of firm and unyielding soil. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 0 to 4 percent above the optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed flatwork, parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within these areas. The grading recommendations presented above do not mitigate the extent of undocumented fill or compressible/collapsible native alluvium in the flatwork, parking and drive areas. As such, some settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the flatwork.

<u>Fill Placement</u>

- 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 County of Riverside.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

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

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the County of Riverside. 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 (horizontal to vertical) 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.

Any soils used to backfill voids around subsurface utility structures, such as manholes or vaults, should be placed as compacted structural fill. If it is not practical to place compacted fill in these areas, then such void spaces may be backfilled with lean concrete slurry. Uncompacted pea gravel or sand is not recommended for backfilling these voids since these materials have a potential to settle and thereby cause distress of pavements placed around these subterranean structures.

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of moderate strength silty sands, sandy silts, and sands. These materials may be subject to minor to moderate caving within shallow excavations. Where caving does occur, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 2h:1v within sandy soils. 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. If grading occurs during a period of relatively wet weather, an increase in subgrade instability should also be expected. The

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site should, therefore, be graded to prevent ponding of surface water and to prevent water from running into excavations.

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

Groundwater

The groundwater table is considered to exist at a depth of 34 to 41± feet below the existing grades at the time of drilling. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pads will be underlain by structural fill soils extending to depths of at least 3 feet below foundation bearing grade, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, the proposed structures may be supported on conventional shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2.

- Maximum, net allowable soil bearing pressure: 1,500 lbs/ft^2 if the full recommended lateral extent of remedial grading cannot be achieved.

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

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

<u>Foundation Construction</u>

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 or suitable native alluvium (where reduced bearing pressures are utilized), 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.

<u>Estimated Foundation Settlements</u>

Post-construction total and differential static settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively, under static conditions. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch. These settlements are in addition to the liquefaction-induced settlements previously discussed in Section 6.1 of this report. However, the likelihood of these two settlements combining is considered remote. The static settlements are expected to occur in a relatively short period of time after the building loads are 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.

<u>Lateral Load Resistance</u>

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

- Passive Earth Pressure: 300 lbs/ft^3
- Friction Coefficient: 0.30

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 2,500 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 3 feet below finished pad grade. Based on geotechnical considerations, the floor slabs may be designed as follows:

- Minimum slab thickness: 6 inches.

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

- Minimum slab reinforcement: Minimum slab reinforcement: No. 3 bars at 16 inches on-center, in both directions, due to the presence of potentially liquefiable soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed loading, and the liquefaction-induced settlements.

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

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

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

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

necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements discussed in Section 6.1.

6.7 Retaining Wall Design and Construction

Although not indicated on the site plan, some small (less than 6 feet in height) retaining walls may be required to facilitate the new site grades. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near-surface soils generally consist of silty sands, sandy silts, and sands. Based on their classification, the sandy materials are expected to possess a friction angle of at least 30 degrees when compacted to at least 90 percent of the ASTM D-1557 maximum dry density.

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 Silty Sands and Sandy Silts
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

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

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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.

Retaining Wall Foundation Design

The retaining wall foundations should be underlain by at least 3 feet of newly placed structural fill. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

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.

Backfill Material

On-site soils may be used to backfill the retaining walls, provided that they are very low expansive (EI < 20). All backfill material placed within 3 feet of the back wall-face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

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

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

Subsurface Drainage

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

- A weep hole drainage system typically consisting of a series of 2-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 10-foot on-center spacing. Alternatively, 4-inch diameter holes at an approximate 20-foot on-center spacing can be used for this type of drainage system. In addition, the weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system. The actual design of this type of system should be determined by the civil engineer to verify that the drainage system possesses the adequate capacity and slope for its intended use.

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

6.8 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface soils generally consist of silty sands, sandy silts, and sands. These soils are generally considered to possess poor to fair pavement support characteristics with estimated R-values ranging from 40 to 50. The subsequent pavement design is therefore 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. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

<u>Asphaltic Concrete</u>

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

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

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

ASPHALT PAVEMENTS (R = 40)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	4	6	7	8	10
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 batch plant-reported maximum density. 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)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI =7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

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

7.0 GENERAL COMMENTS

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

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

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

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

8.0 REFERENCES

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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

SITE

San Diego Ca...

SMITH RD
1500
ARVIN HULL RD
1500
1600
1600
CALIFORNIA AVE
AQUEDUCT RD
EL FUEGO RD
SIMPSON RD
1500
1500
DOMENIGONI PKWY
PATTON RD
...anal

CAM. SUENO
PSO FAMOSA
RTE ALAMONTE
VIA ROJAS
OAK TREE WAY
S. CAWSTON AVE
REX
W COM...
W HARRISON AVE
FISHER ST
COVE ST
N WARREN RD
1500
POPLAR ST
DUSKY WING RD
CLOUDYWING RD
009
ALAMAR MESA DR
TRIOLO TRL
WARREN RD
Lakeview
1700
1800
2000
2100
1900
2000
Domenigoni Mountains
2100
1600
1800
2000
Domenigoni
Mountains
Diamond Valley
1900
1700
1900
2000
1800
2100
2200
1800
Lakeview Trl
900

Diamond
Valley Lake

Diamond Valley

SOURCE: USGS TOPOGRAPHIC MAPS OF THE WINCHESTER
QUADRANGLE, RIVERSIDE COUNTY, CALIFORNIA, 2021.

SITE LOCATION MAP		
PROPOSED WAREHOUSE DEVELOPMENT		
WINCHESTER, CALIFORNIA		
SCALE: 1" = 2000'		SOUTHERN
DRAWN: MD CHKD: DN	SoCalGeo	CALIFORNIA
SCG PROJECT 22G127-1R		GEOTECHNICAL
PLATE 1		

NORTH
SoCalGeo

SIMPSON ROAD

EL FUEGO ROAD EASEMENT

WARREN ROAD

OLIVE AVENUE EASEMENT

B-14

B-1

B-2

B-3

B-4   PROPOSED BUILDING 1
       882,700 SF

B-5

B-6

B-7

B-8

B-9

B-10

B-11

PROPOSED
BUILDING 2
309,077 SF

B-12

B-13

B-16

B-15

B-17

B-18

GEOTECHNICAL LEGEND
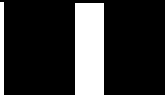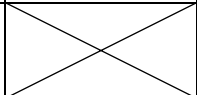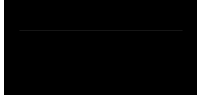
PROPOSED BORING LOCATION

NOTE: CONCEPTUAL SITE PLAN PREPARED BY WARE MALCOMB.

INFILTRATION TEST LOCATION PLAN
PROPOSED WAREHOUSE DEVELOPMENT
RIVERSIDE COUNTY (WINCHESTER AREA), CALIFORNIA

SCALE: 1" = 250'	
DRAWN: DK	SoCalGeo
CHKD: RGT	
SCG PROJECT 22G127-1R	SOUTHERN CALIFORNIA GEOTECHNICAL
PLATE 2	

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

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

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

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

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

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

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

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

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

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

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
SoCalGeo
A California Corporation

JOB NO.: 22G127-1
PROJECT: Proposed Warehouse Development
LOCATION: Winchester, California

DRILLING DATE: 3/7/22
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Esparza

WATER DEPTH: 40 feet
CAVE DEPTH: 20 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
		4			FILL: Gray Brown Silty fine to medium Sand, trace coarse Sand, trace fine Root Fibers, very loose to loose-moist		10					
		7			FILL: Brown Silty fine to coarse Sand, trace fine Root Fibers, loose-damp to moist		9					
5		10			ALLUVIUM: Brown fine to coarse Sand, little Silt, loose to medium dense-damp to moist		6					
		9			Gray Brown fine Sandy Silt, trace medium Sand, loose-very moist		18					
10												
		21			Gray fine to coarse Sandy Silt, medium dense-moist		10					
15												
		24			Light Gray Brown fine to coarse Sand, medium dense-damp		3					
20												
		21			Gray Brown fine Silty Sand, trace medium Sand, medium dense-very moist		15					
25												
		14			Gray Brown to Brown Silty fine Sand, trace coarse Sand, medium dense-very moist		15			32		
30												
		20			Gray Brown Silty fine to medium Sand, little coarse Sand, medium dense-very moist		15			24		

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG

PLATE B-1a

SOUTHERN CALIFORNIA GEOTECHNICAL
SoCalGeo
A California Corporation

BORING NO.
B-1

JOB NO.: 22G127-1	DRILLING DATE: 3/7/22
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger
LOCATION: Winchester, California	LOGGED BY: Michelle Esparza

WATER DEPTH: 40 feet
CAVE DEPTH: 20 feet
READING TAKEN: At Completion

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
					Gray Brown Silty fine to medium Sand, little coarse Sand, medium dense-very moist							
					Gray Brown Silty fine Sand, medium dense-wet							
40	⊠	19					24			34		
45	⊠	20			Gray Brown Silty fine Sand to fine Sandy Silt, trace Calcareous veining, medium dense-wet		28			51		
50	⊠	14			Gray Brown fine Sandy Silt, medium dense-wet		28			72		
					Boring Terminated at 50'							

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG

PLATE B-1b

SOUTHERN CALIFORNIA GEOTECHNICAL

A California Corporation

SoCalGeo

JOB NO.: 22G127-1	DRILLING DATE: 3/9/22	WATER DEPTH: Dry
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 22 feet
LOCATION: Winchester, California	LOGGED BY: Michelle Esparza	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
	▨	15			FILL: Dark Brown Silty fine Sand, little medium Sand, trace to little fine Root Fibers, medium dense-moist to very moist	107	19				13.8	
	▨	12			@ 3', mottled	107	8				3.8	
5	▨	18			ALLUVIUM: Light Brown fine to medium Sand, little coarse Sand, trace fine Root Fibers, medium dense-damp	107	4				0.6	
	▨	14				100	3				10.1	
10	▨	23			Dark Brown fine Sandy Silt, trace Clay, medium dense-very moist	116	17					
15	⊠	13			Gray Brown Silty fine to medium Sand, trace coarse Sand, medium dense-damp to moist		8					
20	⊠	16			Dark Brown fine Sandy Silt, trace to little medium Sand, medium dense-moist		13					
25	⊠	28					10					
					Boring Terminated at 25'							

TEST BORING LOG　　　　　　　　　　　　　　　　　　　**PLATE B-2**

JOB NO.: 22G127-1	DRILLING DATE: 3/7/22	WATER DEPTH: 36 feet
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 46 feet
LOCATION: Winchester, California	LOGGED BY: Jamie Hayward	READING TAKEN: 4½ Hours After 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
	⊠	6			FILL: Gray Brown Silty fine Sand, trace medium Sand, loose-moist		8					
	⊠	41			@ 3½ feet, weakly cemented, dense		11					
5	⊠	19			ALLUVIUM: Gray Brown fine Sandy Silt, trace to little medium to coarse Sand, slightly cemented, little to some Calcareous nodules/veining, medium dense-moist to very moist		17					
	⊠	12					18					
10												
	⊠	20					21					
15												
	⊠	30					12					
20												
	⊠	24			Gray Brown Silty fine to medium Sand, medium dense-moist to very moist		13					
25												
	⊠	12			@ 28½ feet, little Calcareous nodules.veining		23			40		
30												
	⊠	13			Gray Brown Silty fine Sand to fine Sandy Silt, trace Calcareous nodules/veining, medium dense-very moist to wet		23			56		

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG

PLATE B-3a

JOB NO.: 22G127-1	DRILLING DATE: 3/7/22	WATER DEPTH: 36 feet
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 46 feet
LOCATION: Winchester, California	LOGGED BY: Jamie Hayward	READING TAKEN: 4½ Hours After Completion

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
	⊠	16					24			58		
40												
	⊠	19			@ 43½ feet, trace to little Clay		18			49		
45												
	⊠	20					21			47		
50												
					Boring Terminated at 50'							

TEST BORING LOG

PLATE B-3b

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

BORING NO. B-4

JOB NO.: 22G127-1	DRILLING DATE: 3/7/22	WATER DEPTH: Dry
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Winchester, California	LOGGED BY: Michelle Esparza	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
	⊠	12			FILL: Dark Brown Silty fine to medium Sand, trace fine Root Fibers, trace fine Root Fibers, trace coarse Sand, slightly mottled, loose-moist to very moist	112	14				3.1	
	⊠	5			FILL: Brown Silty fine to coarse Sand, trace fine Root Fibers, loose-damp	109	8				1.1	
5	⊠	12			ALLUVIUM: Brown fine to medium Sand, little coarse Sand, little Silt, loose-damp	108	3					
	⊠	13			Light Gray Brown fine to coarse Sand, trace fine Gravel, loose-damp	97	4					
10	⊠	35			Gray Brown Silty fine to coarse Sand, trace Clay, medium dense-damp	119	4					
15	⊠	16			Gray Brown fine Sandy Silt, trace medium Sand, medium dense-moist to very moist		14					
20	⊠	21			Gray Brown fine to medium Sandy Silt, trace coarse Sand, medium dense-moist to very moist		14					

Boring Terminated at 20'

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG

PLATE B-4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

BORING NO.
B-5

JOB NO.: 22G127-1	DRILLING DATE: 3/8/22	WATER DEPTH: Dry
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 14 feet
LOCATION: Winchester, California	LOGGED BY: Michelle Esparza	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
	☒	25			FILL: Dark Brown Silty fine Sand, trace medium Sand, trace fine Root Fibers, mottled, medium dense-moist to very moist	119	12					
	☒	16				98	18					
5	☒	15				92	13					
	☒	12			ALLUVIUM: Brown fine Sandy Silt, trace to little medium to coarse Sand, loose to medium dense-moist	114	11					
10	☒	16				115	9					
	☒	16			Gray Brown Silty fine to medium Sand to fine to medium Sandy Silt, medium dense-moist to very moist		12					
15												
20	☒	18					10					
					Boring Terminated at 20'							

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG

PLATE B-5

SOUTHERN CALIFORNIA GEOTECHNICAL
SoCalGeo
A California Corporation

JOB NO.: 22G127-1	DRILLING DATE: 3/7/22	WATER DEPTH: 40 feet
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 48 feet
LOCATION: Winchester, California	LOGGED BY: Jamie Hayward	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
	☒	10			FILL: Gray Brown Silty fine Sand, trace to little Clay, mottled, slightly cemented, loose to medium dense-very moist		13					
	☒	8					14					
5	☒	11			ALLUVIUM: Gray Brown fine Sandy Silt, trace medium Sand, trace to little Calcareous veining, medium dense-moist to very moist		13					
	☒	17			Light Gray Brown fine to medium Sand, trace coarse Sand, medium dense-damp		3					
10												
	☒	22			Light Gray Brown fine to coarse Sand, 1-inch fine Sandy Silt lense, medium dense-damp		3					
15												
	☒	29			Gray Brown fine Sandy Silt, little medium Sand, medium dense-very moist		16					
					Light Gray fine to coarse Sand, medium dense-damp		3					
20					Gray Brown fine Sandy Silt, little to some Clay, medium dense-moist to very moist		14					
	☒	20			@ 23½ feet, little medium Sand		13					
25												
	☒	25			Gray Brown Silty fine Sand, medium dense-very moist		13			35		
30												
	☒	22					13			36		

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG   **PLATE B-6a**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

BORING NO.
B-6

JOB NO.: 22G127-1	DRILLING DATE: 3/7/22	WATER DEPTH: 40 feet
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 48 feet
LOCATION: Winchester, California	LOGGED BY: Jamie Hayward	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
					Gray Brown Silty fine Sand, medium dense-moist							
40	✕	34			@ 38½ feet, trace to little Clay, dense		10					
45	✕	23			Brown to Red Brown Silty fine to medium Sand, medium dense-wet		14			31		
50	✕	37					13					
					Boring Terminated at 50'							

TEST BORING LOG

PLATE B-6b

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

JOB NO.: 22G127-1	DRILLING DATE: 3/7/22	WATER DEPTH: Dry
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 15 feet
LOCATION: Winchester, California	LOGGED BY: Michelle Esparza	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
	☒	16			FILL: Gray Brown Silty fine to medium Sand, trace fine Root Fibers, medium dense-moist to very moist	108	19					
	☒	33			FILL: Brown Silty fine to coarse Sand, trace Clay, medium dense-moist	113	10					
5	☒	22				119	10					
	☒	13			ALLUVIUM: Brown fine to coarse Sand, loose to medium dense-damp	106	3					
10	☒	24				110	4					
15	☒	23			Brown fine to medium Sand, trace coarse Sand, trace Silt, medium dense-damp		5					
20	☒	29			Dark Brown fine to medium Sandy Silt, trace coarse Sand, medium dense-moist		10					
25	☒	15			Dark Brown fine Sandy Silt, trace coarse Sand-very moist		22					

Boring Terminated at 25'

TEST BORING LOG **PLATE B-7**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 22G127-1	DRILLING DATE: 3/8/22	WATER DEPTH: 34 feet
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: ---
LOCATION: Winchester, California	LOGGED BY: Michelle Esparza	READING TAKEN: 4 Hours After 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
	X	15	3.5		FILL: Dark Gray Brown fine Sandy Clay, trace medium Sand, trace fine Root Fibers, stiff-very moist		17					
	X	10			FILL: Gray Brown Silty fine Sand to fine Sandy Silt to fine Sandy Silt, loose to medium dense-very moist		14					
5	X	14			FILL: Brown Silty fine to medium Sand, trace coarse Sand, medium dense-very moist		13					
	X	26			ALLUVIUM: Brown Silty fine to medium Sand, trace Gravelly Sand, trace coarse Sand, medium dense-moist		11					
10												
	X	31			Light Brown fine to medium Sand, trace coarse Sand, dense-damp		4					
15												
	X	15			Dark Brown fine Sandy Silt, trace to little Clay, medium dense-very moist		21					
20												
	X	21			Brown to Red Brown Silty fine Sand, trace to little medium to coarse Sand, medium dense to dense-very moist to wet		12					
25												
	X	24					13			35		
30												
	X	33					13					

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG

PLATE B-8a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

JOB NO.: 22G127-1	DRILLING DATE: 3/8/22	WATER DEPTH: 34 feet
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: ---
LOCATION: Winchester, California	LOGGED BY: Michelle Esparza	READING TAKEN: 4 Hours After Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Brown to Red Brown Silty fine Sand, trace to little medium to coarse Sand, medium dense to dense-moist to wet							
40	⊠	19					13			26		
45	⊠	29					15					
50	⊠	30					21					
					Boring Terminated at 50'							

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-9

JOB NO.: 22G127-1	DRILLING DATE: 3/8/22	WATER DEPTH: 40 feet
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 47 feet
LOCATION: Winchester, California	LOGGED BY: Jamie Hayward	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
	⊠	25			FILL: Gray Brown Silty fine Sand, trace medium Sand, slightly cemented, slightly mottled, medium dense-very moist		14					
	⊠	16			ALLUVIUM: Gray Brown Silty fine Sand to fine Sandy Silt, little Calcareous nodules/veining, medium dense-damp to very moist		21					
5	⊠	15					7					
	⊠	13			Gray Brown fine to medium Sand, 3-inch fine Sandy Silt lense, medium dense-damp to moist		7					
10												
	⊠	15			Gray Brown Silty fine to medium Sand, medium dense-damp to very moist		12					
15												
	⊠	18					9					
20												
	⊠	30			@ 23½ feet, dense		7					
25												
	⊠	21					11			23		
30												
	⊠	23					13			25		

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG

PLATE B-9a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

BORING NO.
B-9

JOB NO.: 22G127-1	DRILLING DATE: 3/8/22	WATER DEPTH: 40 feet
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 47 feet
LOCATION: Winchester, California	LOGGED BY: Jamie Hayward	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
40	⊠	43			@ 38½ feet, dense-wet		16					
45	⊠	18			Gray Brown Silty fine Sand to fine Sandy Silt, medium dense-wet		19			53		
50	⊠	25			Brown Silty fine Sand, medium dense-wet		22			44		
					Boring Terminated at 50'							

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG

PLATE B-9b

SOUTHERN CALIFORNIA GEOTECHNICAL
SoCalGeo
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JOB NO.: 22G127-1	DRILLING DATE: 3/9/22
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger
LOCATION: Winchester, California	LOGGED BY: Michelle Esparza

WATER DEPTH: Dry
CAVE DEPTH: 17½
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
	(hand)				FILL: Dark Gray Brown Silty fine Sand, little fine Root Fibers, medium dense-very moist		18				15.3	
	X	17			FILL: Gray Brown fine Sandy Silt, little Clay, trace medium Sand, trace fine Root Fibers, mottled, medium dense-moist	115	11				1.4	
5	X	16			ALLUVIUM: Brown fine Sandy Silt, medium dense-moist	110	12					
	X	22			Brown Silty fine to medium Sand, trace coarse Sand, medium dense-moist	112	11					
10	X	18			Gray Brown fine Sandy Silt, trace medium Sand, slightly porous, trace Calcareous nodules, medium dense-moist to very moist	113	14					
15	X	16			Brown Silty fine to medium Sand to fine to medium Sandy Silt, trace coarse Sand, medium dense-very moist		19					
20	X	18			Dark Brown fine Sandy Silt, trace medium Sand, medium dense-moist to very moist		14					

Boring Terminated at 20'

TEST BORING LOG **PLATE B-10**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO.
B-11

JOB NO.: 22G127-1	DRILLING DATE: 3/8/22
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger
LOCATION: Winchester, California	LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

FIELD RESULTS | LABORATORY RESULTS

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	⊠	43			<u>FILL:</u> Gray to Brown to Brown Silty fine Sand, little medium Sand, slightly cemented, mottled, medium dense-moist	119	8					
	⊠	18			<u>ALLUVIUM:</u> Gray Brown Silty fine Sand to fine Sandy Silt, little Clay, trace medium Sand, medium dense-very moist	109	18					
5	⊠	28			Gray Brown Silty fine Sand, trace medium Sand, medium dense-moist	114	10					
	⊠	33				116	9					
10	⊠	40			Light Gray fine to coarse Sand, medium dense-damp	114	4					
					Gray Brown Silty fine Sand, trace medium Sand, medium dense-moist							
15	⊠	40				121	11					

Boring Terminated at 15'

TEST BORING LOG

PLATE B-11

SOUTHERN CALIFORNIA GEOTECHNICAL
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SoCalGeo

JOB NO.: 22G127-1	DRILLING DATE: 3/8/22	WATER DEPTH: Dry
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 16 feet
LOCATION: Winchester, California	LOGGED BY: Jamie Hayward	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
	⊠	16			FILL: Brown Silty fine to coarse Sand, mottled, medium dense-damp to moist	112	6					
	⊠	16				113	8					
5	⊠	15			ALLUVIUM: Gray Brown fine to coarse Sand, trace Silt, medium dense-damp to moist	116	4					
	⊠	8			@ 7 feet, loose	105	6					
10	⊠	19			Gray Brown Silty fine Sand to fine Sandy Silt, medium dense-damp	113	6					
					Light Gray Brown fine to medium Sand, little coarse Sand, medium dense-damp							
15	⊠	29					4					
20	⊠	34			@ 18½ feet, dense		3					
					Gray Brown Silty fine to medium Sand, trace coarse Sand, dense-damp to moist							
25	⊠	32					7					
					Boring Terminated at 25'							

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
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SoCalGeo

JOB NO.: 22G127-1
PROJECT: Proposed Warehouse Development
LOCATION: Winchester, California

DRILLING DATE: 3/7/22
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: 41 feet
CAVE DEPTH: 46 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
	⊠	42			**FILL:** Gray Brown Silty fine Sand, trace medium Sand, weakly cemented, slightly mottled, dense-damp to moist		9					
	⊠	20					7					
5	⊠	16			**ALLUVIUM:** Light Gray Brown Silty fine Sand, medium dense-damp		6					
	⊠	16			Gray Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, medium dense-moist		9					
10												
	⊠	23			Gray Brown Silty fine to medium Sand, medium dense-moist		8					
15												
	⊠	37			@ 18½ feet, dense		8					
20												
	⊠	42			Gray Brown Silty fine Sand, dense-damp to very moist		6					
25												
	⊠	15			@ 28½', trace Clay		17			46		
30												
	⊠	17			Brown to Red Brown Silty fine Sand, trace Clay, trace medium Sand, very stiff-moist	16		23	15	49		

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
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SoCalGeo

JOB NO.: 22G127-1	DRILLING DATE: 3/7/22	WATER DEPTH: 41 feet
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 46 feet
LOCATION: Winchester, California	LOGGED BY: Jamie Hayward	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
					Brown to Red Brown Clayey Silt, little fine Sand, trace medium Sand, medium dense-moist							
40	⊠	27			Gray Brown to Brown Silty fine to medium Sand, trace fine to coarse Gravel, medium dense-moist		13			31		
45	⊠	27			Brown to Red Brown Silty fine Sand to fine Sandy Silt, trace to little Clay, medium dense-very moist to wet		16			47		
50	⊠	35			Brown fine Sandy Silt, trace medium sand, dense-medium dense to wet		18					

Boring Terminated at 50'

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG   **PLATE B-13b**

SOUTHERN CALIFORNIA GEOTECHNICAL
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SoCalGeo

JOB NO.: 22G127-1	DRILLING DATE: 3/9/22	WATER DEPTH: Dry
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 8½
LOCATION: Winchester, California	LOGGED BY: Michelle Esparza	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
	☒	4			FILL: Dark Brown fine Sandy Silt, trace medium Sand, trace fine Root Fibers, loose-moist		12					
	☒	13			FILL: Brown Silty fine to medium Sand, trace coarse Sand, medium dense-very moist		21					
5	☒	8					17					
	☒	10	2.5		ALLUVIUM: Dark Brown Clayey Silt to Silty Clay, loose to medium dense-very moist		30					
10												

Boring Terminated at 10'

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG   **PLATE B-14**

SOUTHERN CALIFORNIA GEOTECHNICAL
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SoCalGeo

JOB NO.: 22G127-1	DRILLING DATE: 3/7/22
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger
LOCATION: Winchester, California	LOGGED BY: Michelle Esparza

WATER DEPTH: Dry
CAVE DEPTH: 7 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
	☒	5			FILL: Gray Brown to Brown Silty fine Sand to fine Sandy Silt, trace coarse Sand, trace fine Root Fibers, loose to medium dense-damp to very moist		16					
	☒	8					8					
5	☒	18					10					
	☒	30			ALLUVIUM: Brown fine to medium Sand, trace coarse Sand, trace Silt, dense-damp		3					
10												

Boring Terminated at 10'

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG   **PLATE B-15**

SOUTHERN CALIFORNIA GEOTECHNICAL
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SoCalGeo

JOB NO.: 22G127-1	DRILLING DATE: 3/8/22	WATER DEPTH: Dry
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 8 feet
LOCATION: Winchester, California	LOGGED BY: Michelle Esparza	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
	⊠	9	3.5		FILL: Gray Brown fine Sandy Clay, little Silt, trace medium Sand, stiff-moist to very moist		13					
	⊠	7			FILL: Light Gray Brown fine to medium Sandy Silt, trace Clay, loose-very moist		18					
5	⊠	13			ALLUVIUM: Light Gray Brown Silty fine to medium Sand to fine to medium Sandy Silt, trace coarse Sand, medium dense-moist to very moist		12					
	⊠	18					10					
10					Boring Terminated at 10'							

TBL 22G127-1.GPJ SOCALGEO.GDT 5/2/22

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
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SoCalGeo

JOB NO.: 22G127-1	DRILLING DATE: 3/9/22
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger
LOCATION: Winchester, California	LOGGED BY: Michelle Esparza

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

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG		DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	⊠	9			**SURFACE ELEVATION: --- MSL** FILL: Dark Brown Silty fine Sand, trace medium Sand, trace coarse Sand, trace fine Root Fibers, loose-moist		8					
	⊠	8			FILL: Brown Silty fine to medium Sand, trace coarse Sand, loose-damp		7					
5	⊠	9			ALLUVIUM: Light Brown fine to coarse Sand, trace fine Silt, trace Gravelly Sand, loose-damp		7					
	⊠	15					4					
10					Boring Terminated at 10'							

TEST BORING LOG   **PLATE   B-17**

SOUTHERN CALIFORNIA GEOTECHNICAL
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BORING NO.
B-18

JOB NO.: 22G127-1	DRILLING DATE: 3/8/22	WATER DEPTH: Dry
PROJECT: Proposed Warehouse Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 9 feet
LOCATION: Winchester, California	LOGGED BY: Jamie Hayward	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
	⊠	7			FILL: Brown Silty fine Sand, trace medium Sand, loose-damp		5					
	⊠	22			FILL: Brown fine Sandy Silt, slightly mottled, medium dense-moist		15					
5	⊠	10			ALLUVIUM: Gray Brown fine Sandy Silt, trace Iron Oxide staining, trace Calcareous nodules/veining, loose to medium dense-moist to very moist		22					
	⊠	8					15					
10					Boring Terminated at 10'							

TEST BORING LOG

PLATE B-18

APPENDIX C

Consolidation/Collapse Test Results
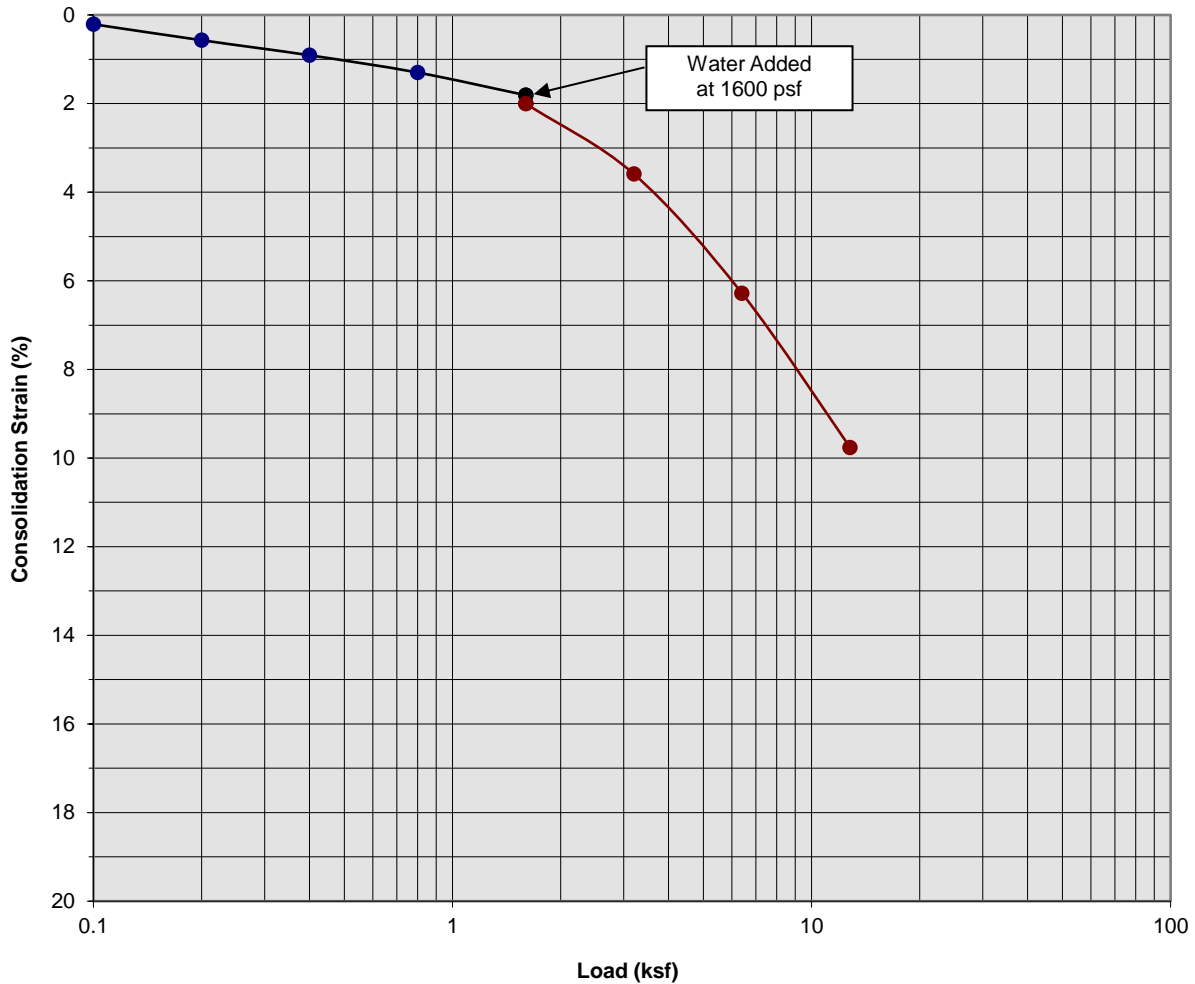
Consolidation Strain (%) vs Load (ksf)

Water Added at 1600 psf

Classification: FILL: Dark Brown Silty fine Sand, trace medium Sand

Boring Number:	B-5	Initial Moisture Content (%)	18
Sample Number:	---	Final Moisture Content (%)	24
Depth (ft)	3 to 4	Initial Dry Density (pcf)	97.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.19

Proposed Warehouse Development
Winchester, California
Project No. 22G127-1

PLATE C- 1

SoCalGeo

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Consolidation/Collapse Test Results

Consolidation Strain (%) vs Load (ksf). Data points with "Water Added at 1600 psf" annotation.
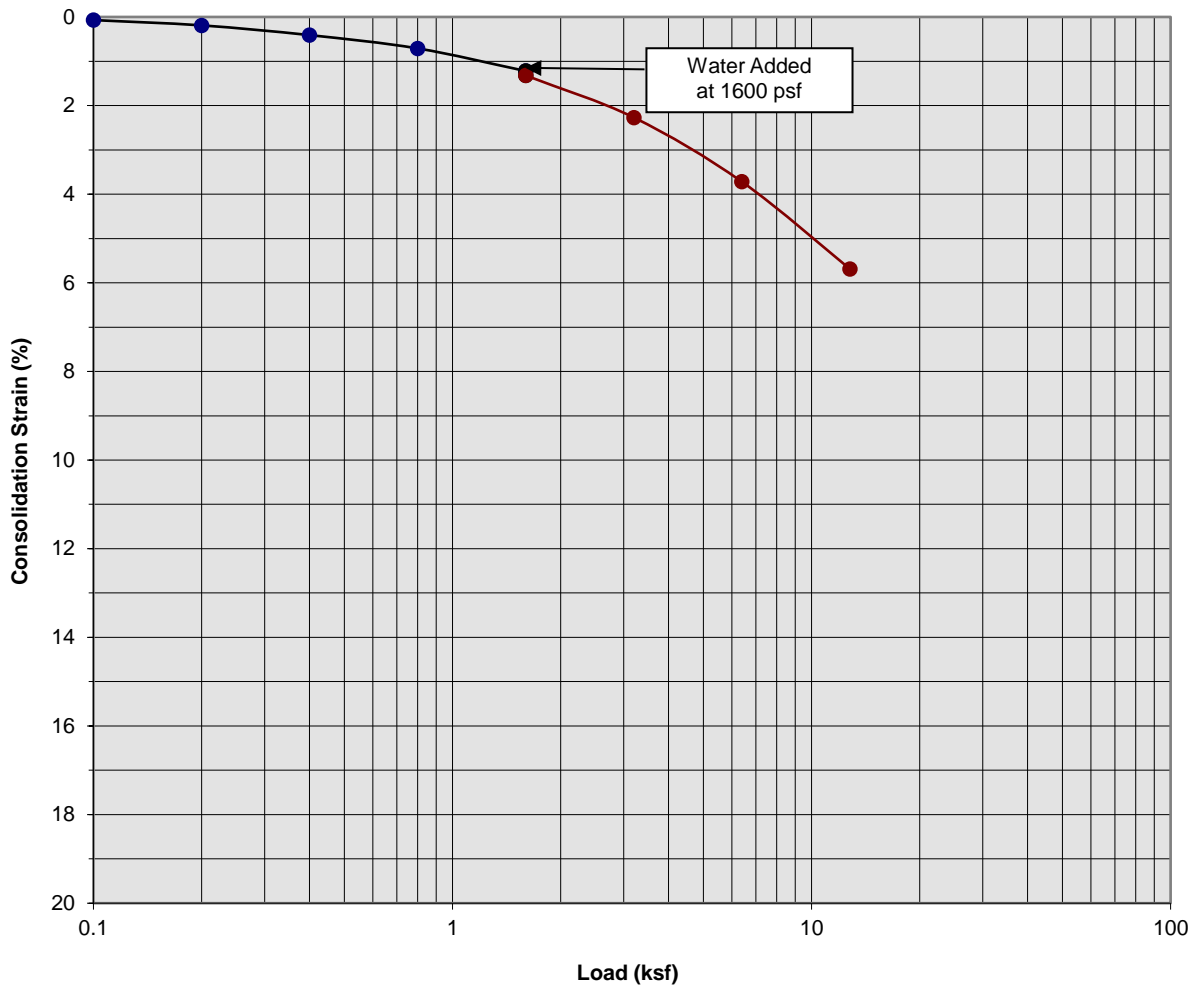
Classification: FILL: Dark Brown Silty fine Sand, trace medium Sand

Boring Number:	B-5	Initial Moisture Content (%)	13
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	5 to 6	Initial Dry Density (pcf)	92.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	98.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.10

Proposed Warehouse Development
Winchester, California
Project No. 22G127-1

PLATE C- 2

SoCalGeo

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Consolidation/Collapse Test Results

Water Added
at 1600 psf

Consolidation Strain (%)

Load (ksf)
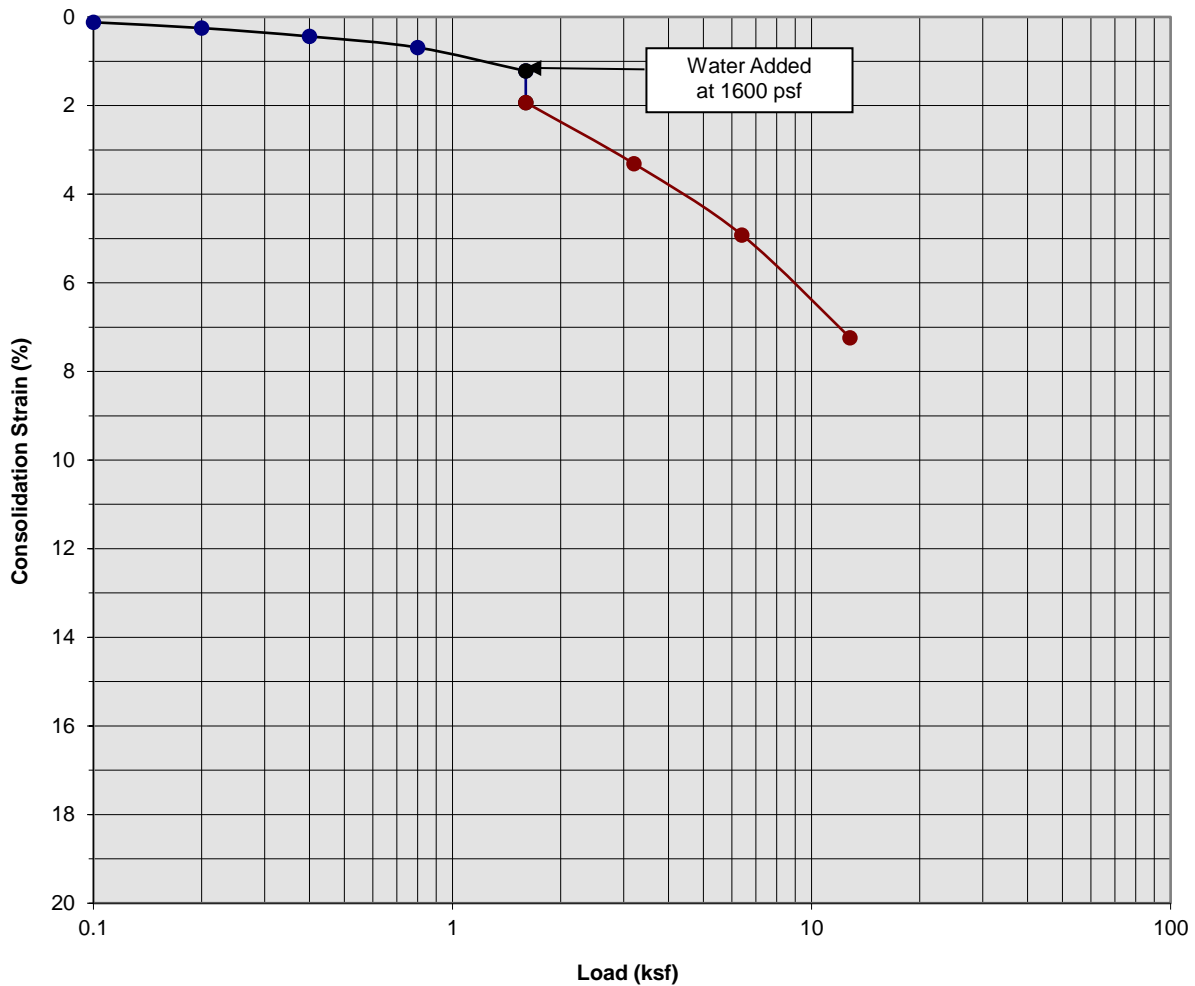
Classification: Brown fine Sandy Silt, trace to little medium to coarse Sand

Boring Number:	B-5	Initial Moisture Content (%)		11
Sample Number:	---	Final Moisture Content (%)		15
Depth (ft)	7 to 8	Initial Dry Density (pcf)		113.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)		122.5
Specimen Thickness (in)	1.0	Percent Collapse (%)		0.71

Proposed Warehouse Development
Winchester, California
Project No. 22G127-1

PLATE C- 3

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Consolidation/Collapse Test Results

Consolidation Strain (%)

Load (ksf)

Water Added at 1600 psf
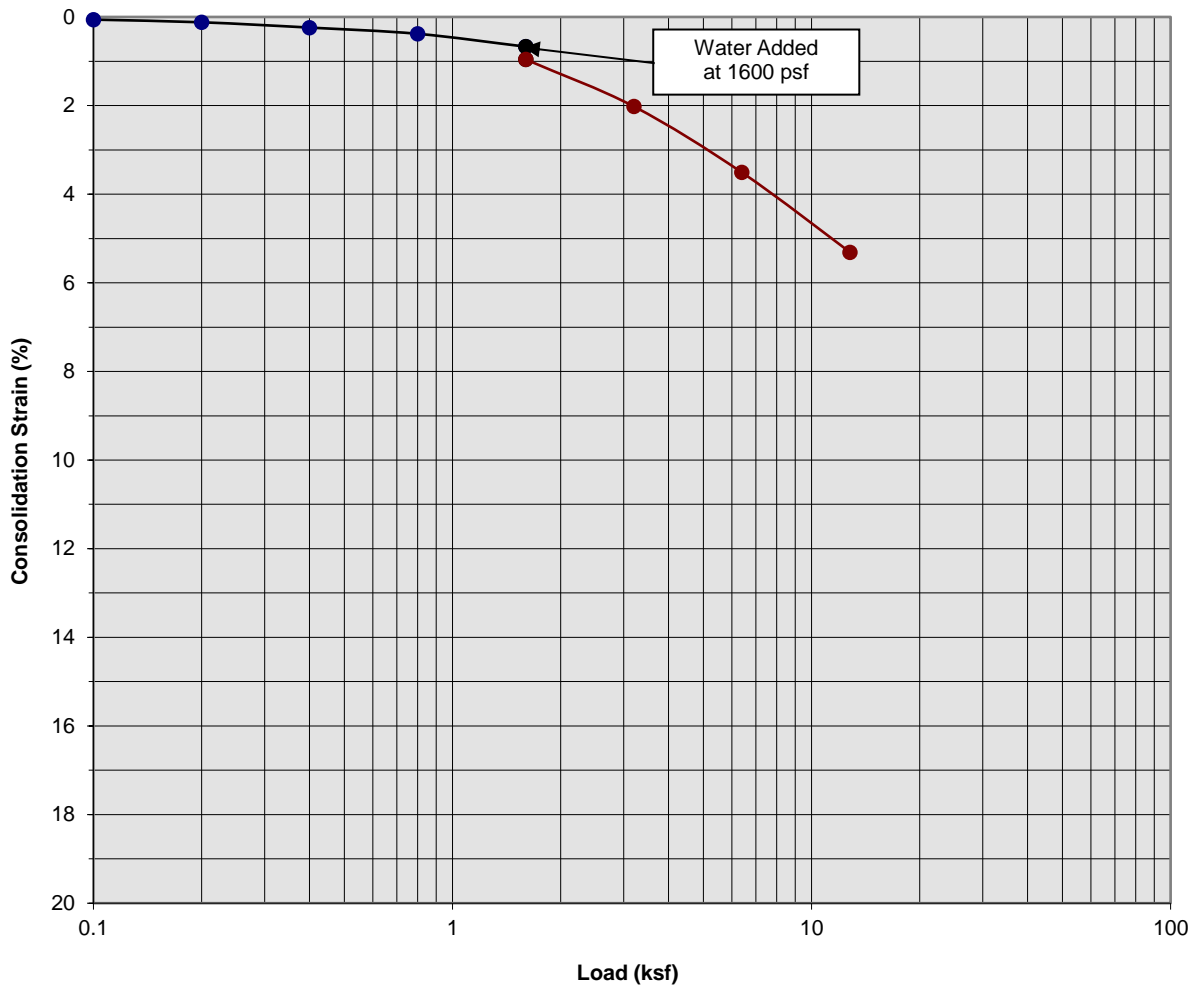
Classification: Brown fine Sandy Silt, trace to little medium to coarse Sand

Boring Number:	B-5	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	9 to 10	Initial Dry Density (pcf)	114.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	121.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.29

Proposed Warehouse Development
Winchester, California
Project No. 22G127-1
PLATE C- 4

SoCalGeo

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Consolidation/Collapse Test Results

Water Added
at 1600 psf

Consolidation Strain (%)

Load (ksf)
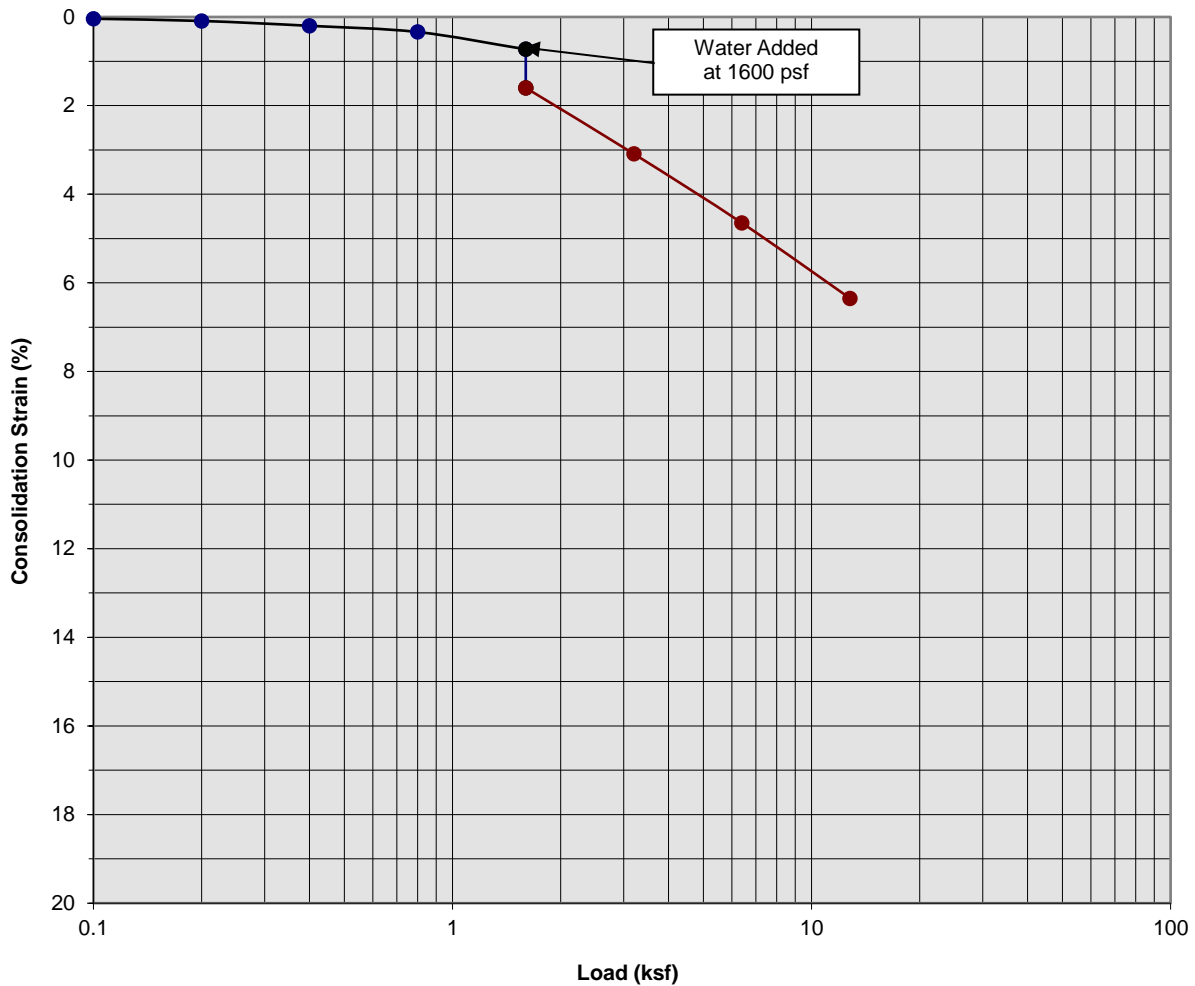
Classification: FILL: Brown Silty fine Sand

Boring Number:	B-12	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	3 to 4	Initial Dry Density (pcf)	113.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	120.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.87

Proposed Warehouse Development
Winchester, California
Project No. 22G127-1
PLATE C- 5

SoCalGeo

SOUTHERN
CALIFORNIA
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Consolidation/Collapse Test Results
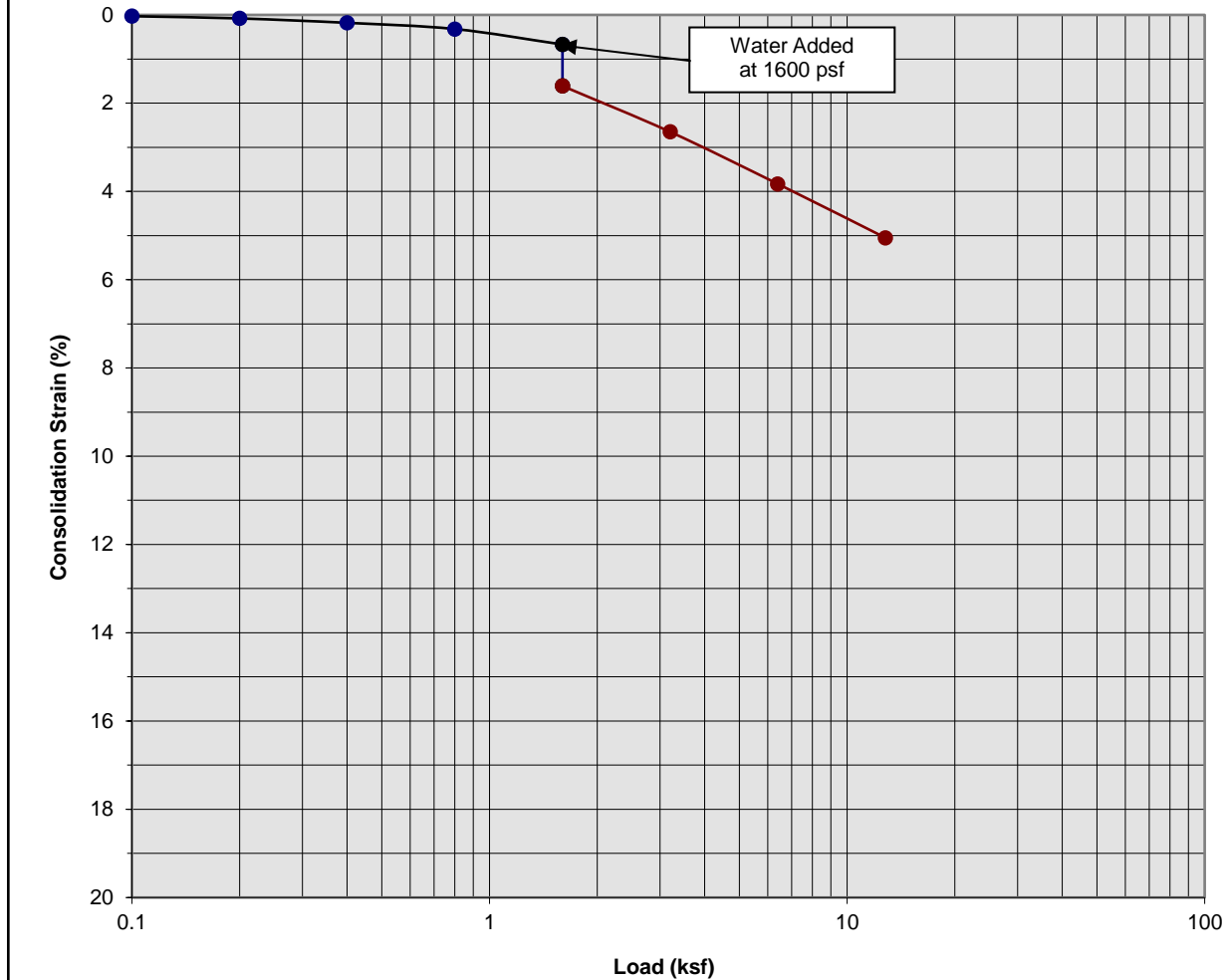
Classification: Gray Brown fine to coarse Sand, trace Silt

Boring Number:	B-12	Initial Moisture Content (%)	4
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	5 to 6	Initial Dry Density (pcf)	116.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	122.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.94

Proposed Warehouse Development
Winchester, California
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PLATE C- 6

SoCalGeo

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Consolidation/Collapse Test Results

Water Added at 1600 psf

Consolidation Strain (%) (y-axis: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20)

Load (ksf) (x-axis: 0.1, 1, 10, 100)
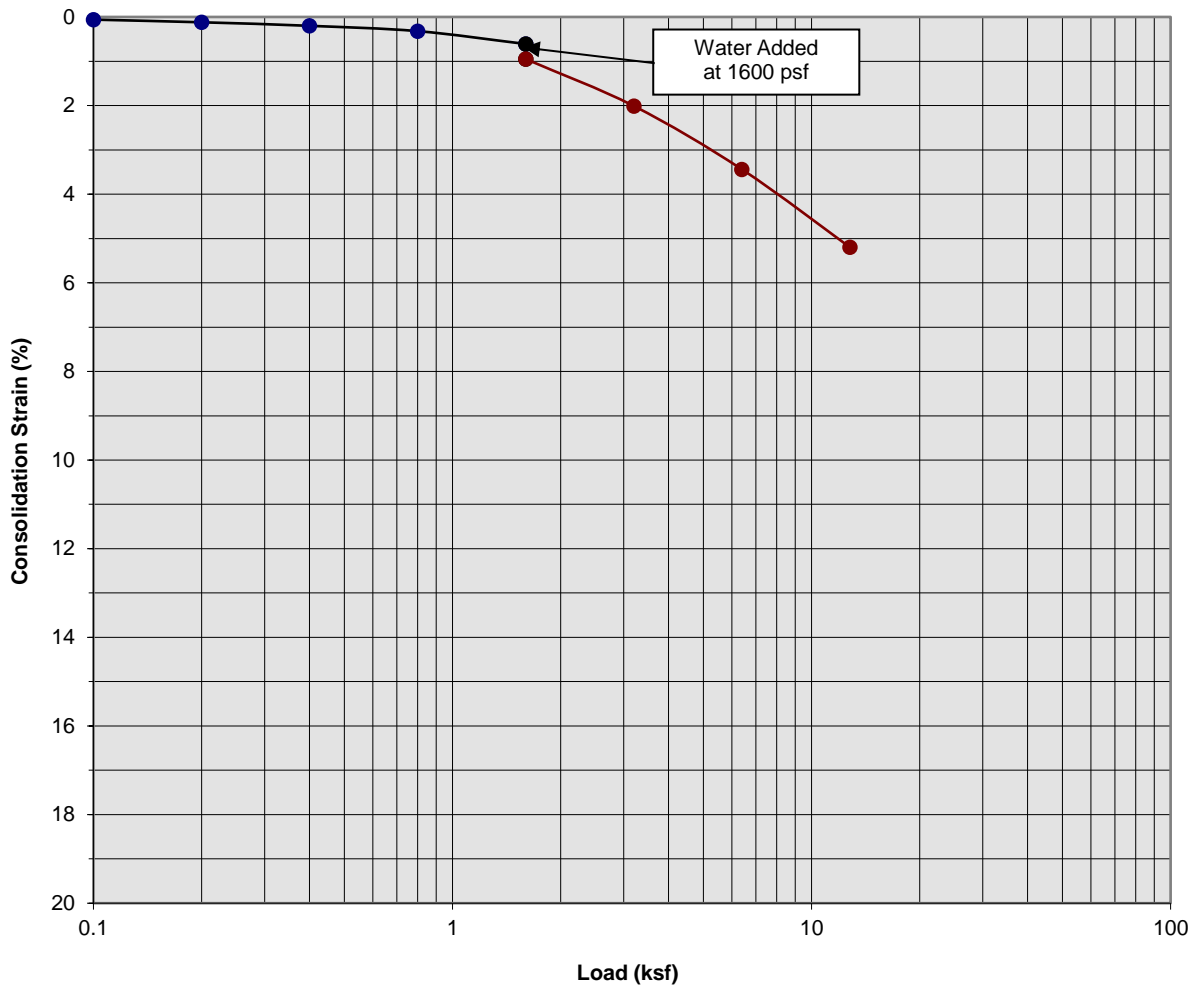
Classification: Gray Brown fine to coarse Sand, trace Silt

Boring Number:	B-12	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	7 to 8	Initial Dry Density (pcf)	105.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	110.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.34

Proposed Warehouse Development
Winchester, California
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PLATE C- 7

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Consolidation/Collapse Test Results

Chart: X-axis "Load (ksf)" logarithmic from 0.1 to 100. Y-axis "Consolidation Strain (%)" from 0 to 20. Annotation box: "Water Added at 1600 psf"
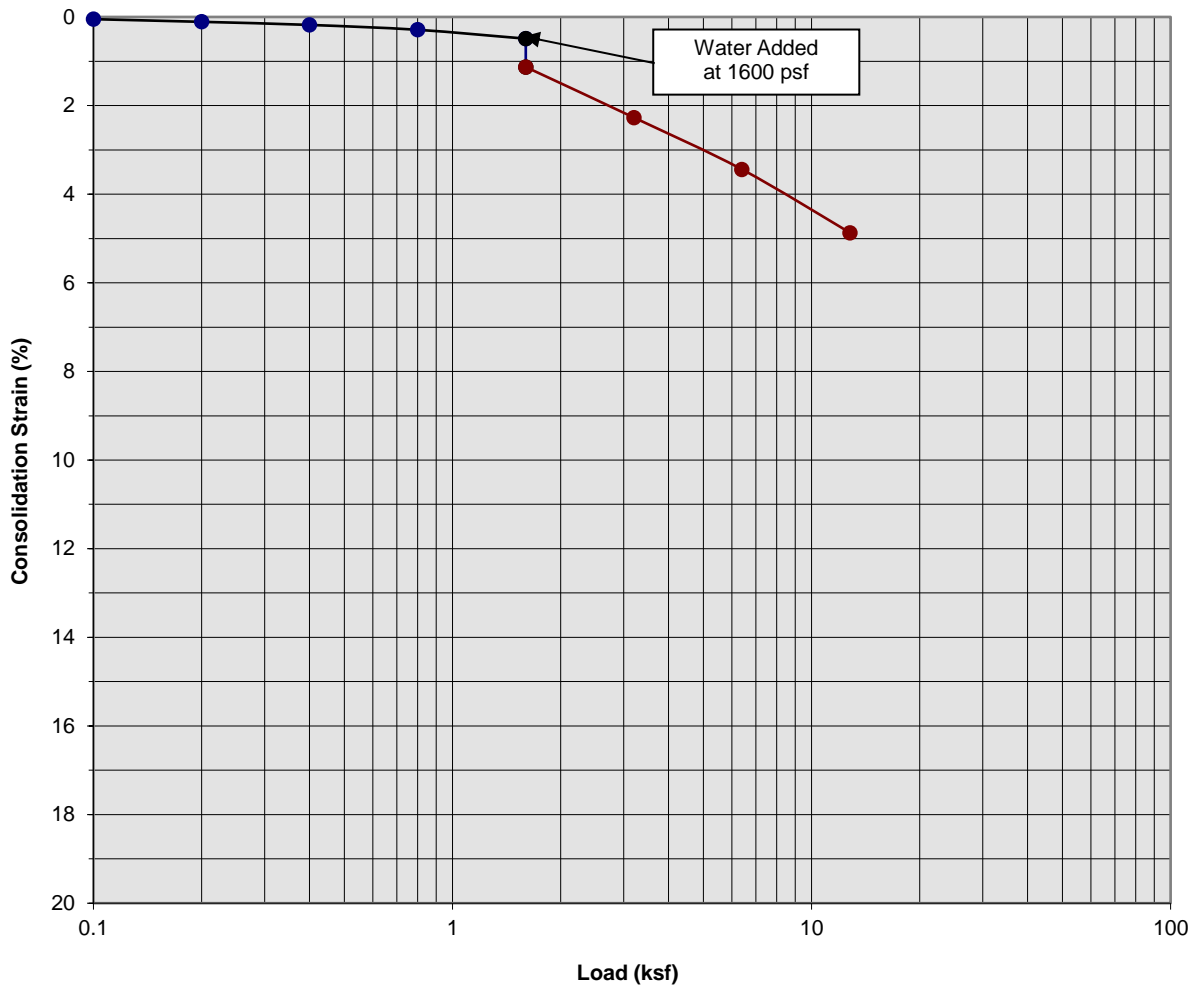
Classification: Gray Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-12	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	9 to 10	Initial Dry Density (pcf)	113.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	119.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.64

Proposed Warehouse Development
Winchester, California
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PLATE C- 8

Moisture/Density Relationship
ASTM D-1557

Zero Air Voids Curve:
Specific Gravity = 2.7

Dry Density (lbs/ft³)

Moisture Content (%)
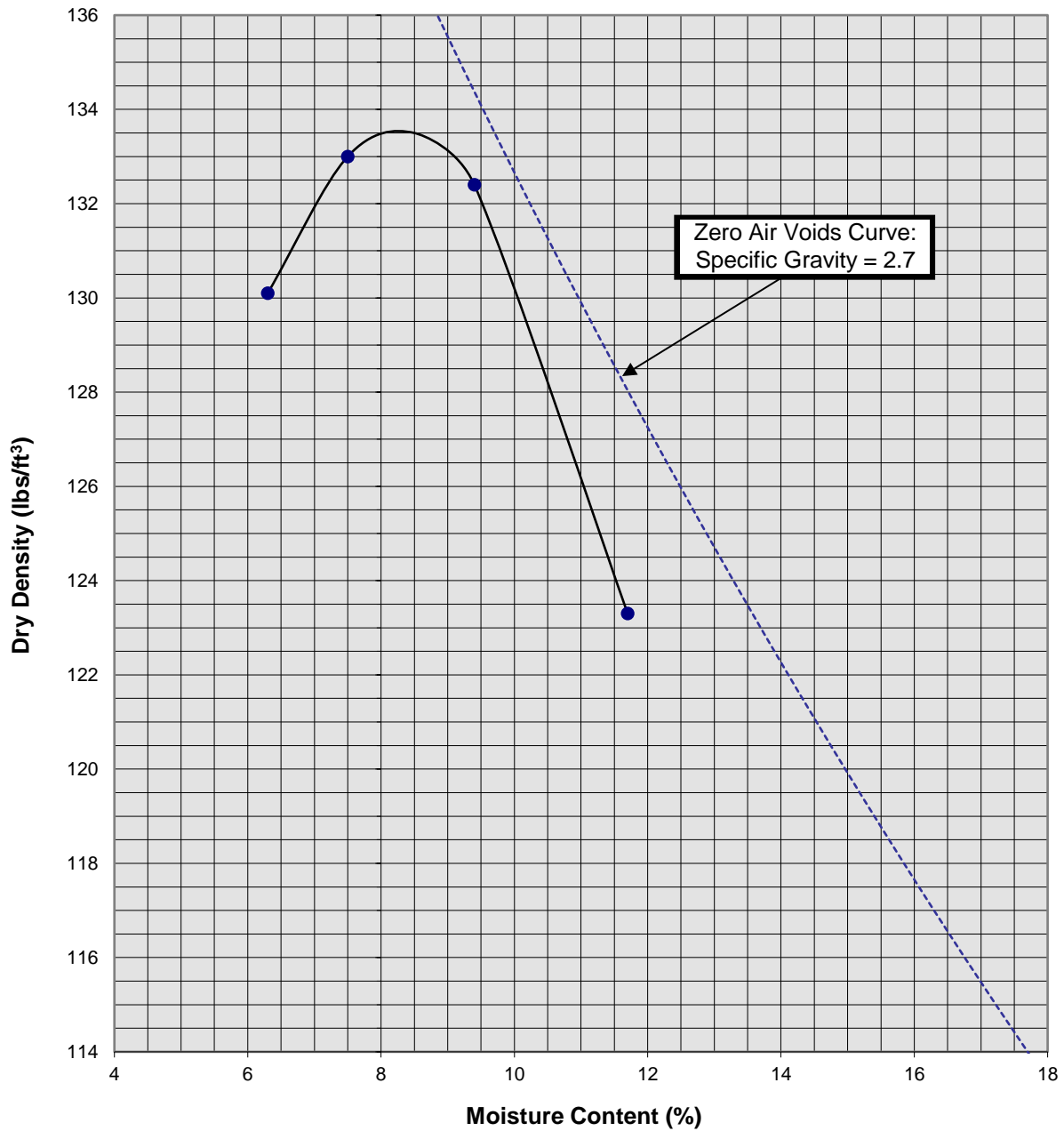
Soil ID Number	B-4 @ 0-5'
Optimum Moisture (%)	8.5
Maximum Dry Density (pcf)	133.5
Soil Classification	Brown Silty fine to medium Sand

Proposed Warehouse Development
Winchester, California
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PLATE C-9

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Moisture/Density Relationship
ASTM D-1557

Zero Air Voids Curve:
Specific Gravity = 2.7

Dry Density (lbs/ft³)

Moisture Content (%)
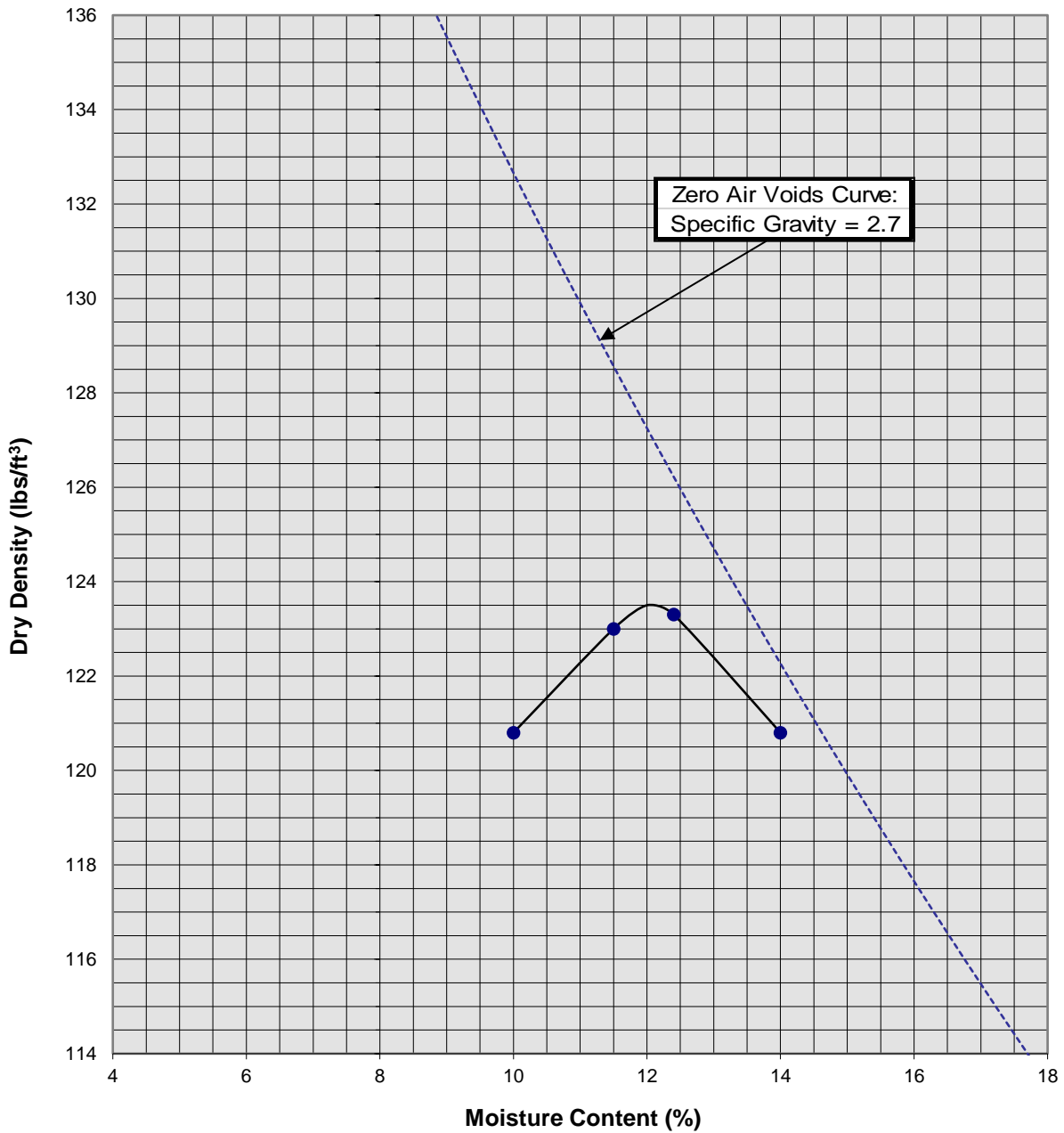
Soil ID Number	B-8 @ 0-5'
Optimum Moisture (%)	12
Maximum Dry Density (pcf)	123.5
Soil Classification	Dark Gray to Gray Brown Silty fine Sand, little Clay, trace medium Sand

Proposed Warehouse Development
Winchester, California
Project No. 22G127-1
PLATE C-10

SoCalGeo

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Moisture/Density Relationship
ASTM D-1557

Zero Air Voids Curve:
Specific Gravity = 2.7

Dry Density (lbs/ft³)

Moisture Content (%)
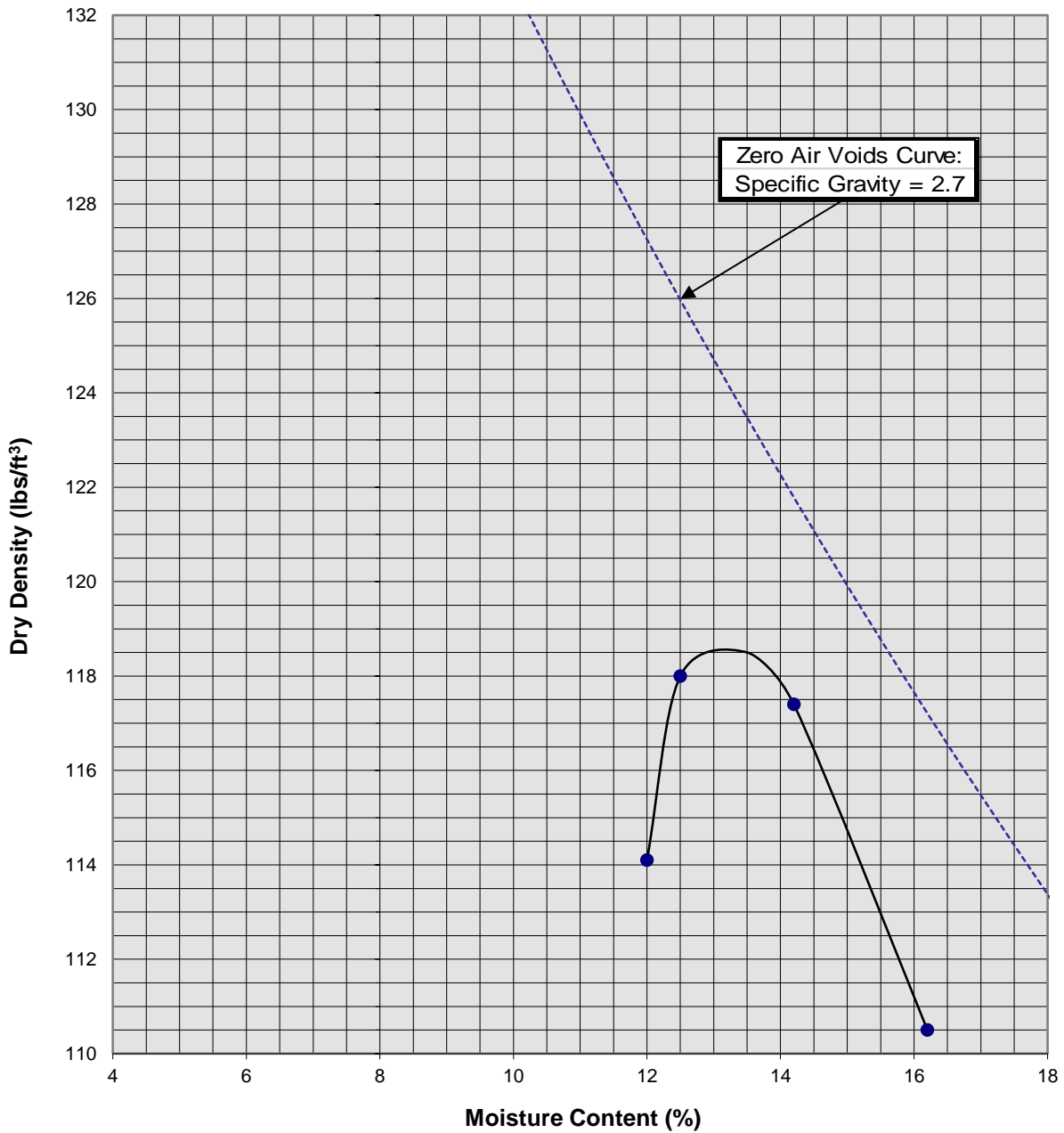
Soil ID Number	B-9 @ 0-5'
Optimum Moisture (%)	13.5
Maximum Dry Density (pcf)	118.5
Soil Classification	Gray Brown Silty fine Sand to fine Sandy Silt, trace medium Sand

Proposed Warehouse Development
Winchester, California
Project No. 22G127-1
PLATE C-11

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APPENDIX

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.

<u>General</u>

- 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.

<u>Site Preparation</u>

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

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

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

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

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

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

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

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

Compacted Fills

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

<u>Subdrains</u>

- 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

NATURAL GRADE

UNSUITABLE
MATERIAL

5' MIN.

3' MIN.*

COMPACTED FILL

OVEREXCAVATE AND
RECOMPACT

COMPETENT MATERIAL, AS APPROVED
BY THE GEOTECHNICAL ENGINEER

CUT/FILL LOT (TRANSITION)

NATURAL GRADE

5' MIN.

3' MIN.*

COMPACTED FILL

UNSUITABLE
MATERIAL

OVEREXCAVATE AND
RECOMPACT

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

COMPETENT MATERIAL, AS APPROVED
BY THE GEOTECHNICAL ENGINEER

*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		

NEW COMPACTED FILL

COMPETENT MATERIAL

CUT/FILL CONTACT SHOWN
ON GRADING PLAN

CUT/FILL CONTACT TO BE
SHOWN ON "AS-BUILT"

9' MIN.

REMOVE UNSUITABLE MATERIAL

NATURAL GRADE

10' TYP.

4' TYP.

CUT SLOPE

BENCHING DIMENSIONS IN ACCORDANCE
WITH PLAN OR AS RECOMMENDED
BY THE GEOTECHNICAL ENGINEER

MINIMUM 1' TILT BACK
OR 2% SLOPE
(WHICHEVER IS GREATER)

BEDROCK OR APPROVED
COMPETENT MATERIAL

CUT SLOPE TO BE CONSTRUCTED
PRIOR TO PLACEMENT OF FILL

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

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

NATURAL GROUND

COMPACTED FILL

CLEANOUT EXCAVATION

6" MIN.

FIRM NATIVE SOIL/BEDROCK

18" MIN.

24" MIN.

18" MIN.

4" MIN.

MINUS 1" CRUSHED ROCK COMPLETELY SURROUNDED BY FILTER FABRIC, OR CLASS II PERMEABLE MATERIAL

6" DIAMETER PERFORATED PIPE - MINIMUM 1% SLOPE
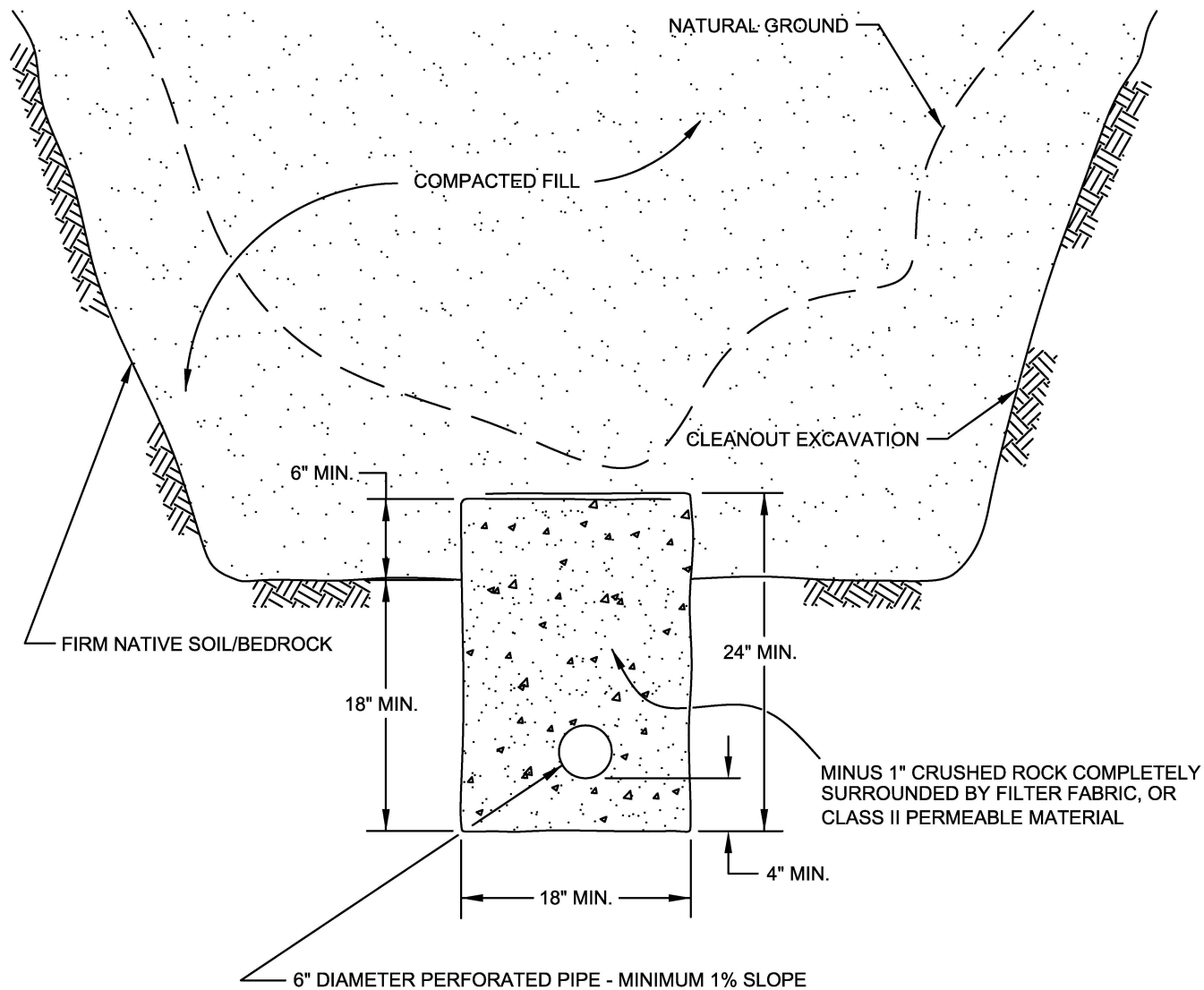
PIPE MATERIAL	DEPTH OF FILL OVER SUBDRAIN
ADS (CORRUGATED POLETHYLENE)	8
TRANSITE UNDERDRAIN	20
PVC OR ABS: SDR 35	35
SDR 21	100

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

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SOUTHERN CALIFORNIA GEOTECHNICAL

PLATE D-3

FINISHED SLOPE FACE

OVERFILL REQUIREMENTS
PER GRADING GUIDE SPECIFICATIONS

NEW COMPACTED FILL

COMPETENT MATERIAL

TOE OF SLOPE SHOWN
ON GRADING PLAN

PROJECT SLOPE GRADIENT
(1:1 MAX.)

PLACE COMPACTED BACKFILL
TO ORIGINAL GRADE

BACKCUT - VARIES

REMOVE UNSUITABLE MATERIAL

10' TYP.

4' TYP.

BENCHING DIMENSIONS IN ACCORDANCE
WITH PLAN OR AS RECOMMENDED
BY THE GEOTECHNICAL ENGINEER

2' MINIMUM
KEY DEPTH

MINIMUM 1' TILT BACK
OR 2% SLOPE
(WHICHEVER IS GREATER)

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

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

| **FILL ABOVE NATURAL SLOPE DETAIL** |
| GRADING GUIDE SPECIFICATIONS |

NOT TO SCALE	
DRAWN: JAS CHKD: GKM	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
PLATE D-4	

3' TYPICAL
BLANKET FILL IF RECOMMENDED
BY THE GEOTECHNICAL ENGINEER

TOP WIDTH OF FILL
AS SPECIFIED BY THE
GEOTECHNICAL ENGINEER

COMPETENT MATERIAL ACCEPTABLE
TO THE SOIL ENGINEER

COMPACTED FILL

FACE OF FINISHED SLOPE

10' TYP.

4' TYP.

BENCHING DIMENSIONS IN ACCORDANCE
WITH PLAN OR AS RECOMMENDED
BY THE GEOTECHNICAL ENGINEER

MINIMUM 1' TILT BACK
OR 2% SLOPE
(WHICHEVER IS GREATER)

2' MINIMUM
KEY DEPTH

KEYWAY WIDTH, AS SPECIFIED
BY THE GEOTECHNICAL ENGINEER

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

DESIGN FINISH SLOPE

OUTLETS TO BE SPACED
AT 100' MAXIMUM INTERVALS.
EXTEND 12 INCHES
BEYOND FACE OF SLOPE
AT TIME OF ROUGH GRADING
CONSTRUCTION.

BLANKET FILL IF RECOMMENDED
BY THE GEOTECHNICAL ENGINEER

BUTTRESS OR
SIDEHILL FILL

10' MIN.
25' MAX.

15' MAX.

DETAIL "A"

2%

4-INCH DIAMETER NON-PERFORATED
OUTLET PIPE TO BE LOCATED IN FIELD
BY THE SOIL ENGINEER.

2' CLEAR

"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 CON-
NECTED TO SUBDRAIN PIPE
WITH TEE OR ELBOW
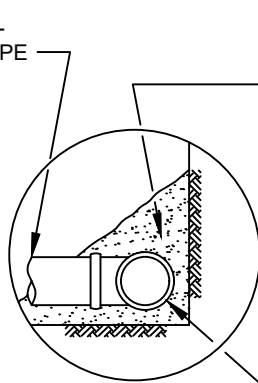
FILTER MATERIAL - MINIMUM OF FIVE
CUBIC FEET PER FOOT OF PIPE. SEE
ABOVE FOR FILTER MATERIAL SPECIFICATION.

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.

DETAIL "A"

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.

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

WATERPROOFING AT FACE OF WALL IN ACCORDANCE
WITH ARCHITECTURAL AND/OR STRUCTURAL DETAILS

MINIMUM ONE FOOT THICK LAYER OF
LOW PERMEABLILITY SOIL IF NOT
COVERED WITH AN IMPERMEABLE SURFACE

MINIMUM ONE FOOT WIDE LAYER OF
FREE DRAINING MATERIAL
(LESS THAN 5% PASSING THE #200 SIEVE)
OR
PROPERLY INSTALLED PREFABRICATED DRAINAGE COMPOSITE
(MiraDRAIN 6000 OR APPROVED EQUIVALENT).
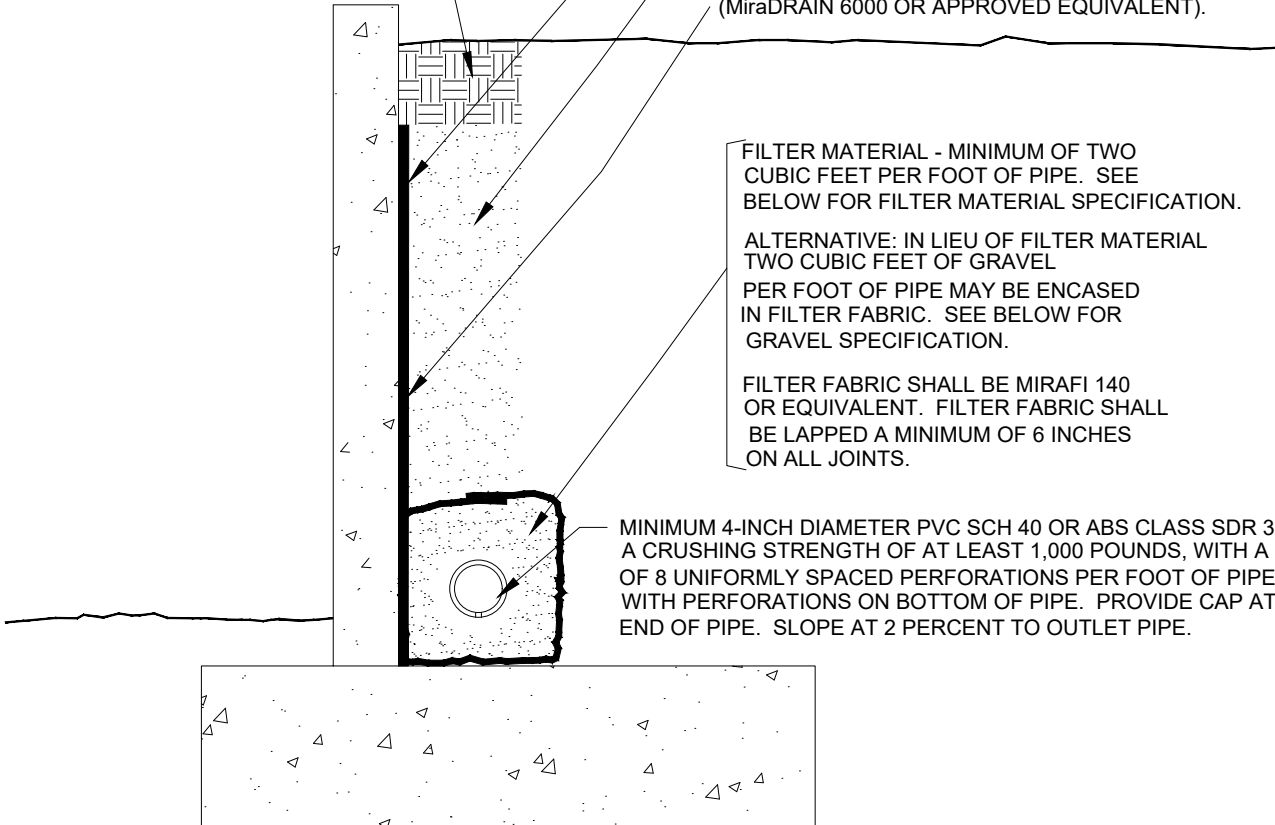
FILTER MATERIAL - MINIMUM OF TWO
CUBIC FEET PER FOOT OF PIPE. SEE
BELOW FOR FILTER MATERIAL SPECIFICATION.

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

FILTER FABRIC SHALL BE MIRAFI 140
OR EQUIVALENT. FILTER FABRIC SHALL
BE LAPPED A MINIMUM OF 6 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.

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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR
APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL

Section View

10 FEET MINIMUM

15 FEET MINIMUM

5 FEET MINIMUM OFFSET

3 FEET MINIMUM

15 FEET MINIMUM

Typical Row of Oversize Rock Fragments

Plan View

Typical Row of Oversize Rock Fragments

15 FEET MINIMUM

Fill Slope

APPENDIX E

Latitude, Longitude: 33.705684, -117.03764
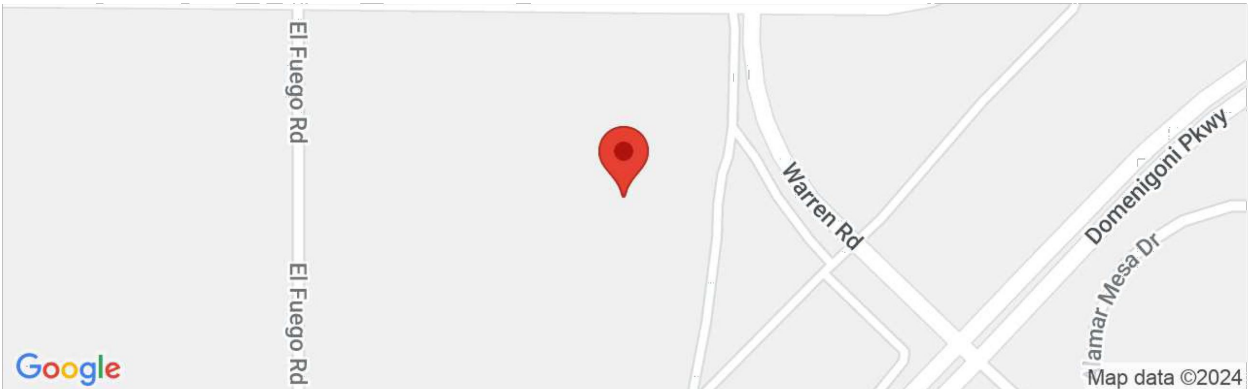
Date	4/10/2024, 3:21:59 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.6	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1	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	0.571	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.628	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.592	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.739	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.605	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.674	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.571	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.695	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.916	Mapped value of the risk coefficient at short periods

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

SEISMIC DESIGN PARAMETERS - 2022 CBC
PROPOSED WAREHOUSE DEVELOPMENT
RIVERSIDE COUNTY (WINCHESTER), CALIFORNIA

DRAWN: MK CHKD: DN	SoCalGeo	SOUTHERN CALIFORNIA GEOTECHNICAL
SCG PROJECT 22G127-1R		
PLATE E-1		

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Warehouse Development	
Project Location	Winchester, California	
Project Number	22G127-1	
Engineer	DWN	

MCE_G Design Acceleration	0.571	(g)
Design Magnitude	7.06	
Historic High Depth to Groundwater	30	(ft)
Depth to Groundwater at Time of Drilling	40	(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=7.06)	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	30	15		120		1.3	1.05	1.1	1.14	0.75	0.0	0.0	1800	1800	1800	0.95	1.01	1.01	0.06	0.06	N/A	N/A	Above Water Table
29.5	30	32	31	14	120	32	1.3	1.05	1.167	0.79	0.95	16.7	22.1	3720	3658	3720	0.88	1.09	0.92	0.23	0.24	0.33	0.71	**Liquefiable**
34.5	32	37	34.5	20	120	24	1.3	1.05	1.276	0.79	1	27.6	32.6	4140	3859	4140	0.86	1.18	0.86	0.71	0.72	0.34	2.11	Nonliquefiable
39.5	37	42	39.5	19	120	34	1.3	1.05	1.239	0.74	1	23.9	29.4	4740	4147	4740	0.83	1.15	0.87	0.45	0.45	0.35	1.26	**Liquefiable**
44.5	42	47	44.5	20	120	51	1.3	1.05	1.249	0.73	1	24.9	30.5	5340	4435	5059	0.80	1.16	0.84	0.52	0.51	0.36	1.42	Nonliquefiable
49.5	47	50	48.5	14	120	72	1.3	1.05	1.147	0.67	1	14.7	20.2	5820	4666	5290	0.78	1.08	0.89	0.21	0.20	0.36	0.55	**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	Warehouse Development
Project Location	Winchester, California
Project Number	22G127-1
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 content	$(N_1)_{60-cs}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	30	15	0.0	0.0	0.0	N/A	0.50	0.95	0.00	30.00		0.000		0.00	Above Water Table
29.5	30	32	31	16.7	5.4	22.1	0.71	0.12	0.40	0.09	2.00		0.021		0.51	**Liquefiable**
34.5	32	37	34.5	27.6	5.0	32.6	2.11	0.03	-0.27	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	23.9	5.5	29.4	1.26	0.05	-0.05	0.02	5.00		0.004		0.25	**Liquefiable**
44.5	42	47	44.5	24.9	5.6	30.5	1.42	0.04	-0.13	0.01	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	14.7	5.6	20.2	0.55	0.15	0.51	0.15	3.00		0.023		0.82	**Liquefiable**
											Total Deformation (in)			1.58	

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	Warehouse Development
Project Location	Winchester, California
Project Number	22G127-1
Engineer	DWN

MCE_G Design Acceleration	0.571 (g)
Design Magnitude	7.06
Historic High Depth to Groundwater	30 (ft)
Depth to Groundwater at Time of Drilling	36 (ft)
Borehole Diameter	6 (in)

Boring No. **B-3**

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60cs}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	K_σ	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.06)	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	30	15		120		1.3	1.05	1.1	1.14	0.75	0.0	0.0	1800	1800	1800	0.95	1.01	1.01	0.06	0.06	N/A	N/A	Above Water Table
29.5	30	32	31	12	120	40	1.3	1.05	1.138	0.78	0.95	13.8	19.3	3720	3658	3720	0.88	1.07	0.93	0.20	0.20	0.33	0.60	**Liquefiable**
34.5	32	37	34.5	13	120	56	1.3	1.05	1.153	0.75	1	15.3	20.9	4140	3859	4140	0.86	1.08	0.92	0.22	0.22	0.34	0.63	**Liquefiable**
39.5	37	42	39.5	16	120	58	1.3	1.05	1.192	0.74	1	19.2	24.8	4740	4147	4522	0.83	1.11	0.89	0.29	0.28	0.35	0.80	**Liquefiable**
44.5	42	47	44.5	19	120	49	1.3	1.05	1.237	0.74	1	23.7	29.3	5340	4435	4810	0.80	1.15	0.85	0.45	0.44	0.36	1.22	**Liquefiable**
49.5	47	50	48.5	20	120	47	1.3	1.05	1.25	0.73	1	25.0	30.6	5820	4666	5040	0.78	1.16	0.83	0.52	0.51	0.36	1.40	Nonliquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}
(2) Borehole Diameter Correction (Skempton, 1986)
(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)
(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)
(5) Rod Length Correction for Samples <10 m in depth
(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden
(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)
(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)
(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)
(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)
(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)
(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Warehouse Development
Project Location	Winchester, California
Project Number	22G127-1
Engineer	DWN

Boring No. **B-3**

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60-CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	30	15	0.0	0.0	0.0	N/A	0.50	0.95	0.00	30.00		0.000		0.00	Above Water Table
29.5	30	32	31	13.8	5.6	19.3	0.60	0.17	0.55	0.17	2.00		0.024		0.57	**Liquefiable**
34.5	32	37	34.5	15.3	5.6	20.9	0.63	0.14	0.47	0.14	5.00		0.022		1.33	**Liquefiable**
39.5	37	42	39.5	19.2	5.6	24.8	0.80	0.09	0.24	0.06	5.00		0.014		0.82	**Liquefiable**
44.5	42	47	44.5	23.7	5.6	29.3	1.22	0.05	-0.04	0.02	5.00		0.005		0.28	**Liquefiable**
49.5	47	50	48.5	25.0	5.6	30.6	1.40	0.04	-0.13	0.02	3.00		0.000		0.00	Nonliquefiable
													Total Deformation (in)		3.00	

Notes:

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

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

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

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

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

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

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

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

LIQUEFACTION EVALUATION

Project Name	Warehouse Development		MCE$_G$ Design Acceleration	0.571	(g)
Project Location	Winchester, California		Design Magnitude	7.06	
Project Number	22G127-1		Historic High Depth to Groundwater	30	(ft)
Engineer	DWN		Depth to Groundwater at Time of Drilling	40	(ft)
			Borehole Diameter	6	(in)

Boring No. **B-6**

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=7.06)	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	30	15		120		1.3	1.05	1.1	1.14	0.75	0.0	0.0	1800	1800	1800	0.95	1.01	1.01	0.06	0.06	N/A	N/A	Above Water Table
29.5	30	32	31	25	120	35	1.3	1.05	1.3	0.85	0.95	35.8	41.3	3720	3658	3720	0.88	1.18	0.84	2.00	1.98	0.33	5.99	Nonliquefiable
34.5	32	37	34.5	22	120	36	1.3	1.05	1.3	0.81	1	31.6	37.1	4140	3859	4140	0.86	1.18	0.82	1.79	1.74	0.34	5.10	Nonliquefiable
39.5	37	42	39.5	34	120		1.3	1.05	1.3	0.82	1	49.6	49.6	4740	4147	4740	0.83	1.18	0.8	2.00	1.89	0.35	5.37	Nonliquefiable
44.5	42	47	44.5	23	120	31	1.3	1.05	1.3	0.76	1	30.8	36.2	5340	4435	5059	0.80	1.18	0.79	1.45	1.36	0.36	3.79	Nonliquefiable
49.5	47	50	48.5	37	120		1.3	1.05	1.3	0.82	1	53.6	53.6	5820	4666	5290	0.78	1.18	0.76	2.00	1.81	0.36	5.00	Nonliquefiable

Notes:

(1) Energy Correction for N$_{90}$ of automatic hammer to standard N$_{60}$

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name | Warehouse Development
Project Location | Winchester, California
Project Number | 22G127-1
Engineer | DWN

Boring No.   B-6

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60-cs}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	30	15	0.0	0.0	0.0	N/A	0.50	0.95	0.00	30.00		0.000		0.00	Above Water Table
29.5	30	32	31	35.8	5.5	41.3	5.99	0.01	-0.90	0.00	2.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	31.6	5.5	37.1	5.10	0.02	-0.59	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	49.6	0.0	49.6	5.37	0.00	-1.56	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	30.8	5.4	36.2	3.79	0.02	-0.52	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	53.6	0.0	53.6	5.00	0.00	-1.88	0.00	3.00		0.000		0.00	Nonliquefiable
												Total Deformation (in)		0.00	

Notes:

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

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

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

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

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

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

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

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

LIQUEFACTION EVALUATION

Project Name	Warehouse Development	
Project Location	Winchester, California	
Project Number	22G127-1	
Engineer	DWN	

MCE$_G$ Design Acceleration	0.571	(g)
Design Magnitude	7.06	
Historic High Depth to Groundwater	30	(ft)
Depth to Groundwater at Time of Drilling	34	(ft)
Borehole Diameter	6	(in)

Boring No. B-8

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	K_s	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.06)	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	30	15		120		1.3	1.05	1.1	1.14	0.75	0.0	0.0	1800	1800	1800	0.95	1.01	1.01	0.06	0.06	N/A	N/A	Above Water Table
29.5	30	32	31	24	120	35	1.3	1.05	1.3	0.84	0.95	34.2	39.7	3720	3658	3720	0.88	1.18	0.84	2.00	1.98	0.33	5.99	Nonliquefiable
34.5	32	37	34.5	33	120		1.3	1.05	1.3	0.85	1	49.9	49.9	4140	3859	4109	0.86	1.18	0.82	2.00	1.94	0.34	5.69	Nonliquefiable
39.5	37	42	39.5	19	120	26	1.3	1.05	1.248	0.77	1	24.8	30.0	4740	4147	4397	0.83	1.15	0.86	0.48	0.48	0.35	1.36	Nonliquefiable
44.5	42	47	44.5	29	120		1.3	1.05	1.3	0.79	1	40.7	40.7	5340	4435	4685	0.80	1.18	0.78	2.00	1.85	0.36	5.14	Nonliquefiable
49.5	47	50	48.5	30	120		1.3	1.05	1.3	0.78	1	41.8	41.8	5820	4666	4915	0.78	1.18	0.76	2.00	1.81	0.36	5.00	Nonliquefiable

Notes:

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

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name Warehouse Development
Project Location Winchester, California
Project Number 22G127-1
Engineer DWN

Boring No. B-8

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60-CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ϵ_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	30	15	0.0	0.0	0.0	N/A	0.50	0.95	0.00	30.00		0.000		0.00	Above Water Table
29.5	30	32	31	34.2	5.5	39.7	5.99	0.01	-0.78	0.00	2.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	49.9	0.0	49.9	5.69	0.00	-1.58	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	24.8	5.1	30.0	1.36	0.05	-0.09	0.02	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	40.7	0.0	40.7	5.14	0.01	-0.86	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	41.8	0.0	41.8	5.00	0.01	-0.94	0.00	3.00		0.000		0.00	Nonliquefiable
										Total Deformation (in)			0.00		

Notes:

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

LIQUEFACTION EVALUATION

Project Name	Warehouse Development
Project Location	Winchester, California
Project Number	22G127-1
Engineer	DWN
Boring No.	B-9

MCE_G Design Acceleration	0.571 (g)
Design Magnitude	7.06
Historic High Depth to Groundwater	30 (ft)
Depth to Groundwater at Time of Drilling	40 (ft)
Borehole Diameter	6 (in)

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction (1)	C_B (2)	C_S (3)	C_N (4)	Rod Length Correction (5)	$(N_1)_{60}$ (6)	$(N_1)_{60cs}$ (7)	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) (8)	MSF (9)	K_s (10)	Cyclic Resistance Ratio (M=7.5) (11)	Cyclic Resistance Ratio (M=7.06) (12)	Cyclic Stress Ratio Induced by Design Earthquake (13)	Factor of Safety	Comments
7	0	30	15		120		1.3	1.05	1.1	1.14	0.75	0.0	0.0	1800	1800	1800	0.95	1.01	1.01	0.06	0.06	N/A	N/A	Above Water Table
29.5	30	32	31	21	120	23	1.3	1.05	1.291	0.83	0.95	29.1	34.0	3720	3658	3720	0.88	1.18	0.86	0.90	0.92	0.33	2.78	Nonliquefiable
34.5	32	37	34.5	23	120	25	1.3	1.05	1.3	0.81	1	33.2	38.2	4140	3859	4140	0.86	1.18	0.82	2.00	1.94	0.34	5.69	Nonliquefiable
39.5	37	42	39.5	43	120		1.3	1.05	1.3	0.88	1	67.4	67.4	4740	4147	4740	0.83	1.18	0.8	2.00	1.89	0.35	5.37	Nonliquefiable
44.5	42	47	44.5	18	120	53	1.3	1.05	1.213	0.71	1	21.3	26.9	5340	4435	5059	0.80	1.13	0.87	0.34	0.34	0.36	0.94	**Liquefiable**
49.5	47	50	48.5	25	120	44	1.3	1.05	1.3	0.76	1	33.6	39.2	5820	4666	5290	0.78	1.18	0.76	2.00	1.81	0.36	5.00	Nonliquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}
(2) Borehole Diameter Correction (Skempton, 1986)
(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)
(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)
(5) Rod Length Correction for Samples <10 m in depth
(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden
(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)
(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)
(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)
(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)
(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)
(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Warehouse Development
Project Location	Winchester, California
Project Number	22G127-1
Engineer	DWN

Boring No. B-9

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60-CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	30	15	0.0	0.0	0.0	N/A	0.50	0.95	0.00	30.00		0.000		0.00	Above Water Table
29.5	30	32	31	29.1	4.9	34.0	2.78	0.03	-0.36	0.00	2.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	33.2	5.1	38.2	5.69	0.01	-0.67	0.00	5.00		0.000		0.00	**Nonliquefiable**
39.5	37	42	39.5	67.4	0.0	67.4	5.37	0.00	-3.07	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	21.3	5.6	26.9	0.94	0.07	0.11	0.04	5.00		0.009		0.53	Liquefiable
49.5	47	50	48.5	33.6	5.6	39.2	5.00	0.01	-0.74	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)			0.53	

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	Warehouse Development	
Project Location	Winchester, California	
Project Number	22G127-1	
Engineer	DWN	

Boring No.	B-13	

MCE$_G$ Design Acceleration	0.571 (g)
Design Magnitude	7.06
Historic High Depth to Groundwater	30 (ft)
Depth to Groundwater at Time of Drilling	41 (ft)
Borehole Diameter	6 (in)

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60cs}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	K_σ	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.06)	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	30	15		120		1.3	1.05	1.1	1.14	0.75	0.0	0.0	1800	1800	1800	0.95	1.01	1.01	0.06	0.06	N/A	N/A	Above Water Table
29.5	30	32	31	15	120	46	1.3	1.05	1.183	0.79	0.95	18.3	23.9	3720	3658	3720	0.88	1.10	0.91	0.27	0.27	0.33	0.81	**Liquefiable**
34.5	32	37	34.5	17	120	49	1.3	1.05	1.219	0.77	1	21.9	27.5	4140	3859	4140	0.86	1.13	0.89	0.36	0.37	0.34	1.08	**Liquefiable**
39.5	37	42	39.5	27	120	31	1.3	1.05	1.3	0.80	1	38.3	43.8	4740	4147	4740	0.83	1.18	0.8	2.00	1.89	0.35	5.37	Nonliquefiable
44.5	42	47	44.5	27	120	47	1.3	1.05	1.3	0.78	1	37.4	43.0	5340	4435	5122	0.80	1.18	0.78	2.00	1.85	0.36	5.14	Nonliquefiable
49.5	47	50	48.5	35	120		1.3	1.05	1.3	0.80	1	49.6	49.6	5820	4666	5352	0.78	1.18	0.76	2.00	1.81	0.36	5.00	Nonliquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}
(2) Borehole Diameter Correction (Skempton, 1986)
(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)
(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)
(5) Rod Length Correction for Samples <10 m in depth
(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden
(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)
(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)
(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)
(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)
(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)
(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name   Warehouse Development
Project Location Winchester, California
Project Number   22G127-1
Engineer   DWN

Boring No.   B-13

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60-cs}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ϵ_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	30	15	0.0	0.0	0.0	N/A	0.50	0.95	0.00	30.00		0.000		0.00	Above Water Table
29.5	30	32	31	18.3	5.6	23.9	0.81	0.10	0.30	0.06	2.00		0.014		0.34	**Liquefiable**
34.5	32	37	34.5	21.9	5.6	27.5	1.08	0.06	0.07	0.03	5.00		0.006		0.39	**Liquefiable**
39.5	37	42	39.5	38.3	5.4	43.8	5.37	0.00	-1.09	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	37.4	5.6	43.0	5.14	0.00	-1.03	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	49.6	0.0	49.6	5.00	0.00	-1.55	0.00	3.00		0.000		0.00	Nonliquefiable
												Total Deformation (in)		0.73	

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)