

**GEOTECHNICAL INVESTIGATION  
PROPOSED WAREHOUSE**

NEC Alabama Street and Pioneer Avenue  
Redlands, California  
for  
First Industrial Realty Trust, Inc.



**SOUTHERN  
CALIFORNIA  
GEOTECHNICAL**  
*A California Corporation*

December 13, 2019

First Industrial Realty Trust, Inc.  
898 North Pacific Coast Highway, Suite 175  
El Segundo, California 90245



**SOUTHERN  
CALIFORNIA  
GEOTECHNICAL**  
*A California Corporation*

Attention: Mr. Matt Pioli  
Investment Officer

Project No.: **19G237-1**

Subject: **Geotechnical Investigation**  
Proposed Warehouse  
NEC Alabama Street and Pioneer Avenue  
Redlands, 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 Robert G. Trazo in blue ink.

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



Handwritten signature of Gregory K. Mitchell in blue ink.

Gregory K. Mitchell, GE 2364  
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Distribution: (1) Addressee

# TABLE OF CONTENTS

<b>1.0 EXECUTIVE SUMMARY</b>	<b>1</b>
<b>2.0 SCOPE OF SERVICES</b>	<b>3</b>
<b>3.0 SITE AND PROJECT DESCRIPTION</b>	<b>4</b>
3.1 Site Conditions	4
3.2 Proposed Development	4
<b>4.0 SUBSURFACE EXPLORATION</b>	<b>5</b>
4.1 Scope of Exploration/Sampling Methods	5
4.2 Geotechnical Conditions	5
<b>5.0 LABORATORY TESTING</b>	<b>7</b>
<b>6.0 CONCLUSIONS AND RECOMMENDATIONS</b>	<b>9</b>
6.1 Seismic Design Considerations	9
6.2 Geotechnical Design Considerations	12
6.3 Site Grading Recommendations	14
6.4 Construction Considerations	17
6.5 Foundation Design and Construction	17
6.6 Floor Slab Design and Construction	19
6.7 Retaining Wall Design and Construction	20
6.8 Pavement Design Parameters	22
<b>7.0 GENERAL COMMENTS</b>	<b>24</b>
<b>APPENDICES</b>	
A Plate 1: Site Location Map Plate 2: Boring Location Plan	
B Boring Logs	
C Laboratory Test Results	
D Grading Guide Specifications	
E Seismic Design Parameters	

# 1.0 EXECUTIVE SUMMARY

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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.

## Site Preparation Recommendations

- The subject site is currently being used for agricultural purposes. The majority of the site is planted with citrus trees. The remaining portions of the site contain minor to moderate grass and weed growth or exposed soil. Site stripping will be necessary to remove these materials. All vegetation, including root masses, and any organic topsoil should be stripped and removed from the site.
- The near-surface soils consist of native alluvium. These soils generally consist of loose silty sands and fine sands.
- The underlying alluvium in the upper 12± feet possesses loose to medium dense relative densities. Based on the results of laboratory testing, some of the native alluvium present within the upper 6 to 8± feet possesses significant potential for consolidation settlement and hydrcollapse.
- Based on their variable relative densities and the potential for consolidation settlement and hydrocollapse, remedial grading is recommended to be performed within the proposed building pad area. The existing soils within the proposed building area should be overexcavated to a depth of 5 feet below existing grade and to a depth of 5 feet below proposed pad grade. The proposed foundation influence zones should be overexcavated to a depth of at least 4 feet below proposed foundation bearing grade. The materials exposed at the base of overexcavations should possess a minimum relative compaction of 85 percent of the maximum dry density as determined by ASTM D-1557 maximum dry density. The overexcavation should also extend to a sufficient depth to remove any artificial fill soils encountered during grading.
- Additional overexcavation may be necessary in localized areas, based on the presence of loose and collapsible soils present to depths of up to 8± feet at Boring No. B-1 and up to 6± feet at Boring No. B-4.
- After overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated. The resulting soils should be scarified and moisture conditioned to achieve a moisture content of 0 to 4 percent above optimum moisture, to a depth of at least 12 inches. The overexcavation subgrade soils should then be recompacted under the observation of the geotechnical engineer. The previously excavated soils may then be replaced as compacted structural fill.
- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, thoroughly moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

## Building Foundation Recommendations

- Spread footing foundations, supported in newly placed structural fill soils.
- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft<sup>2</sup>.

- Reinforcement consisting of at least two (2) No. 5 rebars (1 top and 1 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

### Building Floor Slab Recommendations

- Conventional Slab on Grade, at least 6 inches thick
- Modulus of Subgrade Reaction:  $k = 150$  psi/in
- Reinforcement is not considered to be necessary for geotechnical considerations.
- The actual thickness and reinforcement of the floor slabs should be determined by the structural engineer, based on the imposed slab loading.

### Pavements

ASPHALT PAVEMENTS (R=50)					
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	3	4	5	5	7
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R=50)				
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**

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The scope of services performed for this project was in accordance with our Proposal No. 19P188, dated March 29, 2019. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor 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**

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### **3.1 Site Conditions**

The subject site is located at the northeast corner of Alabama Street and Pioneer Avenue in an unincorporated portion of San Bernardino County, near Redlands, California. The site is bounded to the north by Palmetto Avenue, to the west by Alabama Street, to the south by Pioneer Avenue, and to the east by an existing commercial/industrial building. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The site consists of a nearly rectangular-shaped parcel, 22.7± acres in size. The site is presently utilized as an orange orchard. A residential structure, 1,300± ft<sup>2</sup> in size is located in the south-central area of the site. The ground surface cover in the northwestern portion of the site consists of wood chips. Ground surface cover throughout the remaining areas of the site generally consists of exposed soil, orange trees, and sparse to moderate native weed growth. Historical aerial photographs, readily available on the internet, show that citrus trees have been present throughout the site since 1938. During the time between 2016 and 2018, citrus trees were removed from the northwestern area of the site.

Detailed topographic information was not available at the time of this report. However, based on topographic information obtained from Google Earth, the site topography ranges from 1224± feet mean sea level (msl) in the northwest corner of the site, to 1251± feet msl in the east-central area of the site. The site topography generally slopes to the west at a gradient of 3± percent.

### **3.2 Proposed Development**

Based on the preliminary site plan (Scheme 5) provided to our office, the site will be developed with one (1) new warehouse located in the central area of the site. The building will be 451,110± ft<sup>2</sup> in size and dock-high doors will be constructed along the east and west sides of 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 area, 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 a conventional shallow foundation system with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 4 to 7 kips per linear foot, respectively.

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 2 to 3± feet are expected to be necessary to achieve the proposed site grades.

## **4.0 SUBSURFACE EXPLORATION**

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### **4.1 Scope of Exploration/Sampling Methods**

The subsurface exploration conducted for this project consisted of eight (8) borings advanced to depths of 15 to 25± feet below existing site grades. All of the borings were logged during drilling by a member of our staff.

The borings were advanced with hollow-stem augers, by a limited access track-mounted drilling rig. Representative bulk and relatively undisturbed soil samples were taken during drilling. Relatively undisturbed 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**

#### Wood Chips

A 6±-inch-thick layer of loose wood chips was encountered at the ground surface of Boring No. B-1.

#### Alluvium

Native alluvial soils were encountered beneath the wood chips at Boring No. B-1 and at the ground surface at the remaining boring locations, extending to at least the maximum depth explored of 25± feet. The native alluvial soils generally consist of loose to medium dense fine sands with variable amounts of silt, and silty fine sands with variable amounts of medium to coarse sand. Occasional layers of loose to medium dense fine to medium sand were encountered between depths of 4½ to 12± feet. Boring No. B-7 encountered a layer of dense well-graded sand with trace amounts of fine to coarse gravel from depths of 12 to 20± feet. Boring Nos. B-2 and B-6 encountered a fine sandy silt layer from 2½ to 5½± feet, and 17 to 22± feet, respectively.

## Groundwater

Free water was not encountered during the drilling of any of the borings. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater is considered to have existed at a depth in excess of 25± feet at the time of the subsurface exploration.

As part of our research, we reviewed readily available groundwater data in order to determine regional groundwater depths. Recent water level data was obtained from the California State Water Resources Control Board, GeoTracker, website, <http://geotracker.waterboards.ca.gov/>. The nearest monitoring well in this database with available groundwater depth data is located approximately 3,550± feet northwest of the site. Water level readings within this monitoring well indicate a groundwater level of 170± feet below the ground surface in August 2019.

## **5.0 LABORATORY TESTING**

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

### Maximum Dry Density and Optimum Moisture Content

Two representative bulk samples have been tested for their maximum dry densities and optimum moisture contents. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plates C-9 and C-10 in Appendix C of this report. This test is generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

### Soluble Sulfates

Representative samples of the near-surface soils were submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes

into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

<b><u>Sample Identification</u></b>	<b><u>Soluble Sulfates (%)</u></b>	<b><u>Sulfate Classification</u></b>
B-1 @ 0 to 5 feet	0.001	Not Applicable (S0)
B-6 @ 0 to 5 feet	<0.001	Not Applicable (S0)

### Corrosivity Testing

Representative bulk samples of the near-surface soils were submitted to a subcontracted corrosion engineering laboratory to determine if the near-surface soils possess 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.

<b><u>Sample Identification</u></b>	<b><u>Saturated Resistivity (ohm-cm)</u></b>	<b><u>pH</u></b>	<b><u>Chlorides (mg/kg)</u></b>	<b><u>Nitrates (mg/kg)</u></b>
B-1 @ 0 to 5 feet	13,600	7.5	4.9	17
B-6 @ 0 to 5 feet	12,000	7.1	1.9	25

## **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 in this report 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.

#### Seismic Design Parameters

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

Based on standards in place at the time of this report, the proposed development is expected to be designed in accordance with the requirements of the 2016 edition of the California Building Code (CBC). However, it is also possible that the proposed development may be designed using the 2019 CBC, which will be adopted on January 1, 2020. Therefore, this report provides design parameters for both the 2016 CBC and the 2019 CBC. Other design consultants should verify the version of the code under which the proposed development will be submitted.

The 2016 and 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](http://www.seismicmaps.org). This software application calculates seismic design parameters in accordance with several building code reference documents, including ASCE 7-10 and ASCE 7-16, upon which the 2016 CBC and 2019 CBC are based, respectively. 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 Plates E-1A (2016 CBC) and E-1B (2019 CBC) in Appendix E of this report. Based on this output, the following parameters may be utilized for the subject site:

### 2016 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	$S_s$	1.810
Mapped Spectral Acceleration at 1.0 sec Period	$S_1$	0.864
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	$S_{MS}$	1.810
Site Modified Spectral Acceleration at 1.0 sec Period	$S_{M1}$	1.296
Design Spectral Acceleration at 0.2 sec Period	$S_{DS}$	1.207
Design Spectral Acceleration at 1.0 sec Period	$S_{D1}$	0.864

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 11.8.4 applies to the proposed structures 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 $MCE_R$ Acceleration at 0.2 sec Period	$S_S$	1.966
Mapped $MCE_R$ Acceleration at 1.0 sec Period	$S_1$	0.762
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	$S_{MS}$	1.966
Site Modified Spectral Acceleration at 1.0 sec Period	$S_{M1}$	1.295
Design Spectral Acceleration at 0.2 sec Period	$S_{DS}$	1.310
Design Spectral Acceleration at 1.0 sec Period	$S_{D1}$	0.864

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 California Geological Survey (CGS) has not yet conducted detailed seismic hazards mapping in the area of the subject site. The general liquefaction susceptibility of the site was determined by research of the San Bernardino County Official Land Use Plan, General Plan, Geologic Hazard Overlay. Map FH31 for the Redlands Quadrangle indicates that the subject site is not located within an area of liquefaction susceptibility. Based on the mapping performed by the county of San Bernardino and the subsurface conditions encountered at the boring locations, including the lack of a static ground water table, liquefaction is not considered to be a design concern for this project.

## **6.2 Geotechnical Design Considerations**

### General

Based on the results of the laboratory testing, the upper portion of the near-surface soils generally possess loose relative densities, are compressible when loaded and may be subject to hydrocollapse when inundated with water. Slight porosity was also observed in occasional samples of the near-surface soils. The near-surface soils, in their present condition, are not considered suitable to support the foundation loads of the new building, and could result in excessive post-construction settlements. The native soils at greater depths generally exhibit more favorable consolidation/collapse characteristics and will experience less influence from the new foundation loads. Based on these considerations, remedial grading is warranted within the proposed building area in order to remove and replace the near surface soils as compacted structural fill.

As discussed in Section 3.1 of this report, our review of historical aerial photographs indicates that the site has been planted with citrus trees, since approximately 1938. Although only trace root fibers were encountered at the boring locations not currently occupied by citrus trees, it is unknown whether or not the root masses were removed with the former citrus trees. If any significant tree root masses are encountered during grading, they should be removed in their entirety, prior to the replacement of any soils as compacted structural fill. It should be noted that the volume loss from any tree root removals would be in addition to any volume loss due to shrinkage from soil compaction. It may be beneficial to perform some additional subsurface exploration at the subject site prior to the start of rough grading in order to determine if any significant root masses remain at the subject site.

### Settlement

Laboratory testing indicates that some of the near surface soils possess a potential for collapse when exposed to moisture infiltration. Some of these soils also possess a potential for consolidation when exposed to load increases in the range of those that will be exerted by the foundations of the new structure. The recommended remedial grading will remove most of these soils from within the zone of influence of the new foundations. The native alluvium that will remain in place below the recommended depth of overexcavation will not be significantly influenced by the foundation loads of the new structure. Provided that the recommended remedial grading is completed, the post construction settlements of the proposed structure are expected to be within tolerable limits.

### Soluble Sulfates

The results of the soluble sulfate testing indicate that the selected samples of the on-site soils 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 generally consist of sands and silty sands with no appreciable clay content. These materials have been visually classified as non-expansive. Therefore, no design considerations related to expansive soils are considered warranted for this site.

## Soluble Sulfates

The result of the soluble sulfate testing indicated a sulfate concentration of approximately 0.001 percent for the selected samples of the on-site soils. This concentration is considered to be not applicable (S0) with respect to the American Concrete Institute (ACI) Publication 318-14 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building area.

## Corrosion Potential

The results of laboratory testing indicate that the on-site soils possess saturated resistivity values of 12,000 to 13,600 ohm-cm, and pH values of 7.1 to 7.5. 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 not considered to be corrosive to ductile iron pipe. **However, SCG does not practice in the area of corrosion engineering. Therefore, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.**

Relatively low concentrations (1.9 to 4.9 mg/kg) of chlorides were detected in the samples submitted for corrosivity testing. In general, soils possessing chloride concentrations in excess of 350 to 500 parts per million (ppm) are considered to be corrosive with respect to steel reinforcement within reinforced concrete. Based on the lack of any significant chlorides in the tested sample, the site is considered to have a C1 chloride exposure in accordance with the American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary. Therefore, a specialized concrete mix design for reinforced concrete for protection against chloride exposure is not considered warranted.

## Shrinkage/Subsidence

Removal and recompaction of the artificial fill and near-surface native soils is estimated to result in an average shrinkage of 13 to 18 percent. Shrinkage estimates for the individual samples range between 3 and 25 percent based on the results of density testing and the assumption that the onsite soils will be compacted to about 92 percent of the ASTM D-1557 maximum dry density. It should be noted that the shrinkage estimate is based on the results of dry density testing performed on small-diameter samples of the existing soils taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving

several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1 to 0.15± feet. This estimate may be used for grading in areas that are underlain by native alluvial soils.

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

### Grading and Foundation Plan Review

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

### **6.3 Site Grading Recommendations**

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

#### Site Stripping

Stripping off the existing trees, dense grass and weeds present throughout the majority of the site should be stripped and disposed of off-site. Stripping should include any organic soils and any root masses from the existing trees as well as the trees previously removed from the 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 area in order to remove a portion of the near-surface native alluvial soils. Based on conditions encountered at the boring locations, the existing soils within the proposed building area are recommended to be overexcavated to a depth of at least 5 feet below existing grade and to a depth of at least 5 feet below proposed building pad subgrade elevation, whichever is greater. The depth of overexcavation should also be sufficient to remove any artificial fill soils.

Where not encompassed within the general building pad overexcavations, additional overexcavation should be performed within the influence zones of the new foundations, to provide

for a new layer of compacted structural fill extending to a depth of 4 feet below proposed bearing grade.

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

As discussed in previous sections of this report, it is unknown whether or not the root masses of the former citrus trees were removed at the time the trees were stripped from the site. If any significant tree root masses are encountered during grading, they should be removed in their entirety, prior to the replacement of any soils as compacted structural fill.

Following completion of the overexcavation, the subgrade soils within the building area should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. The materials exposed at the base of overexcavations should possess a minimum relative compaction of 85 percent of the maximum dry density as determined by ASTM D-1557 maximum dry density. Some localized areas of deeper excavation may be required if loose, porous, or low density native soils are encountered at the base of the overexcavation. **Based on the results of our laboratory testing, additional overexcavation will likely be necessary in the area of Boring Nos. B-1 and B-4, and perhaps other areas of the site, based on the presence of loose and collapsible soils present to depths of up to 8± feet and 6± feet, respectively, at these boring locations.**

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 to achieve a moisture content of 0 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The building pad area may then be raised to grade with previously excavated soils or imported structural fill.

#### Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining walls and non-retaining 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 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. Please note that any erection pads used to construct the walls are considered to be part of the foundation system. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

#### Treatment of Existing Soils: Parking and Drive Areas

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

Subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength alluvial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed 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 loose native soils 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± inches), near-horizontal lifts, moisture conditioned to within 0 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2016 CBC and the grading code of the city of Redlands and/or the county of San Bernardino.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

#### Imported Structural Fill

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

#### Utility Trench Backfill

In general, all utility trench backfill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. It is recommended that materials in excess of 3 inches in size not be used for utility trench backfill. Compacted trench backfill should conform to the requirements

of the local grading code, and more restrictive requirements may be indicated by city of Redlands and/or the county of San Bernardino. 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 are predominately granular in composition. These materials will likely be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 2h:1v. 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.

### Groundwater

The static groundwater table at this site is considered to exist at a depth greater than 25± feet. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

## **6.5 Foundation Design and Construction**

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by structural fill soils used to replace existing near-surface, potentially compressible and collapsible alluvial soils. These new structural fill soils are expected to extend to depths of at least 4 feet below proposed foundation bearing grade. Based on this subsurface profile, the proposed structure may be supported on conventional shallow foundations.

### Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft<sup>2</sup>.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Two (2) No. 5 rebars (1 top and 1 bottom).

- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressure presented above may be increased by one-third when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on geotechnical considerations; additional reinforcement 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 to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

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

### Estimated Foundation Settlements

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

### Lateral Load Resistance

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

- Passive Earth Pressure: 300 lbs/ft<sup>3</sup>
- Friction Coefficient: 0.30

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume

that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 2,500 lbs/ft<sup>2</sup>.

## **6.6 Floor Slab Design and Construction**

Subgrades which will support 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 new structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill soils. These fill soils are expected to extend to a depth of at least 5 feet below finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.
- Modulus of Subgrade Reaction:  $k = 150$  psi/in
- Minimum slab reinforcement: Reinforcement is not required for geotechnical conditions. 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 the minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area where such moisture sensitive floor coverings are anticipated. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. 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.
- Moisture condition the floor slab subgrade soils to 0 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.
- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

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

## **6.7 Retaining Wall Design and Construction**

Small retaining walls are expected to be necessary in the area of the new truck loading docks and may also be required to facilitate the new site grades. The parameters recommended for use in the design of these walls are presented below.

### Retaining Wall Design Parameters

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

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**

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

The walls should be designed using a soil-footing coefficient of friction of 0.30 and an equivalent passive pressure of 300 lbs/ft<sup>3</sup>. 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.

### Retaining Wall Foundation Design

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

### Seismic Lateral Earth Pressures

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

### Backfill Material

On-site soils may be used to backfill the retaining walls. However, all backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

It is recommended that a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls, be placed against the face on the back side 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. 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.

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 D1557-91). 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.

## **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 and sands. Based on their classification, these materials are expected to possess good to excellent pavement support characteristics, with R-values in the range of 50 to 60. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon an assumed R-value of 50. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

### Asphaltic Concrete

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

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

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

<b>ASPHALT PAVEMENTS (R=50)</b>					
<b>Materials</b>	<b>Thickness (inches)</b>				
	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	3	4	5	5	7
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:

<b>PORTLAND CEMENT CONCRETE PAVEMENTS (R=50)</b>				
<b>Materials</b>	<b>Thickness (inches)</b>			
	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**

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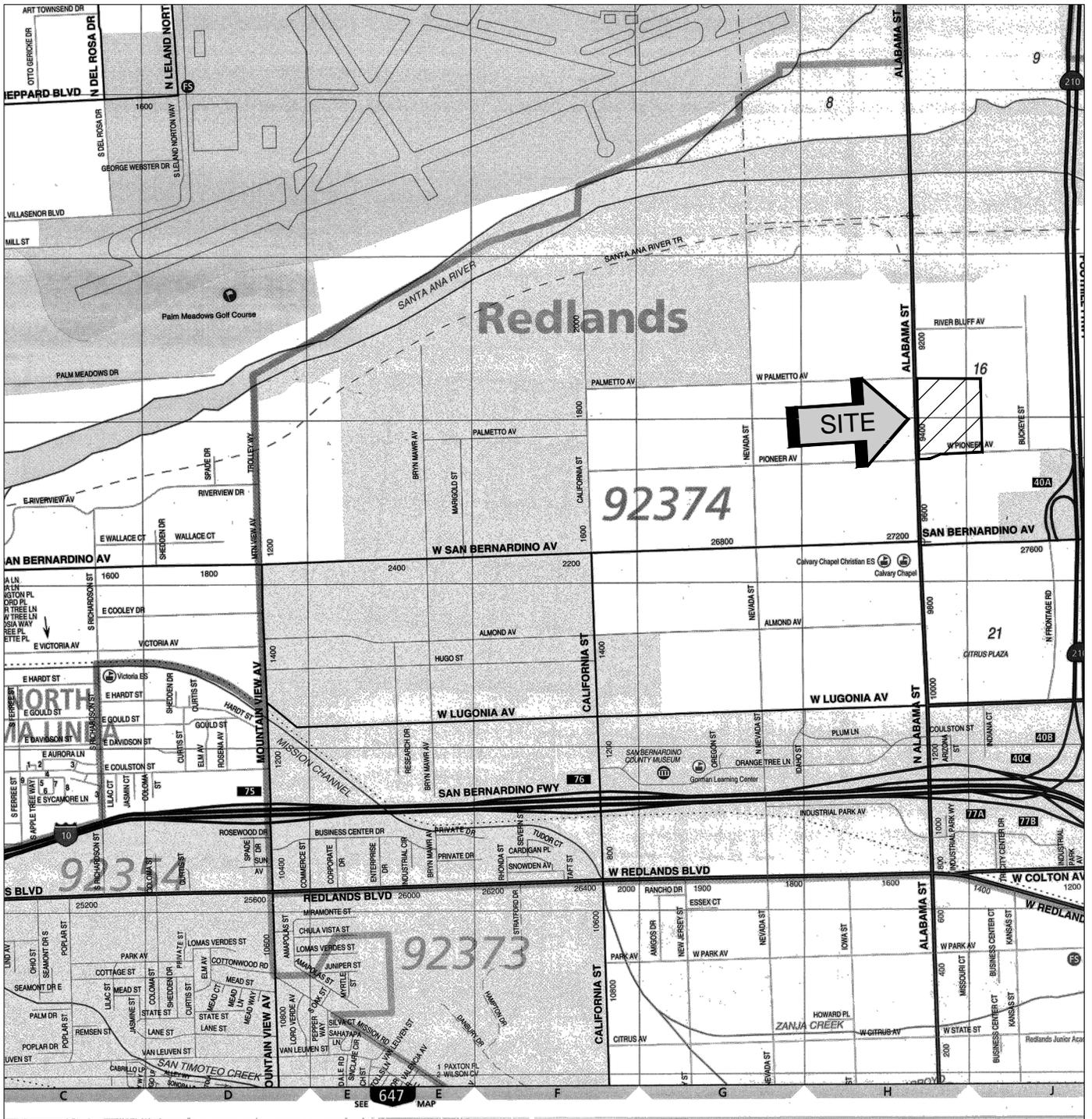
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other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

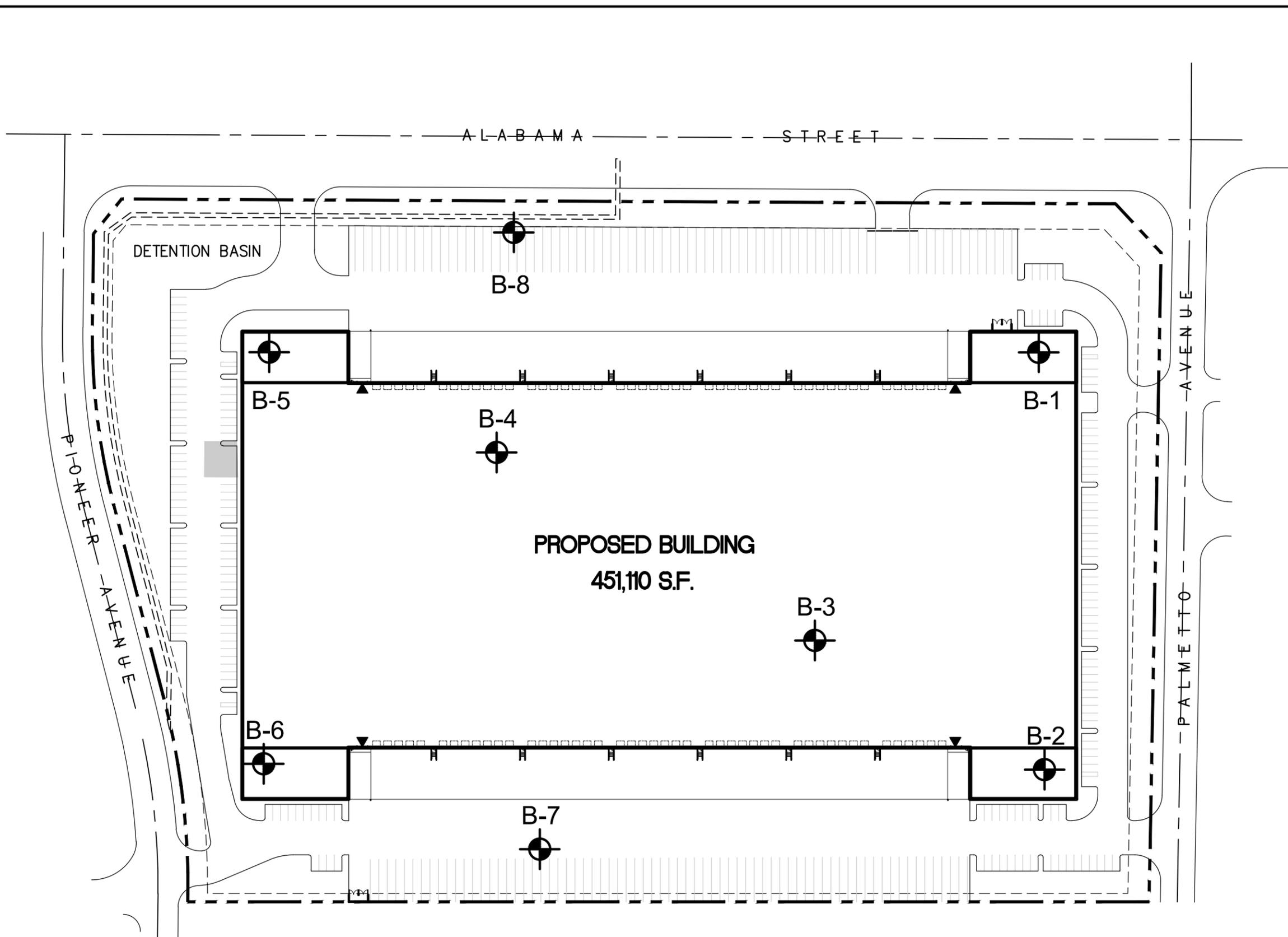
# APPENDIX A



SOURCE: SAN BERNARDINO COUNTY  
THOMAS GUIDE, 2009



<b>SITE LOCATION MAP</b>	
PROPOSED WAREHOUSE	
REDLANDS, CALIFORNIA	
SCALE: 1" = 2400'	
DRAWN: JAH	
CHKD: RGT	
SCG PROJECT 19G237-1	<b>SOUTHERN CALIFORNIA GEOTECHNICAL</b>
PLATE 1	



**GEOTECHNICAL LEGEND**

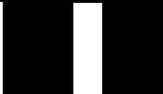
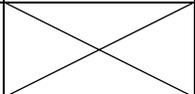
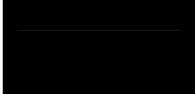
- APPROXIMATE BORING LOCATION
- EXISTING STRUCTURE TO BE DEMOLISHED

NOTE: CONCEPTUAL SITE PLAN PROVIDED BY HPA ARCHITECTURE.

<b>BORING LOCATION PLAN - SCHEME 5</b>	
PROPOSED WAREHOUSE	
REDLANDS, CALIFORNIA	
SCALE: 1" = 120'	 <b>SOUTHERN CALIFORNIA GEOTECHNICAL</b>
DRAWN: JAH	
CHKD: RGT	
SCG PROJECT 19G237-1	
<b>PLATE 2</b>	

# APPENDIX B

# BORING LOG LEGEND

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

## COLUMN DESCRIPTIONS

### DEPTH:

Distance in feet below the ground surface.

### SAMPLE:

Sample Type as depicted above.

### BLOW COUNT:

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

### POCKET PEN.:

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

### GRAPHIC LOG:

Graphic Soil Symbol as depicted on the following page.

### DRY DENSITY:

Dry density of an undisturbed or relatively undisturbed sample in lbs/ft<sup>3</sup>.

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS



JOB NO.: 19G237-1	DRILLING DATE: 12/2/19	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Redlands, California	LOGGED BY: Jamie Hayward	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													
				6± inch wood chips									
	X	11			ALLUVIUM: Gray Brown Silty fine Sand, trace medium Sand, trace mica, loose-damp to moist	100	7						
	X	11				92	8						
5	X	7				91	10						
	X	7			Gray Brown fine Sand, trace Silt, trace medium to coarse Sand, loose-damp	96	7						
10	X	9			Gray Brown Silty fine Sand, trace mica, loose to medium dense-damp	96	5						
15	X	10					9						
20	X	23			Gray Brown fine Sand, little Silt, medium dense-damp to moist		13						
25	X	20			@ 23½ feet, little medium to coarse Sand		6						
Boring Terminated at 25'													

TBL\_19G237.GPJ\_SOCALGEO.GDT\_12/13/19



JOB NO.: 19G237-1	DRILLING DATE: 12/2/19	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17½
LOCATION: Redlands, 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											
		6		ALLUVIUM: Gray Brown fine Sand, little Silt, trace medium Sand, loose-damp		6					
		5		Gray Brown Silty fine Sand, trace calcareous veining, loose-moist		11					
5		7		Gray Brown fine to medium Sand, trace Silt, loose-damp		7					
		6		Gray Brown Silty fine Sand, loose-moist		10					
10		12		Gray Brown fine Sand, little Silt, medium dense-moist	93	14					
		12		Gray Brown fine Sand, little Silt, medium dense-moist		10					
15		13		Gray Brown Silty fine Sand, medium dense-moist		10					
20				Boring Terminated at 20'							

TBL\_19G237.GPJ\_SOCALGEO.GDT\_12/13/19



JOB NO.: 19G237-1	DRILLING DATE: 12/2/19	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 12½
LOCATION: Redlands, California	LOGGED BY: Jamie Hayward	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												
				ALLUVIUM: Gray Brown fine Sand, trace Silt, loose-damp								
12	X	12			95	3						
9	X	9			94	5						
5	X	10		Gray Brown Silty fine Sand, trace to little mica, trace fine root fibers, trace pores, loose-very moist	79	23						
	X	11		Gray Brown Silty fine to medium Sand, trace coarse Sand, trace Iron oxide staining, loose-very moist	85	16						
10	X	11		Gray Brown fine Sand, little Silt, loose to medium dense-moist	91	11						
	X	10				11						
15	X											
Boring Terminated at 15'												

TBL\_19G237.GPJ\_SOCALGEO.GDT\_12/13/19



JOB NO.: 19G237-1	DRILLING DATE: 12/2/19	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 13 feet
LOCATION: Redlands, California	LOGGED BY: Jamie Hayward	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	7			ALLUVIUM: Gray Brown fine Sand, little Silt, trace fine root fibers, loose-damp	94	5					Organic Content = 0.8%
	X	9			Gray Brown Silty fine Sand, trace medium Sand, trace to little mica, trace fine root fibers, loose-damp to very moist	97	6					Organic Content = 0.8%
5	X	10			@ 5 feet, porous	89	15					
	X	12			Light Gray Brown fine Sand, loose-damp	92	5					
10	X	11			@ 9 feet, trace Silt, trace mica	90	7					
	X	13		@ 13½ feet, medium dense		8						
15					Boring Terminated at 15'							

TBL\_19G237.GPJ\_SOCALGEO.GDT\_12/13/19



JOB NO.: 19G237-1	DRILLING DATE: 12/2/19	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 7 feet
LOCATION: Redlands, California	LOGGED BY: Jamie Hayward	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												
				ALLUVIUM: Gray Brown fine Sand, trace medium to coarse Sand, loose to medium dense-damp		2						
5	X	10				2						
				@ 6 feet, 1-inch lense of Silty Clay		6						
10	X	15				7						
				Gray Brown Silty fine Sand, medium dense-damp to very moist		14						
15	X	10				17						
				Gray Brown fine Sand, medium dense-damp		4						
20	X	27										
				Boring Terminated at 20'								

TBL\_19G237.GPJ\_SOCALGEO.GDT\_12/13/19



JOB NO.: 19G237-1	DRILLING DATE: 12/2/19	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Redlands, California	LOGGED BY: Jamie Hayward	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												
				ALLUVIUM: Gray Brown fine Sand, trace to some medium Sand, medium dense-dry to damp	101	1						
				@ 5 to 6 feet, trace fine to coarse Gravel	100	2						
5	X	16		@ 5 to 6 feet, trace fine to coarse Gravel	101	2						
				Gray Brown Silty fine Sand, trace fine root fibers, medium dense-damp to moist	92	4						
				@ 18½ to 25 feet, very moist	90	9						
10	X	15		@ 18½ to 25 feet, very moist								
						8						
15	X	12										
20	X	17				16						
25	X	17				16						
					Boring Terminated at 25'							

TBL\_19G237.GPJ\_SOCALGEO.GDT\_12/13/19



JOB NO.: 19G237-1	DRILLING DATE: 12/2/19	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 8 feet
LOCATION: Redlands, California	LOGGED BY: Jamie Hayward	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												
				ALLUVIUM: Gray Brown fine Sand, trace Silt, loose-damp		5						
5	X	5		Gray Brown Silty fine Sand, loose-moist to very moist		7						
7	X	7				17						
10	X	14		Gray Brown fine Sand, trace fine Gravel, medium dense-damp to moist		9						
15	X	37		Gray Brown fine to coarse Sand, trace fine to coarse Gravel, medium dense to dense-damp		3						
20	X	23				5						
Boring Terminated at 20'												

TBL\_19G237.GPJ\_SOCALGEO.GDT\_12/13/19



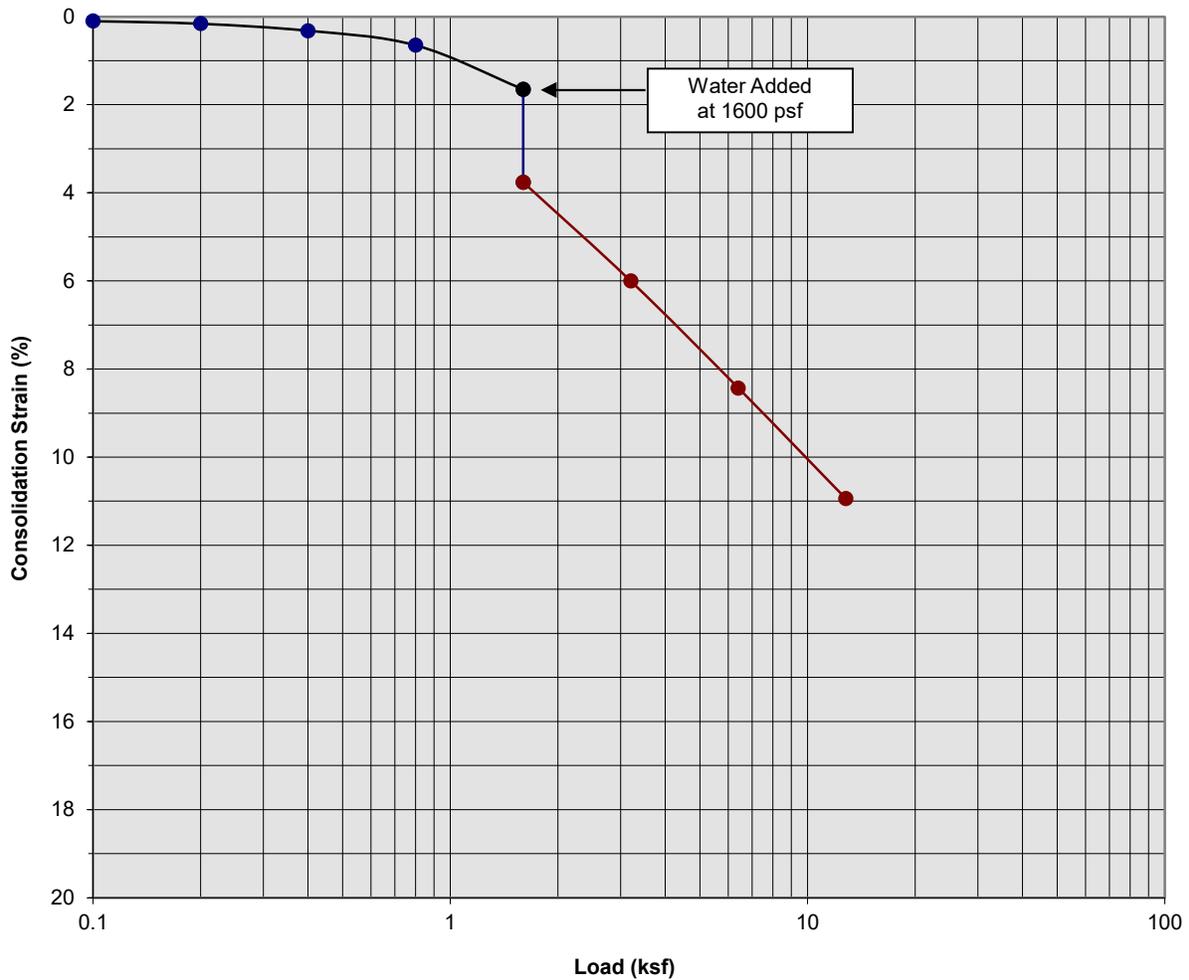
JOB NO.: 19G237-1	DRILLING DATE: 12/2/19	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 7 feet
LOCATION: Redlands, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

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											
		6		ALLUVIUM: Gray Brown fine Sand, trace Silt, trace fine root fibers, loose-damp		2					Organic Content = 1.0%
5		5		@ 3½ feet, some Silt		4					Organic Content = 1.2%
		7		@ 6 to 7½ feet, very moist		16					
10		15		Light Gray Brown fine to medium Sand, trace coarse Sand, medium dense-damp		2					
15		15		Gray Brown fine Sand, little Silt, medium dense-damp		7					
20		25		@ 18½ feet, moist		8					
Boring Terminated at 20'											

TBL\_19G237.GPJ\_SOCALGEO.GDT\_12/13/19

# APPENDIX C

### Consolidation/Collapse Test Results



Classification: Gray Brown Silty fine Sand, trace medium Sand

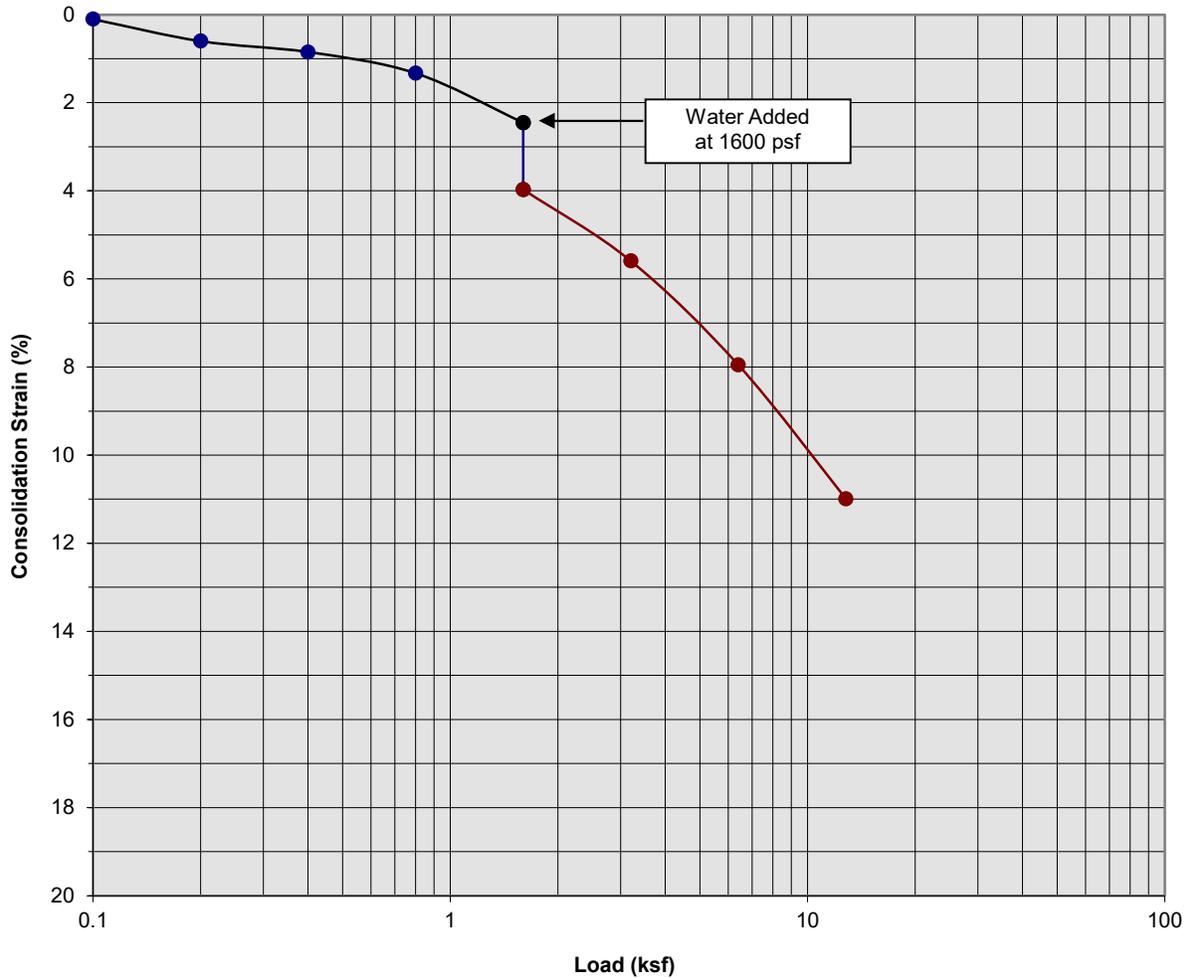
Boring Number:	B-1	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	3 to 4	Initial Dry Density (pcf)	92.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	103.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.11

Proposed Warehouse  
 Redlands, California  
 Project No. 19G237-1  
**PLATE C- 1**



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### Consolidation/Collapse Test Results



Classification: Gray Brown Silty fine Sand

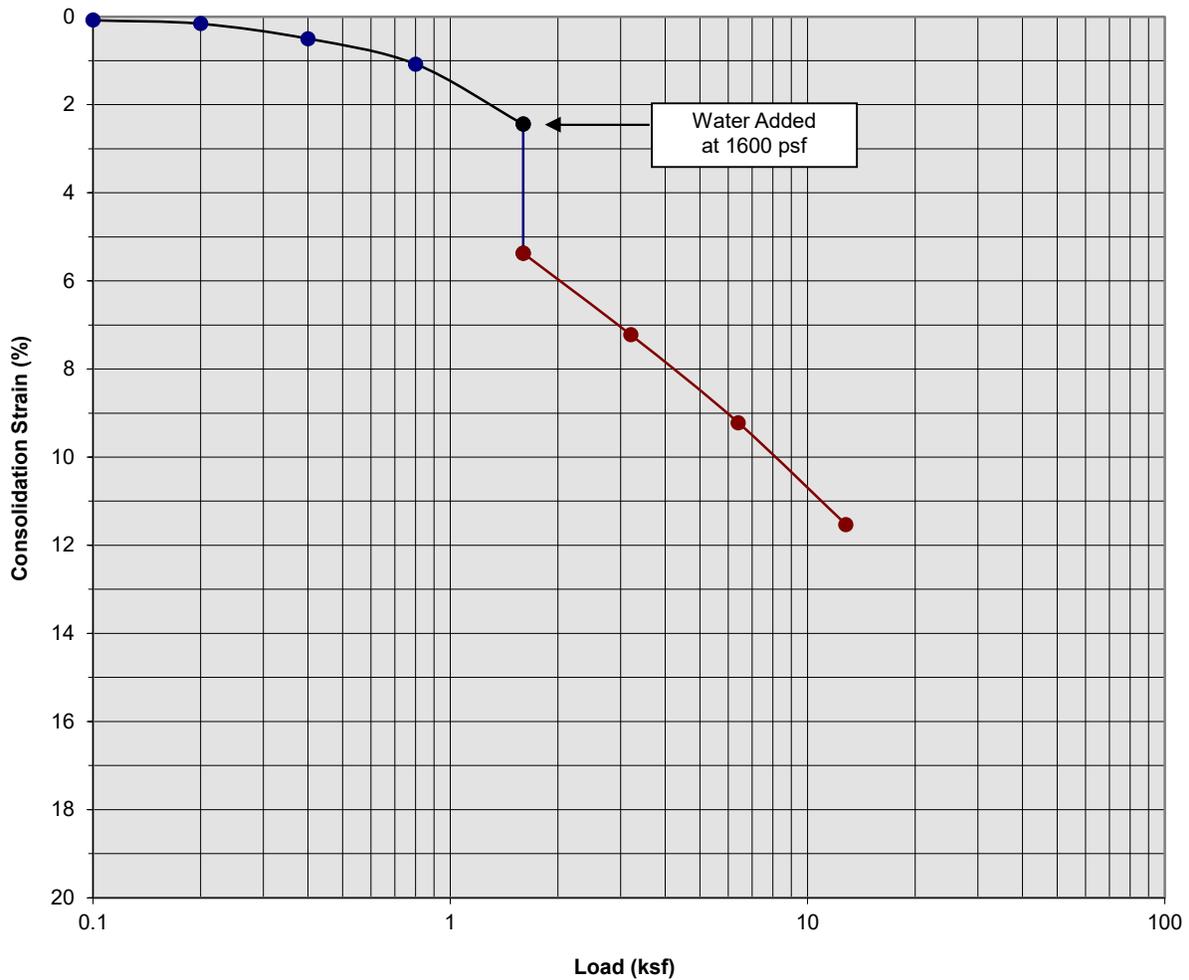
Boring Number:	B-1	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	25
Depth (ft)	5 to 6	Initial Dry Density (pcf)	91.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	102.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.52

Proposed Warehouse  
 Redlands, California  
 Project No. 19G237-1  
**PLATE C- 2**



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### Consolidation/Collapse Test Results



Classification: Gray Brown fine Sand, trace Silt, trace medium to coarse Sand

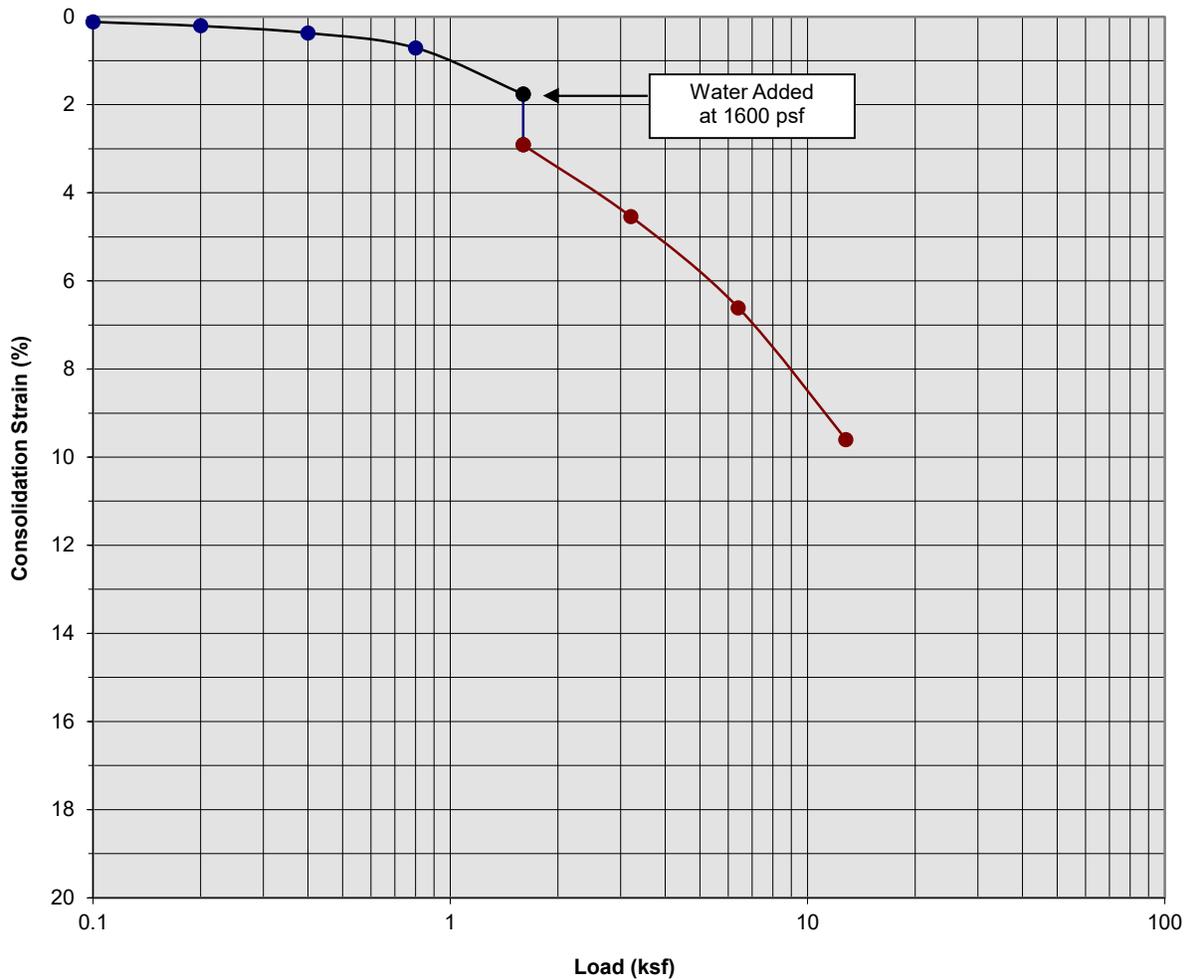
Boring Number:	B-1	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	7 to 8	Initial Dry Density (pcf)	95.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.93

Proposed Warehouse  
Redlands, California  
Project No. 19G237-1  
**PLATE C- 3**



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### Consolidation/Collapse Test Results



Classification: Gray Brown fine Sand and Silty fine Sand

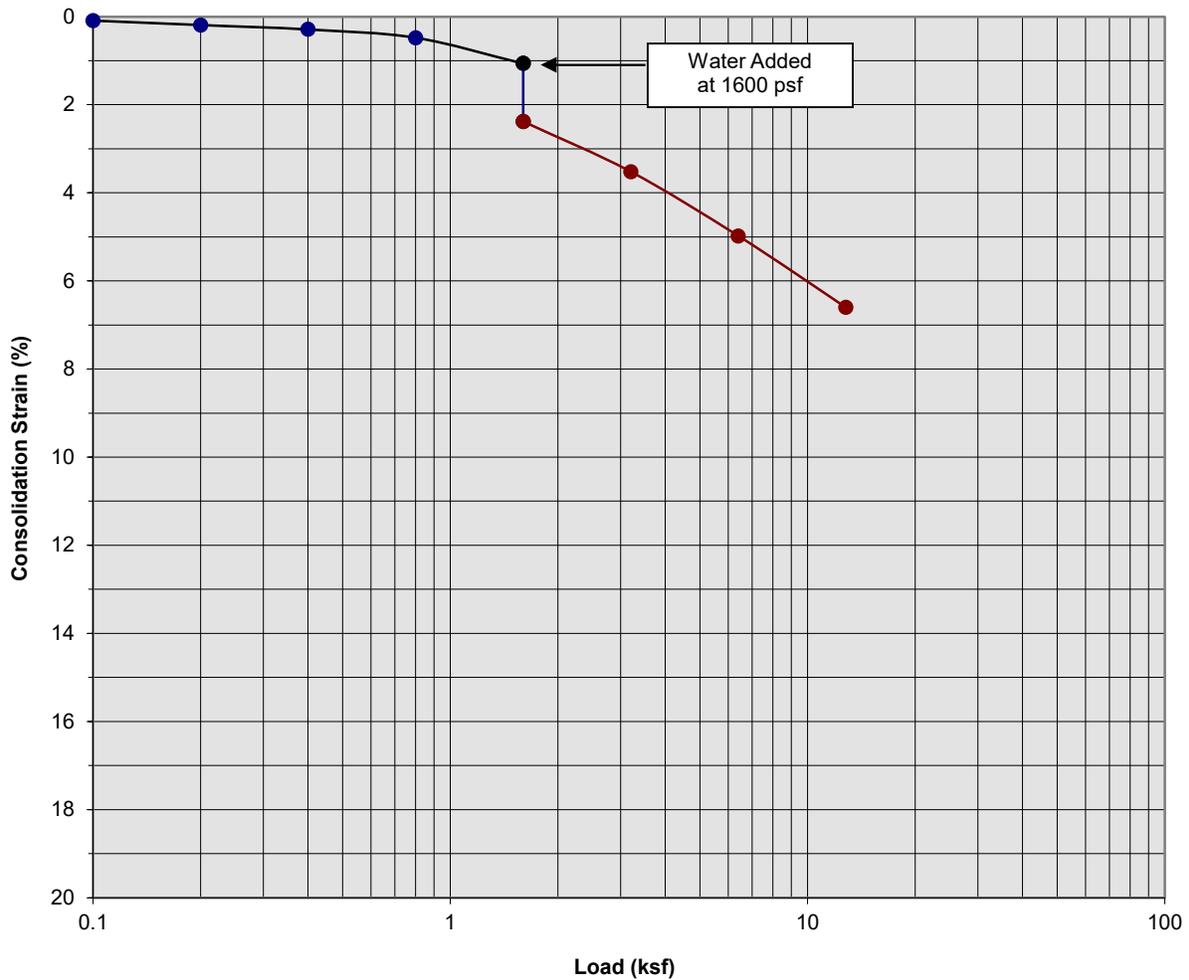
Boring Number:	B-1	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	27
Depth (ft)	9 to 10	Initial Dry Density (pcf)	96.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	106.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.15

Proposed Warehouse  
 Redlands, California  
 Project No. 19G237-1  
**PLATE C- 4**



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### Consolidation/Collapse Test Results



Classification: Gray Brown Silty fine Sand, trace medium Sand

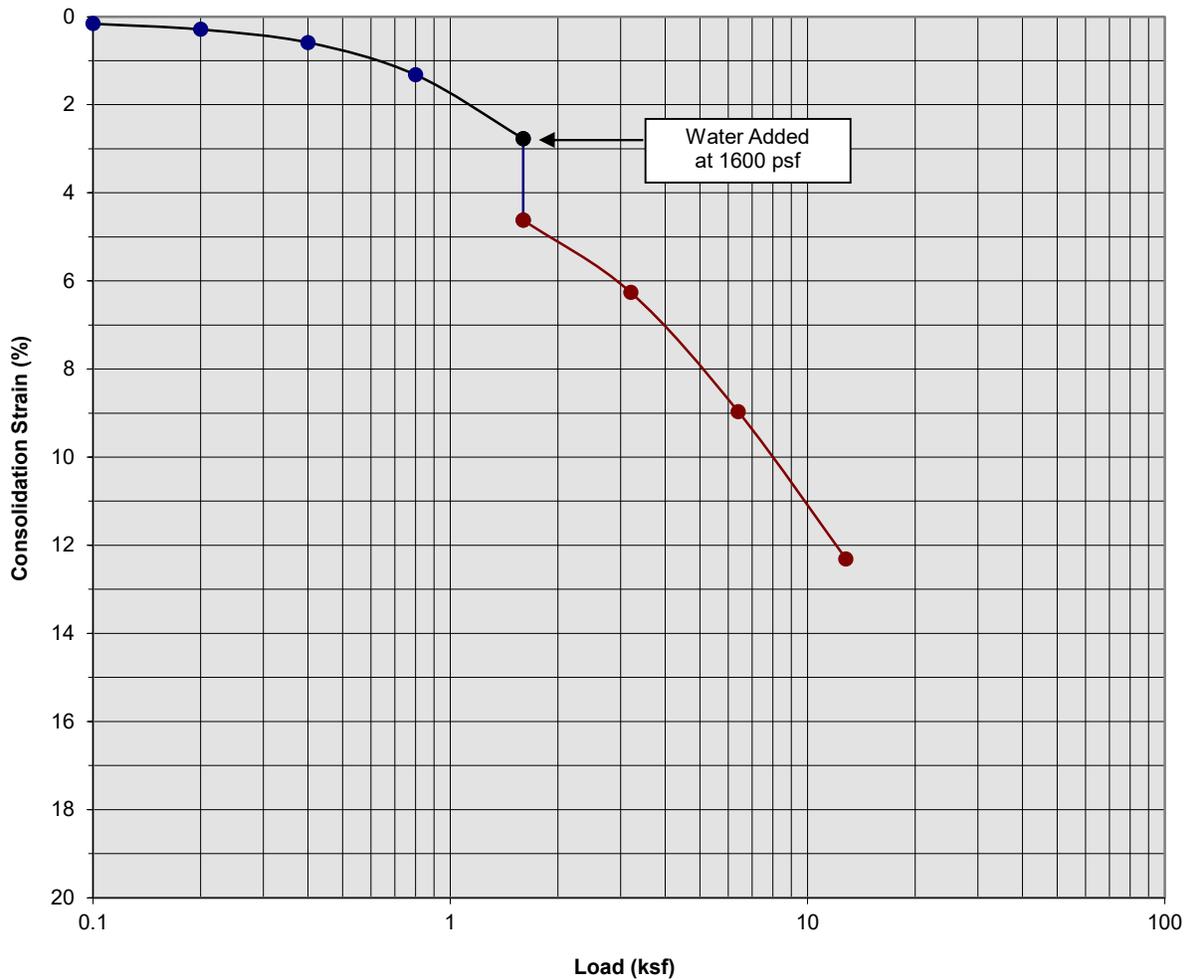
Boring Number:	B-4	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	3 to 4	Initial Dry Density (pcf)	97.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	104.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.32

Proposed Warehouse  
 Redlands, California  
 Project No. 19G237-1  
**PLATE C- 5**



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### Consolidation/Collapse Test Results



Classification: Gray Brown Silty fine Sand, trace medium Sand

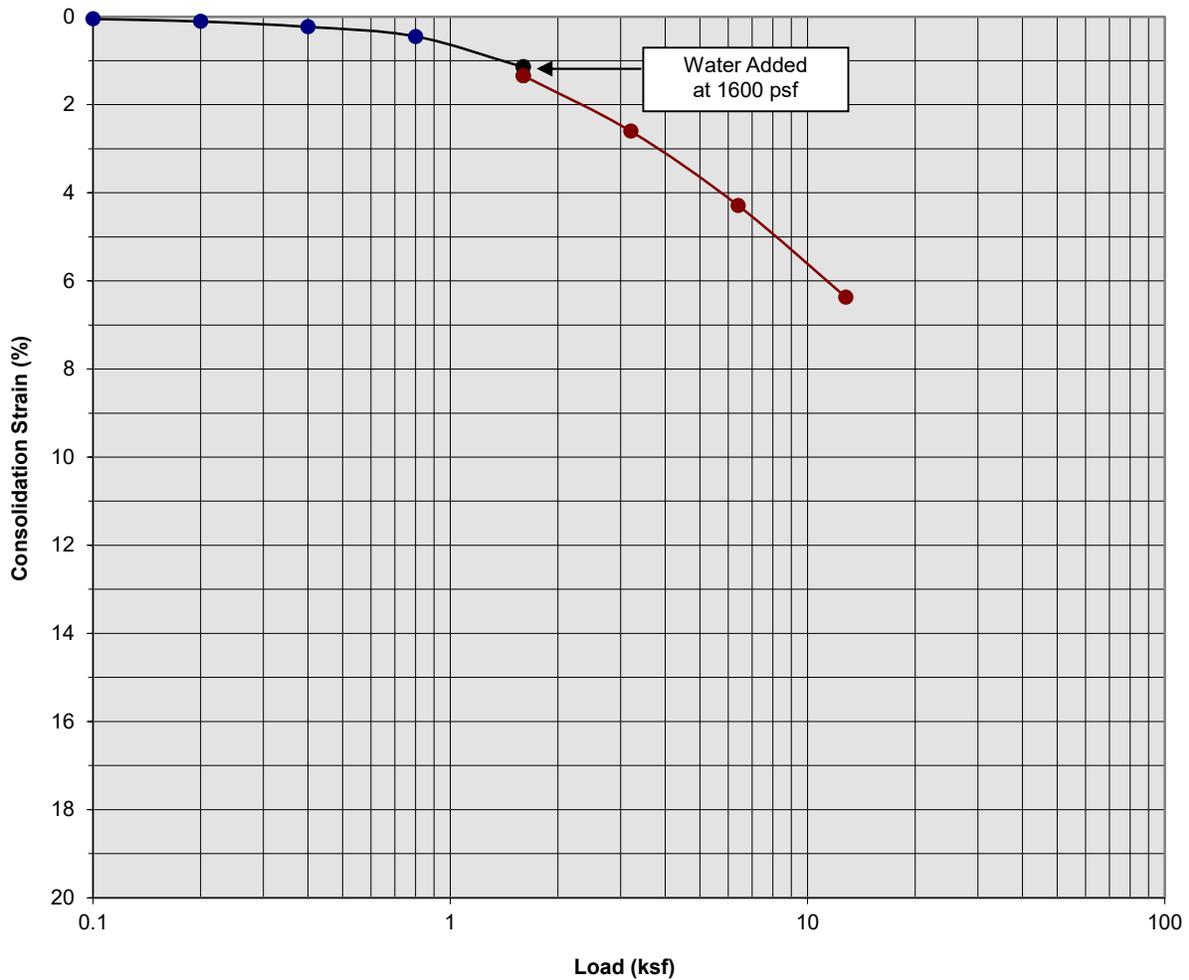
Boring Number:	B-4	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	27
Depth (ft)	5 to 6	Initial Dry Density (pcf)	89.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	101.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.85

Proposed Warehouse  
 Redlands, California  
 Project No. 19G237-1  
**PLATE C- 6**



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### Consolidation/Collapse Test Results



Classification: Light Gray Brown fine Sand

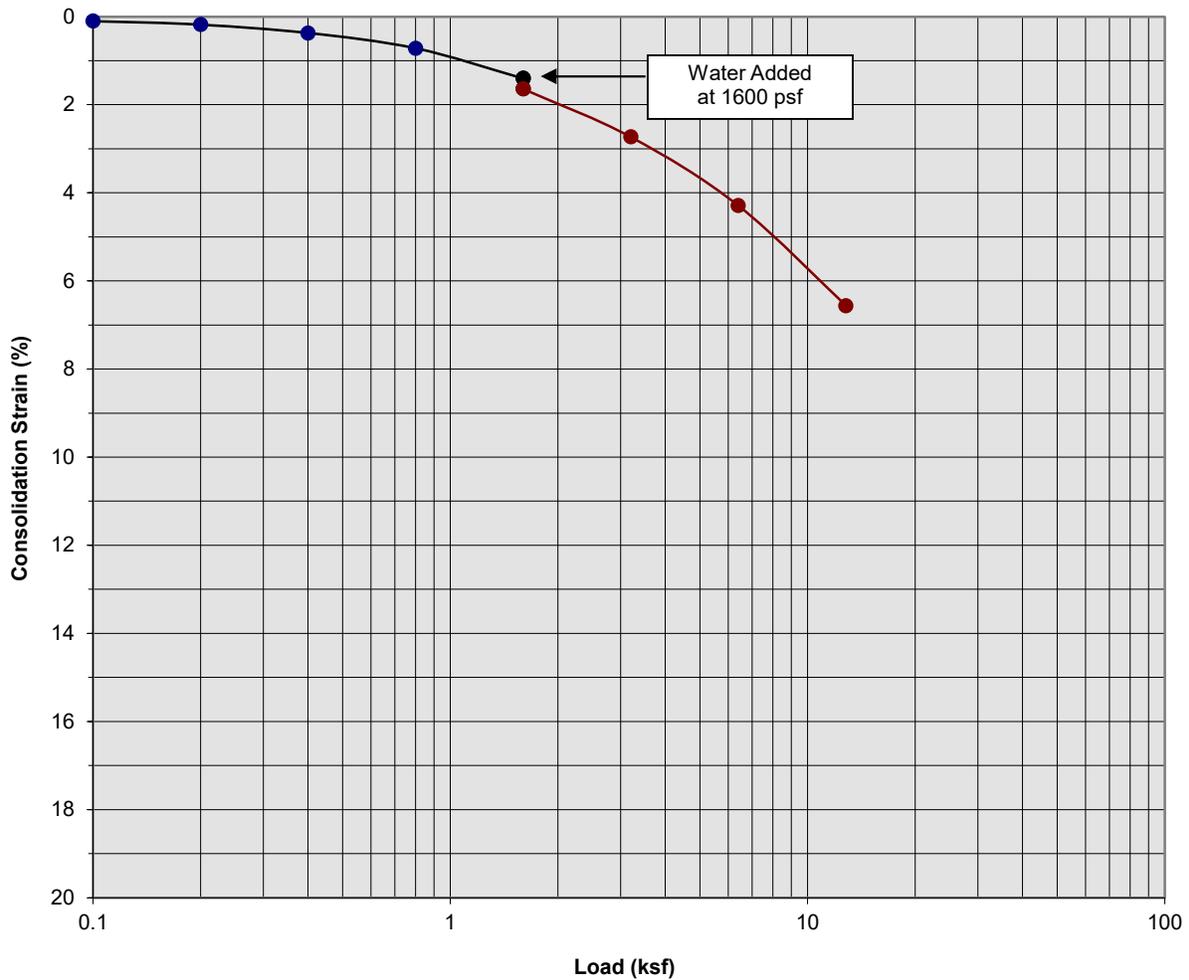
Boring Number:	B-4	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	7 to 8	Initial Dry Density (pcf)	92.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	98.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.20

Proposed Warehouse  
 Redlands, California  
 Project No. 19G237-1  
**PLATE C- 7**



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### Consolidation/Collapse Test Results



Classification: Light Gray Brown fine Sand

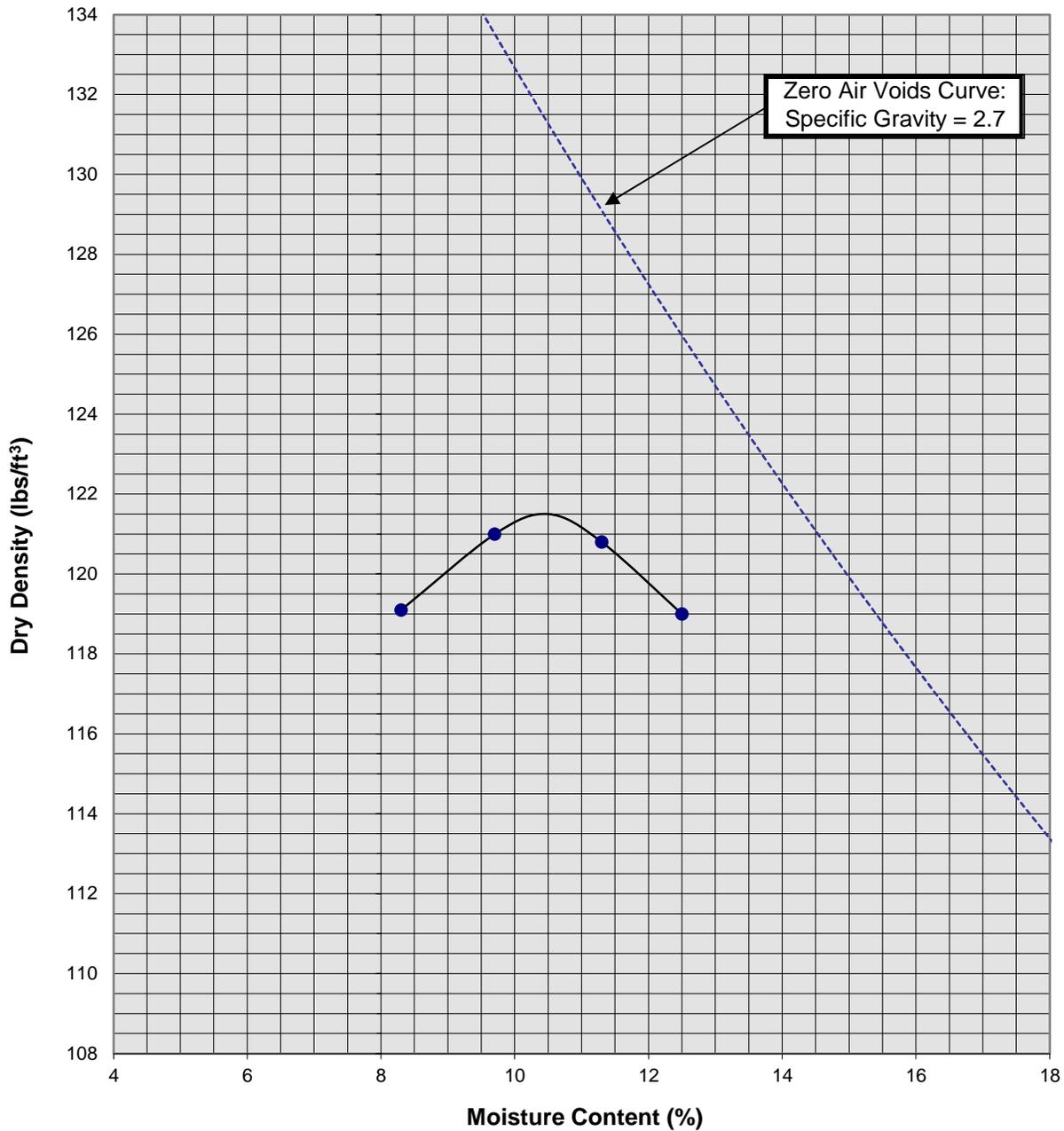
Boring Number:	B-4	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	23
Depth (ft)	9 to 10	Initial Dry Density (pcf)	90.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	96.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.24

Proposed Warehouse  
 Redlands, California  
 Project No. 19G237-1  
**PLATE C- 8**



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### Moisture/Density Relationship ASTM D-1557



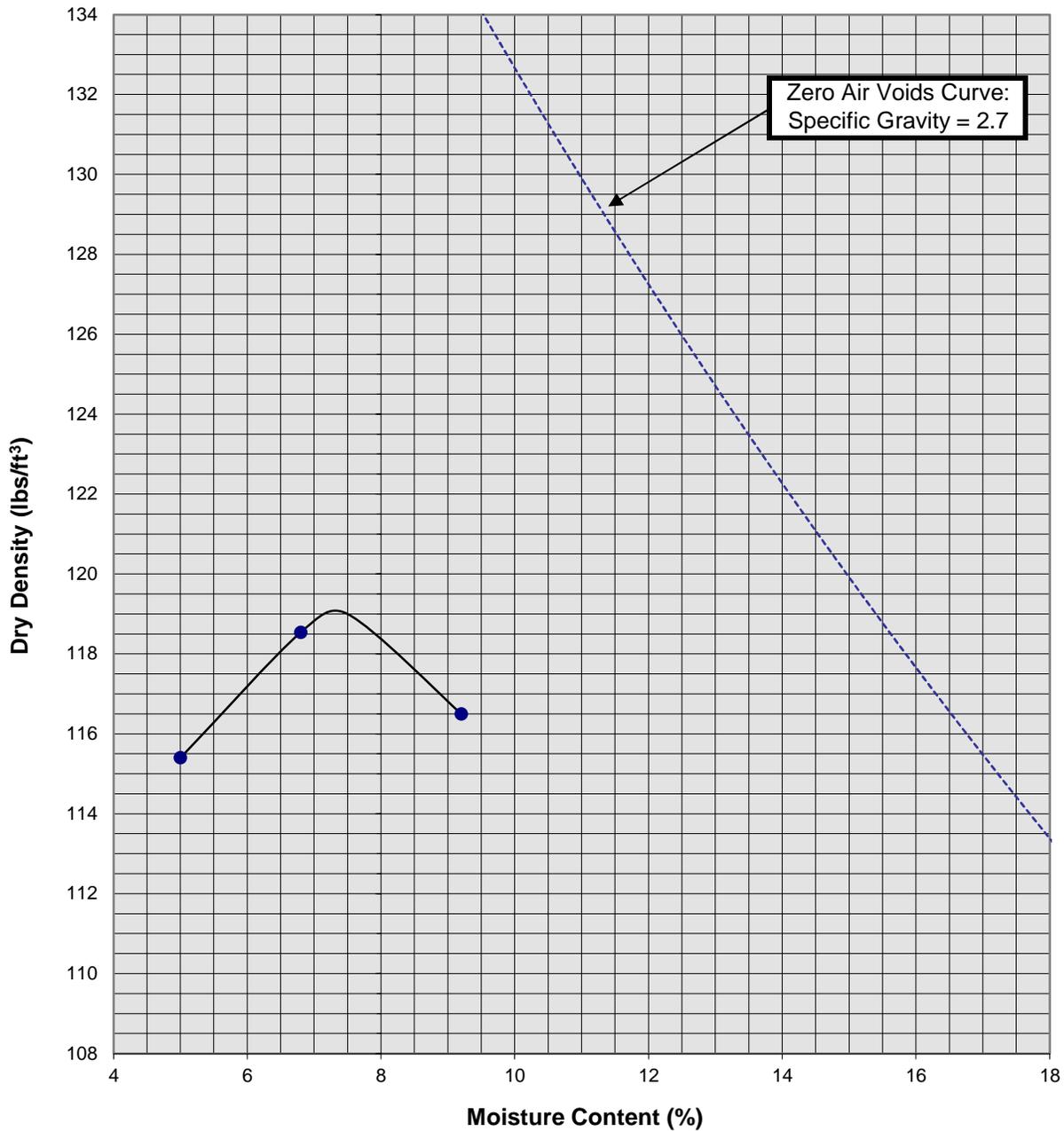
Soil ID Number	B-1 @ 0-5'
Optimum Moisture (%)	10.5
Maximum Dry Density (pcf)	121.5
Soil Classification	Gray Brown Silty fine Sand, trace medium Sand

Proposed Warehouse  
Redlands, California  
Project No. 19G237-1  
**PLATE C-9**



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### Moisture/Density Relationship ASTM D-1557



Soil ID Number	B-6 @ 0-5'
Optimum Moisture (%)	7.5
Maximum Dry Density (pcf)	119
Soil Classification	Gray Brown Silty Sand, trace medium Sand

Proposed Warehouse  
Redlands, California  
Project No. 19G237-1  
**PLATE C-10**



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# APPENDIX

## **GRADING GUIDE SPECIFICATIONS**

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

### 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

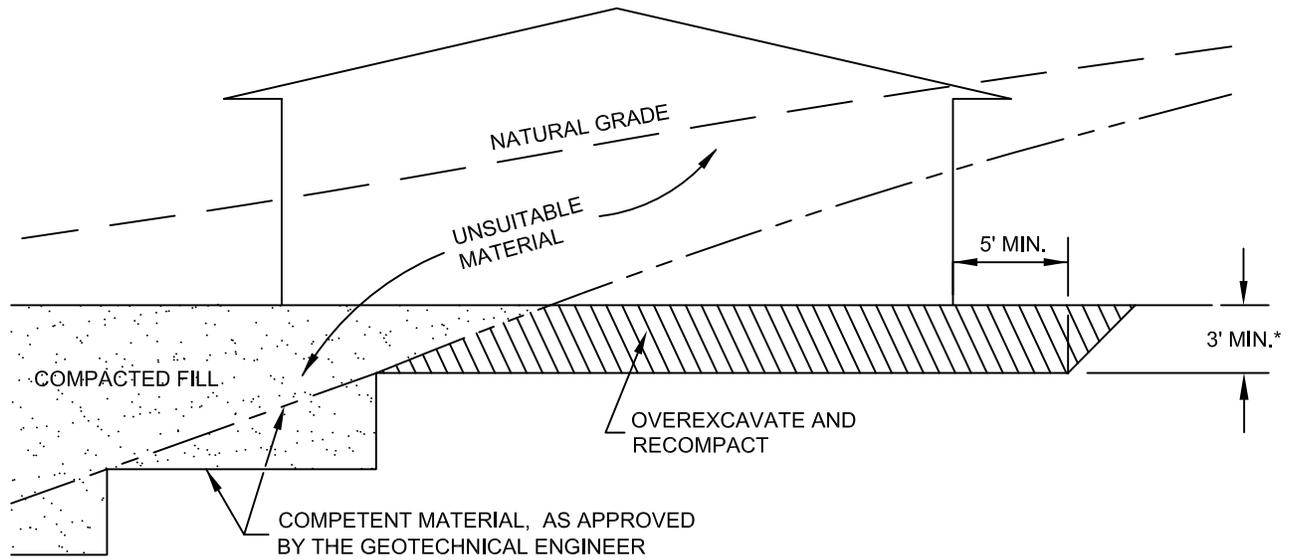
- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.
-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

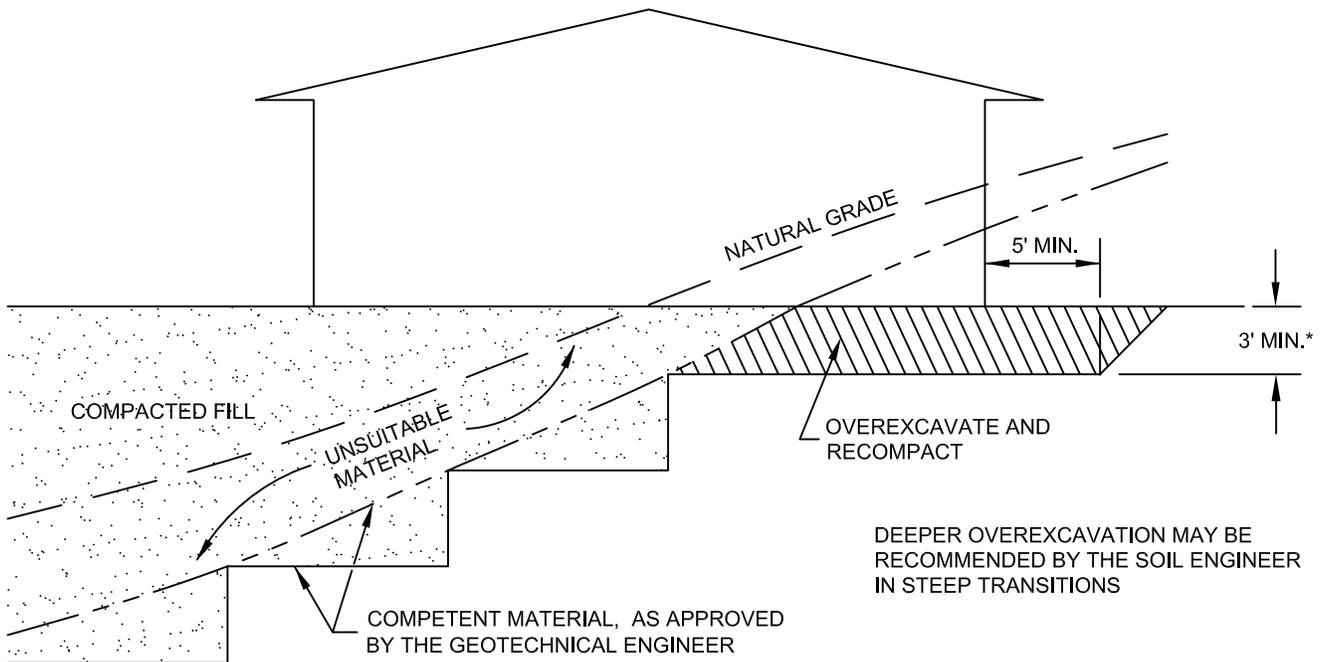
#### Subdrains

- 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

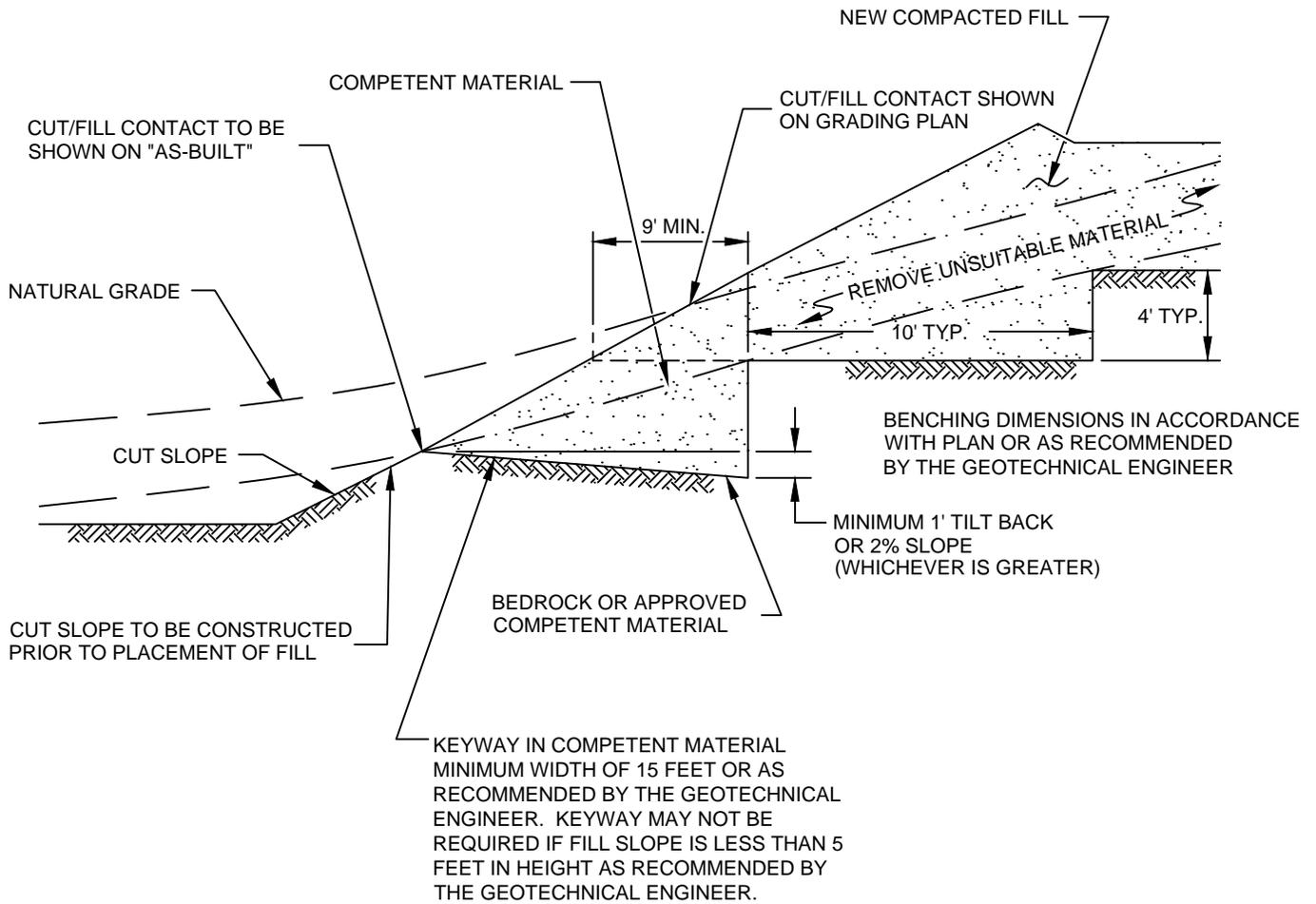


CUT/FILL LOT (TRANSITION)

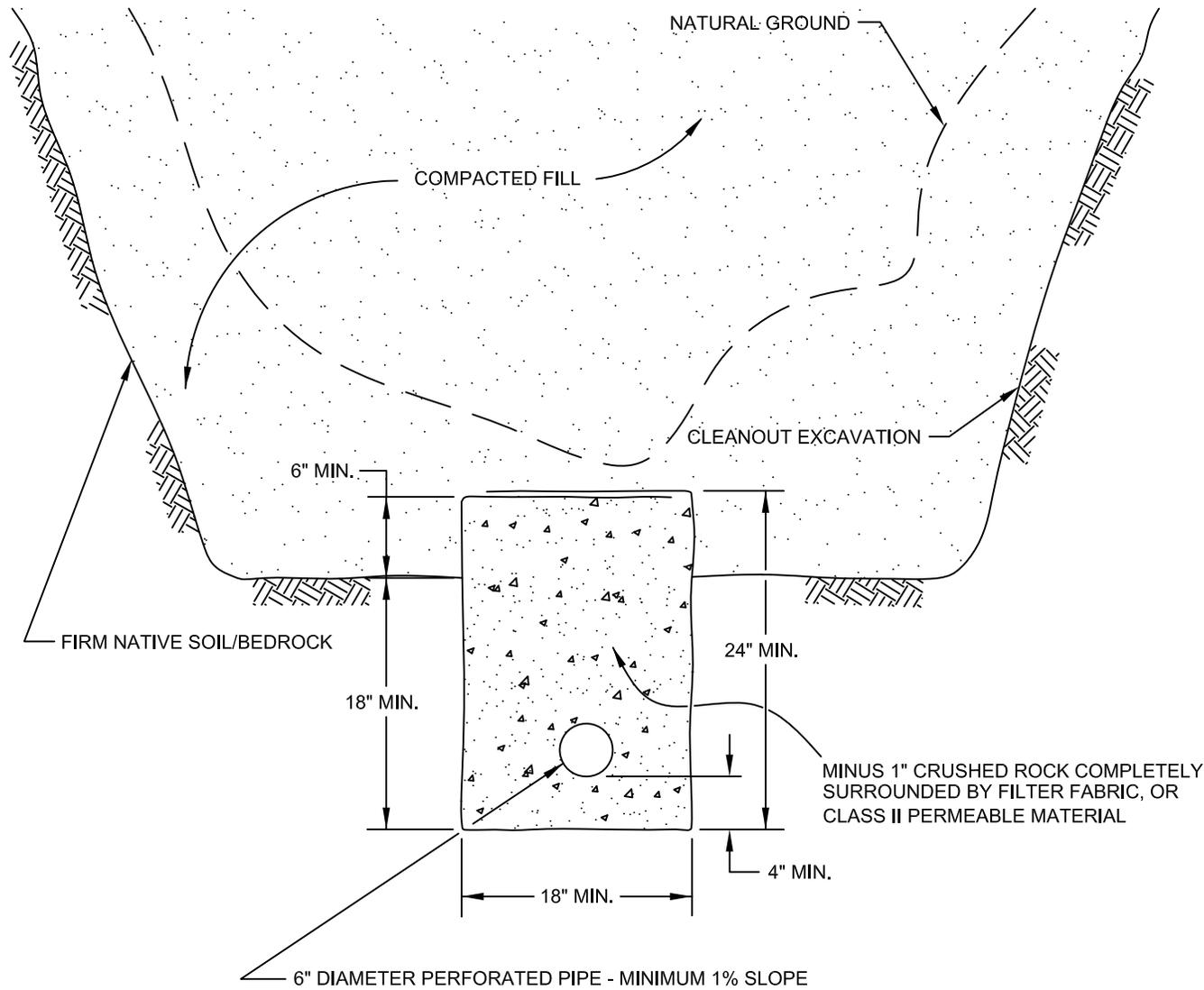


\*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION.  
ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

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



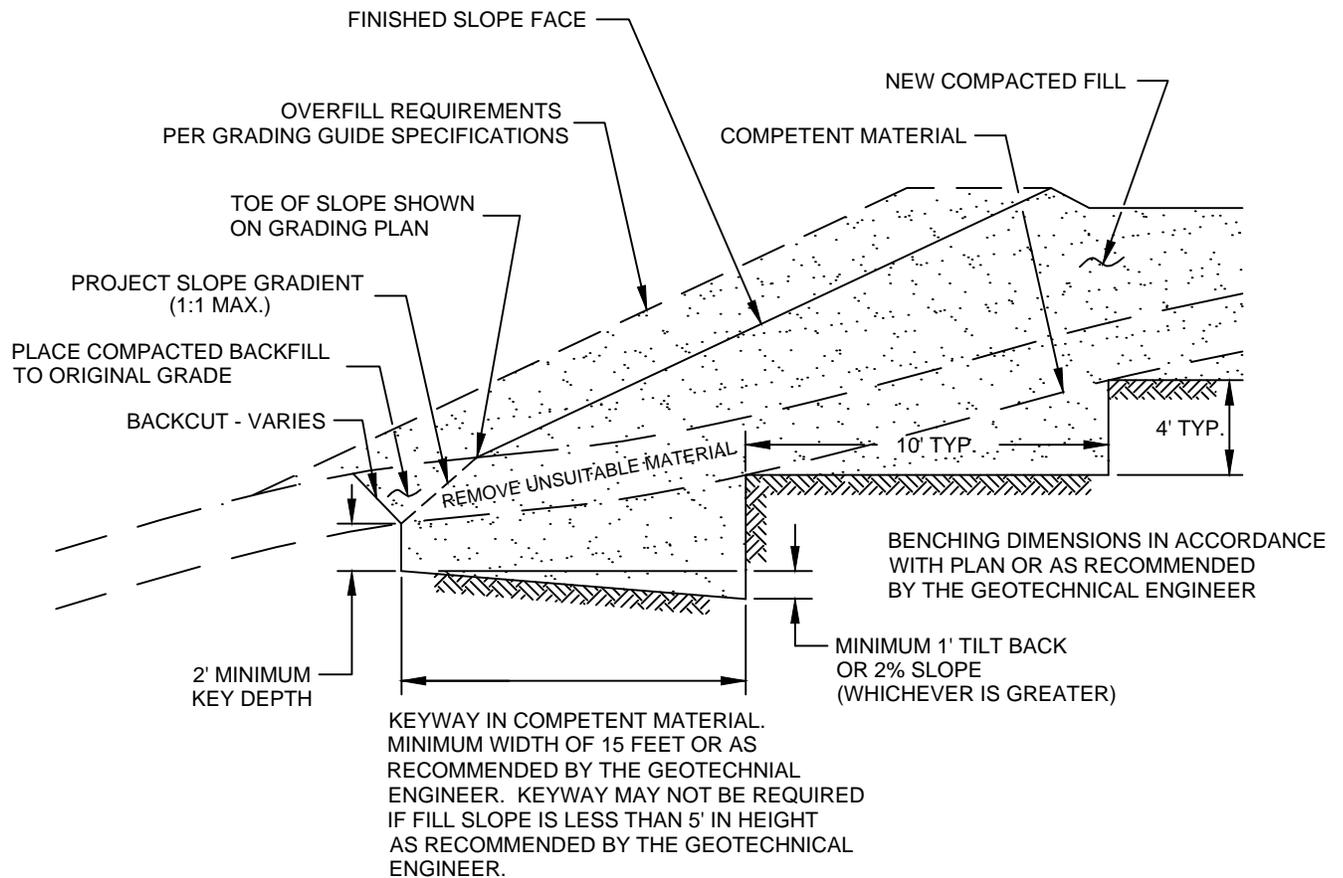
<b>FILL ABOVE CUT SLOPE DETAIL</b>	
<b>GRADING GUIDE SPECIFICATIONS</b>	
NOT TO SCALE	 <b>SOUTHERN CALIFORNIA GEOTECHNICAL</b>
DRAWN: JAS CHKD: GKM	
<b>PLATE D-2</b>	



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

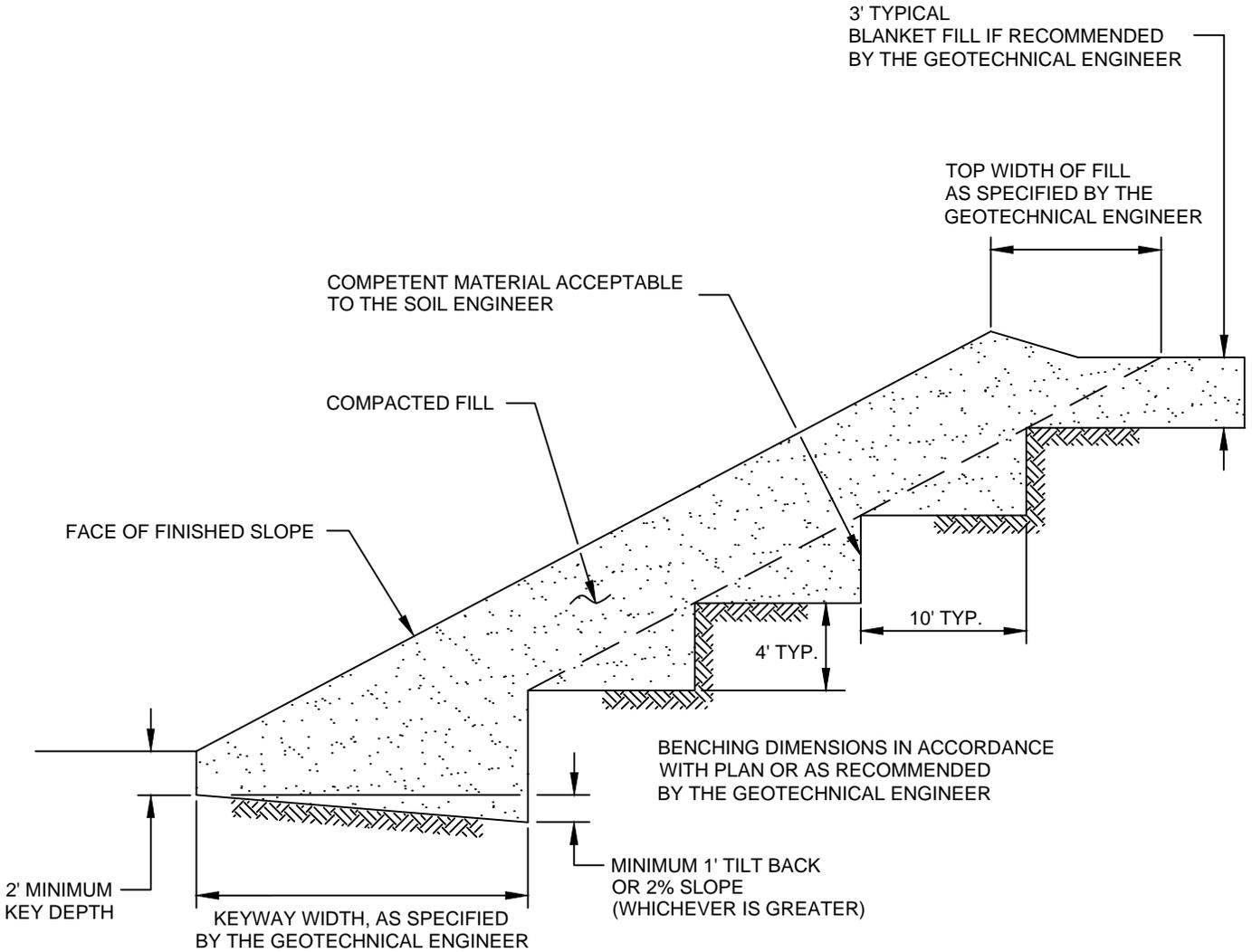
**SCHEMATIC ONLY  
NOT TO SCALE**

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

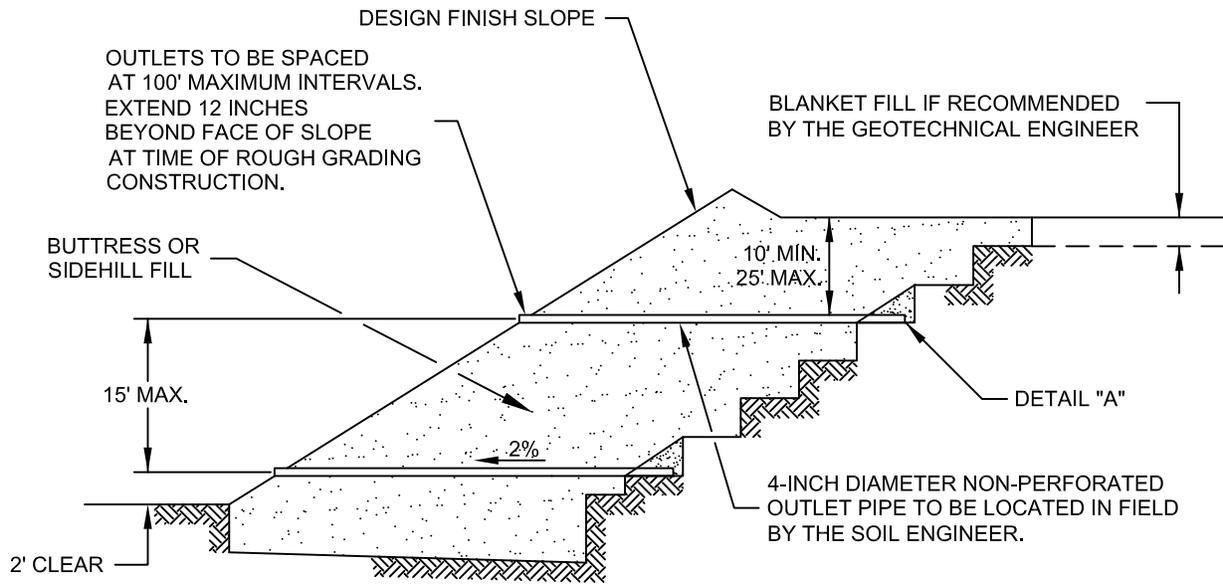


NOTE:  
 BENCHING SHALL BE REQUIRED  
 WHEN NATURAL SLOPES ARE  
 EQUAL TO OR STEEPER THAN 5:1  
 OR WHEN RECOMMENDED BY  
 THE GEOTECHNICAL ENGINEER.

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



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



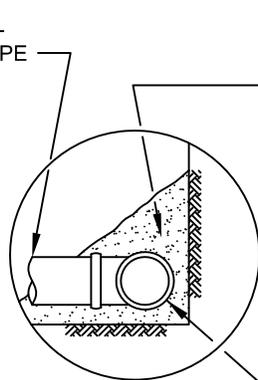
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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:

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

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

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

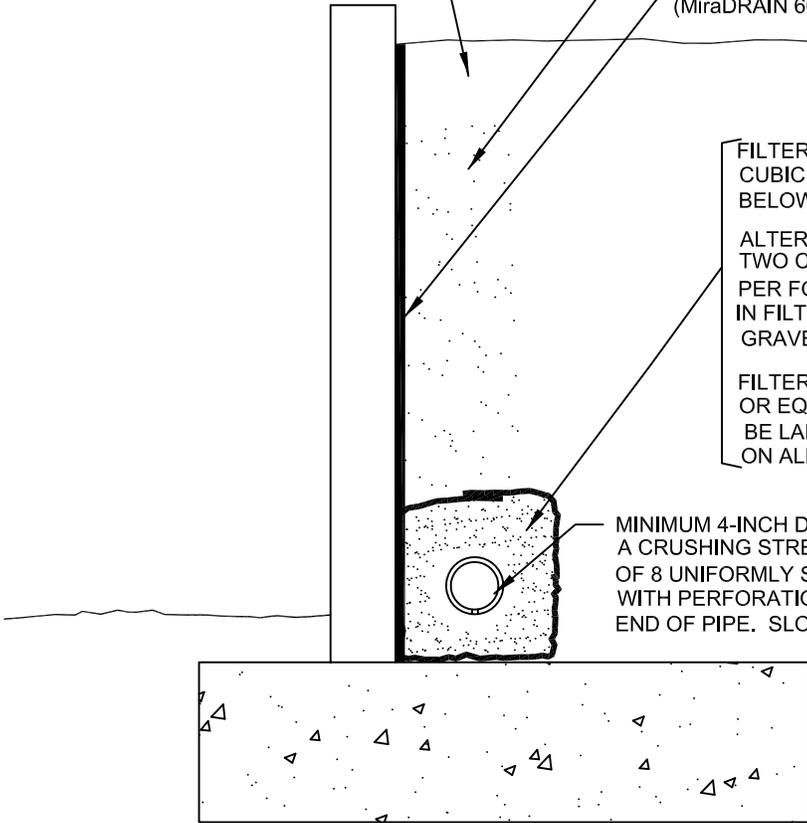
MINIMUM ONE FOOT WIDE LAYER OF FREE DRAINING MATERIAL (LESS THAN 5% PASSING THE #200 SIEVE) OR PROPERLY INSTALLED PREFABRICATED DRAINAGE COMPOSITE (MiraDRAIN 6000 OR APPROVED EQUIVALENT).

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 MIRAFI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 6 INCHES ON ALL JOINTS.

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



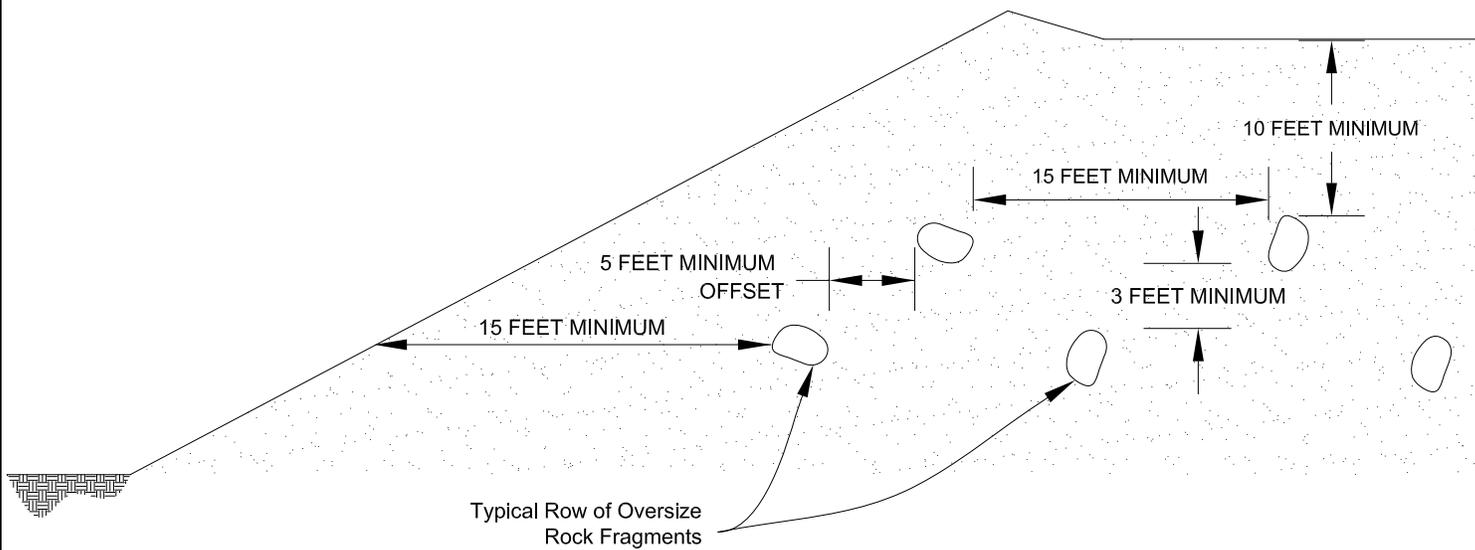
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

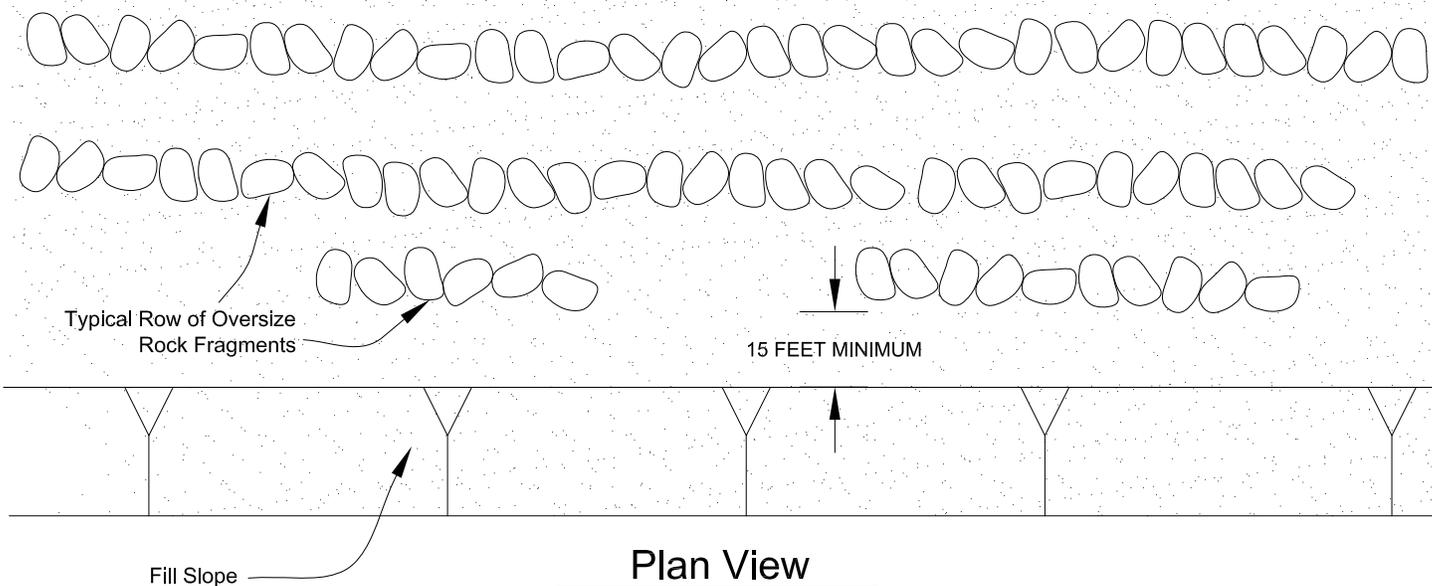
"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

RETAINING WALL BACKDRAINS	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	 <b>SOUTHERN CALIFORNIA GEOTECHNICAL</b>
DRAWN: JAS CHKD: GKM	
PLATE D-7	



**Section View**



**Plan View**

**PLACEMENT OF OVERSIZED MATERIAL  
GRADING GUIDE SPECIFICATIONS**

NOT TO SCALE

DRAWN: PM  
CHKD: GKM

PLATE D-8



**SOUTHERN  
CALIFORNIA  
GEOTECHNICAL**

# APPENDIX E



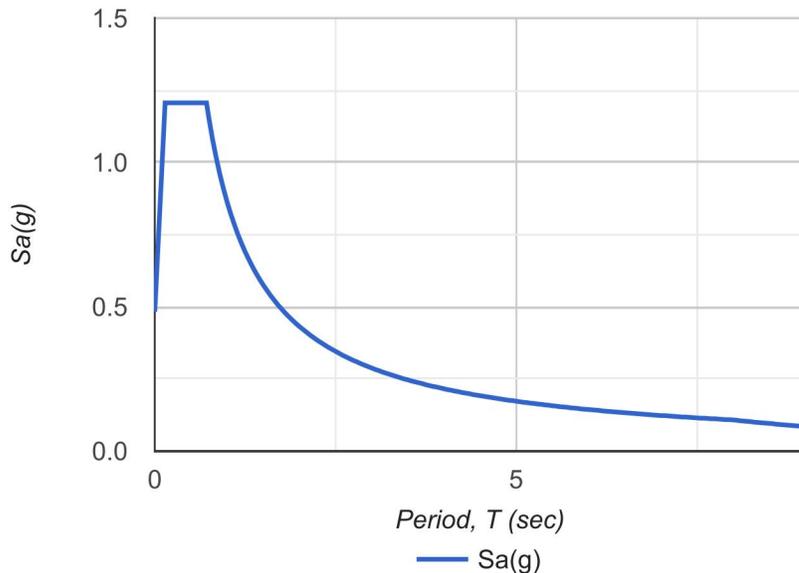
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<b>Date</b>	12/6/2019, 2:44:05 PM
<b>Design Code Reference Document</b>	ASCE7-10
<b>Risk Category</b>	III
<b>Site Class</b>	D - Stiff Soil

Type	Value	Description	Type	Value	Description
$S_S$	1.81	$MCE_R$ ground motion. (for 0.2 second period)	SDC	E	Seismic design category
$S_1$	0.864	$MCE_R$ ground motion. (for 1.0s period)	$F_a$	1	Site amplification factor at 0.2 second
$S_{MS}$	1.81	Site-modified spectral acceleration value	$F_v$	1.5	Site amplification factor at 1.0 second
$S_{M1}$	1.296	Site-modified spectral acceleration value	PGA	0.72	$MCE_G$ peak ground acceleration
$S_{DS}$	1.207	Numeric seismic design value at 0.2 second SA	$F_{PGA}$	1	Site amplification factor at PGA
$S_{D1}$	0.864	Numeric seismic design value at 1.0 second SA	$PGA_M$	0.72	Site modified peak ground acceleration

**Design Response Spectrum**



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



<b>SEISMIC DESIGN PARAMETERS - 2016 CBC</b>	
PROPOSED WAREHOUSE	
REDLANDS, CALIFORNIA	
DRAWN: JAH CHKD: RGT SCG PROJECT 19G237-1 <b>PLATE E-1A</b>	 <b>SOUTHERN CALIFORNIA GEOTECHNICAL</b>



Latitude, Longitude: 34.083088, -117.207239



Date	12/6/2019, 2:44:29 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
$S_S$	1.966	$MCE_R$ ground motion. (for 0.2 second period)
$S_1$	0.762	$MCE_R$ ground motion. (for 1.0s period)
$S_{MS}$	1.966	Site-modified spectral acceleration value
$S_{M1}$	null -See Section 11.4.8	Site-modified spectral acceleration value
$S_{DS}$	1.31	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	0.81	$MCE_G$ peak ground acceleration
$F_{PGA}$	1.1	Site amplification factor at PGA
$PGA_M$	0.891	Site modified peak ground acceleration
$T_L$	8	Long-period transition period in seconds
$S_{sRT}$	2.652	Probabilistic risk-targeted ground motion. (0.2 second)
$S_{sUH}$	2.892	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
$S_{sD}$	1.966	Factored deterministic acceleration value. (0.2 second)
$S_{1RT}$	1.047	Probabilistic risk-targeted ground motion. (1.0 second)
$S_{1UH}$	1.175	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
$S_{1D}$	0.762	Factored deterministic acceleration value. (1.0 second)
$PGA_d$	0.81	Factored deterministic acceleration value. (Peak Ground Acceleration)
$C_{RS}$	0.917	Mapped value of the risk coefficient at short periods
$C_{R1}$	0.891	Mapped value of the risk coefficient at a period of 1 s

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



<b>SEISMIC DESIGN PARAMETERS - 2019 CBC</b>	
PROPOSED WAREHOUSE	
REDLANDS, CALIFORNIA	
DRAWN: JAH CHKD: RGT SCG PROJECT 19G237-1	 <b>SOUTHERN CALIFORNIA GEOTECHNICAL</b>
<b>PLATE E-1B</b>	