

**GEOTECHNICAL INVESTIGATION
PROPOSED WAREHOUSE**

NEC Patterson Avenue and Nance Street
Perris, California
for
Duke Realty



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

December 22, 2020
(Report updated August 6, 2021 and December 13, 2021)



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

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Attention: Mr. D.J. Arellano, P.E.
Director, Development Services

Project No.: **20G239-3R**

Subject: **Geotechnical Investigation**
Proposed Warehouse
NEC Patterson Avenue and Nance Street
Perris, California

Gentlemen:

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

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Handwritten signature of Ricardo Frias in blue ink.

Ricardo Frias, RCE 91772
Staff Engineer



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Robert G. Trazo, GE 2655
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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

- Undocumented fill soils were encountered at one of the borings, extending to a depth of $4\frac{1}{2}\pm$ feet. Additional soils identified as possible fill were encountered at one of the borings, extending to a depth of $6\frac{1}{2}\pm$ feet. Younger alluvial soils were encountered at all of the boring locations, with the exception of Boring No. B-9, extending from the ground surface to depths of $1\frac{1}{2}$ to $6\frac{1}{2}\pm$ feet.
- The near-surface alluvial soils possess varying strengths and densities. In addition, some of the younger alluvial soils possess moderate compressibility and a minor potential for hydro-collapse. These soils, in their present condition, are not considered suitable for support of the foundation loads of the new structures.
- Remedial grading will be necessary to remove a portion of the near-surface alluvial soils and replace them as compacted structural fill. Generally, the existing soils may be reused as structural fill.

Site Preparation

- Initial site stripping should include removal of any surficial vegetation from the site. Stripping should include any weeds, grasses, and any organic topsoil.
- Demolition of the existing structures, pavements 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. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2-inch particle size, well mixed with the on-site soils, and incorporated into new structural fills. It may also be crushed and made into crushed miscellaneous base (CMB), if desired.
- We recommend that remedial grading be performed within the proposed building area in order to remove all of the undocumented fill soils and a portion of the near-surface alluvium. The soils present within the proposed building area should be overexcavated to a depth of at least 4 feet below existing grade and to a depth of at least 4 feet below proposed building pad subgrade elevation. The proposed foundation influence zones should also be overexcavated to a depth of at least 3 feet below proposed foundation bearing grade. Additional overexcavation may be necessary in areas where loose or otherwise unsuitable soils are encountered at the base of the overexcavation.
- 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 2 to 4 percent above the optimum moisture content, 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 $12\pm$ inches, thoroughly moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,500 lbs/ft² maximum allowable soil bearing pressure.
- Reinforcement consisting of at least six (6) No. 5 rebars (3 top and 3 bottom), due to the presence of potentially expansive soils. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-grade, 6 inches thick.
- Modulus of Subgrade Reaction: 100 psi/in.
- Minimum slab reinforcement: No. 3 bars at 18 inches on center in both directions due to the presence of medium expansive soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed loading.

Pavements

ASPHALT PAVEMENTS (R = 30)					
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	6	8	10	11	13
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 30)				
Materials	Thickness (inches)			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Change Order Nos. 20G239-CO and 20G239-CO2, dated July 6, 2021 and November 24, 2021, respectively. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the northeast corner of Nance Street and Patterson Avenue in Perris, California, and extends 300± feet south of Nance Street. The site is bounded to the north by existing single-family residences (SFRs), and vacant land, to the west by Patterson Avenue, to the south by an existing commercial/industrial building, and to the east by Nevada Avenue and the March Air Reserve Base. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The site consists of several rectangular to triangular-shaped parcels which total 33.71± acres in size. The site is mostly vacant and undeveloped, except a trailer drop lot in the northwesternmost parcel and a 40± feet by 40± feet concrete slab-on-grade located in the east-central region of the site. Ground surface cover within the trailer drop-lot consists of old/degraded crushed aggregate base (CAB) pavements with areas of open-graded gravel, and isolated areas of exposed soils with sparse native grass and weed growth. In the remainder of the site, ground surface cover consists of exposed soil with moderate to dense native grass and weed growth.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth, and visual observations made at the time of the subsurface investigation, the overall site topography slopes gently downward to the north at a gradient of ½± percent. There is approximately 4± feet of elevation differential across the overall site.

3.2 Proposed Development

SCG was provided with a site plan by the client. Based on this site plan, the site will be developed with one (1) building, 769,668± ft² in size, in the central area of the site. Dock-high doors will be constructed along portions of the eastern and western building walls. Additional parking areas will surround the building. The building will be surrounded by asphaltic concrete pavements in the parking and drive lanes, Portland cement concrete pavements in the loading dock areas, and limited areas of concrete flatwork and landscape planters throughout.

Detailed structural information has not been provided. It is assumed that the new building will be a single-story structure of tilt-up concrete construction, typically supported on conventional shallow foundation systems 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 basements or crawl spaces, are expected to be included in the proposed development. Based on the assumed topography, cuts and fills of up to 2± feet are expected to be necessary to achieve the proposed site grades.

3.3 Previous Study

Southern California Geotechnical, Inc. (SCG) previously performed a geotechnical investigation for the subject sites. The results of this study is presented in the referenced report:

Geotechnical Investigation, Proposed Warehouse, NEC Patterson Avenue and Nance Street, Perris, California, prepared by Southern California Geotechnical, Inc. (SCG) for Duke Realty, SCG Project No. 20G239-1, dated December 22, 2020.

As part of this investigation, a total of seven (7) borings, advanced to depths of 10 to 25± feet below currently existing site grades. Native alluvium was encountered at the ground surface at all of boring locations, extending to the maximum explored depth of 25± feet below existing site grades. The near-surface native alluvial soils extending from the ground surface to depths of 1½ to 6½± feet, were classified as younger alluvium. The younger alluvium generally possesses lower densities than the soils classified as older alluvium. The younger alluvium generally consists of medium dense to dense silty sands, sandy silts, and clayey sands. At Boring No. B-4 a layer of hard sandy clay was encountered. Older native alluvial soils were encountered beneath the younger native alluvial soils at all of the boring locations. The soils classified as older alluvium generally possess higher densities than the younger alluvial soils, many samples were observed to be weakly to moderately cemented. Most of the older alluvial soils encountered at the boring locations consist of medium dense to very dense silty sands, sandy silts, and clayey sands. Older alluvial soils also consisted of stiff to hard sandy clays and varying amounts of silt. Older native alluvial soils extended to at least the maximum depths explored at all of the boring locations. Groundwater was not encountered at any of the boring locations.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of three (3) borings advanced to depths of 20 to 25± feet below the existing site grades. Including the subsurface exploration discussed in the referenced report, a total of ten (10) borings were advanced to depths of 10 to 30± feet below existing site grades. All of the borings were logged during drilling by a member of our staff.

All of the borings were advanced with hollow-stem augers by a conventional truck-mounted drilling rig. Representative bulk and relatively undisturbed soil samples were taken during drilling. Relatively undisturbed soil samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. 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

Possible Fill

Possible fill soils were encountered at Boring No. B-10, extending to depths of 6½± feet below existing site grades. The possible fill soils consist of loose to medium dense silty fine to medium sands with trace quantities of clay. The possible fill soils possessed a disturbed appearance, was surrounded by surficial trash debris, and possessed uncharacteristic blow counts. Historic aerials indicate that the area drilled was disturbed in 2009 during the development of the northern parcel.

Artificial Fill

Artificial fill soils were encountered at the ground surface of Boring No. B-8, extending to a depth of 4½± feet below existing site grades. The artificial fill soils consisted of medium dense silty fine sands with trace quantities of clay and medium to coarse sands. The fill soils

possessed a disturbed appearance. Review of historic aerial photographs indicate that a building was present within this portion of the site in 2014.

Older Alluvium

Older alluvial soils were encountered at the ground surface at Boring No. B-9, and beneath the artificial fill soils and possible fill soils at all of the boring locations, extending to at least the maximum depth explored of 25± feet below existing site grades. The older alluvial soils generally consist of medium dense to dense silty fine sands, medium dense to very dense fine to medium sands, and very dense silty fine to coarse sands. Variable clay and calcareous nodule/veining were encountered within the older alluvial strata.

Groundwater

Groundwater was not encountered at any of the boring locations. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 25± feet below existing site grades, at the time of the subsurface investigation.

Recent water level data was obtained from the California Department of Water Resources Water Data Library website, <http://wdl.water.ca.gov/>. The nearest monitoring well on record is located 60± feet south of the site. Water level readings within this monitoring well indicate a groundwater level of 72± feet below the ground surface in March 2020.

5.0 LABORATORY TESTING

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

Classification

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

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 and collapse 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-3 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample from our previous study has been tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557, and are included in Appendix F of this report. These tests are generally used to with compare the dry 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.

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829. The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample. The sample is initially remolded to 50± 1 percent saturation and then loaded with a

surcharge equivalent to 144 pounds per square foot. The sample is then inundated with water, and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The results of the EI testing are as follows:

<u>Sample Identification</u>	<u>Expansion Index</u>	<u>Expansive Potential</u>
B-4 @ 0 to 5 feet (Previous Study)	53	Medium

Soluble Sulfates

A representative sample of the near-surface soils has been 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.

<u>Sample Identification</u>	<u>Soluble Sulfates (%)</u>	<u>Sulfate Classification</u>
B-2 @ 0 to 5 feet (Previous Study)	0.005	Not Applicable (S0)
B-10 @ 0 to 5 feet	0.003	Not Applicable (S0)

Corrosivity Testing

A representative sample of the near-surface soils has been 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.

<u>Sample Identification</u>	<u>Saturated Resistivity (ohm-cm)</u>	<u>pH</u>	<u>Chlorides (mg/kg)</u>	<u>Nitrates (mg/kg)</u>
B-2 @ 0 to 5 feet (Previous Study)	2,280	7.8	42	42
B-10 @ 0 to 5 feet	3,640	7.5	14	23

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

6.1 Seismic Design Considerations

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

Faulting and Seismicity

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

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

Seismic Design Parameters

The 2019 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

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

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

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

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_s	1.500
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.576
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.500
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.993
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.000
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.662

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

Liquefaction

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 Riverside County GIS website indicates that the subject site is located within a zone of low liquefaction susceptibility. In addition, the subsurface conditions encountered at the boring locations are not considered to be conducive to liquefaction. These conditions consist of moderate to high strength older native alluvial soils and no evidence of a long-term groundwater table within the depths explored by the borings. Based on these considerations, liquefaction is not considered to be a design concern for this project.

6.2 Geotechnical Design Considerations

General

Undocumented fill soils were encountered at Boring No. B-8 extending to a depth of $4\frac{1}{2}\pm$ feet. Possible fill soils were encountered at Boring No. B-10 extending to a depth of $6\frac{1}{2}\pm$ feet. All of the other borings encountered native alluvium at the ground surface. The near-surface native alluvial soils extending from the ground surface to depths of $1\frac{1}{2}$ to $6\frac{1}{2}\pm$ feet, were classified as younger alluvium, except for Boring No. B-9. The younger alluvium generally consists of medium dense to dense silty sands, sandy silts, and clayey sands. Some of the borings encountered relatively lower strength younger soils within the upper 3 to $6\frac{1}{2}\pm$ feet below the ground surface. The results of consolidation/collapse testing indicate that some of the near-surface alluvium encountered with the upper $6\pm$ feet possess minor collapse potential and moderate compressibility when inundated with water. All of the borings also encountered older alluvial soils directly beneath artificial fill soils, possible fill soils, and the younger alluvium. The older alluvial soils generally possess moderate to high strengths and favorable consolidation/collapse characteristics. Remedial grading is considered warranted within the proposed building area in order to remove all of the undocumented fill soils in their entirety, and a portion of the near-surface alluvium, and any soils disturbed during the demolition process, and replace these materials as compacted structural fill soils.

Settlement

The recommended remedial grading will remove all of the undocumented fill soils and a portion of the near-surface native alluvium, including potentially collapsible/compressible soils. The native soils that will remain in place below the recommended depth of overexcavation will not be subject to significant load increases from the foundations of the new structure. Provided that the recommended remedial grading is completed, the post-construction static settlements of the proposed structure is expected to be within tolerable limits.

Soluble Sulfates

The results of the soluble sulfate testing indicate that the selected samples of the on-site soils contain negligible concentrations of soluble sulfates, in accordance with American Concrete Institute (ACI) guidelines. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building area.

Expansion

The near-surface soils at this site generally consist of silty sands and sandy silts with varying clay content and occasional sandy clays. Laboratory testing indicates that the on-site soils possess medium expansion potential (EI=53). The foundation and floor slab design recommendations contained within this report are made in consideration of the expansion index test results. It is recommended that additional expansion index testing be conducted at the completion of rough grading to verify the expansion potential of the as-graded building pad.

Corrosion Potential

The results of laboratory testing indicate that the on-site soils possess saturated resistivity values of 2,280 and 3,640 ohm-cm, and a pH values of 7.5 and 7.8. 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. Sulfides, and redox potential are factors that are also used in the evaluation procedure. 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 slightly corrosive to ductile iron pipe. Therefore, polyethylene encasement or some other appropriate method of protection may be required for iron pipes.** Since SCG does not practice in the area of corrosion engineering, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.

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 of 14 and 42 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. Since SCG does not practice in the area of corrosion engineering, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.

Nitrates

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested sample possesses a nitrate concentration of 23 and 42 mg/kg. Based on this test result, the on-site soils are not 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 a more thorough evaluation.

Shrinkage/Subsidence

Based on the results of the laboratory testing, removal and recompaction of the near-surface native alluvium will result in an average shrinkage of 2 to 12 percent. However, the estimated shrinkage of the individual soil layers at the site is highly variable, locally ranging from a minimum shrinkage value of 1 percent to a maximum shrinkage of 16 percent at varying sample depths and locations. It should be noted that the potential shrinkage estimate is based on dry density testing performed on small-diameter samples taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

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

It is recommended that we be provided with copies of the 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

Demolition of the existing structures and pavements will be necessary in order to facilitate the construction of the proposed development. Demolition should include all foundations, floor slabs, utilities and any other subsurface improvements that will not remain in place with the new development. Debris resultant from demolition should be disposed of off-site. Alternatively, concrete and asphalt debris may be crushed to a maximum 2-inch particle size, well mixed with the on-site soils, and incorporated into new structural fills.

Initial site preparation should include stripping of any surficial vegetation and organic soils. Based on conditions encountered at the time of the subsurface exploration, stripping of native grass and weed growth is expected to be necessary throughout the majority of the site. Any trash should also be disposed of prior to site grading. These materials should be 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 Pad

Remedial grading should be performed within the proposed building pad area in order to remove all of the undocumented fill soils and a portion of the existing younger alluvial soils. In general, it is recommended that the overexcavation extend to a depth of at least 4 feet below existing grade, and to a depth of at least 4 feet below proposed grade, whichever is greater. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade.

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

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

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 2 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

The building pad area 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 area 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 pad. 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 5 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 2 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 necessary precautions to protect the adjacent structures 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.

Treatment of Existing Soils: Parking Areas

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

Subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

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

and drive areas should be overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill.

Fill Placement

- Fill soils should be placed in thin ($6\pm$ inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2019 CBC and the grading code of the city of Perris.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

All imported structural fill should consist of low expansive ($EI < 50$), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Perris. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

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

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of silty sands, clayey sands, sandy silts and sandy clays. Some of these materials will likely be subject to minor caving within shallow excavations. Where

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

Moisture Sensitive Subgrade Soils

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

Expansive Soils

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

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

- Ponding and areas of low flow gradients in unpaved walkways, grass and planter areas should be avoided. In general, minimum drainage gradients of 2 percent should be maintained in unpaved areas.
- Bare soil within five feet of proposed structures should be sloped at a minimum five percent gradient away from the structures (about three inches of fall in five feet), or the same area could be paved with a minimum surface gradient of one percent. Pavement is preferable.
- Decorative gravel ground cover tends to provide a reservoir for surface water and may hide areas of ponding or poor drainage. Decorative gravel is, therefore, not recommended and should not be utilized for landscaping unless equipped with a subsurface drainage system designed by a licensed landscape architect.
- Positive drainage devices, such as graded swales, paved ditches, and catch basins should be installed at appropriate locations within the area of proposed development.

- Concrete walks and flatwork should not obstruct the free flow of surface water to the appropriate drainage devices.
- Area drains should be recessed below grade to allow free flow of water into the drain. Concrete or brick flatwork joints should be sealed with mortar or flexible mastic.
- Gutter and downspout systems should be installed to capture all discharge from roof areas. Downspouts should discharge directly into a pipe or paved surface system to be conveyed offsite.
- Enclosed planters adjoining, or in close proximity to proposed structures, should be sealed at the bottom and provided with subsurface collection systems and outlet pipes.
- Depressed planters should be raised with soil to promote runoff (minimum drainage gradient two percent or five percent, see above), and/or equipped with area drains to eliminate ponding.
- Drainage outfall locations should be selected to avoid erosion of slopes and/or properly armored to prevent erosion of graded surfaces. No drainage should be directed over or towards adjoining slopes.
- All drainage devices should be maintained on a regular basis, including frequent observations during the rainy season to keep the drains free of leaves, soil and other debris.
- Landscape irrigation should conform to the recommendations of the landscape architect and should be performed judiciously to preclude either soaking or excessive drying of the foundation soils. This should entail regular watering during the drier portions of the year and little or no irrigation during the rainy season. Automatic sprinkler systems should, therefore, be switched to manual operation during the rainy season. Good irrigation practice typically requires frequent application of limited quantities of water that are sufficient to sustain plant growth, but do not excessively wet the soils. Ponding and/or run-off of irrigation water are indications of excessive watering.

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

Groundwater

The static groundwater table is considered to exist at a depth greater than 25± feet or more below existing grade. 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 pad will be underlain by newly placed structural fill soils extending to depths of at least 3 feet below foundation bearing grade. Based on this subsurface profile, the proposed structure may be supported on shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft².
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom), due to the presence of potentially expansive soils.

- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 24 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill compacted at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

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

Estimated Foundation Settlements

Post-construction total and differential static settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively, under static conditions. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch.

Lateral Load Resistance

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

- Passive Earth Pressure: 300 lbs/ft³
- Friction Coefficient: 0.28

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill soils. The maximum allowable passive pressure is 2,500 lbs/ft².

6.6 Floor Slab Design and Construction

Subgrades which will support the new floor slab should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floor of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, extending to a depth of at least 4 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: 100 psi/in.
- Minimum slab reinforcement: No. 3 bars at 18-inches on-center, in both directions, due to the presence of medium expansive soils at this site. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.
- 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 15 mil Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.
- Moisture condition the floor slab subgrade soils to 2 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.
- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.
- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement.

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 in truck court area and 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 on-site soils generally consist of silty sands, sandy silts, clayey sands and sandy clays. Based on their classifications, the on-site soils consisting of silty sands and sandy silts are generally expected to possess a friction angle of at least 30 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density. **However, clayey sands and sandy clay soils should not be used to backfill retaining walls because they likely possess higher expansion potential and lower strengths than the majority of the near-surface soils at the site.**

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		136 lbs/ft ³
Equivalent Fluid Pressure:	Active Condition (level backfill)	46 lbs/ft ³
	Active Condition (2h:1v backfill)	73 lbs/ft ³
	At-Rest Condition (level backfill)	68 lbs/ft ³

Regardless of the backfill type, the walls should be designed using a soil-footing coefficient of friction of 0.28 and an equivalent passive pressure of 300 lbs/ft³. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

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

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

Seismic Lateral Earth Pressures

In accordance with the 2019 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 3 feet below proposed foundation bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

With the exception of sandy clays and clayey sands, on-site soils may be used for retaining wall backfill. However, all backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

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

All retaining wall backfill should be placed and compacted under engineering-controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557).

Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

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

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.
- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system.

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

6.8 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 on-site soils generally consist of silty sands, sandy silts, clayey sands, and sandy clays. These materials are expected to exhibit fair to good pavement support characteristics, with estimated R-values between 30 and 50. Therefore the subsequent pavement design is based upon a conservative R-value of 30. 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 additional R-value testing be performed after completion of rough grading to verify the pavement support characteristics of the pavement subgrades following site grading.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. The pavement designs are based on the

traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20 year design life, assuming six operational traffic days per week.

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

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

ASPHALT PAVEMENTS (R=30)					
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	6	8	10	11	13
Compacted Subgrade	12	12	12	12	12

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

Portland Cement Concrete

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

PORTLAND CEMENT CONCRETE PAVEMENTS (R=30)				
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. 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

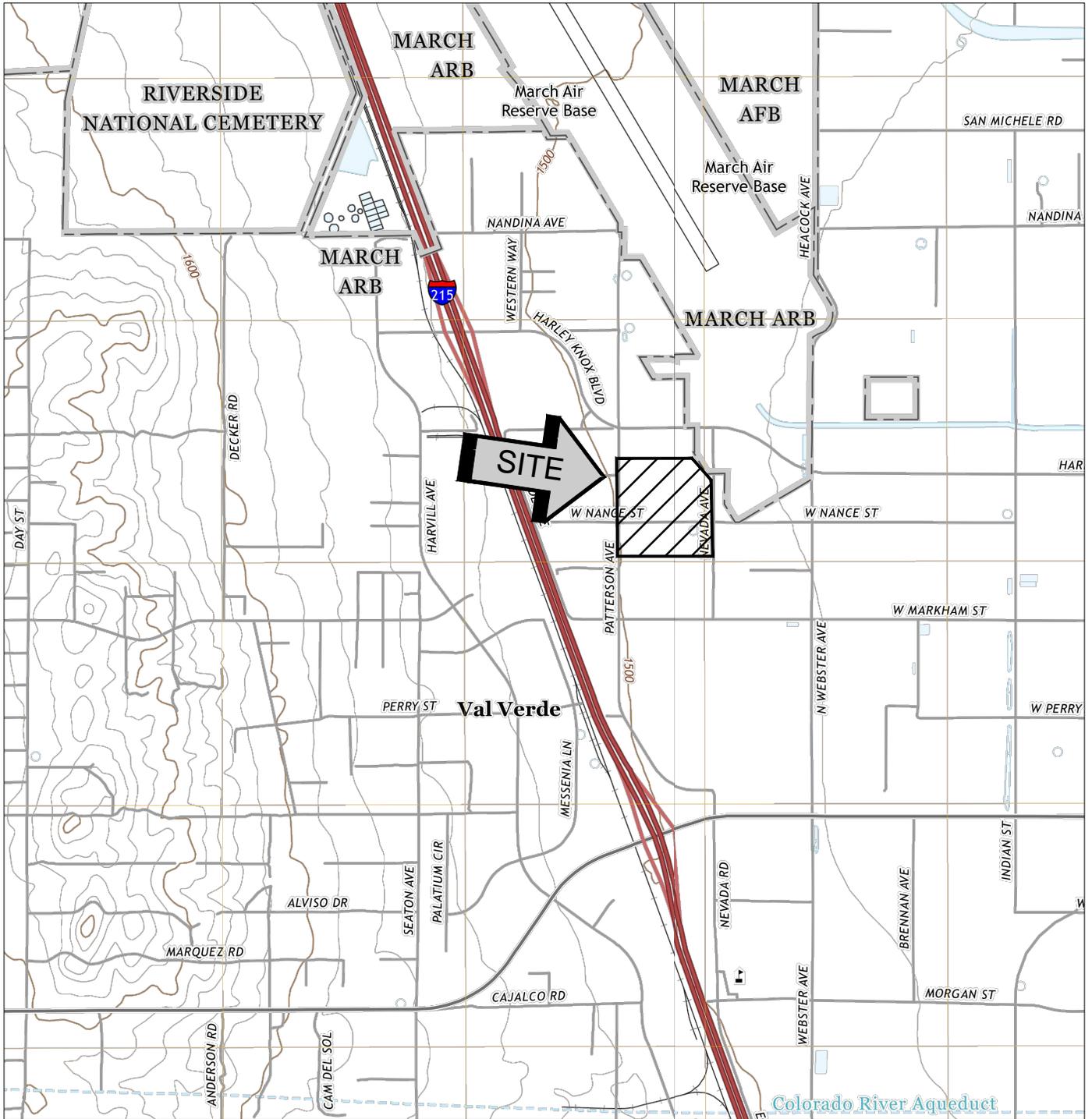
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other 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.

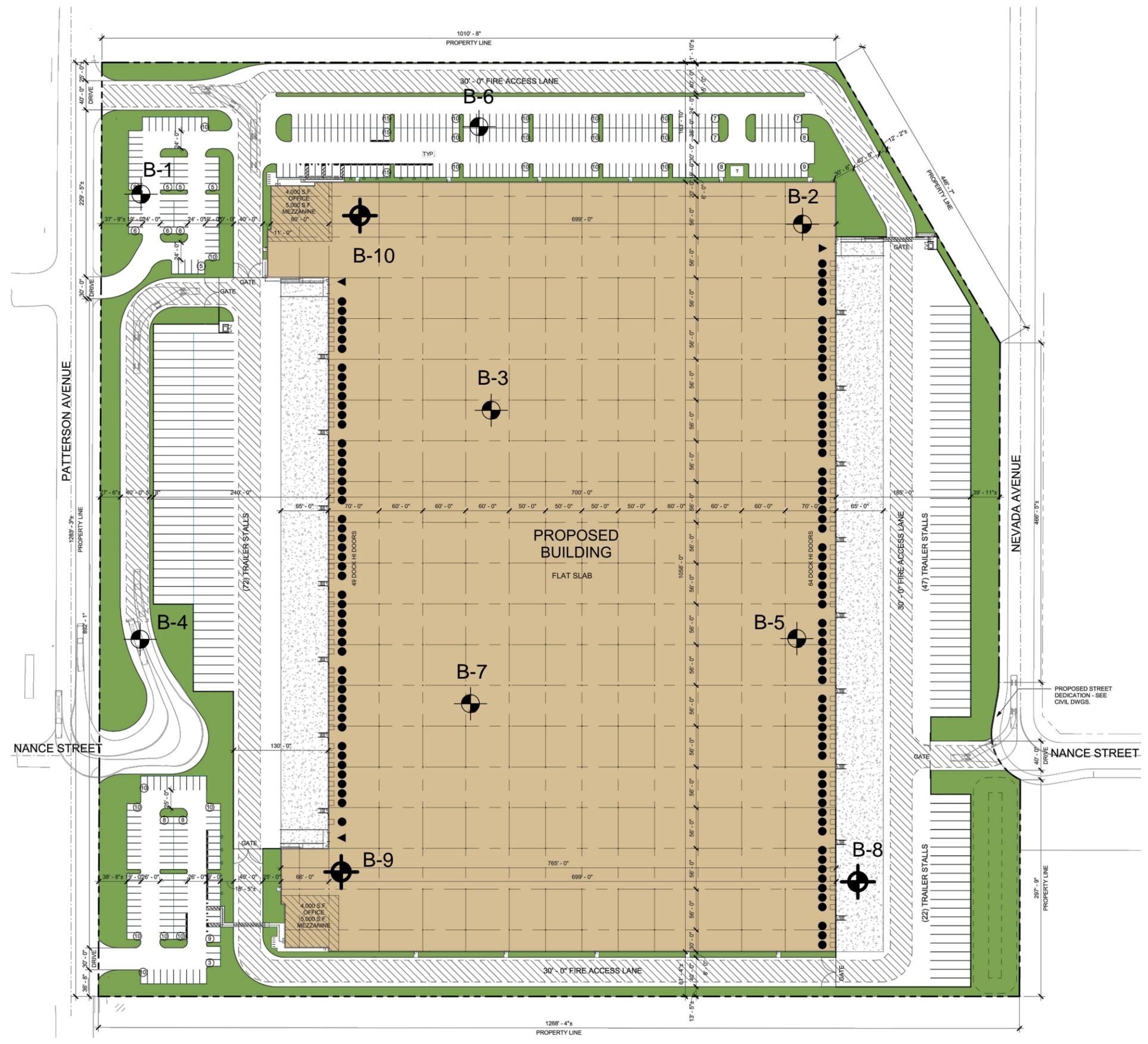
APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAPS OF THE STEELE PEAK AND PERRIS QUADRANGLES, RIVERSIDE COUNTY, CALIFORNIA, 2018



SITE LOCATION MAP	
PROPOSED WAREHOUSE	
PERRIS, CALIFORNIA	
SCALE: 1" = 2000'	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: RF	
CHKD: RGT	
SCG PROJECT 20G239-3R	
PLATE 1	



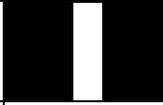
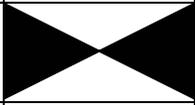
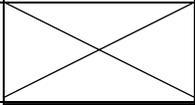
GEOTECHNICAL LEGEND

- APPROXIMATE BORING LOCATION
- PREVIOUS BORING LOCATION (SCG PROJECT NO. 20G239-1)

BORING LOCATION PLAN	
PROPOSED WAREHOUSE	
PERRIS, CALIFORNIA	
SCALE: 1" = 150'	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: RF	
CHKD: RGT	
SCG PROJECT 20G239-3R	
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/ft³.

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.

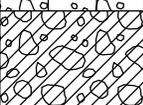
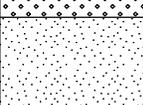
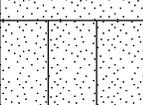
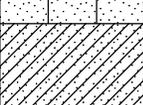
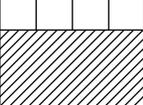
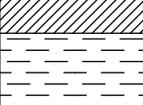
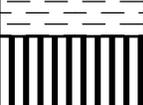
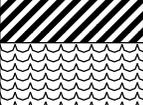
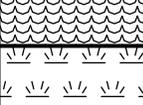
PASSING #200 SIEVE:

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS



JOB NO.: 20G239-3	DRILLING DATE: 7/7/21	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: ---
LOCATION: Perris, California	LOGGED BY: Ryan Bremer	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
	X	24			FILL: Brown Silty fine Sand, trace Clay, trace to little medium to coarse Sand, trace Calcareous nodules, mottled, micaceous, medium dense-damp	126	6					
	X	44			@ 3', trace medium to coarse Sand	126	6					
5	X	50			OLDER ALLUVIUM: Brown Silty fine Sand, trace to little medium to coarse Sand, trace Calcareous nodules, micaceous, medium dense to dense-damp	114	6					
	X	39			@ 7', no Calcareous nodules	121	8					
10	X	58			@ 9', trace to little Clay	128	10					
	X	33			Brown Silty fine to medium Sand, trace coarse Sand, micaceous, trace Calcareous nodules, medium dense-damp to moist		8					
15												
	X	30				10						
20												
Boring Terminated at 20'												

TBL_20G239-3.GPJ_SOCALGEO.GDT_8/6/21



JOB NO.: 20G239-3	DRILLING DATE: 7/7/21	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: ---
LOCATION: Perris, California	LOGGED BY: Ryan Bremer	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS					COMMENTS	
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)		ORGANIC CONTENT (%)
SURFACE ELEVATION: --- MSL												
	X	70			OLDER ALLUVIUM: Brown Silty fine to medium Sand, trace Clay, trace fine root fibers, trace Calcareous nodules, micaceous, medium dense-damp	120	5					
	X	29				122	4					
5	X	39			Brown Silty fine Sand, some Calcareous veining, micaceous, dense-damp	121	7					
	X	58				111	5					
10	X	58			@ 9', trace to little medium Sand, trace Calcareous nodules, micaceous, trace Clay, very dense-damp	129	6					
	X	40			Brown Silty fine to medium Sand, trace Calcareous nodules, micaceous dense-damp to moist	9						
15	X	66			Brown Silty fine to coarse Sand, trace Calcareous nodules, micaceous, dense to very dense-damp	7						
	X	39				5						
25	X											
Boring Terminated at 25'												

TBL_20G239-3.GPJ_SOCALGEO.GDT_8/6/21



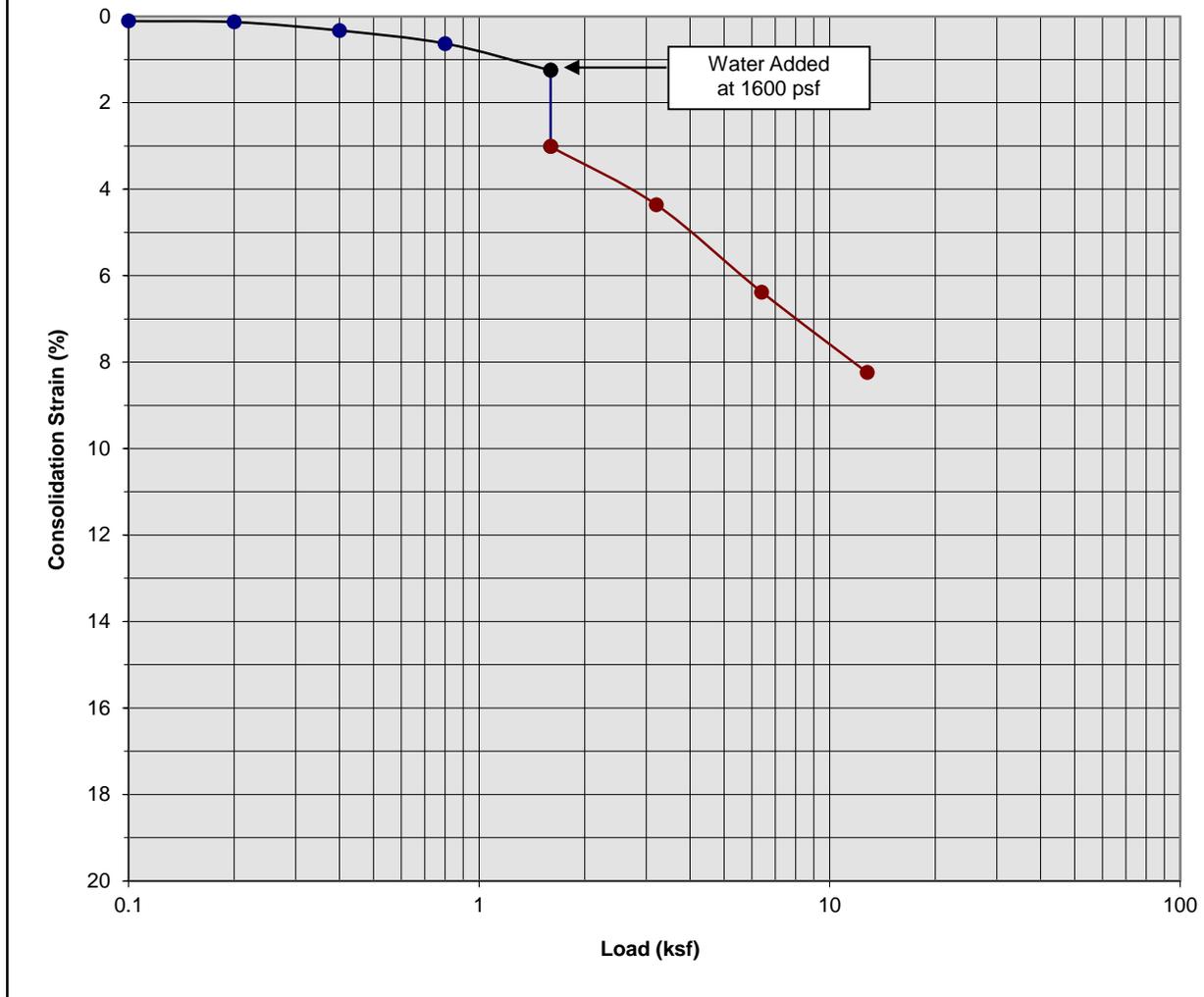
JOB NO.: 20G239-3	DRILLING DATE: 7/7/21	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: ---
LOCATION: Perris, California	LOGGED BY: Ryan Bremer	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
	X	34			POSSIBLE FILL: Dark Brown Silty fine to medium Sand, trace Clay, trace fine root fibers, micaceous, loose to medium dense-damp to moist	114	4					
	X	25			@ 3', trace to little coarse Sand, little Calcareous nodules	120	7					
5	X	11				120	10					
	X	50/4"			OLDER ALLUVIUM: Brown Silty fine to medium Sand, trace Clay, trace coarse Sand, some Calcareous veining, micaceous very dense-moist	12						Disturbed Sample
10	X	49			Brown Silty fine Sand, trace medium to coarse Sand, trace Calcareous nodules, micaceous, dense-moist	126	11					
15	X	23			Brown Silty fine Sand, trace medium to coarse Sand, trace Calcareous nodules, micaceous, medium dense to dense-damp		13					
20	X	36			@ 18.5', no Calcareous nodules		10					
Boring Terminated at 20'												

TBL_20G239-3.GPJ_SOCALGEO.GDT_8/6/21

APPENDIX C

Consolidation/Collapse Test Results



Classification: POSS. FILL: Silty fine to medium Sand, trace Clay, trace to little coarse Sand

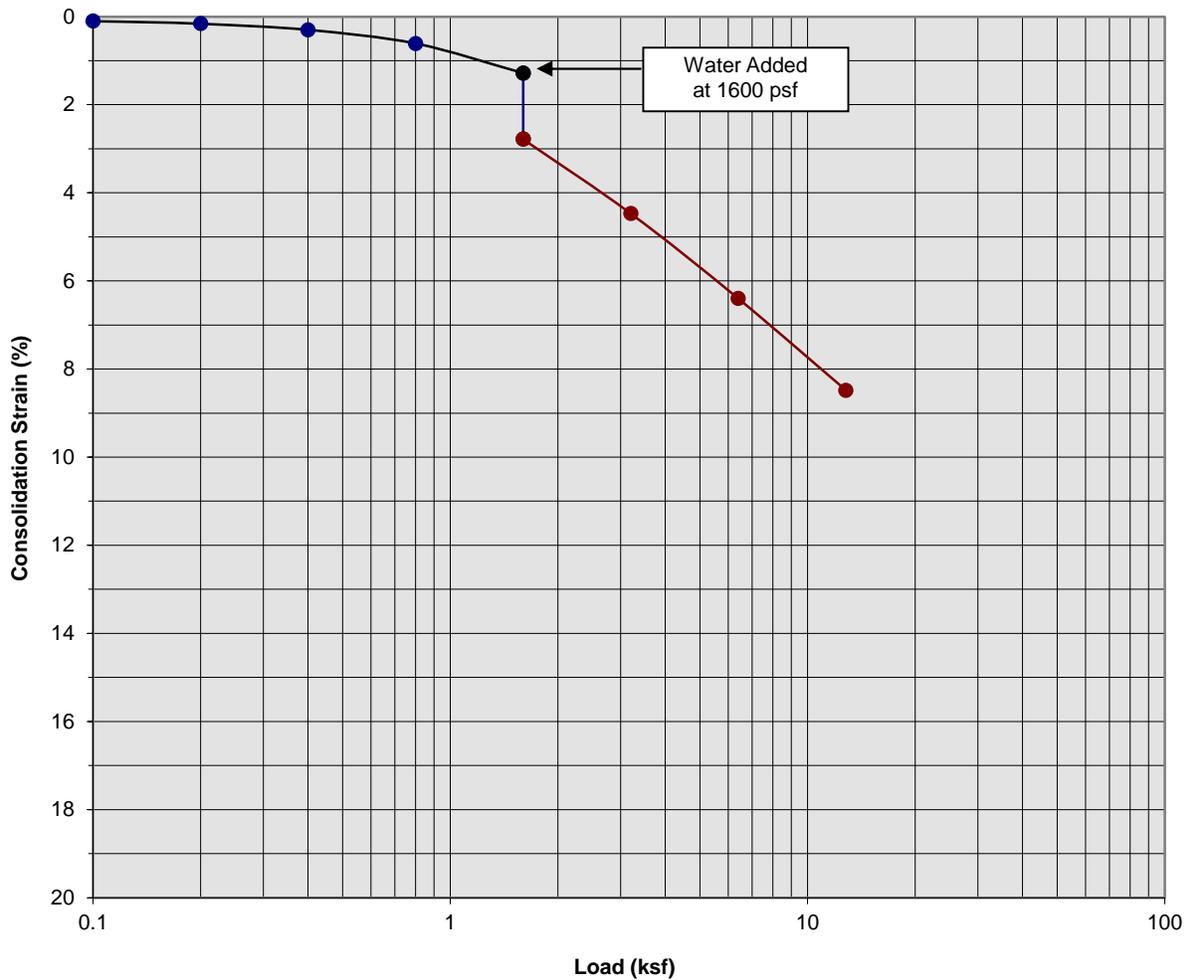
Boring Number:	B-10	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	10
Depth (ft)	3 to 4	Initial Dry Density (pcf)	120.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	130.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.76

Proposed Warehouse
 Perris, California
 Project No. 20G239-3
PLATE C- 1



SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: POSS. FILL: Silty fine to medium Sand, trace Clay

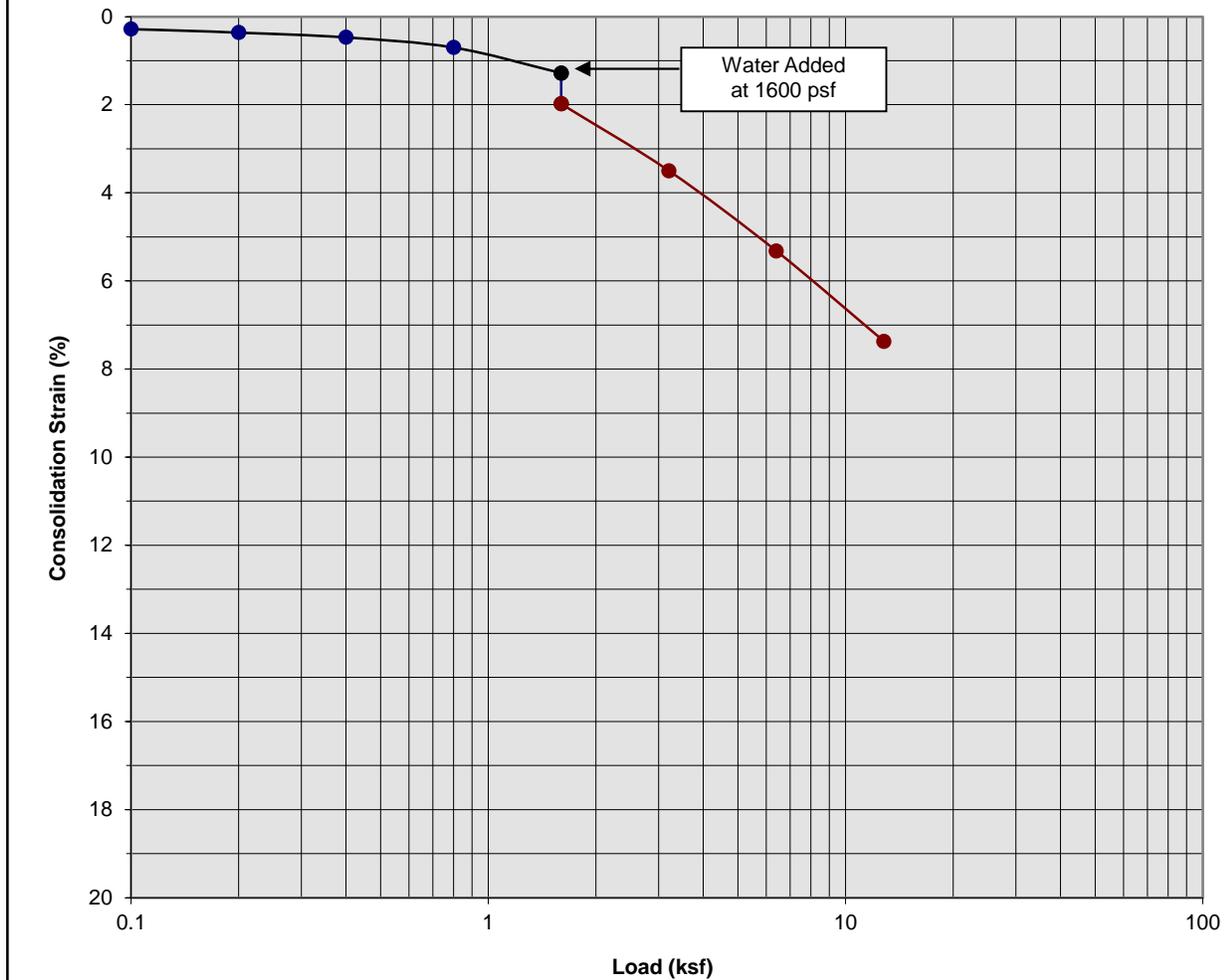
Boring Number:	B-10	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	5 to 6	Initial Dry Density (pcf)	120.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	131.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.50

Proposed Warehouse
 Perris, California
 Project No. 20G239-3
PLATE C- 2



**SOUTHERN
 CALIFORNIA
 GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: OLDER ALLUVIUM: Silty fine to medium Sand, trace Clay, trace coarse Sand

Boring Number:	B-10	Initial Moisture Content (%)	12
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	9 to 10	Initial Dry Density (pcf)	126.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	136.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.70

Proposed Warehouse
 Perris, California
 Project No. 20G239-3
PLATE C- 3



**SOUTHERN
 CALIFORNIA
 GEOTECHNICAL**
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.
- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.
- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.
- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.
- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.
- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.
- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

- Major vegetation should be stripped and disposed of off-site. This includes trees, brush, heavy grasses and any materials considered unsuitable by the Geotechnical Engineer.
- Underground structures such as basements, cesspools or septic disposal systems, mining shafts, tunnels, wells and pipelines should be removed under the inspection of the Geotechnical Engineer and recommendations provided by the Geotechnical Engineer and/or city, county or state agencies. If such structures are known or found, the Geotechnical Engineer should be notified as soon as possible so that recommendations can be formulated.
- Any topsoil, slopewash, colluvium, alluvium and rock materials which are considered unsuitable by the Geotechnical Engineer should be removed prior to fill placement.
- Remaining voids created during site clearing caused by removal of trees, foundations basements, irrigation facilities, etc., should be excavated and filled with compacted fill.
- Subsequent to clearing and removals, areas to receive fill should be scarified to a depth of 10 to 12 inches, moisture conditioned and compacted
- The moisture condition of the processed ground should be at or slightly above the optimum moisture content as determined by the Geotechnical Engineer. Depending upon field conditions, this may require air drying or watering together with mixing and/or discing.

Compacted Fills

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated," and shall be very low to non-expansive with a maximum expansion index (EI) of 50. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.
- All soils should be evaluated and tested by the Geotechnical Engineer. Materials with high expansion potential, low strength, poor gradation or containing organic materials may require removal from the site or selective placement and/or mixing to the satisfaction of the Geotechnical Engineer.
- Rock fragments or rocks less than 6 inches in their largest dimensions, or as otherwise determined by the Geotechnical Engineer, may be used in compacted fill, provided the distribution and placement is satisfactory in the opinion of the Geotechnical Engineer.
- Rock fragments or rocks greater than 12 inches should be taken off-site or placed in accordance with recommendations and in areas designated as suitable by the Geotechnical Engineer. These materials should be placed in accordance with Plate D-8 of these Grading Guide Specifications and in accordance with the following recommendations:
 - Rocks 12 inches or more in diameter should be placed in rows at least 15 feet apart, 15 feet from the edge of the fill, and 10 feet or more below subgrade. Spaces should be left between each rock fragment to provide for placement and compaction of soil around the fragments.
 - Fill materials consisting of soil meeting the minimum moisture content requirements and free of oversize material should be placed between and over the rows of rock or

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

- Subsequent rows of rocks should be placed such that they are not directly above a row placed in the previous lift of fill. A minimum 5-foot offset between rows is recommended.
- To facilitate future trenching, oversized material should not be placed within the range of foundation excavations, future utilities or other underground construction unless specifically approved by the soil engineer and the developer/owner representative.
- Fill materials approved by the Geotechnical Engineer should be placed in areas previously prepared to receive fill and in evenly placed, near horizontal layers at about 6 to 8 inches in loose thickness, or as otherwise determined by the Geotechnical Engineer for the project.
- Each layer should be moisture conditioned to optimum moisture content, or slightly above, as directed by the Geotechnical Engineer. After proper mixing and/or drying, to evenly distribute the moisture, the layers should be compacted to at least 90 percent of the maximum dry density in compliance with ASTM D-1557-78 unless otherwise indicated.
- Density and moisture content testing should be performed by the Geotechnical Engineer at random intervals and locations as determined by the Geotechnical Engineer. These tests are intended as an aid to the Earthwork Contractor, so he can evaluate his workmanship, equipment effectiveness and site conditions. The Earthwork Contractor is responsible for compaction as required by the Geotechnical Report(s) and governmental agencies.
- Fill areas unused for a period of time may require moisture conditioning, processing and recompaction prior to the start of additional filling. The Earthwork Contractor should notify the Geotechnical Engineer of his intent so that an evaluation can be made.
- Fill placed on ground sloping at a 5-to-1 inclination (horizontal-to-vertical) or steeper should be benched into bedrock or other suitable materials, as directed by the Geotechnical Engineer. Typical details of benching are illustrated on Plates D-2, D-4, and D-5.
- Cut/fill transition lots should have the cut portion overexcavated to a depth of at least 3 feet and rebuilt with fill (see Plate D-1), as determined by the Geotechnical Engineer.
- All cut lots should be inspected by the Geotechnical Engineer for fracturing and other bedrock conditions. If necessary, the pads should be overexcavated to a depth of 3 feet and rebuilt with a uniform, more cohesive soil type to impede moisture penetration.
- Cut portions of pad areas above buttresses or stabilizations should be overexcavated to a depth of 3 feet and rebuilt with uniform, more cohesive compacted fill to impede moisture penetration.
- Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure that excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls must be properly tested and approved by the Geotechnical Engineer with consideration of the lateral earth pressure used in the design.

Foundations

- The foundation influence zone is defined as extending one foot horizontally from the outside edge of a footing, and proceeding downward at a ½ horizontal to 1 vertical (0.5:1) inclination.
- Where overexcavation beneath a footing subgrade is necessary, it should be conducted so as to encompass the entire foundation influence zone, as described above.
- Compacted fill adjacent to exterior footings should extend at least 12 inches above foundation bearing grade. Compacted fill within the interior of structures should extend to the floor subgrade elevation.

Fill Slopes

- The placement and compaction of fill described above applies to all fill slopes. Slope compaction should be accomplished by overfilling the slope, adequately compacting the fill in even layers, including the overfilled zone and cutting the slope back to expose the compacted core
- Slope compaction may also be achieved by backrolling the slope adequately every 2 to 4 vertical feet during the filling process as well as requiring the earth moving and compaction equipment to work close to the top of the slope. Upon completion of slope construction, the slope face should be compacted with a sheepsfoot connected to a sideboom and then grid rolled. This method of slope compaction should only be used if approved by the Geotechnical Engineer.
- Sandy soils lacking in adequate cohesion may be unstable for a finished slope condition and therefore should not be placed within 15 horizontal feet of the slope face.
- All fill slopes should be keyed into bedrock or other suitable material. Fill keys should be at least 15 feet wide and inclined at 2 percent into the slope. For slopes higher than 30 feet, the fill key width should be equal to one-half the height of the slope (see Plate D-5).
- All fill keys should be cleared of loose slough material prior to geotechnical inspection and should be approved by the Geotechnical Engineer and governmental agencies prior to filling.
- The cut portion of fill over cut slopes should be made first and inspected by the Geotechnical Engineer for possible stabilization requirements. The fill portion should be adequately keyed through all surficial soils and into bedrock or suitable material. Soils should be removed from the transition zone between the cut and fill portions (see Plate D-2).

Cut Slopes

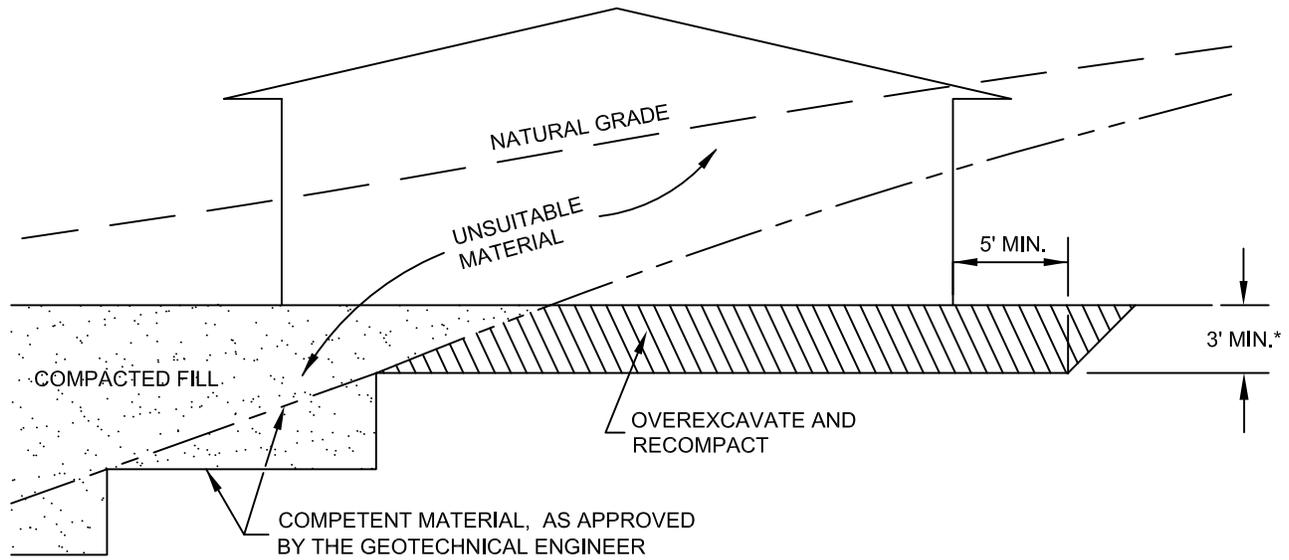
- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.
- Cut slopes exposing loose, cohesionless sands should be reported to the Geotechnical Engineer for possible stabilization recommendations.
- All stabilization excavations should be cleared of loose slough material prior to geotechnical inspection. Stakes should be provided by the Civil Engineer to verify the location and dimensions of the key. A typical stabilization fill detail is shown on Plate D-5.

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

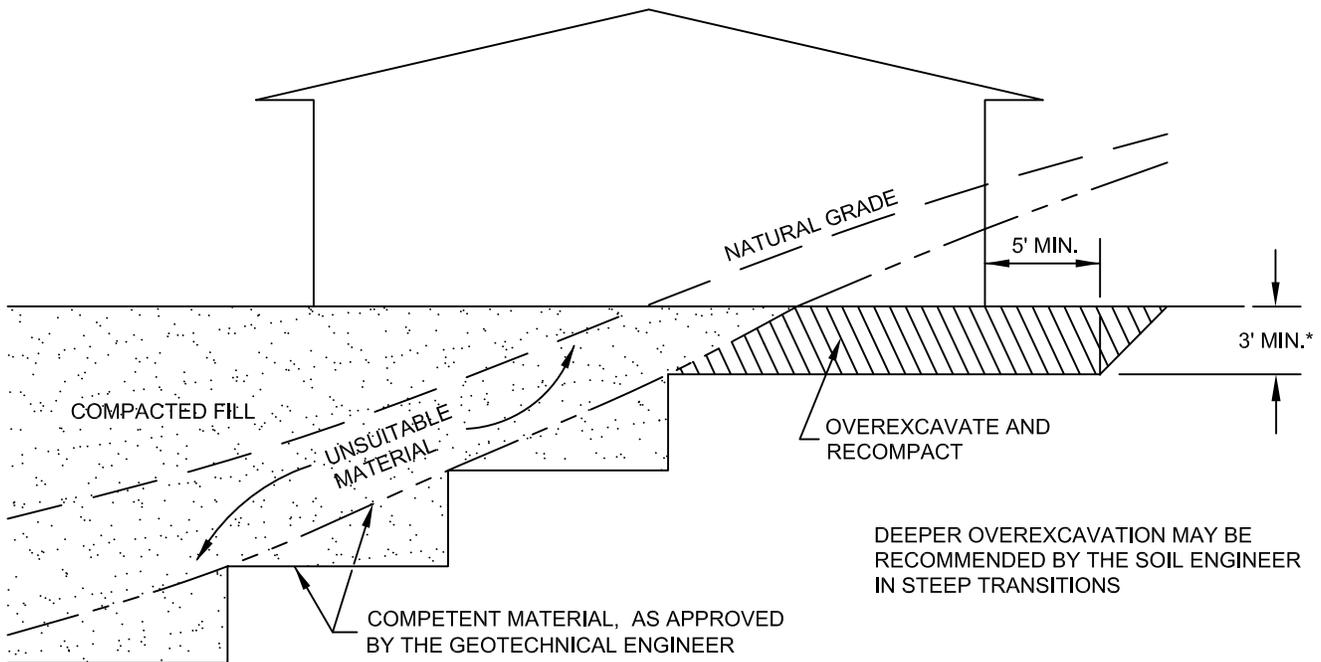
Subdrains

- Subdrains may be required in canyons and swales where fill placement is proposed. Typical subdrain details for canyons are shown on Plate D-3. Subdrains should be installed after approval of removals and before filling, as determined by the Soils Engineer.
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.
- Filter material for subdrains should conform to CALTRANS Specification 68-1.025 or as approved by the Geotechnical Engineer for the specific site conditions. Clean $\frac{3}{4}$ -inch crushed rock may be used provided it is wrapped in an acceptable filter cloth and approved by the Geotechnical Engineer. Pipe diameters should be 6 inches for runs up to 500 feet and 8 inches for the downstream continuations of longer runs. Four-inch diameter pipe may be used in buttress and stabilization fills.

CUT LOT

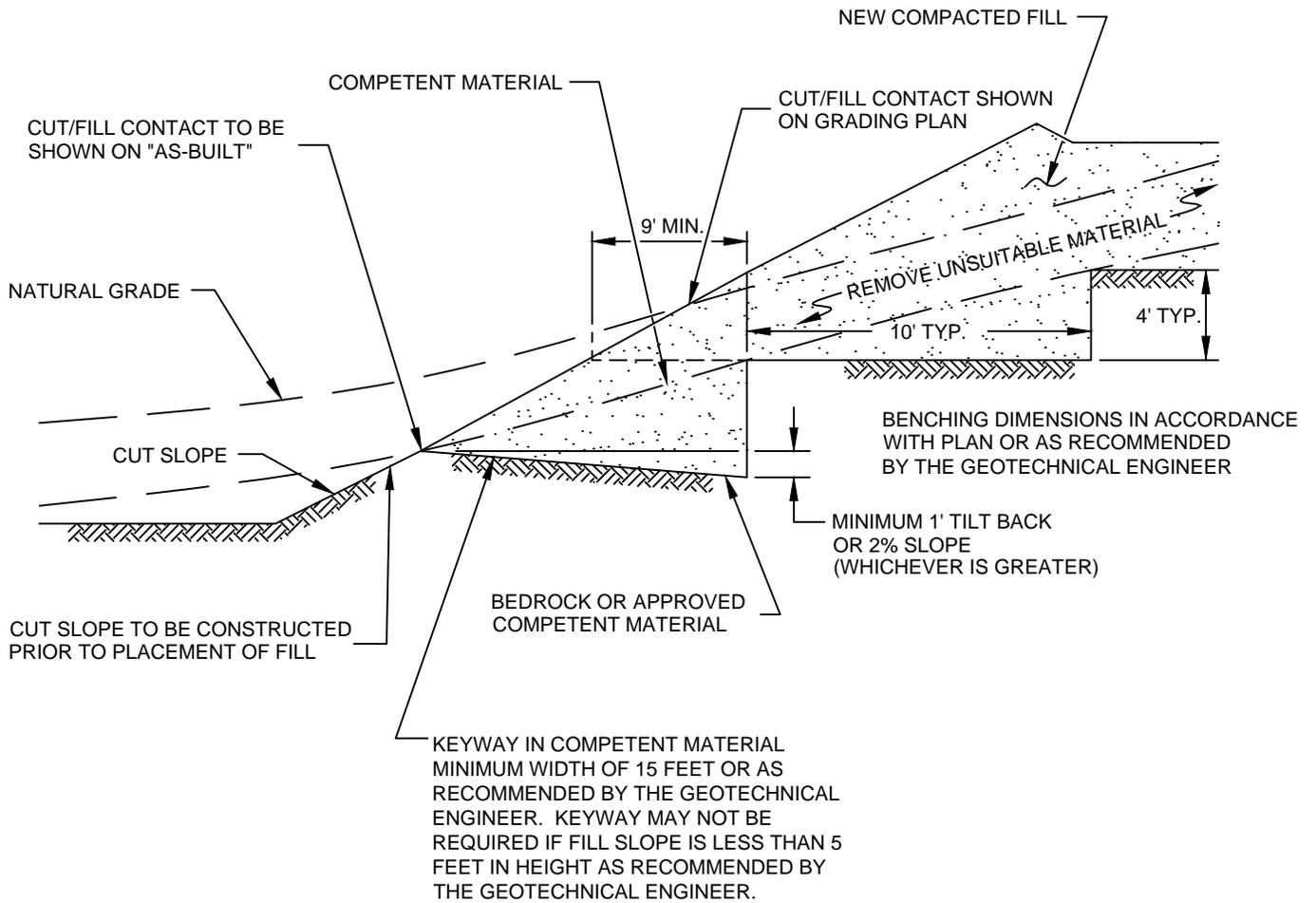


CUT/FILL LOT (TRANSITION)

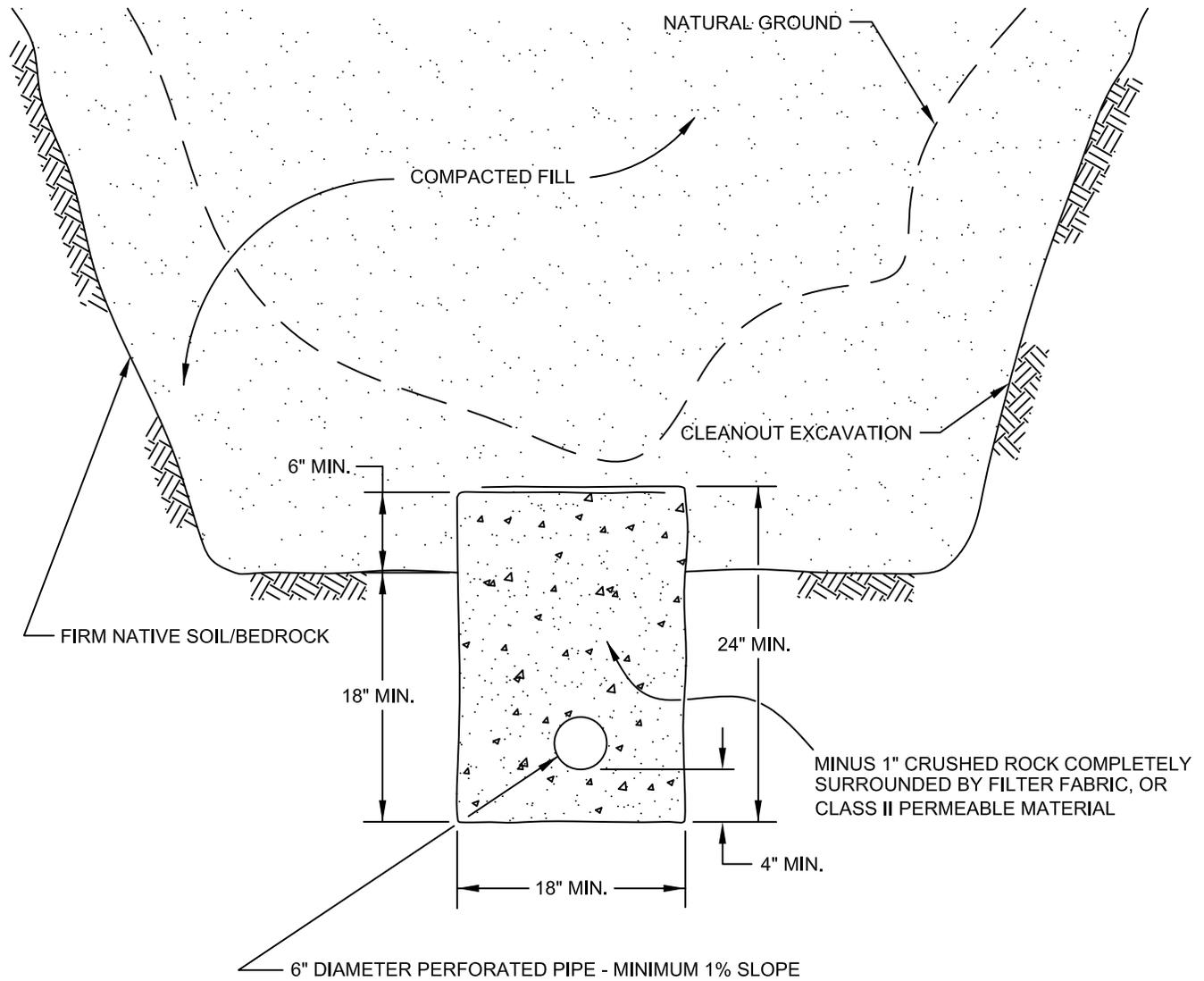


*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION.
ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

TRANSITION LOT DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-1	



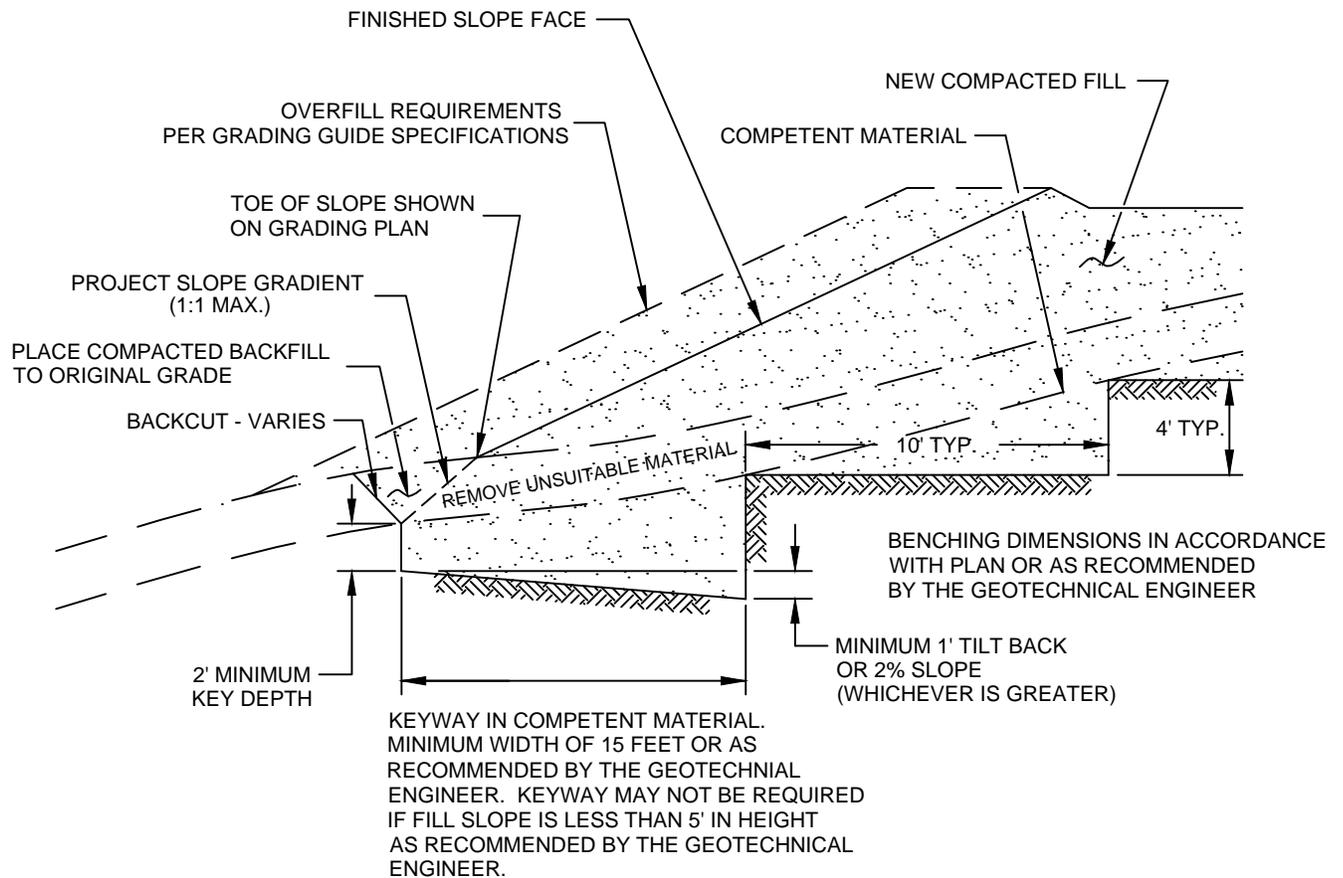
FILL ABOVE CUT SLOPE DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-2	



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

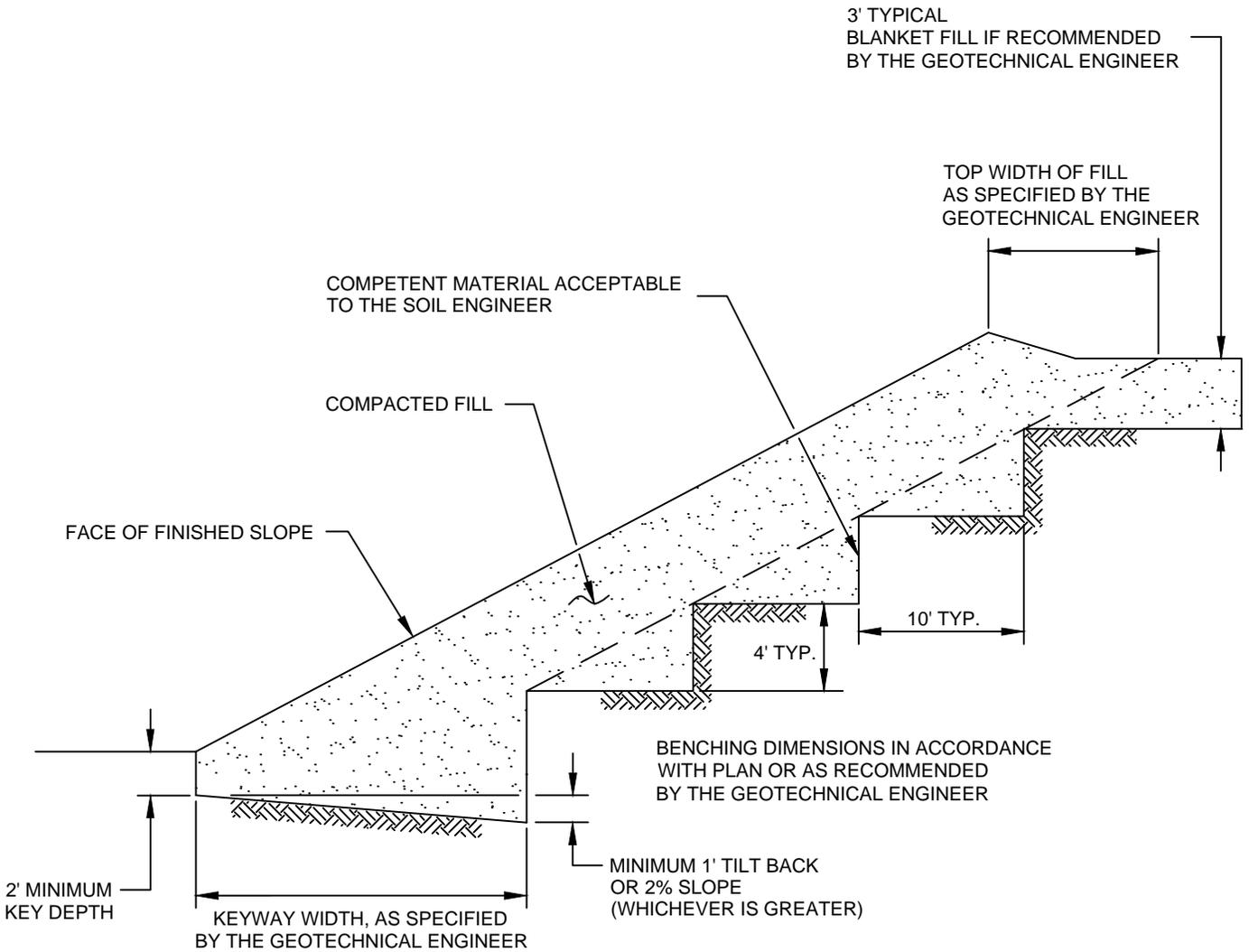
**SCHEMATIC ONLY
NOT TO SCALE**

CANYON SUBDRAIN DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-3	

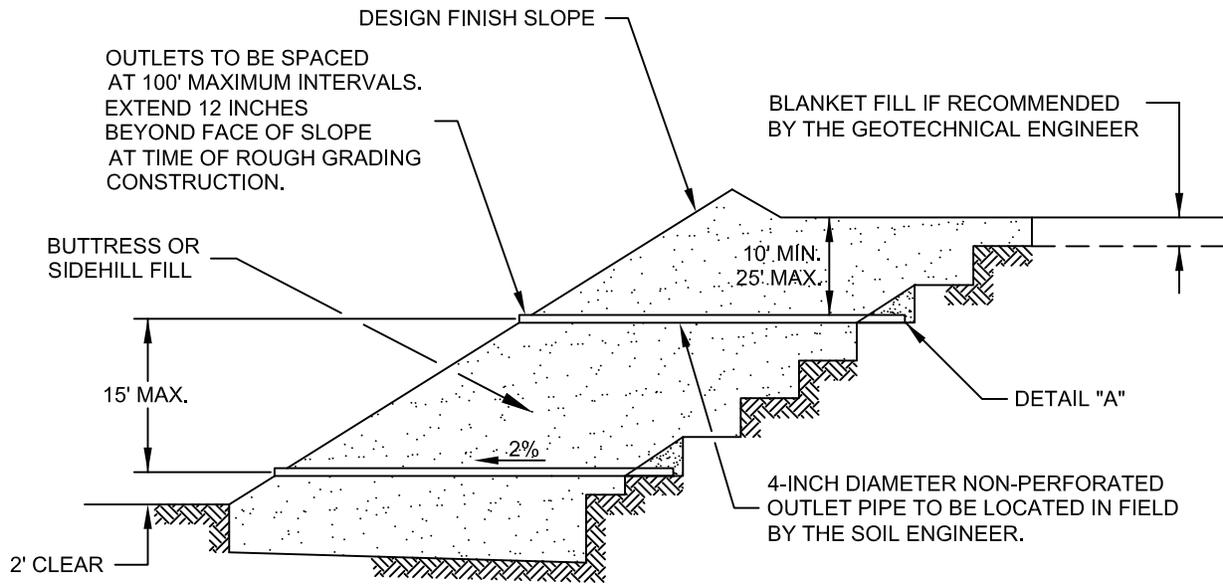


NOTE:
 BENCHING SHALL BE REQUIRED
 WHEN NATURAL SLOPES ARE
 EQUAL TO OR STEEPER THAN 5:1
 OR WHEN RECOMMENDED BY
 THE GEOTECHNICAL ENGINEER.

FILL ABOVE NATURAL SLOPE DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-4	



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



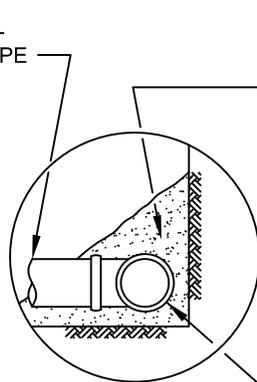
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8
SAND EQUIVALENT = MINIMUM OF 50	

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



DETAIL "A"

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

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

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

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

NOTES:

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

SLOPE FILL SUBDRAINS	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-6	

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

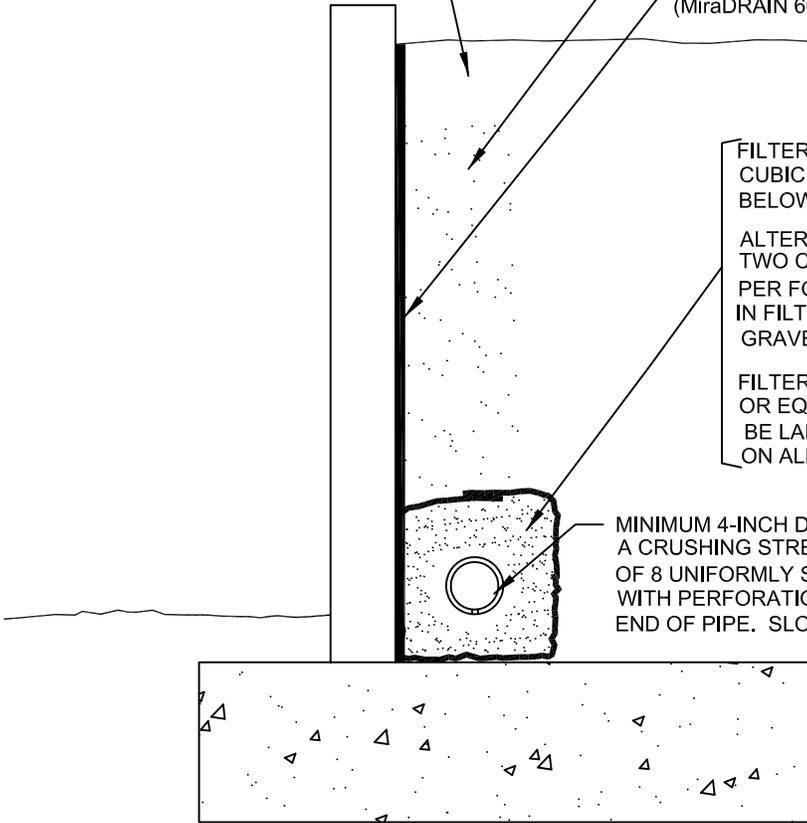
MINIMUM ONE FOOT WIDE LAYER OF FREE DRAINING MATERIAL (LESS THAN 5% PASSING THE #200 SIEVE) OR PROPERLY INSTALLED PREFABRICATED DRAINAGE COMPOSITE (MiraDRAIN 6000 OR APPROVED EQUIVALENT).

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 MIRAFAI 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.



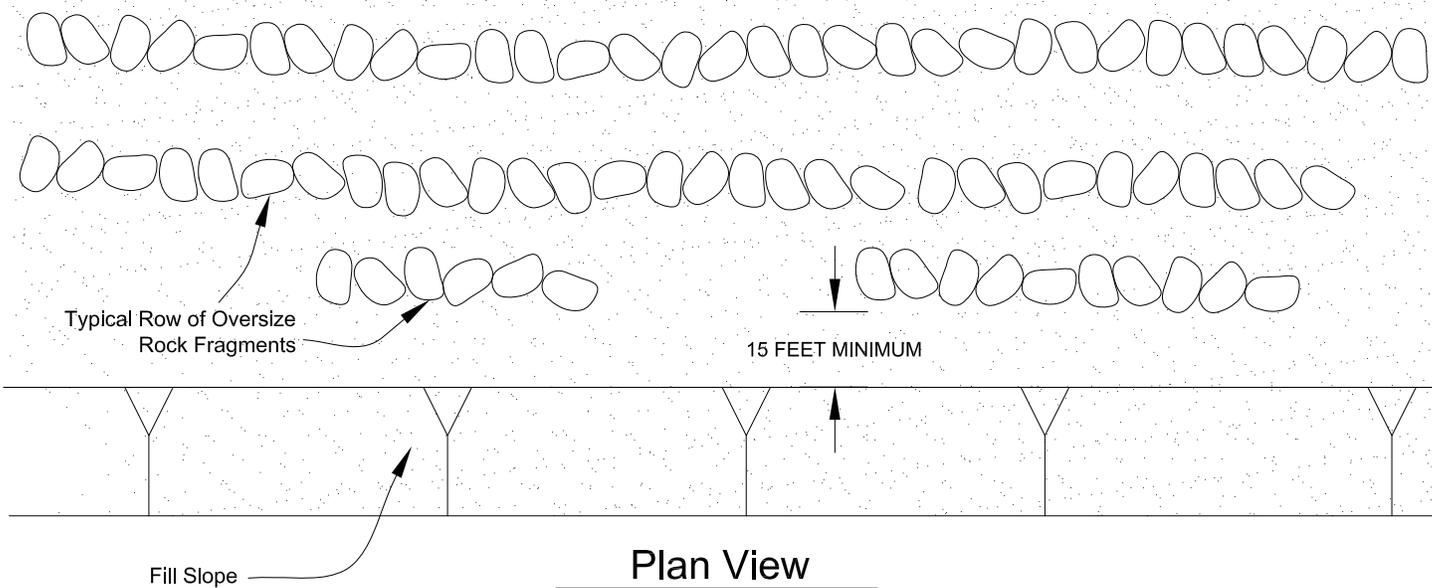
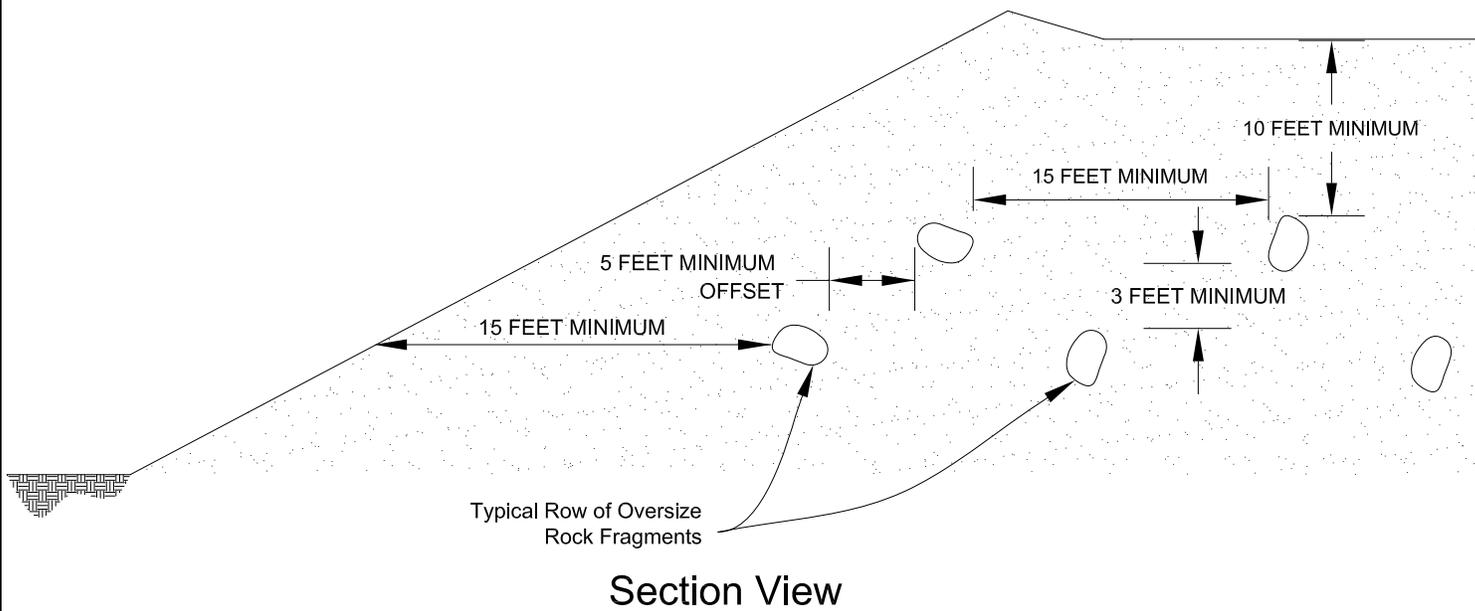
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-7	



**PLACEMENT OF OVERSIZED MATERIAL
GRADING GUIDE SPECIFICATIONS**

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

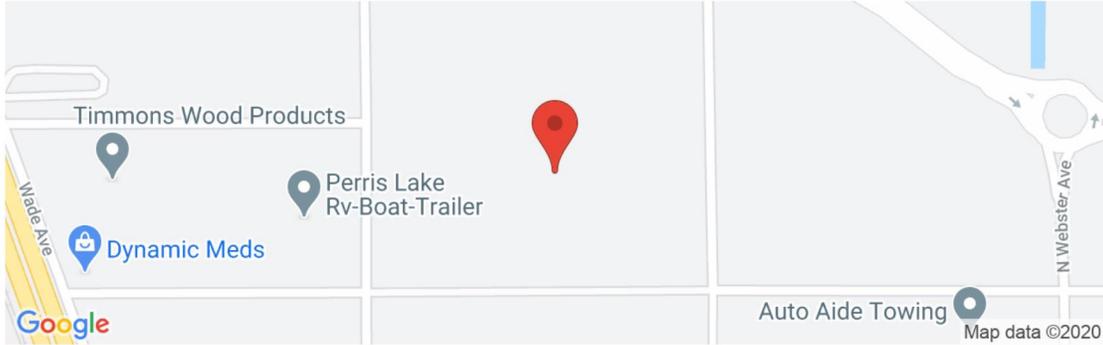


**SOUTHERN
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APPENDIX E



Latitude, Longitude: 33.856727, -117.250189



Date	12/17/2020, 2:39:54 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	0.576	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	0.5	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.55	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
$SsRT$	1.546	Probabilistic risk-targeted ground motion. (0.2 second)
$SsUH$	1.653	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
$S1RT$	0.576	Probabilistic risk-targeted ground motion. (1.0 second)
$S1UH$	0.631	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
$S1D$	0.6	Factored deterministic acceleration value. (1.0 second)
$PGAd$	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.935	Mapped value of the risk coefficient at short periods
C_{R1}	0.912	Mapped value of the risk coefficient at a period of 1 s

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



SEISMIC DESIGN PARAMETERS - 2019 CBC	
PROPOSED WAREHOUSE	
PERRIS, CALIFORNIA	
DRAWN: JAH CHKD: RF SCG PROJECT 20G239-3 PLATE E-1	 SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX



JOB NO.: 20G239-1	DRILLING DATE: 12/4/20	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Perris, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS				DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)		GRAPHIC LOG	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	
SURFACE ELEVATION: MSL											
					YOUNGER ALLUVIUM: Brown Silty fine Sand, trace medium Sand, medium dense-damp		5				
					OLDER ALLUVIUM: Brown Silty fine to medium Sand, trace to little coarse Sand, dense to very dense-damp		5				
5											
							5				
					@ 8½ feet, trace Clay, moist		8				
10											
					Brown fine Sandy Silt, dense-moist						
15							13				
					Dark Brown fine Sandy Silt, with interbedded layers of Silty Clay, very dense-damp						
20							8				
					Brown Silty fine Sand to fine Sandy Silt, dense-damp						
25							9				
Boring Terminated at 25'											

TBL_20G239-1.GPJ_SOCALGEO.GDT_12/24/20



JOB NO.: 20G239-1	DRILLING DATE: 12/4/20	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 13 feet
LOCATION: Perris, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS				DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)		GRAPHIC LOG	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	
SURFACE ELEVATION: MSL											
					<u>YOUNGER ALLUVIUM:</u> Brown Silty fine Sand, trace medium Sand, dense-dry 115 2						
					<u>OLDER ALLUVIUM:</u> Brown Clayey fine Sand, little Silt, trace medium Sand, very dense-damp 122 5						
5					82/8" 8						
					Red Brown Silty fine to coarse Sand, cemented, very dense-damp 103 8						
10					50/4" 6						
					106 6						
					Red Brown Silty fine Sand, trace Clay, dense-damp 6						
15					39 6						
					Brown Silty fine Sand to fine Sandy Silt, trace Clay, medium dense-moist 12						
20					22 12						
					Boring Terminated at 20'						

TBL_20G239-1.GPJ_SOCALGEO.GDT 12/24/20



JOB NO.: 20G239-1	DRILLING DATE: 12/4/20	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 13 feet
LOCATION: Perris, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS				DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)		GRAPHIC LOG	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	
SURFACE ELEVATION: MSL											
				YOUNGER ALLUVIUM: Brown Silty fine to medium Sand, medium dense-damp							
				OLDER ALLUVIUM: Brown Clayey fine Sand to fine Sandy Clay, medium dense to very dense, stiff to hard-damp to moist							
		24			103	4					
		22	4.5		120	12					
5		65	4.5		125	8					
		32/11"	4.5		131	8					
10		97/9"			121	9					
				Brown Silty fine Sand, little Clay, very dense-damp							
				Brown Clayey fine Sand, little Silt, dense-moist							
15		32				12					
				Brown Silty fine Sand, trace medium to coarse Sand, dense-moist							
20		34				9					
				Boring Terminated at 20'							

TBL_20G239-1.GPJ_SOCALGEO.GDT_12/24/20



JOB NO.: 20G239-1	DRILLING DATE: 12/4/20	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 16 feet
LOCATION: Perris, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS				DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)		GRAPHIC LOG	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	
SURFACE ELEVATION: MSL											
				YOUNGER ALLUVIUM: Brown fine Sandy Clay, little Silt, hard-damp							EI = 53 @ 0-5 feet
				Brown fine Sandy Silt, some Clay, medium dense-damp to moist							
5				OLDER ALLUVIUM: Brown fine Sandy Clay, little Silt, very stiff-damp to moist							
			4.5	Gray Brown Clayey fine to coarse Sand, medium dense to dense-damp							
10			4.5	Brown Silty fine Sand, trace Clay, trace medium to coarse Sand, medium dense-damp to moist							
15				Boring Terminated at 20'							

TBL_20G239-1.GPJ_SOCALGEO.GDT_12/24/20



JOB NO.: 20G239-1	DRILLING DATE: 12/4/20	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 15 feet
LOCATION: Perris, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS				DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)		GRAPHIC LOG	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	
SURFACE ELEVATION: MSL											
		26		YOUNGER ALLUVIUM: Brown Silty fine Sand, trace Clay, trace medium Sand, medium dense-damp		4					
		34		@ 3½ feet, little Clay		6					
5				OLDER ALLUVIUM: Brown Clayey fine Sand, trace medium Sand, slightly cemented, very dense-damp		6					
		50/6"		Brown Silty fine Sand, little Clay, very dense-moist		9					
10											
		68									
		50/5"		@ 13½ feet, trace Clay, cemented		9					
15											
		31		@ 18½ feet, trace Clay, dense-damp		5					
20											
		50/5"				11					
25											
Boring Terminated at 26'											

TBL_20G239-1.GPJ_SOCALGEO.GDT_12/24/20



JOB NO.: 20G239-1	DRILLING DATE: 12/4/20	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 7 feet
LOCATION: Perris, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS				DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)		GRAPHIC LOG	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	
SURFACE ELEVATION: MSL											
		18			<u>YOUNGER ALLUVIUM</u> : Brown Silty fine Sand, trace medium Sand, medium dense-damp		5				
		16			Brown Clayey fine Sand, little Silt, medium dense-damp		8				
5											
		50/4"			<u>OLDER ALLUVIUM</u> : Brown Silty fine Sand, trace Clay, very dense-damp to moist		11				
		50/6"					8				
10					Boring Terminated at 10'						

TBL_20G239-1.GPJ_SOCALGEO.GDT_12/24/20

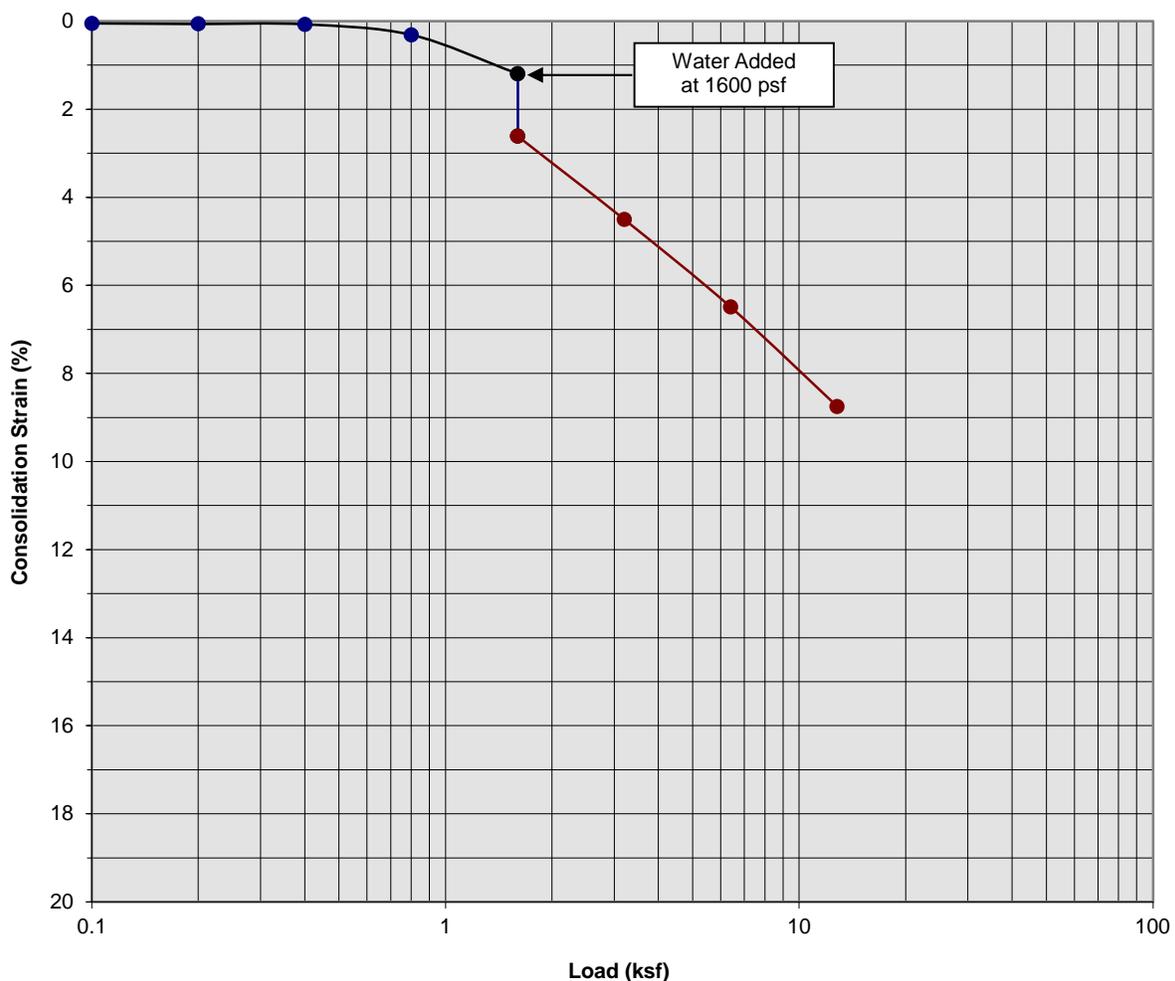


JOB NO.: 20G239-1	DRILLING DATE: 12/4/20	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 6 feet
LOCATION: Perris, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS				DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)		GRAPHIC LOG	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	
SURFACE ELEVATION: MSL											
					YOUNGER ALLUVIUM: Brown Silty fine Sand, trace fine root fibers, medium dense-damp OLDER ALLUVIUM: Brown Clayey fine Sand, trace fine root fibers, medium dense to very dense-damp		4 6				
5		27					6				
		32									
		36					7				
		71					6				
10					Boring Terminated at 10'						

TBL_20G239-1.GPJ_SOCALGEO.GDT_12/24/20

Consolidation/Collapse Test Results



Classification: Brown fine Sandy Silt, some Clay

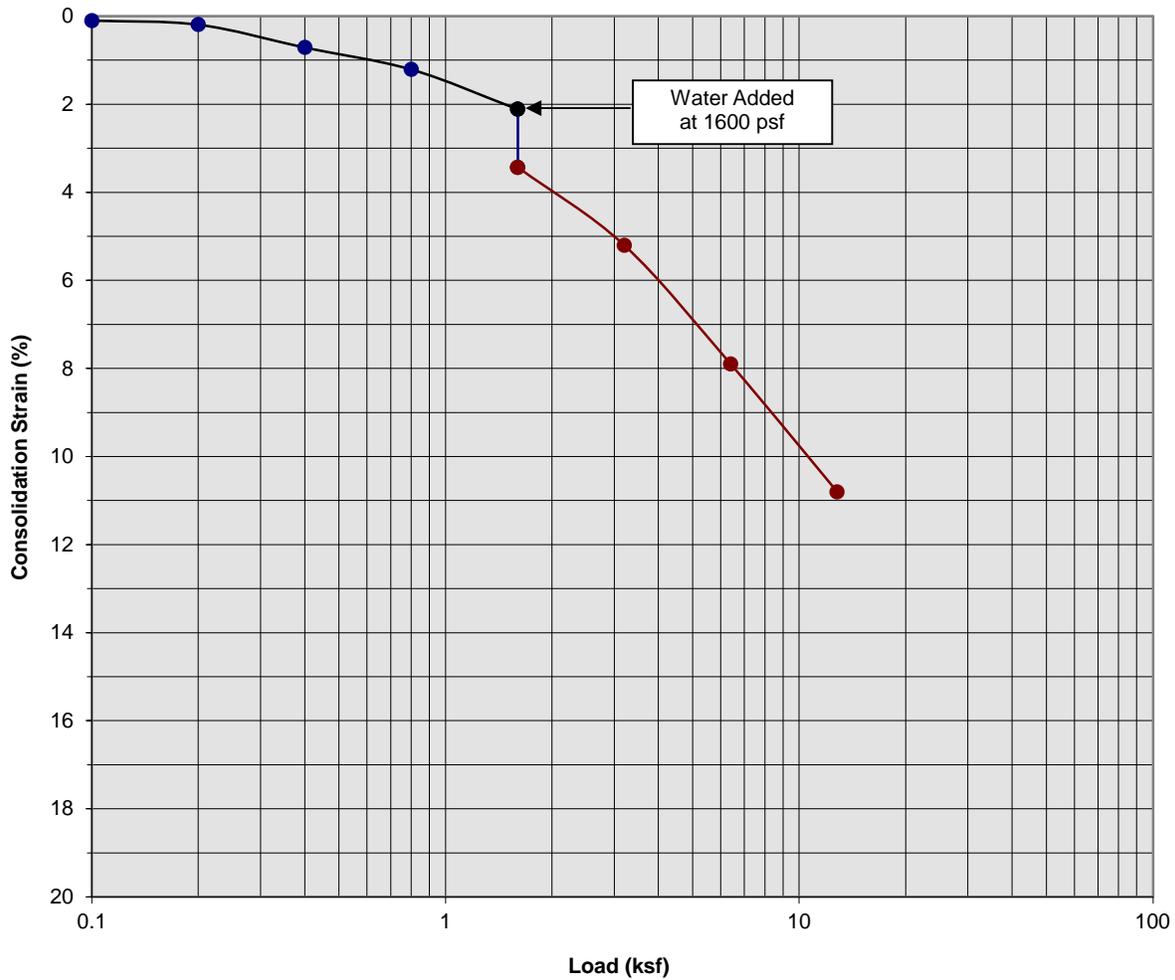
Boring Number:	B-4	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	3 to 4	Initial Dry Density (pcf)	123.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	135.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.42

Proposed Warehouse
 Perris, California
 Project No. 20G239-1
PLATE C-1



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Consolidation/Collapse Test Results



Classification: Brown fine Sandy Silt, some Clay

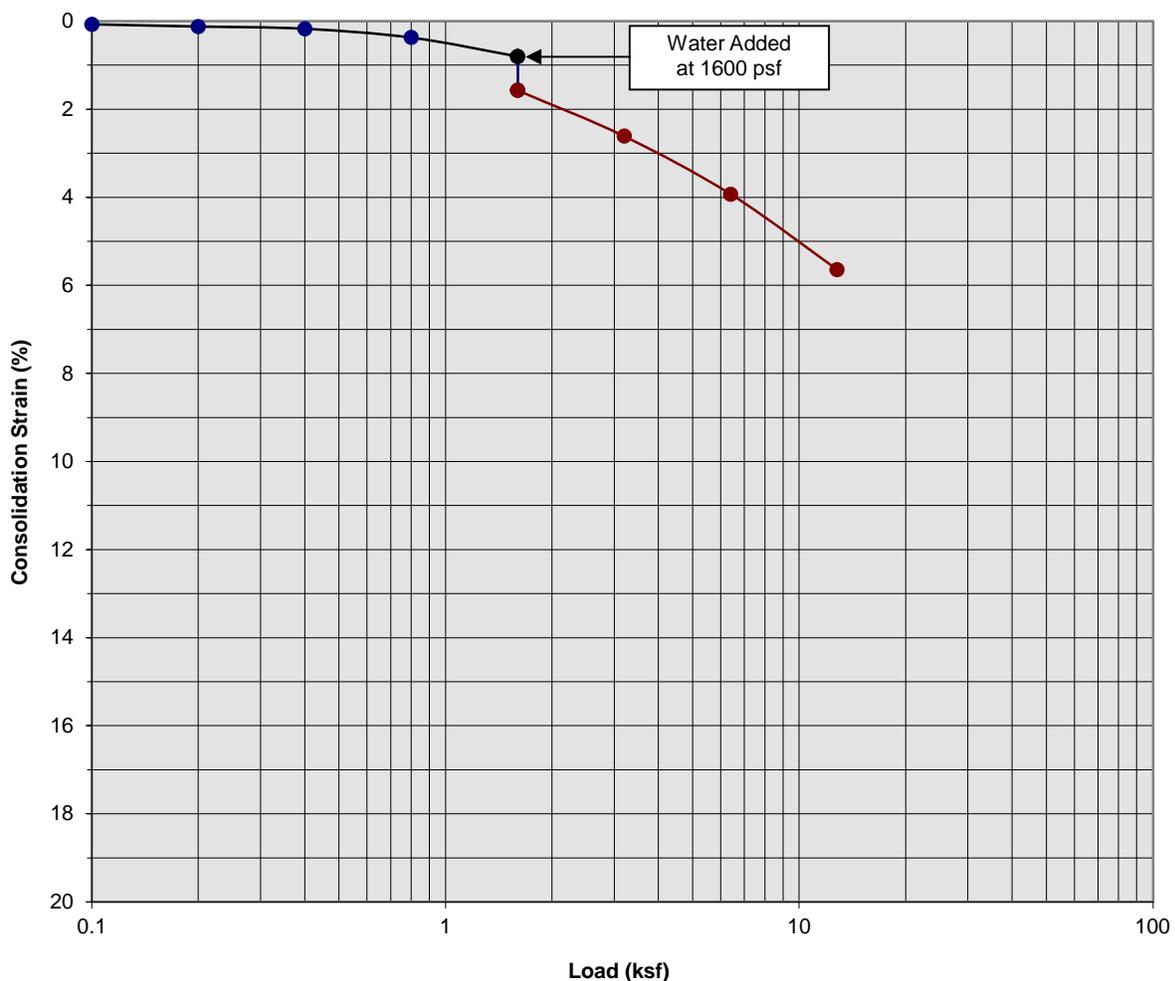
Boring Number:	B-4	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	5 to 6	Initial Dry Density (pcf)	118.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	132.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.32

Proposed Warehouse
 Perris, California
 Project No. 20G239-1
PLATE C-2



**SOUTHERN
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Consolidation/Collapse Test Results



Classification: Brown fine Sandy Clay, little Silt

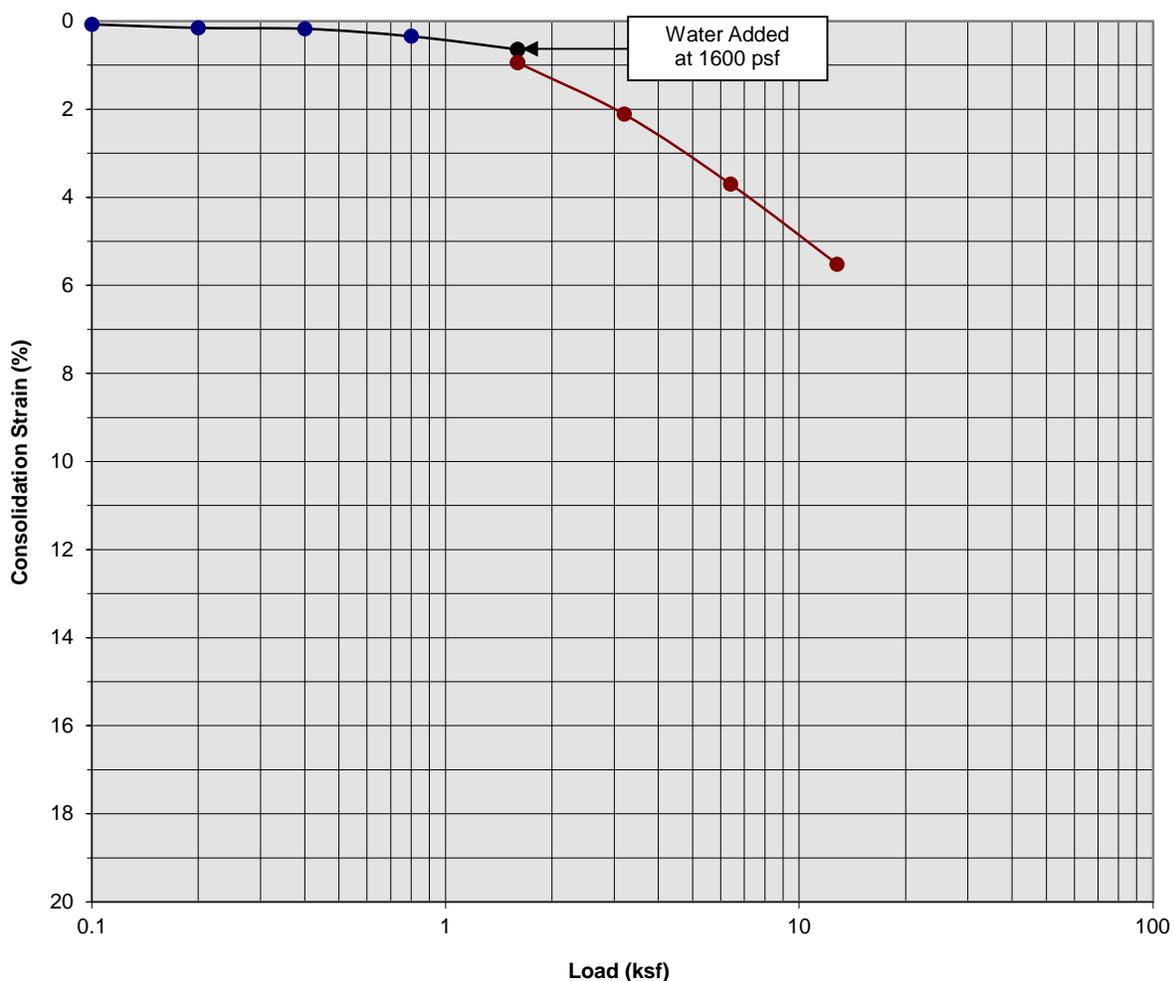
Boring Number:	B-4	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	7 to 8	Initial Dry Density (pcf)	130.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	137.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.77

Proposed Warehouse
 Perris, California
 Project No. 20G239-1
PLATE C-3



**SOUTHERN
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Consolidation/Collapse Test Results



Classification: Brown fine Sandy Clay, little Silt

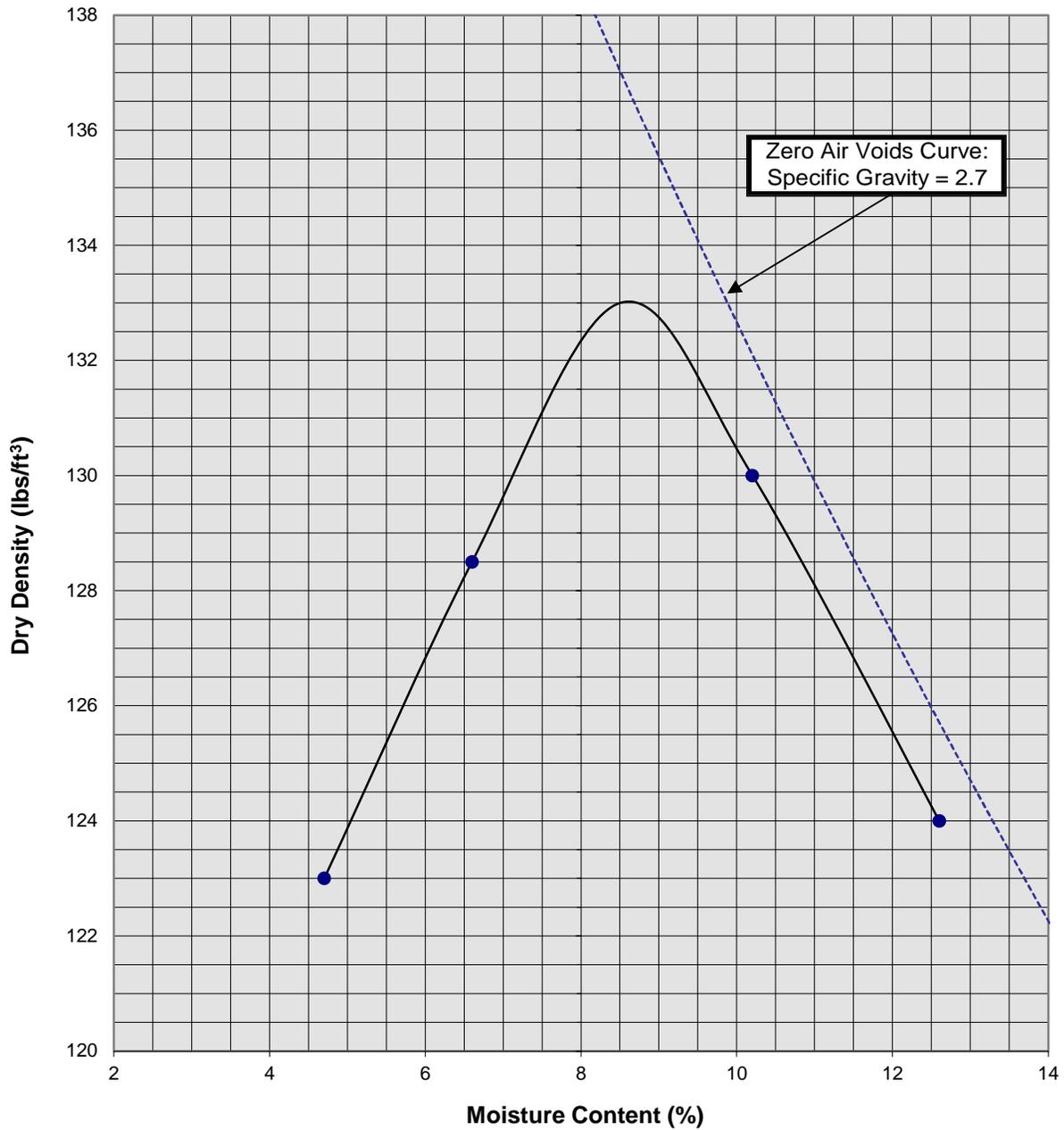
Boring Number:	B-4	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	9 to 10	Initial Dry Density (pcf)	117.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	124.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.30

Proposed Warehouse
 Perris, California
 Project No. 20G239-1
PLATE C-4



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Moisture/Density Relationship ASTM D-1557



Zero Air Voids Curve:
Specific Gravity = 2.7

Soil ID Number	B-2 @ 0 to 5'
Optimum Moisture (%)	8.5
Maximum Dry Density (pcf)	133
Soil Classification	Brown Clayey fine to medium Sand, trace Silt

Proposed Warehouse
Perris, California
Project No. 20G239-1
PLATE C-5



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