April 19, 2021

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Lake Creek Industrial, LLC
1302 Brittany Cross Road
Santa Ana, California 92705

Attention: Mr. Bob Kubichek
Senior Vice President

Project No.: **21G147-2**

Subject: **Results of Infiltration Testing**
Proposed Warehouse
NWC Placentia Avenue at Murrieta Road
Perris, California

Reference: Geotechnical Investigation, Proposed Warehouse, NWC Placentia Avenue at Murrieta Road, Perris, California, prepared for Lake Creek Industrial, LLC, by Southern California Geotechnical, Inc. (SCG), SCG Project No. 21G147-1, dated April 16, 2021.

Mr. Kubichek:

In accordance with your request, we have conducted infiltration testing at the subject site. We are pleased to present this report summarizing the results of the infiltration testing and our design recommendations.

Scope of Services

The scope of services performed for this project was in general accordance with our Proposal No. 21P173, dated February 26, 2021. The scope of services included site reconnaissance, subsurface exploration, field and laboratory testing, and engineering analysis to determine the infiltration rates of the on-site soils. The infiltration testing was performed in general accordance with ASTM Test Method D-3385-03, Standard Test Method for Infiltration Rate of Soils in Field Using Double Ring Infiltrometer.

Site and Project Description

The subject site is located at the northwest corner of Placentia Avenue and Murrieta Road in Perris, California. The site is bounded to the north by an active grading site, to the east by Murrieta Road, to the south by Placentia Avenue and to the west by vacant lots. 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 rectangular-shaped parcel, 18.24± acres in size. The site is presently vacant and undeveloped. The ground surface consists of hummocky soil covered by dense native grass and weed growth. Based on historic aerial photographs obtained from Google Earth, the site was previously tilled for agricultural activities between September of 2002 and June of 2012.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth and visual observations made at the time of the subsurface investigation, the site is relatively level with localized undulations of 1 to 2± feet.

Proposed Development

A conceptual site plan prepared by RGA, was provided to our office. Based on this plan, the site will be developed with one (1) new warehouse, 379,440± ft² in size, located in the central area of the site. Dock-high doors will be constructed in a cross-dock configuration along the west and east building walls. The building is expected to be surrounded by asphaltic concrete pavements in the parking and drive areas, Portland cement concrete pavements in the truck court areas, and limited areas of concrete flatwork and landscape planters throughout.

We understand that the proposed development will include on-site storm water infiltration. However, the type and location of the infiltration system is not known at the time of this report. Based on previous experience, it is expected that the infiltration system will consist of below grade chambers or a shallow basin.

Concurrent Study

SCG recently conducted a geotechnical investigation at the subject site, which is referenced above. As part of this study, seven (7) borings were advanced to depths of 10 to 50± feet below existing site grades. Native alluvium was encountered at the ground surface at all the boring locations, extending to at least the maximum depth explored of 50± feet. The alluvial soils generally consist of interbedded layers of medium dense to very dense silty sands, sandy silts, and clayey sands with varying clay and fine gravel content and very stiff to hard silty clays, clayey silts and sandy clays with varying sand and silt content.

Groundwater

Free water was encountered during the drilling at two (2) of the boring locations. Water was encountered at 29± feet below existing site grades at Boring No. B-1 and at 27± feet below existing site grades at Boring No. B-5. The remaining boreholes were dry at the time of completion of drilling. Therefore, the static groundwater table is considered to have been present at a depth of 27 to 29± feet below the existing site grades at the time of subsurface exploration.

Recent water level data was obtained from the California Department of Water Resources website, https://wdl.water.ca.gov/waterdatalibrary/. The nearest monitoring well is located approximately 3,168± feet northeast from the site. Water level readings within this monitoring well indicate a high groundwater level of 25± feet below the ground surface in November 2020.

Subsurface Exploration

Scope of Exploration

The subsurface exploration for the infiltration testing consisted of four (4) backhoe-excavated trenches, extending to a depth of 10± feet below existing site grades. The trenches were logged during excavation by a member of our staff. The approximate locations of the infiltration trenches (identified as I-1 through I-4) are indicated on the Infiltration Test Location Plan, enclosed as Plate 2 of this report.

Geotechnical Conditions

Native alluvium was encountered beneath at the ground surface at all of the infiltration testing locations, extending to at least the maximum explored depth of 10± feet below existing site grades. The alluvial soils consist of medium dense to very dense silty sands and sandy silts with varying clay content. The Trench Logs, which illustrate the conditions encountered at the infiltration test locations, are presented in this report.

Infiltration Testing

We understand that the results of the testing will be used to prepare a preliminary design for the storm water infiltration system that will be used at the subject site. As previously mentioned, the infiltration testing was performed in general accordance with ASTM Test Method D-3385-03, Standard Test Method for Infiltration Rate of Soils in Field Using Double Ring Infiltrometer.

Two stainless steel infiltration rings were used for the infiltration testing. The outer infiltration ring is 2 feet in diameter and 20 inches in height. The inner infiltration ring is 1 foot in diameter and 20 inches in height. At the test locations, the outer ring was driven 3± inches into the soil at the base of each trench. The inner ring was centered inside the outer ring and subsequently driven 3± inches into the soil at the base of the trench. The rings were driven into the soil using a ten-pound sledge hammer. The soil surrounding the wall of the infiltration rings was only slightly disturbed during the driving process.

Infiltration Testing Procedure

Infiltration testing was performed at all of the trench locations. The infiltration testing consisted of filling the inner ring and the annular space (the space between the inner and outer rings) with water, approximately 3 to 4 inches above the soil. To prevent the flow of water from one ring to the other, the water level in both the inner ring and the annular space between the rings was maintained using constant-head float valves. The volume of water that was added to maintain a constant head in the inner ring and the annular space during each time interval was determined and recorded. A cap was placed over the rings to minimize the evaporation of water during the tests.

The schedule for readings was determined based on the observed soil type at the base of each backhoe-excavated trench. Based on the existing soils at the trench locations, the volumetric measurements were made at 10-minute increments at I-2, at 20-minute increments at I-1 and I-3, and at 30-minute increments at I-4. The water volume measurements are presented on the spreadsheets enclosed with this report. The infiltration rates for each of the timed intervals are also tabulated on these spreadsheets.

The infiltration rates for the infiltration tests are calculated in centimeters per hour and then converted to inches per hour. The rates are summarized below:

Infiltration Test No.	Depth (feet)	Soil Description	Infiltration Rate (inches/hour)
I-1	10	Silty fine to medium Sand	1.1
I-2	10	Silty fine to medium Sand, trace coarse Sand	5.2
I-3	10	Silty fine to medium Sand, trace coarse Sand, little Clay	0.7
I-4	10	fine to medium Sandy Silt, trace Clay	0.5

Laboratory Testing

Moisture Content

The moisture contents for selected soil samples within the trenches were determined in accordance with ASTM D-2216 and are expressed as a percentage of the dry weight. These test results are presented on the Trench Logs in Plates B-1 through B-4 of this report.

Grain Size Analysis

The grain size distribution of selected soils collected from the base of each infiltration test trench has been determined using a range of wire mesh screens. These tests were performed in general accordance with ASTM D-422 and/or ASTM D-1140. The weight of the portion of the sample retained on each screen is recorded and the percentage finer or coarser of the total weight is calculated. The results of the grainsize analysis are presented on Plates C-1 through C-4 of this report.

Design Recommendations

Four (4) infiltration tests were performed at the subject site. As noted above, the infiltration rates at these locations vary from 0.5 to 5.2 inches per hour. Based on the results of Infiltration Test Nos. I-1 through I-4, we recommend infiltration rates as follows:

Infiltration Test	Location	Infiltration Rate (Inches per Hour)
I-1 and I-2	West of Proposed Building	1.1
I-3 and I-4	East of Proposed Building	0.5

We recommend that a representative from the geotechnical engineer be on-site during the construction of the proposed infiltration system to identify the soil classification at the base of the infiltration basin. It should be confirmed that the soils at the base of the proposed infiltration system corresponds with those presented in this report to ensure that the performance of the system will be consistent with the rates reported herein.

The design of the proposed storm water infiltration system should be performed by the project civil engineer, in accordance with the City of Perris and/or County of Riverside guidelines. However, it is recommended that the systems be constructed so as to facilitate removal of silt and clay, or other deleterious materials from any water that may enter the system. The presence of such materials would decrease the effective infiltration rates. **It is recommended that the project civil engineer apply an appropriate factor of safety. The infiltration rate recommended above is based on the assumption that only clean water will be introduced to the subsurface profile. Any fines, debris, or organic materials could significantly impact the infiltration rate.** It should be noted that the recommended infiltration rate is based on infiltration testing at four (4) discrete locations, and the overall infiltration rate of the storm water infiltration system could vary considerably.

Infiltration Rate Considerations

The infiltration rate presented herein was determined in accordance with the Riverside County guidelines and is considered valid only for the time and place of the actual test. Varying subsurface conditions will exist in other areas of the site, which could alter the recommended infiltration rates presented above. The infiltration rates will decline over time between maintenance cycles as silt or clay particles accumulate on the BMP surface. The infiltration rate is highly dependent upon a number of factors, including density, silt and clay content, grainsize distribution throughout the range of particle sizes, and particle shape. Small changes in these factors can cause large changes in the infiltration rates.

Infiltration rates are based on unsaturated flow. As water is introduced into soils by infiltration, the soils become saturated and the wetting front advances from the unsaturated zone to the saturated zone. Once the soils become saturated, infiltration rates become zero, and water can only move through soils by hydraulic conductivity at a rate determined by pressure head and soil permeability. Changes in soil moisture content will affect the infiltration rate. Infiltration rates should be expected to decrease until the soils become saturated. Soil permeability values will then govern groundwater movement. Permeability values may be on the order of 10 to 20 times less than infiltration rates. The system designer should incorporate adequate factors of safety and allow for overflow design into appropriate traditional storm drain systems, which would transport storm water off-site.

Construction Considerations

The infiltration rates presented in this report are specific to the tested locations and tested depths. Infiltration rates can be significantly reduced if the soils are exposed to excessive disturbance or compaction during construction. Compaction of the soils at the bottom of the infiltration system can significantly reduce the infiltration ability of the basins. Therefore, the subgrade soils within proposed infiltration system areas should not be over-excavated, undercut or compacted in any significant manner. **It is recommended that a note to this effect be added to the project plans and/or specifications.**

We recommend that a representative from the geotechnical engineer be on-site during the construction of the proposed infiltration systems to identify the soil classification at the base of each system. It should be confirmed that the soils at the base of the proposed infiltration systems correspond with those presented in this report to ensure that the performance of the systems will be consistent with the rates reported herein.

We recommend that scrapers and other rubber-tired heavy equipment not be operated on the basin bottom, or at levels lower than 2 feet above the bottom of the system, particularly within basins. As such, the bottom 24 inches of the infiltration systems should be excavated with non-rubber-tired equipment, such as excavators.

Basin Maintenance

The proposed project may include infiltration basins. Water flowing into these basins will carry some level of sediment. Wind-blown sediments and erosion of the basin side walls will also contribute to sediment deposition at the bottom of the basin. This layer has the potential to significantly reduce the infiltration rate of the basin subgrade soils. Therefore, a formal basin maintenance program should be established to ensure that these silt and clay deposits are removed from the basin on a regular basis. Appropriate vegetation on the basin sidewalls and bottom may reduce erosion and sediment deposition.

Basin maintenance should also include measures to prevent animal burrows, and to repair any burrows or damage caused by such. Animal burrows in the basin sidewalls can significantly increase the risk of erosion and piping failures.

Location of Infiltration Systems

The use of on-site storm water infiltration systems carries a risk of creating adverse geotechnical conditions. Increasing the moisture content of the soil can cause the soil to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Overlying structures and pavements in the infiltration area could potentially be damaged due to saturation of the subgrade soils. **The proposed infiltration systems for this site should be located at least 25 feet away from any structures, including retaining walls.** Even with this provision of locating the infiltration system at least 25 feet from the building(s), it is possible that infiltrating water into the subsurface soils could have an adverse effect on the proposed or existing structures. It should also be noted that utility trenches which happen to collect storm water can also serve as conduits to transmit storm water toward the structure, depending on the slope of the utility trench. Therefore, consideration should also be given to the proposed locations of underground utilities which may pass near the proposed infiltration system.

The infiltration system designer should also give special consideration to the effect that the proposed infiltration systems may have on nearby subterranean structures, open excavations, or descending slopes. In particular, infiltration systems should not be located near the crest of descending slopes, particularly where the slopes are comprised of granular soils. Such systems will require specialized design and analysis to evaluate the potential for slope instability, piping failures and other phenomena that typically apply to earthen dam design. This type of analysis is beyond the scope of this infiltration test report, but these factors should be considered by the infiltration system designer when locating the infiltration systems.

General Comments

This report has been prepared as an instrument of service for use by the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, structural engineer, and/or civil engineer. The design of the infiltration system is the responsibility of the civil engineer. The role of the geotechnical engineer is limited to determination of infiltration rate only. By using the design infiltration rates contained herein, the civil engineer agrees to indemnify, defend, and hold harmless the geotechnical engineer for all aspects of the design and performance of the infiltration system. 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 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 trench locations and testing 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.

Closure

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.

Ricardo Frias, RCE 91772
Project Engineer

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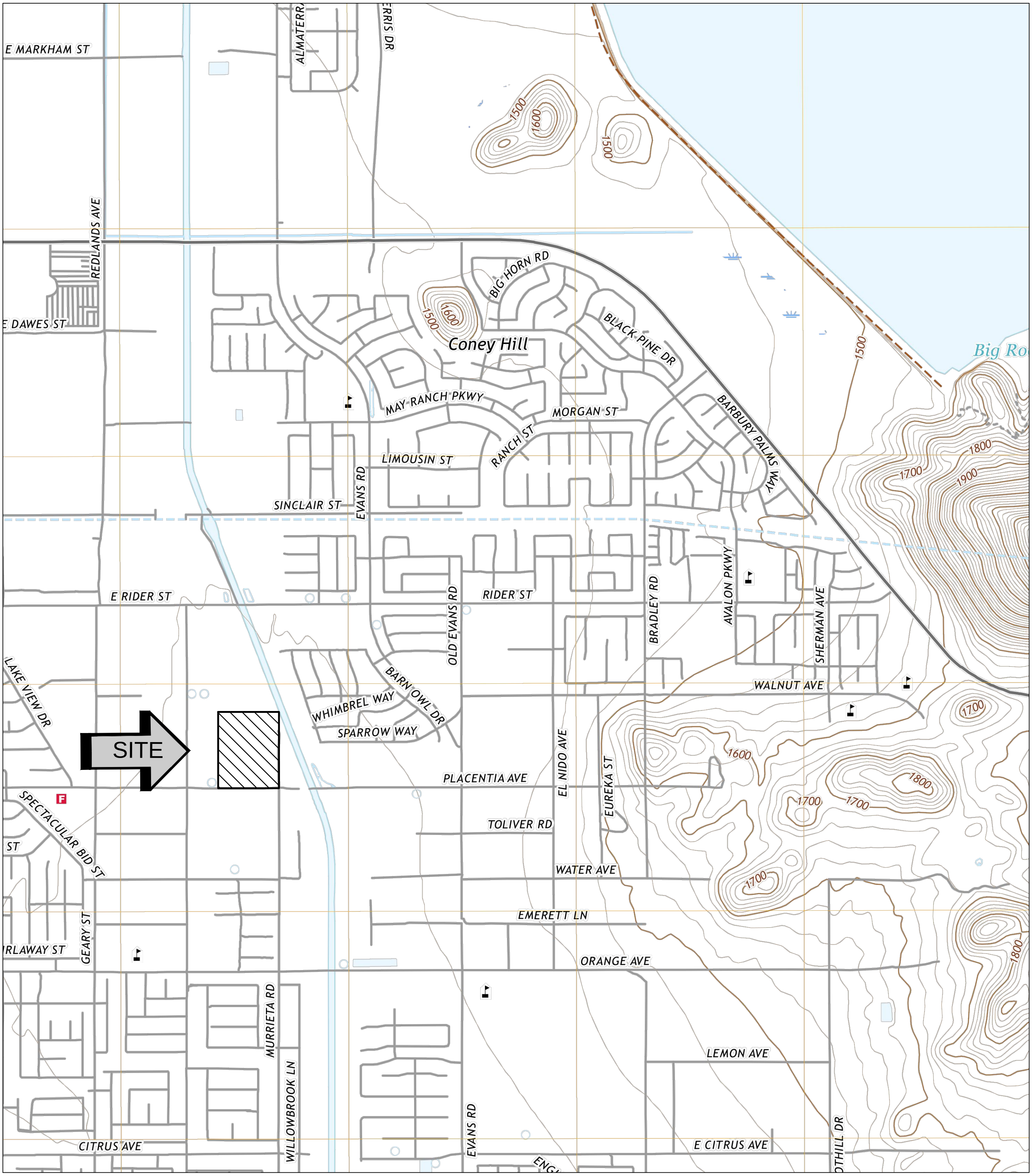
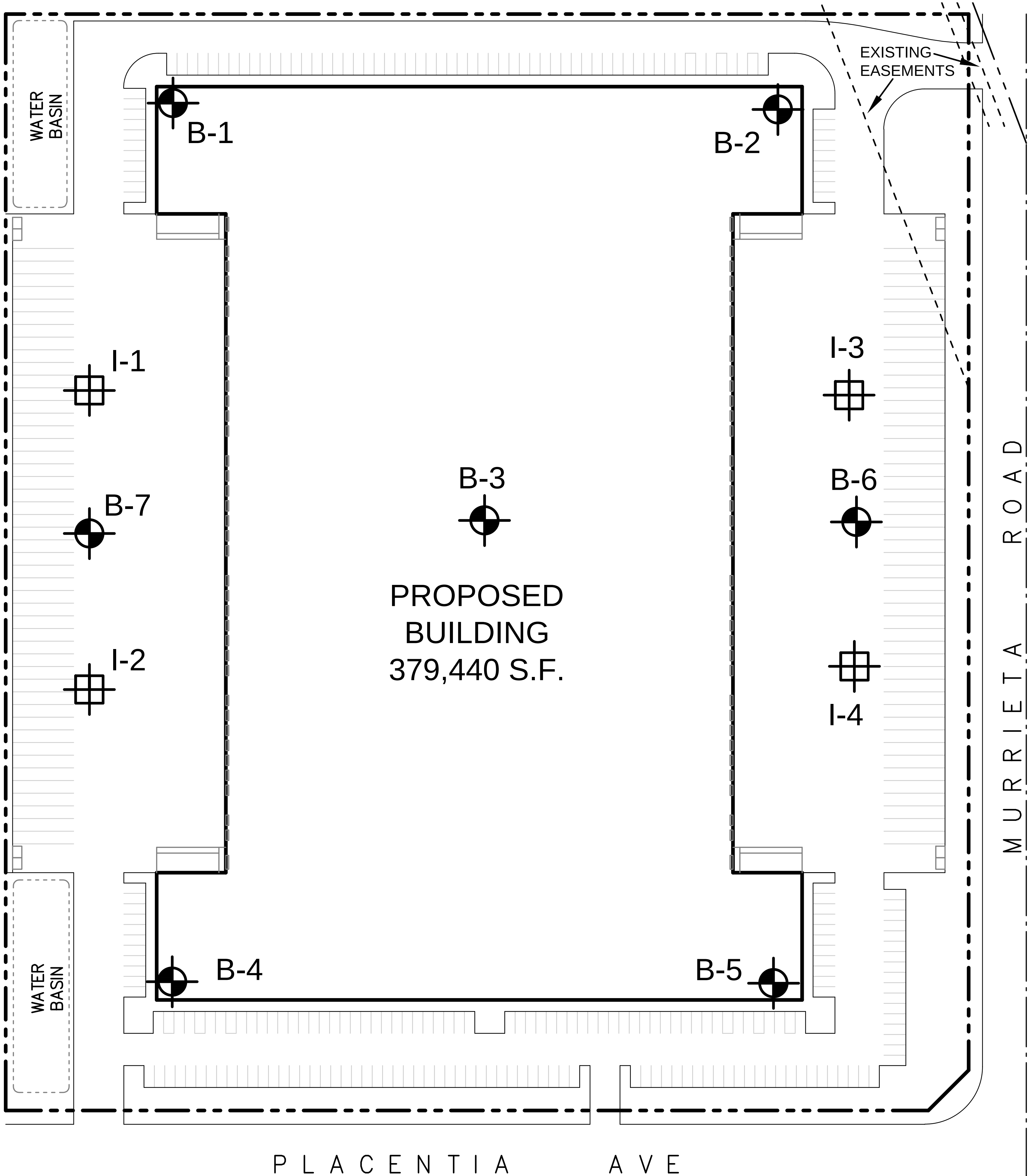
Distribution: (1) Addressee

Enclosures: Plate 1 - Site Location Map
Plate 2 - Infiltration Test Location Plan
Trench Log Legend and Logs (6 pages)
Infiltration Test Results Spreadsheets (4 pages)
Grain Size Distribution Graphs (4 pages)



SOURCE: USGS TOPOGRAPHIC MAP OF THE PERRIS QUADRANGLE, RIVERSIDE COUNTY, CALIFORNIA, 2018.

SITE LOCATION MAP	
PROPOSED WAREHOUSE	
PERRIS, CA	
SCALE: 1" = 2000'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: MD CHKD: RF	
SCG PROJECT 21G147-2	
PLATE 1	



NORTH

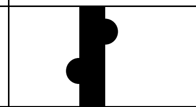
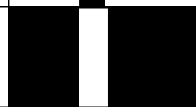
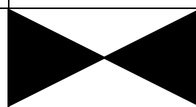
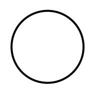
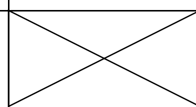
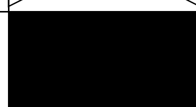
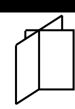
GEOTECHNICAL LEGEND

- APPROXIMATE INFILTRATION TEST LOCATION
- APPROXIMATE BORING LOCATION FROM CONCURRENT STUDY (SCG PROJECT NO. 21G147-1)

NOTE: BASE MAP PREPARED BY RGA.

BORING LOCATION PLAN	
PROPOSED WAREHOUSE	
PERRIS, CALIFORNIA	
SCALE: 1" = 80'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAH CHKD: RF	
SCG PROJECT 21G147-2	
PLATE 2	

TRENCH LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

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

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

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

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

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

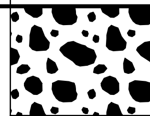
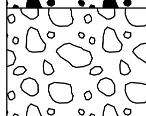
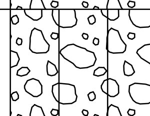
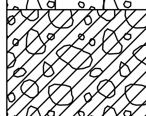
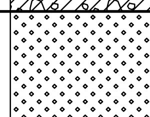
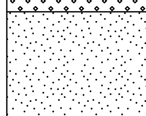
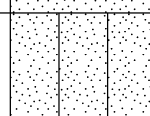
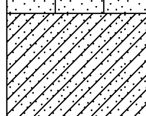
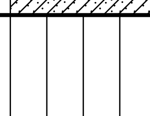
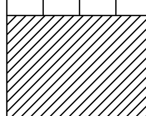
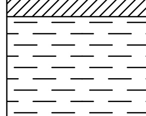
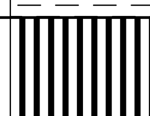
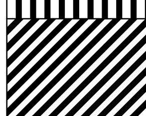
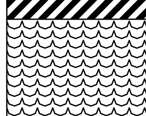
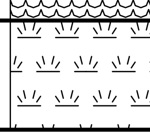
LIQUID LIMIT: The moisture content above which a soil behaves as a liquid.

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

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

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

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

TRENCH NO. I-1

JOB NO.: 21G147-2
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/30/21
EXCAVATION METHOD: Backhoe
LOGGED BY: Caleb Brackett

WATER DEPTH: Dry
CAVE DEPTH: ---
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ALLUVIUM: Red Brown Silty fine to medium Sand, medium dense to dense-damp							
5												
	Bulk (hand)						7					
10												
					Trench Terminated at 10'							

TBL 21G147-2.GPJ SOCALGEO.GDT 4/20/21

TEST TRENCH LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

TRENCH NO. I-2

JOB NO.: 21G147-2
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/30/21
EXCAVATION METHOD: Backhoe
LOGGED BY: Caleb Brackett

WATER DEPTH: Dry
CAVE DEPTH: ---
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
0–10					ALLUVIUM: Light Gray Brown Silty fine to medium Sand, trace coarse Sand, trace calcareous nodules, medium dense to dense-very moist							
5												
10	(hand sample)						13					
					Trench Terminated at 10'							

TBL 21G147-2.GPJ SOCALGEO.GDT 4/20/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

TRENCH NO. I-3

JOB NO.: 21G147-2	DRILLING DATE: 4/1/21	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	EXCAVATION METHOD: Backhoe	CAVE DEPTH: ---
LOCATION: Perris, California	LOGGED BY: Caleb Brackett	READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ALLUVIUM: Light Gray Brown Silty fine to medium Sand, trace coarse Sand, little Clay, very dense-moist							
5												
	Bulk (hand)						11					
10					Trench Terminated at 10'							

TBL 21G147-2.GPJ SOCALGEO.GDT 4/20/21

TEST TRENCH LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

TRENCH NO. I-4

JOB NO.: 21G147-2
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 4/1/21
EXCAVATION METHOD: Backhoe
LOGGED BY: Caleb Brackett

WATER DEPTH: Dry
CAVE DEPTH: ---
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ALLUVIUM: Light Brown fine to medium Sandy Silt, trace Clay, very dense-damp							
5												
10	(hand sample)						6					
					Trench Terminated at 10'							

TBL 21G147-2.GPJ SOCALGEO.GDT 4/20/21

TEST TRENCH LOG **PLATE B-4**

INFILTRATION CALCULATIONS

Project Name	Proposed Warehouse
Project Location	Perris, California
Project Number	21G147-2
Engineer	Caleb Brackett

Infiltration Test No: I-1

Constants	Diameter (ft)	Area (ft^2)	Area (cm^2)
Inner	1	0.79	730
Anlr. Spac	2	2.36	2189

*Note: The infiltration rate was calculated based on current time interval

Test Interval		Time (hr)	Interval **Elapsed** (min)	Flow Readings: Inner Ring (ml)	Ring Flow (cm^3)	Annular Ring (ml)	Space Flow (cm^3)	Infiltration Rates: Inner Ring* (cm/hr)	Annular Space* (cm/hr)	Inner Ring* (in/hr)	Annular Space* (in/hr)
1	Initial	7:00 AM	20	0	1400	0	5700	5.76	7.81	2.27	3.08
	Final	7:20 AM	**20**	1400		5700					
2	Initial	7:20 AM	20	0	900	0	4700	3.70	6.44	1.46	2.54
	Final	7:40 AM	**40**	900		4700					
3	Initial	7:40 AM	20	0	800	0	4300	3.29	5.89	1.30	2.32
	Final	8:00 AM	**60**	800		4300					
4	Initial	8:00 AM	20	0	700	0	4000	2.88	5.48	1.13	2.16
	Final	8:20 AM	**80**	700		4000					
5	Initial	8:20 AM	20	0	700	0	4000	2.88	5.48	1.13	2.16
	Final	8:40 AM	**100**	700		4000					
6	Initial	8:40 AM	20	0	650	0	4000	2.67	5.48	1.05	2.16
	Final	9:00 AM	**120**	650		4000					
7	Initial	9:00 AM	20	0	700	0	4100	2.88	5.62	1.13	2.21
	Final	9:20 AM	**140**	700		4100					
8	Initial	9:20 AM	20	0	650	0	4000	2.67	5.48	1.05	2.16
	Final	9:40 AM	**160**	650		4000					
9	Initial	9:40 AM	20	0	650	0	4000	2.67	5.48	1.05	2.16
	Final	10:00 AM	**180**	650		4000					

INFILTRATION CALCULATIONS

Project Name	Proposed Warehouse
Project Location	Perris, California
Project Number	21G147-2
Engineer	Caleb Brackett

Infiltration Test No: I-2

Constants	Diameter (ft)	Area (ft^2)	Area (cm^2)
Inner	1	0.79	730
Anlr. Spac	2	2.36	2189

*Note: The infiltration rate was calculated based on current time interval

Test Interval		Time (hr)	Interval **Elapsed** (min)	Flow Readings: Inner Ring (ml)	Ring Flow (cm^3)	Annular Ring (ml)	Space Flow (cm^3)	Infiltration Rates: Inner Ring* (cm/hr)	Annular Space* (cm/hr)	Inner Ring* (in/hr)	Annular Space* (in/hr)
1	Initial	10:20 AM	10	0	2800	0	9100	23.03	24.94	9.07	9.82
	Final	10:30 AM	**10**	2800		9100					
2	Initial	10:30 AM	10	0	2350	0	7800	19.32	21.38	7.61	8.42
	Final	10:40 AM	**20**	2350		7800					
3	Initial	10:40 AM	10	0	2000	0	7000	16.45	19.19	6.48	7.55
	Final	10:50 AM	**30**	2000		7000					
4	Initial	10:50 AM	10	0	1750	0	6200	14.39	16.99	5.67	6.69
	Final	11:00 AM	**40**	1750		6200					
5	Initial	11:00 AM	10	0	1750	0	2800	14.39	7.68	5.67	3.02
	Final	11:10 AM	**50**	1750		2800					
6	Initial	11:10 AM	10	0	1800	0	3000	14.80	8.22	5.83	3.24
	Final	11:20 AM	**60**	1800		3000					
7	Initial	11:20 AM	10	0	1650	0	3100	13.57	8.50	5.34	3.35
	Final	11:30 AM	**70**	1650		3100					
8	Initial	11:30 AM	10	0	1600	0	3000	13.16	8.22	5.18	3.24
	Final	11:40 AM	**80**	1600		3000					
9	Initial	11:40 AM	10	0	1550	0	3000	12.75	8.22	5.02	3.24
	Final	11:50 AM	**90**	1550		3000					
10	Initial	11:50 AM	10	0	1600	0	3000	13.16	8.22	5.18	3.24
	Final	12:00 PM	**100**	1600		3000					

INFILTRATION CALCULATIONS

Project Name	Proposed Warehouse
Project Location	Perris, California
Project Number	21G147-2
Engineer	Caleb Brackett

Infiltration Test No: I-3

Constants	Diameter (ft)	Area (ft^2)	Area (cm^2)
Inner	1	0.79	730
Anlr. Spac	2	2.36	2189

*Note: The infiltration rate was calculated based on current time interval

Test Interval		Time (hr)	Interval **Elapsed** (min)	Flow Readings: Inner Ring (ml)	Ring Flow (cm^3)	Annular Ring (ml)	Space Flow (cm^3)	Infiltration Rates: Inner Ring* (cm/hr)	Annular Space* (cm/hr)	Inner Ring* (in/hr)	Annular Space* (in/hr)
1	Initial	7:40 AM	20	0	1100	0	6300	4.52	8.63	1.78	3.40
	Final	8:00 AM	**20**	1100		6300					
2	Initial	8:00 AM	20	0	700	0	3100	2.88	4.25	1.13	1.67
	Final	8:20 AM	**40**	700		3100					
3	Initial	8:20 AM	20	0	550	0	2500	2.26	3.43	0.89	1.35
	Final	8:40 AM	**60**	550		2500					
4	Initial	8:40 AM	20	0	500	0	2100	2.06	2.88	0.81	1.13
	Final	9:00 AM	**80**	500		2100					
5	Initial	9:00 AM	20	0	450	0	2100	1.85	2.88	0.73	1.13
	Final	9:20 AM	**100**	450		2100					
6	Initial	9:20 AM	20	0	450	0	2000	1.85	2.74	0.73	1.08
	Final	9:40 AM	**120**	450		2000					

INFILTRATION CALCULATIONS

Project Name	Proposed Warehouse
Project Location	Perris, California
Project Number	21G147-2
Engineer	Caleb Brackett

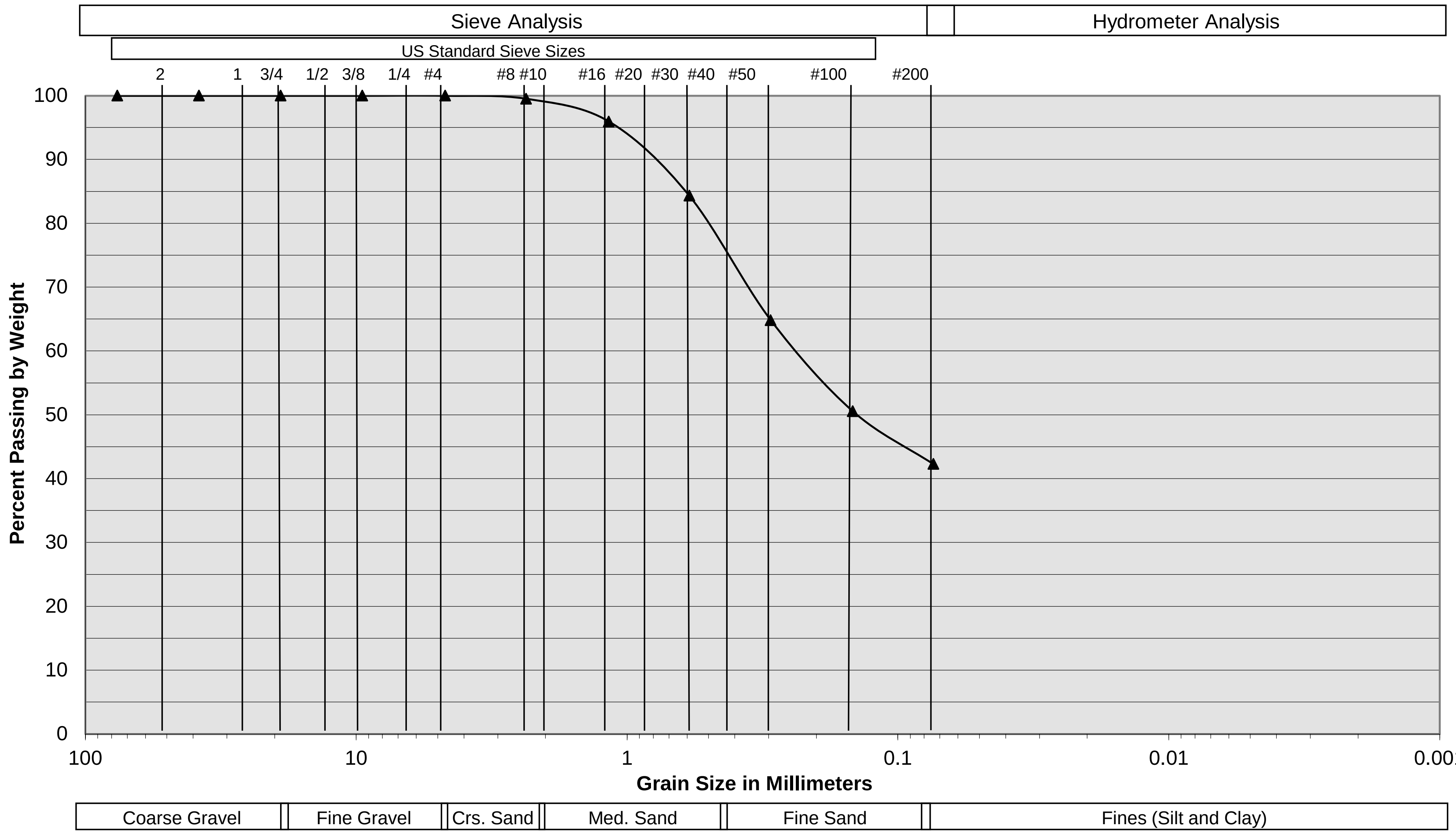
Infiltration Test No: I-4

Constants	Diameter (ft)	Area (ft^2)	Area (cm^2)
Inner	1	0.79	730
Anlr. Spac	2	2.36	2189

*Note: The infiltration rate was calculated based on current time interval

Test Interval		Time (hr)	Interval Elapsed (min)	Flow Readings: Inner Ring (ml)	Flow Readings: Ring Flow (cm^3)	Flow Readings: Annular Ring (ml)	Flow Readings: Space Flow (cm^3)	Infiltration Rates: Inner Ring* (cm/hr)	Infiltration Rates: Annular Space* (cm/hr)	Infiltration Rates: Inner Ring* (in/hr)	Infiltration Rates: Annular Space* (in/hr)
1	Initial	10:00 AM	30	0	0	0	0	0.00	0.00	0.00	0.00
	Final	10:30 AM	**30**	0		0					
2	Initial	10:30 AM	30	0	500	0	300	1.37	0.27	0.54	0.11
	Final	11:00 AM	**60**	500		300					
3	Initial	11:00 AM	30	0	500	0	300	1.37	0.27	0.54	0.11
	Final	11:30 AM	**90**	500		300					
4	Initial	11:30 AM	30	0	500	0	300	1.37	0.27	0.54	0.11
	Final	12:00 PM	**120**	500		300					

Grain Size Distribution

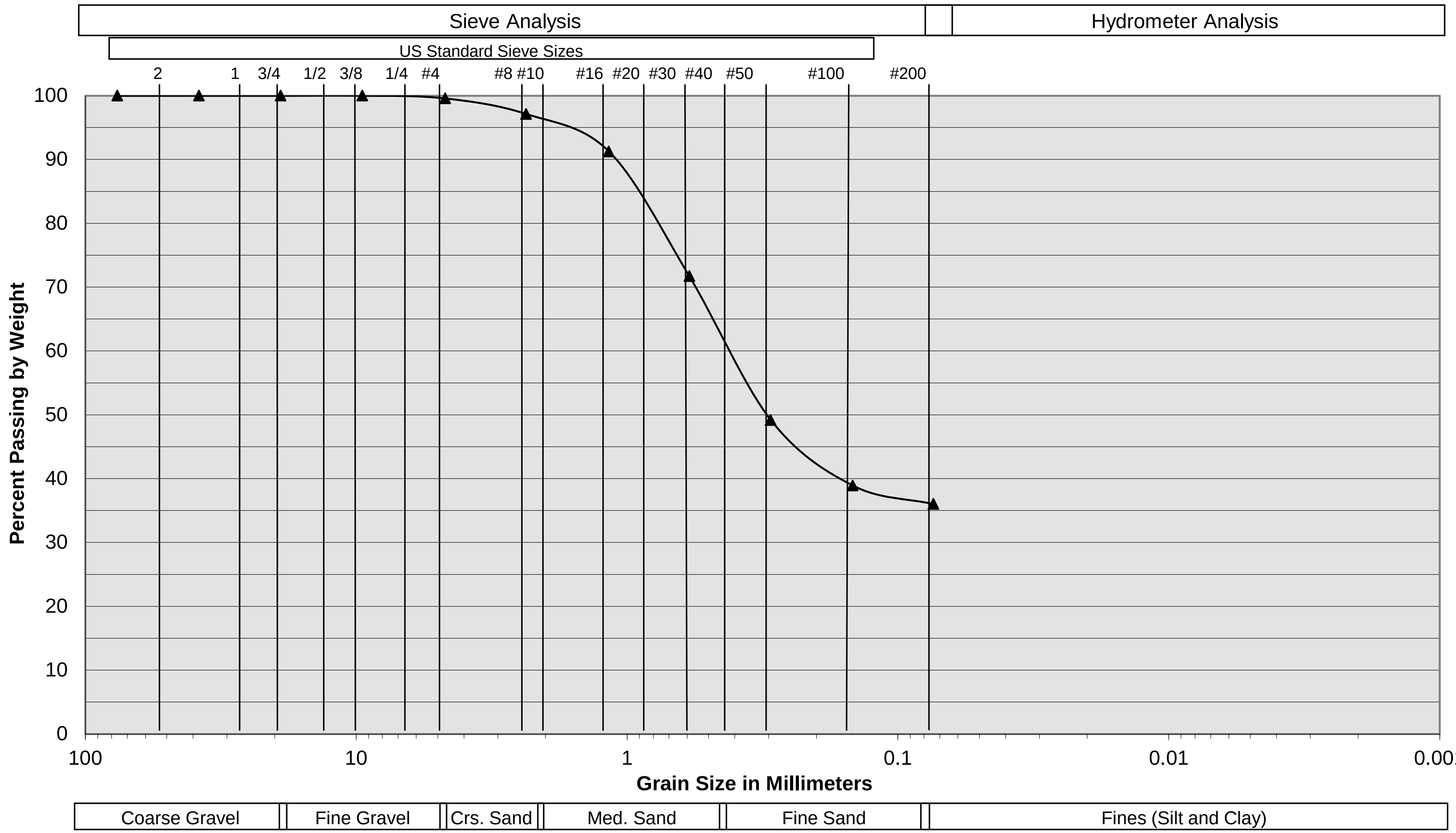


Sample Description	I-1 @ 9'
Soil Classification	Red Brown Silty fine to medium Sand

Proposed Warehouse
Perris, California
Project No. 21G147-2
PLATE C- 1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Grain Size Distribution

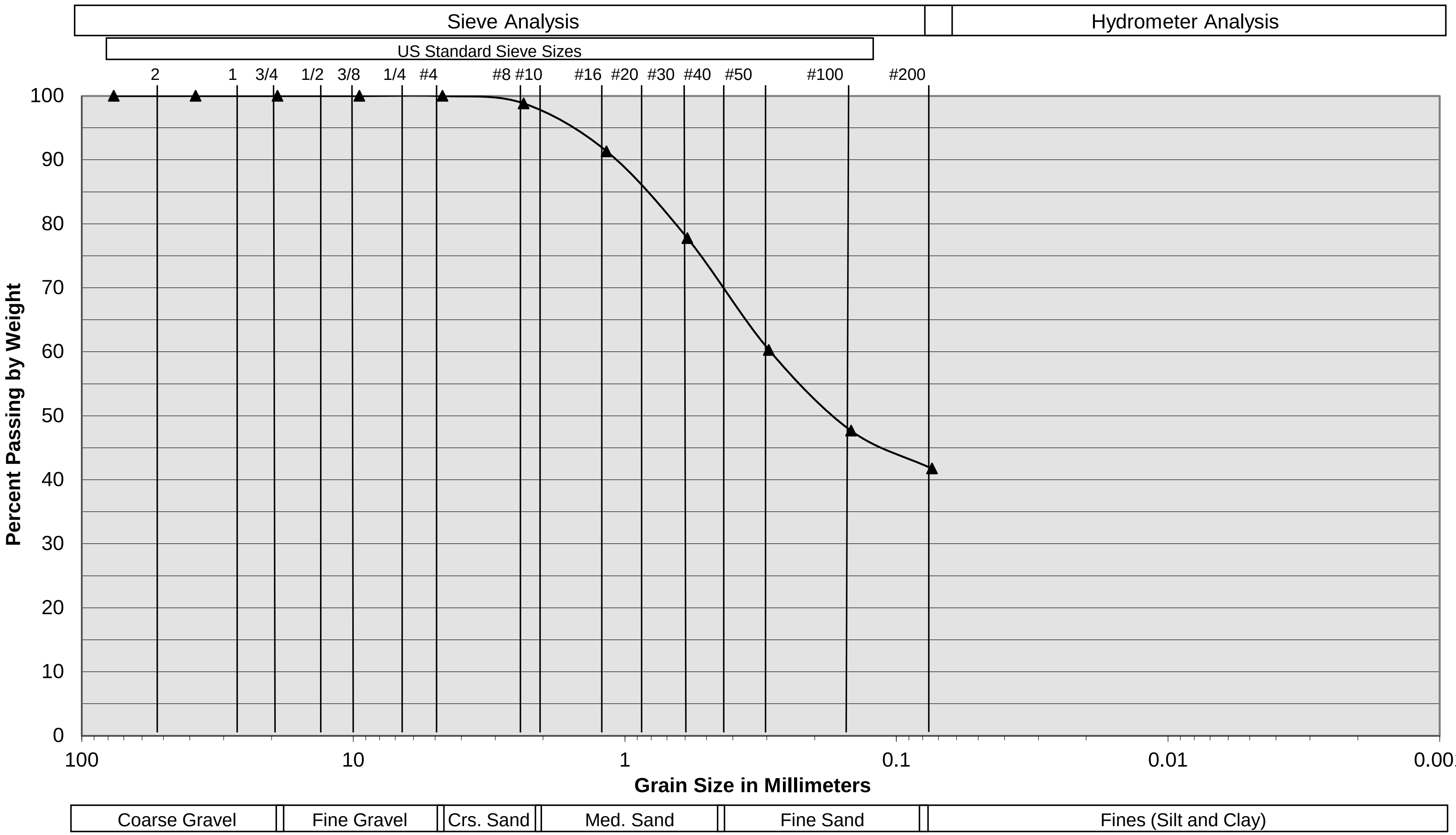


Sample Description	I-2 @ 9'
Soil Classification	Light Gray Brown Silty fine to medium Sand, trace coarse Sand

Proposed Warehouse
Perris, California
Project No. 21G147-2
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Grain Size Distribution



Sample Description	I-3 @ 9'
Soil Classification	Light Gray Brown Silty fine to medium Sand, trace coarse Sand, little Clay

Proposed Warehouse
Perris, California
Project No. 21G147-2
PLATE C- 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Grain Size Distribution

Sample Description	I-4 @ 9'
Soil Classification	Light Brown fine to medium Sandy Silt, trace Clay

Proposed Warehouse
Perris, California
Project No. 21G147-2
PLATE C- 4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

UPDATED GEOTECHNICAL INVESTIGATION
PROPOSED WAREHOUSE

NWC Placentia Avenue at Murrieta Road
Perris, California
for
Lake Creek industrial, LLC

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

August 20, 2021

Lake Creek Industrial, LLC
1302 Brittany Cross Road
Santa Ana, California 92705

Attention: Mr. Bob Kubichek
Senior Vice President

Project No.: **21G147-3**

Subject: **Updated Geotechnical Investigation**
Proposed Warehouse
NWC Placentia Avenue at Murrieta Road
Perris, California

Gentlemen:

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

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Pablo Montes Jr.
Staff Engineer

Robert G. Trazo, GE 2655
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
ROBERT TRAZO
No. 2655
GEOTECHNICAL
STATE OF CALIFORNIA

Distribution: (1) Addressee

TABLE OF CONTENTS

1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report.

Geotechnical Design Considerations

- Our site-specific liquefaction evaluation indicates that some of the on-site soils are subject to liquefaction during the design seismic event.
- The results of the liquefaction evaluation indicate total dynamic settlements ranging between 0.0 and 0.8± inches. The liquefaction induced differential settlements are expected to be on the order of ½± inch.
- The results of the corrosivity testing indicate that the near surface soils are severely corrosive to buried metallic improvements such as ductile iron pipe and to copper pipe.
- The near-surface alluvial soils possess varying strengths. These soils, in their present condition, are not considered suitable for support of the foundation loads of the new structure.
- Remedial grading will be necessary to remove a portion of the near-surface alluvial soils and replace them as compacted structural fill. Generally, the existing soils may be reused as structural fill.

Site Preparation Recommendations

- Initial site preparation should include stripping of any surficial vegetation. This includes the removal of native grass and weeds present within the site. These materials should be disposed of off-site. Concrete and miscellaneous debris material should also be disposed of off-site.
- Remedial grading is recommended to be performed within the proposed building area in order to remove a portion of the near-surface native alluvial soils. The soils within the proposed building area should be overexcavated to a depth of 4 feet below existing grade and to a depth of at least 4 feet below proposed building pad subgrade elevations. Deeper overexcavations, extending to depths of 5 to 6± feet, may be necessary in areas where loose soils are encountered at the base of the planned bottom of overexcavation.
- The proposed foundation influence zones should be overexcavated to a depth of at least 4 feet below proposed foundation bearing grade.
- After overexcavation has been completed, the subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated. The resulting subgrade should then be scarified to a depth of 12 inches, moisture conditioned or air dried to 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.
- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Building Foundation Recommendations

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

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

Building Floor Slab Recommendations

- Conventional slab-on-grade, at least 6 inches thick
- Modulus of subgrade reaction: k = 100 psi/in
- Reinforcement is not considered to be necessary for geotechnical considerations.
- Minimum slab reinforcement: No. 3 bars at 18-inches on-center, in both directions, due to the presence of medium expansive soils at this site. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

Pavements

ASPHALT PAVEMENTS (R=20)					
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	8	10	12	14	16
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R=20)				
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	7	8½
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Change Order No. 21GP147-CO, dated July 20, 2021. The scope of services included the review of the previously prepared geotechnical report 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. Based on the location of this site, this investigation also included a site-specific liquefaction evaluation. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the northwest corner of Placentia Avenue and Murrieta Road in Perris, California. The site is bounded to the north by an active grading site, to the east by Murrieta Road, to the south by Placentia Avenue and to the west by vacant lots. 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, 26.13± acres in size. The site is presently vacant and undeveloped. The ground surface consists of moderate to dense native grass and weed growth throughout. During the site investigation a 4- to 8-foot-high berm was found located in the southern region of the property. The berm consisted of Portland cement concrete debris and stockpiled soils surrounded by native grass and weed growth. Based on historic aerial photographs obtained from Google Earth, the site was previously tilled for agricultural activities between September of 2002 and June of 2012.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth and visual observations made at the time of the subsurface investigation, the site is relatively level with localized undulations of 1 to 2± feet.

3.2 Proposed Development

A conceptual site plan prepared by RGA, was provided to our office. Based on this plan, the site will be developed with one (1) new warehouse, 514,520± ft² in size, located in the central area of the site. Dock-high doors will be constructed in a cross-dock configuration along the north and south building walls. The building is expected to be surrounded by asphaltic concrete pavements in the parking and drive areas, Portland cement concrete pavements in the truck court areas, and limited areas of concrete flatwork and landscape planters throughout.

Detailed structural information has not been provided. It is assumed that the new building will be a single-story structure of tilt-up concrete construction, 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 up to 1 to 2± feet are expected to be necessary to achieve the proposed site grades.

3.3 Previous Studies

SCG recently performed a geotechnical investigation for the project site. The results of this investigation are presented in the above referenced report. As part of this investigation, a total of seven (7) borings were advanced to depths of 10 to 50± feet. Native alluvium was encountered at the ground surface at all the boring locations, extending to at least the maximum depth explored of 50± feet. The alluvial soils generally consist of interbedded layers of medium dense to very dense silty sands, sandy silts, and clayey sands with varying clay and fine gravel content and very stiff to hard silty clays, clayey silts and sandy clays with varying sand and silt content.

Results of laboratory testing indicate the on-site soils to possess a medium expansion potential (EI = 56) and the soils to be corrosive to buried metal pipes. A liquefaction evaluation was performed using data obtained from the two 50-foot-deep borings. The result of the analysis indicated total liquefaction induced settlements of 0 to 0.54± inches.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of ten (10) borings advanced to depths of 10 to 50± feet below the existing site grades. Boring Nos. B-1 through B-7 were performed as part of the previous study. All of the borings were logged during drilling by a member of our staff.

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

Alluvium

Native alluvium was encountered at the ground surface at all the boring locations, extending to at least the maximum depth explored of 50± feet. The alluvial soils generally consist of interbedded layers of medium dense to very dense silty sands, sandy silts, and clayey sands with varying clay and fine gravel content and very stiff to hard silty clays, clayey silts and sandy clays with varying sand and silt content.

Groundwater

Free water was encountered during the drilling at three (3) of the boring locations. Water was encountered at depths of 27 to 30± feet below existing site grades at Boring Nos. B-1, B-5 and B-9. The remaining boreholes were dry at the time of completion of drilling. Therefore, the static groundwater table is considered to have been present at depths of 27 to 30± feet below the existing site grades at the time of subsurface exploration.

Recent water level data was obtained from the California Department of Water Resources website, https://wdl.water.ca.gov/waterdatalibrary/. The nearest monitoring well is located approximately 3,168± feet northeast from the site. Water level readings within this monitoring well indicate a high groundwater level of 25± feet below the ground surface in November 2020.

5.0 LABORATORY TESTING

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

Classification

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

Density and Moisture Content

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

Grain Size Analysis

Limited grain size analyses have been performed on several selected samples, in accordance with ASTM D-1140. These samples were washed over a #200 sieve to determine the percentage of fine-grained material in each sample, which is defined as the material which passes the #200 sieve. The weight of the portion of the sample retained on each screen is recorded and the percentage finer or coarser of the total weight is calculated. The results of these laboratory tests are shown on the attached boring logs.

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-4 in Appendix C of this report. The results of the consolidation testing performed during the referenced geotechnical investigation are plotted on Appendix G of this report.

Maximum Dry Density and Optimum Moisture Content

One representative bulk sample has been tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plate C-5 in Appendix G 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.

Expansion Index

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

Sample Identification	Expansion Index	Expansive Potential
B-2 @ 0 to 5 feet (From Previous Study)	56	Medium

Soluble Sulfates

Representative samples of the near-surface soil were submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	Soluble Sulfates (%)	Sulfate Classification
B-3 @ 0 to 5 feet (From Previous Study)	0.014	Not Applicable (S0)
B-9 @ 0 to 5 feet	0.023	Not Applicable (S0)

Corrosivity Testing

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

Sample Identification	Saturated Resistivity (ohm-cm)	pH	Chlorides (mg/kg)	Nitrates (mg/kg)
B-3 @ 0 to 5 feet (From Previous Study)	840	7.8	22	656
B-9 @ 0 to 5 feet	960	7.8	77	384

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.

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

Seismic Design Parameters

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

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

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

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

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.500
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.572
Site Class	---	D*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.500
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.988
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.000
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.659

*The 2019 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site *coefficients* are to be determined in accordance with Section 11.4.7 of ASCE 7-16. However, Section 20.3.1 of ASCE 7-16 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of the structure(s) is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. Therefore, if the proposed structure has a fundamental period greater than 0.5 seconds, a site-specific seismic hazards analysis will be required and additional subsurface exploration will be necessary.

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.

Ground Motion Parameters

For the purposes of the liquefaction analysis performed for this study, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2019 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-16. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-16. The web-based software application SEAOC/OSHPD Seismic Design Maps Tool (described in the previous section) was used to determine PGA_M, which is 0.550g. A portion of the program output is included as Plate E-1 of this report. An associated earthquake magnitude was obtained from the USGS Unified Hazard Tool, Interactive Deaggregation application available on the USGS website. The deaggregated modal magnitude is 6.97, based on the peak ground acceleration and soil classification D.

Liquefaction

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

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

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and plasticity characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Non-sensitive clayey (cohesive) soils which possess a plasticity index of at least 18 (Bray

and Sancio, 2006) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The liquefaction analysis was conducted in accordance with the requirements of Special Publication 117A (CDMG, 2008), and currently accepted practice (SCEC, 1997). The liquefaction potential of the subject site was evaluated using the empirical method developed by Boulanger and Idriss (Boulanger and Idriss, 2008, 2014). This method predicts the earthquake-induced liquefaction potential of the site based on a given design earthquake magnitude and peak ground acceleration at the subject site. This procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude). CRR is determined as a function of the corrected SPT N-value $(N_1)_{60\text{-}cs}$, adjusted for fines content. The factor of safety against liquefaction is defined as CRR/CSR. Based on Special Publication 117A, a factor of safety of at least 1.3 is required in order to demonstrate that a given soil stratum is non-liquefiable. Additionally, in accordance with Special Publication 117A, clayey soils which do not meet the criteria for liquefiable soils defined by Bray and Sancio (2006), loose soils with a plasticity index (PI) less than 12 and moisture content greater than 85% of the liquid limit, are considered to be insusceptible to liquefaction. Non-sensitive soils with a PI greater than 18 are also considered non-liquefiable.

The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report. The liquefaction analysis was performed for Boring Nos. B-1, B-5 and B-9, which were advanced to depths of 50± feet. The liquefaction potential was analyzed at the boring locations utilizing a PGA_M of 0.550g related to a 6.97 magnitude seismic event. The liquefaction evaluation was performed using the reported high groundwater depth of 25 feet.

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

<u>Conclusions and Recommendations</u>

Potentially liquefiable soils were encountered at one of the 50±-foot deep boring locations. The potentially liquefiable soils were generally encountered at depths between 27 to 37± feet. The remaining soil strata encountered below the reported high groundwater table either possess factors of safety of 1.3 or greater and are considered non-liquefiable with respect to the requirements of Special Publication 117A. Settlement analyses were performed for the potentially liquefiable strata. The results of the settlement analyses indicate the following potential total deformations:

- Boring No. B-1: 0.00 inches
- Boring No. B-5: 0.54 inches
- Boring No. B-9: 0.80 inches

Based on the results of the settlement analyses, total dynamic settlements due to liquefaction are expected to range from 0.0 to 0.8± inches. The resulting differential settlements are expected to be on the order of ½± inches. The estimated differential settlement can be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of less than 0.001 inches per inch.

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

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

The use of a shallow foundation system, as described in this report, is typical for buildings of this type, where they are underlain by the extent of liquefiable soils encountered at this site. The post-liquefaction damage that could occur within the building proposed for this site will also be typical of similar buildings in the vicinity of this project. However, if the owner determines that this level of potential damage is not acceptable, other geotechnical and structural options are available, including the use of ground improvement techniques or mat foundations.

6.2 Geotechnical Design Considerations

General

The site is generally underlain by native alluvium at all of the boring locations. These soils possess variable densities, variable composition, and slightly porous soils were encountered at one of the boring locations. Based on these conditions, the near-surface alluvium, in their present condition, are not considered suitable for support of the foundations and floor slab of the new structure. Remedial grading will be necessary within the proposed building area to remove a portion of the near-surface alluvium, and to replace these soils as compacted structural fill.

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

Settlement

Laboratory testing indicates that some of the near surface soils possess a potential for collapse when inundated with water. 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.

Expansion

The near-surface soils at this site generally consist of silty sands, sandy silts, silty clays, and sandy clays. Laboratory testing indicates that the on-site soils possess medium expansion potential (EI=56). The foundation and floor slab design recommendations contained within this report are made in consideration of the expansion index test results. It is recommended that additional expansion index testing be conducted at the completion of rough grading to verify the expansion potential of the as-graded building pad.

Soluble Sulfates

The results of the soluble sulfate testing indicated sulfate concentrations of approximately 0.014 to 0.023 percent for the selected samples of the near-surface soils. These concentrations are 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 840 and 960 ohm-cm, and a pH value of 7.8. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity and pH are two of the five factors that enter into the evaluation procedure. Redox potential, relative soil moisture content and sulfides are also included. Although sulfide testing was not part of the scope of services for this project, we have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH and moisture content. Based on these factors, and utilizing the DIPRA procedure, **the on-site soils are considered to be highly corrosive to ductile iron pipe. Therefore, polyethylene encasement or some other appropriate method of protection will be required for iron pipes.**

Relatively low concentrations (22 and 77 mg/kg) of chlorides were detected in the samples submitted for corrosivity testing. In general, soils possessing chloride concentrations in excess of 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.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested samples possess nitrate concentrations of 384 and 656 mg/kg. **Based on this test result, the on-site soils are considered to be corrosive to copper pipe.**

Since SCG does not practice in the area of corrosion engineering, we recommend that the client contact a corrosion engineer to provide a more thorough evaluation.

Shrinkage/Subsidence

Removal and recompaction of the artificial fill and near-surface native soils is estimated to result in an average shrinkage of 5 to 13 percent. Shrinkage estimates for the individual samples range between 0 and 22 percent based on the results of density testing and the assumption that the on-site 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.

Existing Easement

An existing easement traverses the northeast corner of the project site. At the time of this report, it is unknown if a utility line or underground structure is located within this easement. If a high-pressure utility line or underground structure that will need to be protected is located within this easement, special design consideration may need to be taken. This may include, but not be limited to, construction of a "bridge" over the utility or structure, eliminating the drive lane in this area, or relocating the utility or structure.

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

Initial site preparation should include stripping of any surficial vegetation and organic soils. Based on conditions encountered at the time of the subsurface exploration, stripping of native grass and weed growth is expected to be necessary throughout the majority of the site. These materials should be disposed of off-site. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building area in order to remove a portion of the near-surface alluvium soils. The existing soils within the proposed building area are recommended to be overexcavated to a depth of at least 4 feet below existing grade and to a depth of at least 4 feet below proposed building pad subgrade elevation, whichever is greater.

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.

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 structures. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if loose, porous, or low density native soils are encountered at the base of the overexcavation. **Zones of deeper excavations, extending to depths of 5 to 6± feet below existing site grades, may be required if loose soils are encountered at the base of the overexcavation.**

Based on the conditions encountered at one of the exploratory boring locations, some zones of very moist soils may be encountered in localized areas at or near the base of the recommended overexcavation. Stabilization of the exposed overexcavation subgrade soils may be necessary. Where encountered at the boring locations, these very moist layers are generally underlain by drier alluvial soils, so it may be feasible to excavate the very moist soils to a layer of drier, more stable soils. Scarification and air drying of these "very moist" soils may also be sufficient to obtain a stable subgrade. However, if highly unstable soils are identified, and if the construction schedule does not allow for delays associated with drying, mechanical stabilization, usually consisting of coarse crushed stone or geotextile, could be necessary. In this event, the geotechnical engineer should be contacted for supplementary recommendations. Typically, an unstable subgrade can be stabilized using a suitable geotextile fabric, such as Mirafi RS580I, and/or a 12 to 18-inch thick layer of coarse (2 to 4-inch particle size) crushed stone. Crushed asphalt and concrete debris resultant from demolition could also be used as a subgrade stabilization material. Other options, including lime or cement treatment are also available.

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 2 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The building pad area may then be raised to grade with previously excavated soils or imported 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. Any erection pads used to construct the walls are considered to be part of the foundation system with respect to these remedial grading recommendations. 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.

Please note that if the lateral and/or vertical extents of overexcavation are not achievable for the project retaining walls or site walls, then additional recommendations including, but not limited to reduced design bearing pressures may be required. Additionally, specialized grading techniques such as slot cutting or shoring may be required in order to facilitate construction.

Treatment of Existing Soils: Parking and Drive Areas

Based on economic considerations, overexcavation of the existing 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 operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength alluvial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within the proposed parking areas. The grading recommendations presented above do not completely mitigate the extent of 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 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2019 CBC and the grading code of the city of Perris.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

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

Utility Trench Backfill

In general, all utility trench backfill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. 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 Perris. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

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

6.4 Construction Considerations

Excavation Considerations

The near surface soils 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 for the onsite clayey soils 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.

Moisture Sensitive Subgrade Soils

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

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

Three samples of the near-surface soils (Boring No. B-1 at a depth of 3½ to 5± feet and 6 to 7½± feet, and B-8 at 6½ to 8½± feet) possesses moisture contents well over the optimum for recompaction. Allowances should be made for costs and delays associated with drying the very moist on-site soils or the import of a drier, less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pad area as well as the need for and/or the thickness of the crushed stone stabilization layer, discussed in Section 6.3 of this report.

Expansive Soils

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

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

- Ponding and areas of low flow gradients in unpaved walkways, grass and planter areas should be avoided. In general, minimum drainage gradients of 2 percent should be maintained in unpaved areas.
- Bare soil within five feet of proposed structures should be sloped at a minimum five percent gradient away from the structures (about three inches of fall in five feet), or the same area could be paved with a minimum surface gradient of one percent. Pavement is preferable.
- Decorative gravel ground cover tends to provide a reservoir for surface water and may hide areas of ponding or poor drainage. Decorative gravel is, therefore, not recommended and should not be utilized for landscaping unless equipped with a subsurface drainage system designed by a licensed landscape architect.
- Positive drainage devices, such as graded swales, paved ditches, and catch basins should be installed at appropriate locations within the area of proposed development.
- Concrete walks and flatwork should not obstruct the free flow of surface water to the appropriate drainage devices.
- Area drains should be recessed below grade to allow free flow of water into the drain. Concrete or brick flatwork joints should be sealed with mortar or flexible mastic.
- Gutter and downspout systems should be installed to capture all discharge from roof areas. Downspouts should discharge directly into a pipe or paved surface system to be conveyed offsite.
- Enclosed planters adjoining, or in close proximity to proposed structures, should be sealed at the bottom and provided with subsurface collection systems and outlet pipes.
- Depressed planters should be raised with soil to promote runoff (minimum drainage gradient two percent or five percent, see above), and/or equipped with area drains to eliminate ponding.
- Drainage outfall locations should be selected to avoid erosion of slopes and/or properly armored to prevent erosion of graded surfaces. No drainage should be directed over or towards adjoining slopes.
- All drainage devices should be maintained on a regular basis, including frequent observations during the rainy season to keep the drains free of leaves, soil and other debris.
- Landscape irrigation should conform to the recommendations of the landscape architect and should be performed judiciously to preclude either soaking or excessive drying of the foundation soils. This should entail regular watering during the drier portions of the year and little or no irrigation during the rainy season. Automatic sprinkler systems should, therefore, be switched to manual operation during the rainy season. Good irrigation practice typically requires frequent application of limited quantities of water that are sufficient to sustain plant growth, but do not excessively wet the soils. Ponding and/or run-off of irrigation water are indications of excessive watering.

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

Groundwater

Ground water table was considered to be present at depths of 27 to 30± feet at the boring locations, at the time of subsurface exploration. Groundwater is therefore 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 near-surface alluvial soils. These new structural fill soils are expected to extend to depths of at least 4 feet below proposed foundation bearing grades, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, and based on the design considerations presented in Section 6.1 of this report, the proposed 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^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom) in strip footings, due to the presence of expansive and potentially liquefiable soils.
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements, as discussed in Section 6.1. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

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

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

<u>Estimated Foundation Settlements</u>

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 50-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch. These settlements are in addition to the liquefaction-induced settlements previously discussed in Section 6.1 of this report.

<u>Lateral Load Resistance</u>

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: 250 lbs/ft^3
- Friction Coefficient: 0.25

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

6.6 Floor Slab Design and Construction

Subgrades which will support 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 4 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 = 100 psi/in

- Minimum slab reinforcement: No. 3 bars at 18-inches on-center, in both directions, due to the presence of medium expansive soils at this site. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

- Slab underlayment: If moisture sensitive floor coverings will be used 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 2 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

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

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

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

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

RETAINING WALL DESIGN PARAMETERS

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

The walls should be designed using a soil-footing coefficient of friction of 0.25 and an equivalent passive pressure of 250 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

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

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

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

Backfill Material

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

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

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

6.8 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The on-site soils generally consist of silty sands, sandy silts, clayey sands, and sandy clays. These materials are expected to exhibit fair to good pavement support characteristics, with estimated R-values between 20 and 40. Therefore the subsequent pavement design is based upon a conservative R-value of 20. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering-controlled conditions. It is recommended that additional R-value testing be performed after completion of rough grading to verify the pavement support characteristics of the pavement subgrades following site grading.

Asphaltic Concrete

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

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

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

ASPHALT PAVEMENTS (R=20)					
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	8	10	12	14	16
Compacted Subgrade	12	12	12	12	12

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

Portland Cement Concrete

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

PORTLAND CEMENT CONCRETE PAVEMENTS (R=20)				
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	7	8½
Compacted Subgrade (95% minimum compaction)	12	12	12	12

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

7.0 GENERAL COMMENTS

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

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

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

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

8.0 REFERENCES

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

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

Idriss, I. M. and Boulanger, R. W., "SPT-Based Liquefaction Triggering Procedures," Earthquake Engineering Research Institute, 2010.

Boulanger, R. W. and Idriss, I. M., "CPT and SPT Based Liquefaction Triggering Procedures," Earthquake Engineering Research Institute, 2014.

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

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

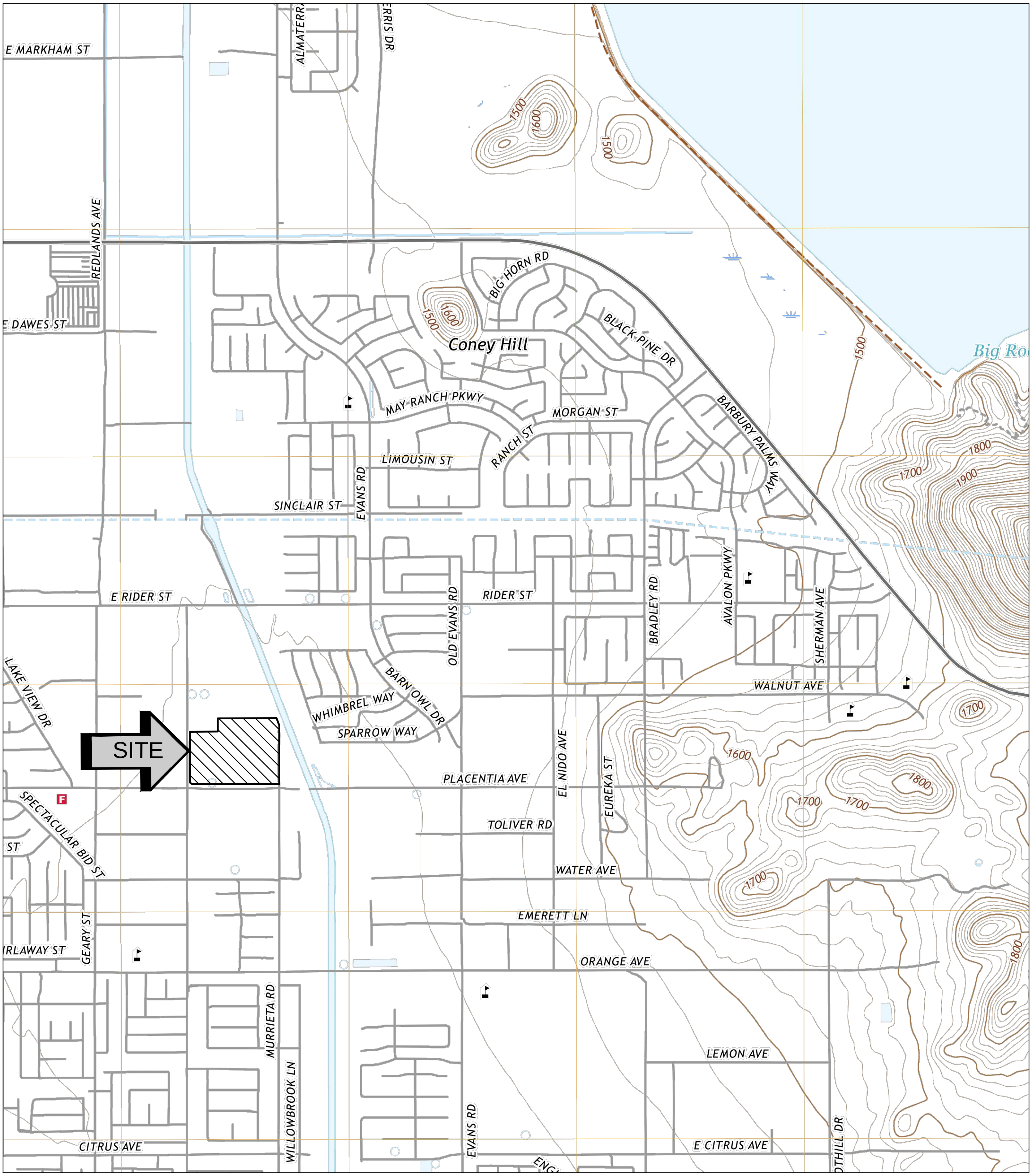
Southern California Earthquake Center (SCEC), University of Southern California, "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California," Committee formed 1997.

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

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

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

APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAP OF THE PERRIS QUADRANGLE, RIVERSIDE COUNTY, CALIFORNIA, 2018.

SITE LOCATION MAP

PROPOSED WAREHOUSE

PERRIS, CALIFORNIA

SCALE: 1" = 2000'

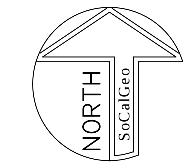
DRAWN: MD
CHKD: RGT

SCG PROJECT
21G147-3

PLATE 1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



GEOTECHNICAL LEGEND

- APPROXIMATE ADDITIONAL BORING LOCATION
- PREVIOUS BORING LOCATION (SCG PROJECT NO. 21G147-1)
- PREVIOUS INFILTRATION TEST LOCATION (SCG PROJECT NO. 21G147-2)

NOTE: SITE PLAN PREPARED BY RGA.

ADDITIONAL BORING LOCATION PLAN

NEC PLACENTIA AVENUE/WILSON AVENUE

PERRIS, CALIFORNIA

SCALE: 1" = 120'

DRAWN: JAZ
CHKD: RGT

SCG PROJECT
21G147-3

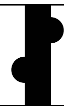
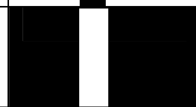
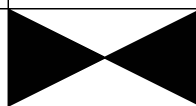
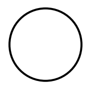
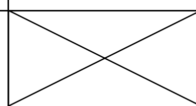
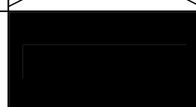
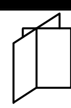
PLATE 2

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

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

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

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

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

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

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

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

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

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

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 21G147-3
PROJECT: Proposed Warehouse
LOCATION: Perris, California
DRILLING DATE: 7/22/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jose Zuniga
WATER DEPTH: Dry
CAVE DEPTH: 16 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
		23		ALLUVIUM: Gray Brown Silty fine Sand, slightly cemented, trace calcareous veining, medium dense-moist	101	9					
		19		Light Brown to Brown Clayey fine Sand, trace to little calcareous veining, loose to medium dense-moist to very moist	94	11					
5		9			90	16					
		15	2.0	Light Gray Silty Clay to Clayey Silt, abundant calcareous veining, stiff-very moist	94	20					
		22		Brown Silty fine Sand, trace Clay, abundant calcareous veining, medium dense-moist	100	8					
10											
		24		Brown Clayey fine Sand, trace medium Sand, cemented, trace calcareous veining, medium dense to dense-moist to very moist		10					
15											
		50				16					
20				Boring Terminated at 20'							

TBL 21G147-3.GPJ SOCALGEO.GDT 8/20/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-9

JOB NO.: 21G147-3
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 7/22/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jose Zuniga

WATER DEPTH: 30 feet
CAVE DEPTH: 25 feet
READING TAKEN: 20 minutes after Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ALLUVIUM: Light Brown Silty fine Sand, trace calcareous veining, medium dense-damp							
	X	24				97	5					
	X	33				94	6					
					Light Gray Brown Silty fine Sand, slightly porous, slightly cemented, abundant calcareous veining, loose to medium dense-damp							
5	X	15				102	6					
					Brown Clayey fine Sand, slightly cemented, trace calcareous veining, medium dense-damp to moist							
	X	22				98	12					
	X	36				127	6					
10												
	X	24					9					
15												
	X	23					11					
20												
					Brown Silty fine Sand, trace Iron oxide staining, medium dense to dense-moist to very moist							
	X	47					8			35		
25												
	X	16					15			47		

TBL 21G147-3.GPJ SOCALGEO.GDT 8/20/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-9

JOB NO.: 21G147-3
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 7/22/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jose Zuniga

WATER DEPTH: 30 feet
CAVE DEPTH: 25 feet
READING TAKEN: 20 minutes after Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Brown Silty fine Sand, trace Iron oxide staining, medium dense to dense-damp to wet							Groundwater @ 30 feet, during drilling
35	X	17					13			44		
					Brown Silty fine to medium Sand, trace Clay, micaceous, dense-wet							
40	X	31					13			20		
					Brown fine to medium Sand, trace coarse Sand, some Silt, very dense-wet							
45	X	51					10			17		
					Brown Silty fine Sand, dense-wet							
50	X	34					14			40		
					Boring Terminated at 50'							

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-10

JOB NO.: 21G147-3
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 7/22/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jose Zuniga

WATER DEPTH: Dry
CAVE DEPTH: 7 feet
READING TAKEN: At Completion

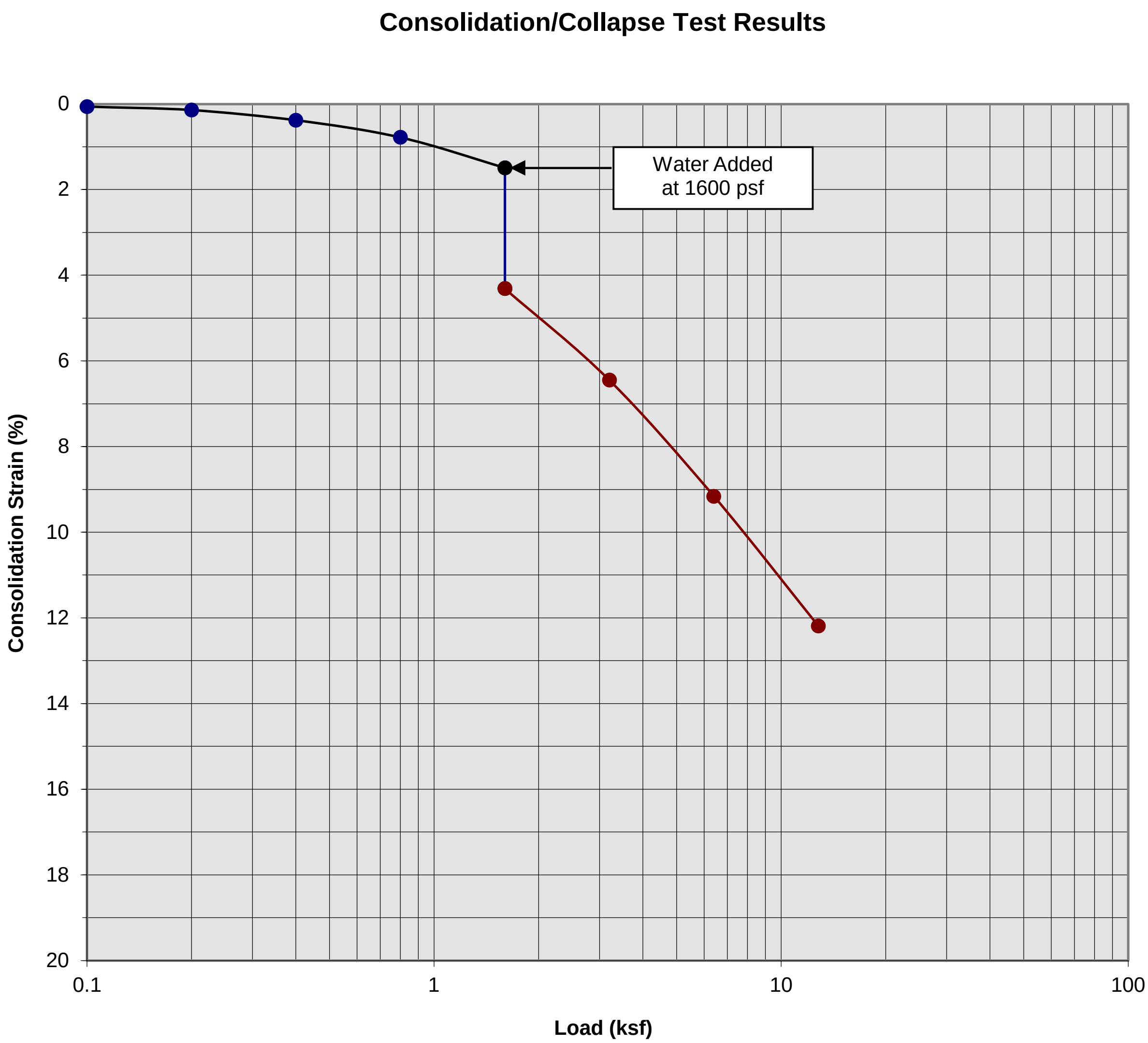
SURFACE ELEVATION: --- MSL

FIELD RESULTS: DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS: DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ALLUVIUM: Brown to Light Brown Silty fine Sand, trace to little calcareous veining, loose to medium dense-damp							
	X	8					7					
	X	22					7					
5					Light Brown fine Sandy Silt, abundant calcareous veining, medium dense-very moist							
	X	23					15					
	X	13					15					
10												

Boring Terminated at 10'

TBL 21G147-3.GPJ SOCALGEO.GDT 8/20/21

APPENDIX C

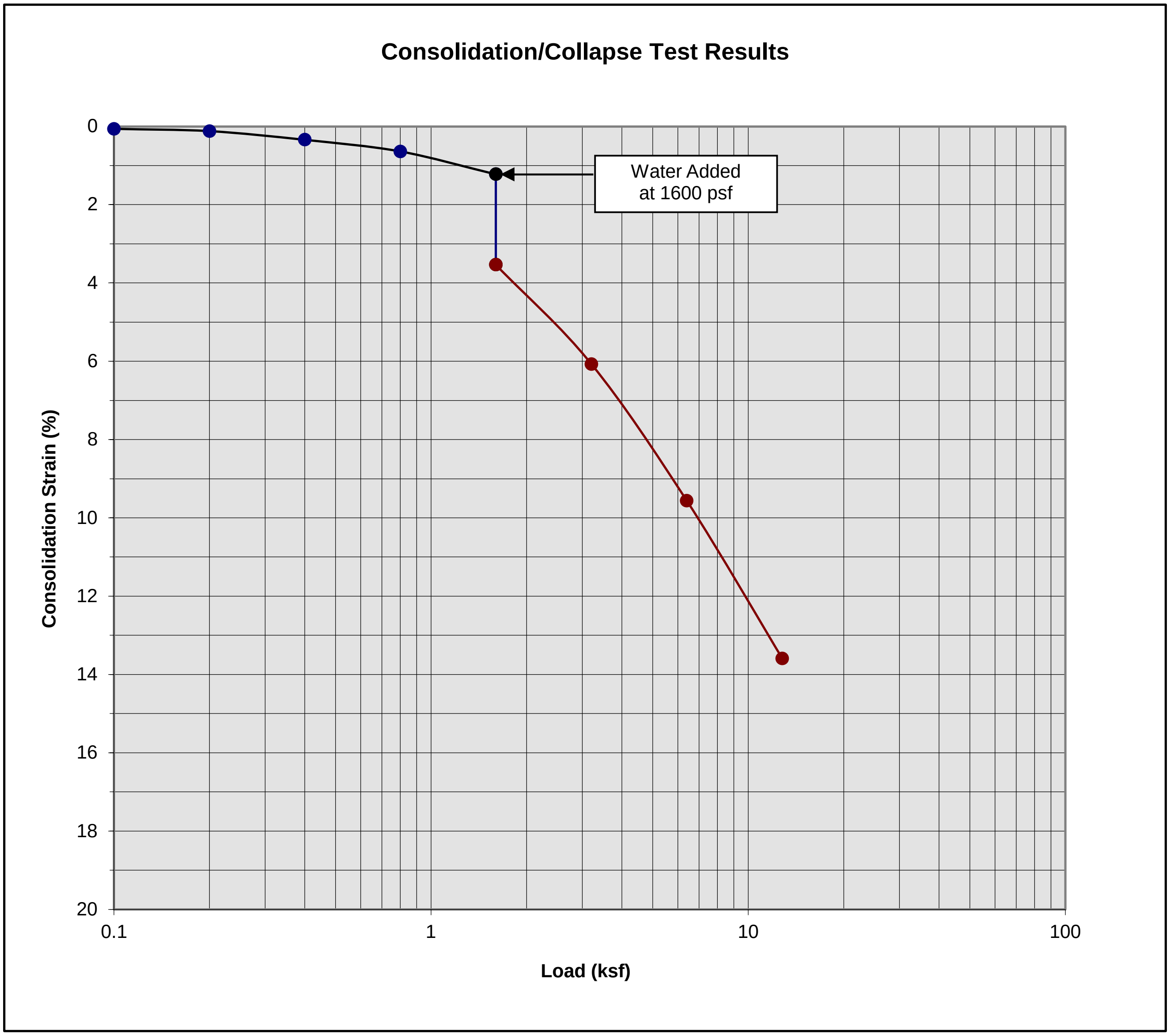


Classification: Gray Brown Silty fine Sand

Boring Number:	B-8	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	1 to 2	Initial Dry Density (pcf)	101.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.82

Proposed Warehouse
Perris, CA
Project No. 21G147-3
PLATE C- 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



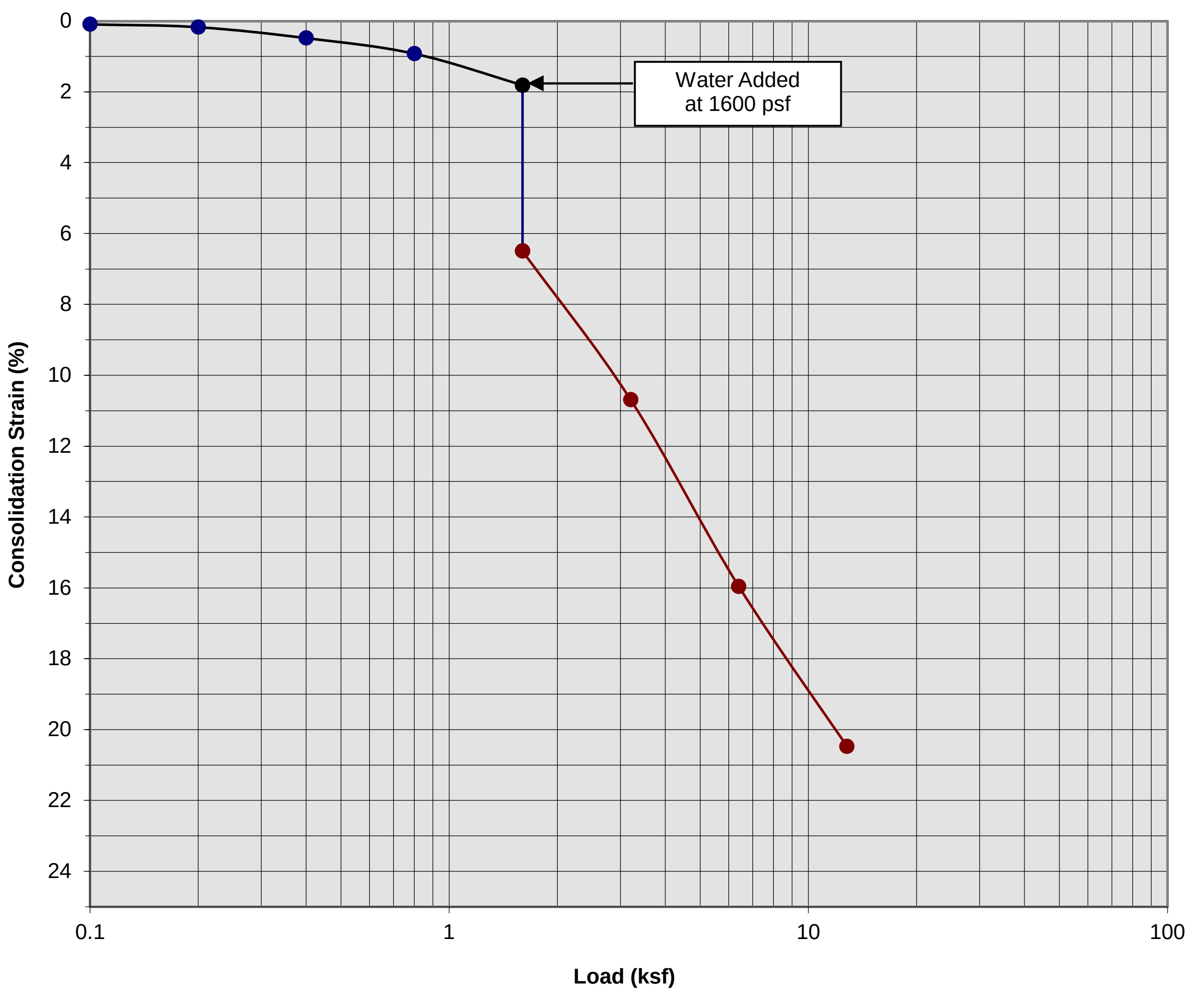
Classification: Light Brown to Brown Clayey fine Sand

Boring Number:	B-8	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	3 to 4	Initial Dry Density (pcf)	94.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.31

Proposed Warehouse
Perris, CA
Project No. 21G147-3
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

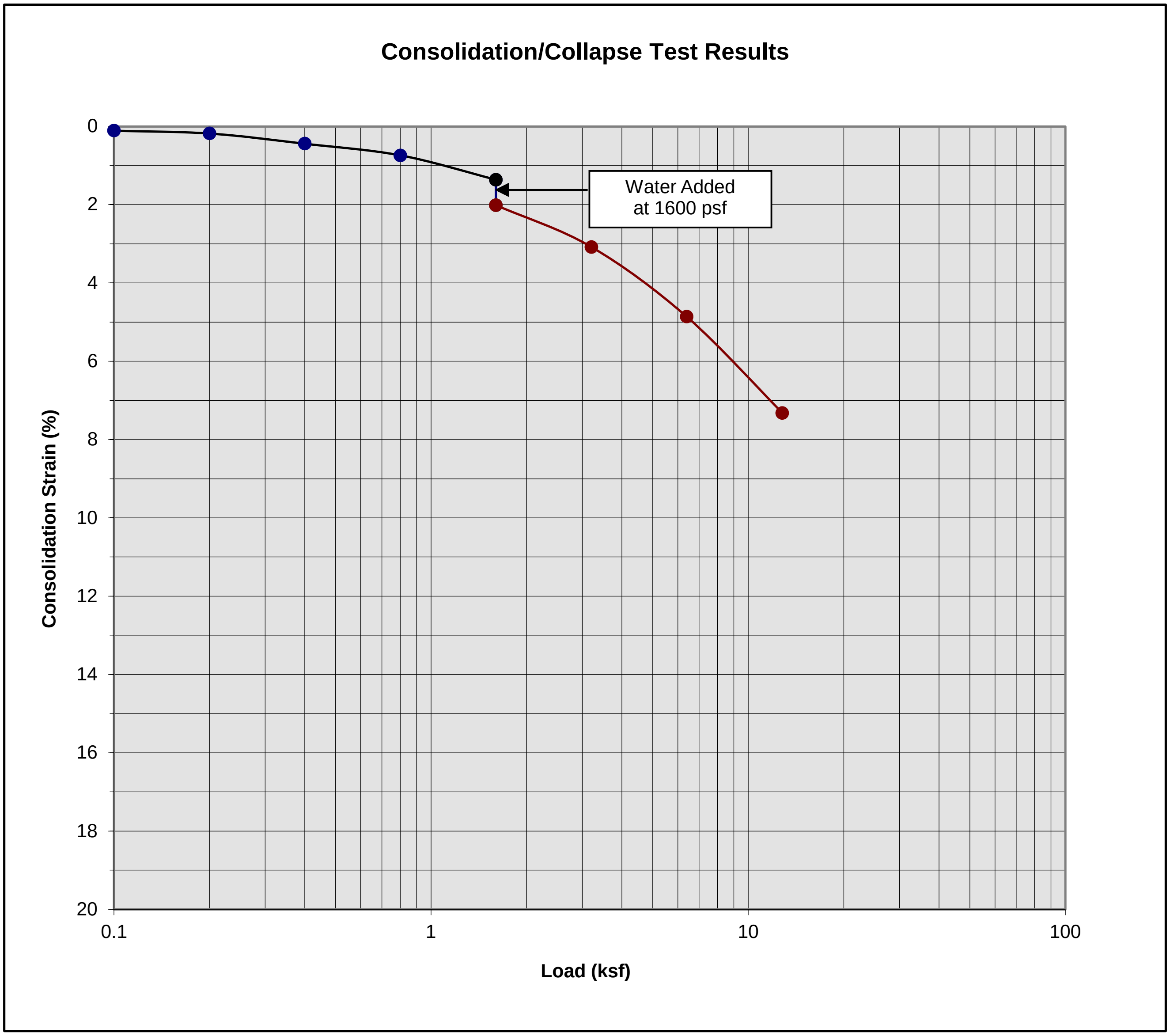


Classification: Light Brown to Brown Clayey fine Sand

Boring Number:	B-8	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	27
Depth (ft)	5 to 6	Initial Dry Density (pcf)	90.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	113.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	4.68

Proposed Warehouse
Perris, CA
Project No. 21G147-3
PLATE C- 3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL *A California Corporation*



Classification: Light Gray Silty Clay to Clayey Silt

Boring Number:	B-8	Initial Moisture Content (%)	20
Sample Number:	---	Final Moisture Content (%)	27
Depth (ft)	7 to 8	Initial Dry Density (pcf)	94.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	101.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.65

Proposed Warehouse
Perris, CA
Project No. 21G147-3
PLATE C- 4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.

- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.

- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.

- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.

- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.

- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

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

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

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

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

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

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

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

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

<u>Compacted Fills</u>

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated," and shall be very low to non-expansive with a maximum expansion index (EI) of 50. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.

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

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

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

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

  - Fill materials consisting of soil meeting the minimum moisture content requirements and free of oversize material should be placed between and over the rows of rock or concrete. Ample water and compactive effort should be applied to the fill materials as they are placed in order that all of the voids between each of the fragments are filled and compacted to the specified density.

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

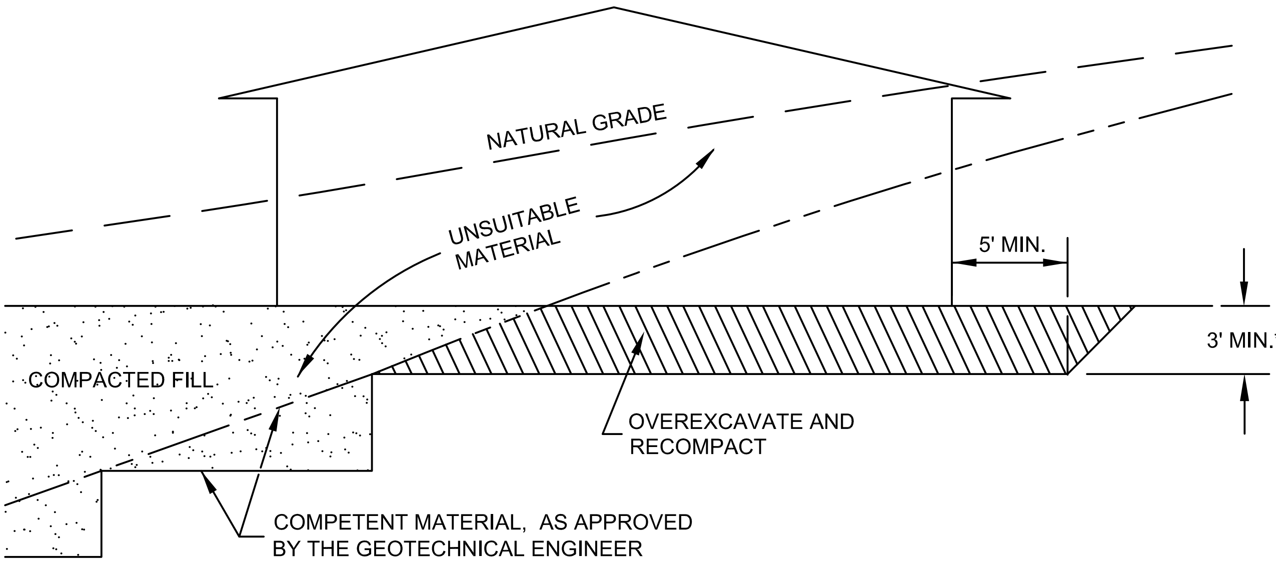
Subdrains

- Subdrains may be required in canyons and swales where fill placement is proposed. Typical subdrain details for canyons are shown on Plate D-3. Subdrains should be installed after approval of removals and before filling, as determined by the Soils Engineer.

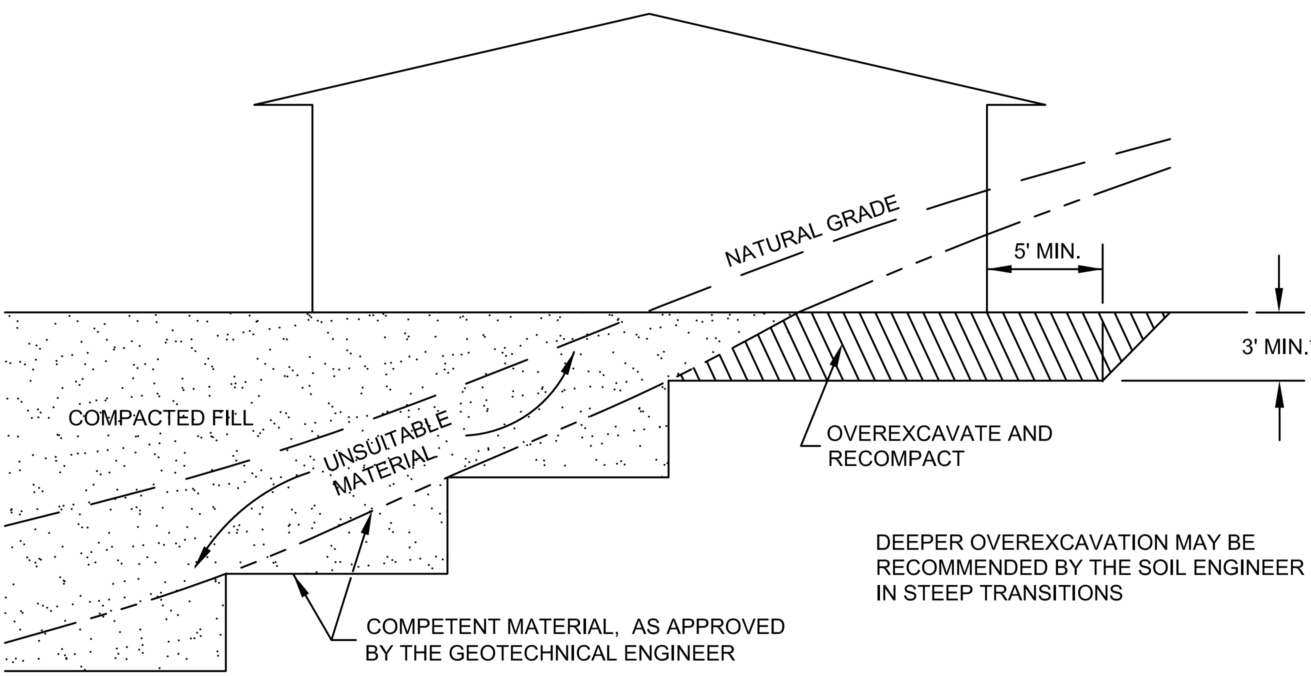
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- Filter material for subdrains should conform to CALTRANS Specification 68-1.025 or as approved by the Geotechnical Engineer for the specific site conditions. Clean ¾-inch crushed rock may be used provided it is wrapped in an acceptable filter cloth and approved by the Geotechnical Engineer. Pipe diameters should be 6 inches for runs up to 500 feet and 8 inches for the downstream continuations of longer runs. Four-inch diameter pipe may be used in buttress and stabilization fills.

CUT LOT



CUT/FILL LOT (TRANSITION)



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

TRANSITION LOT DETAIL

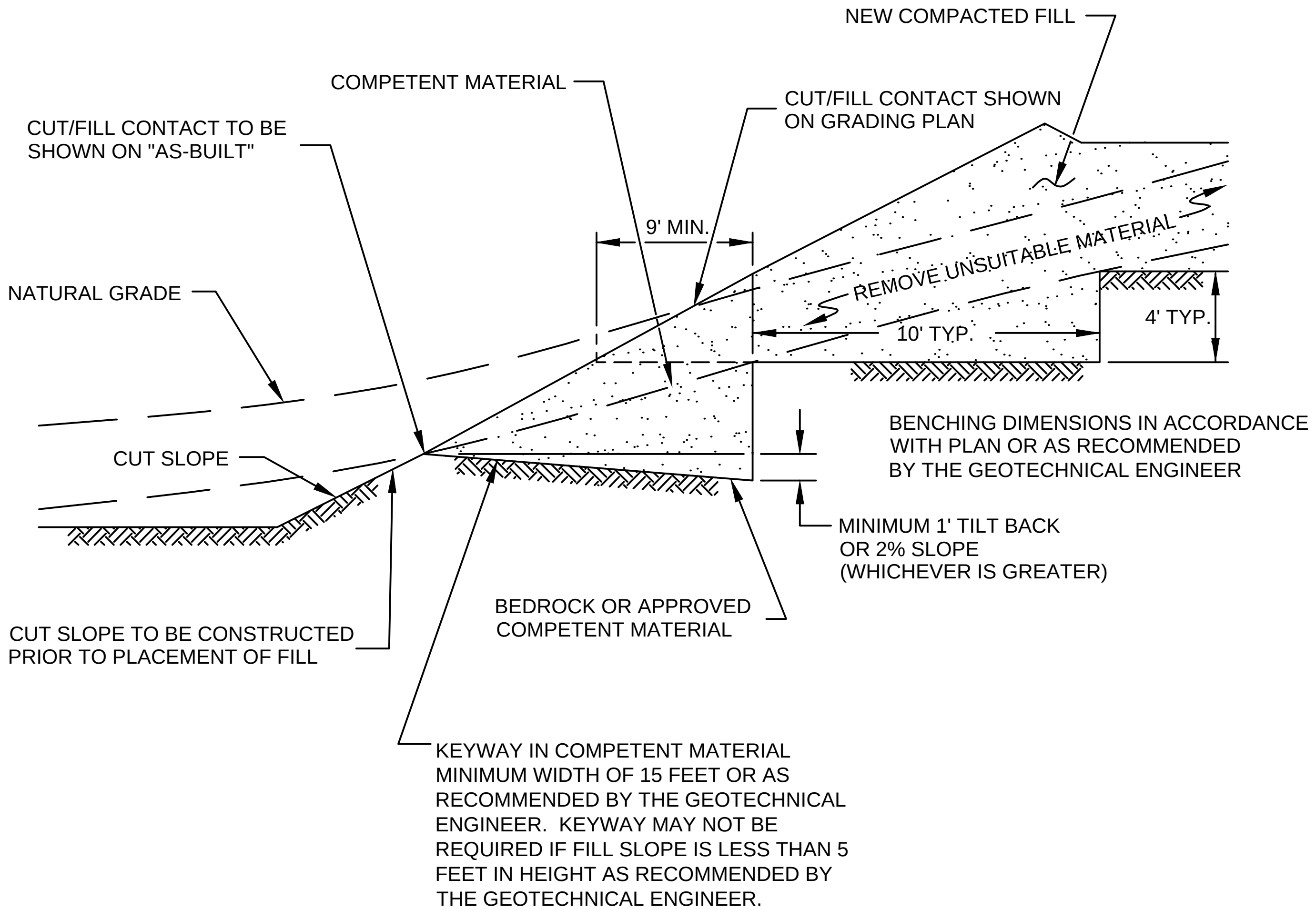
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

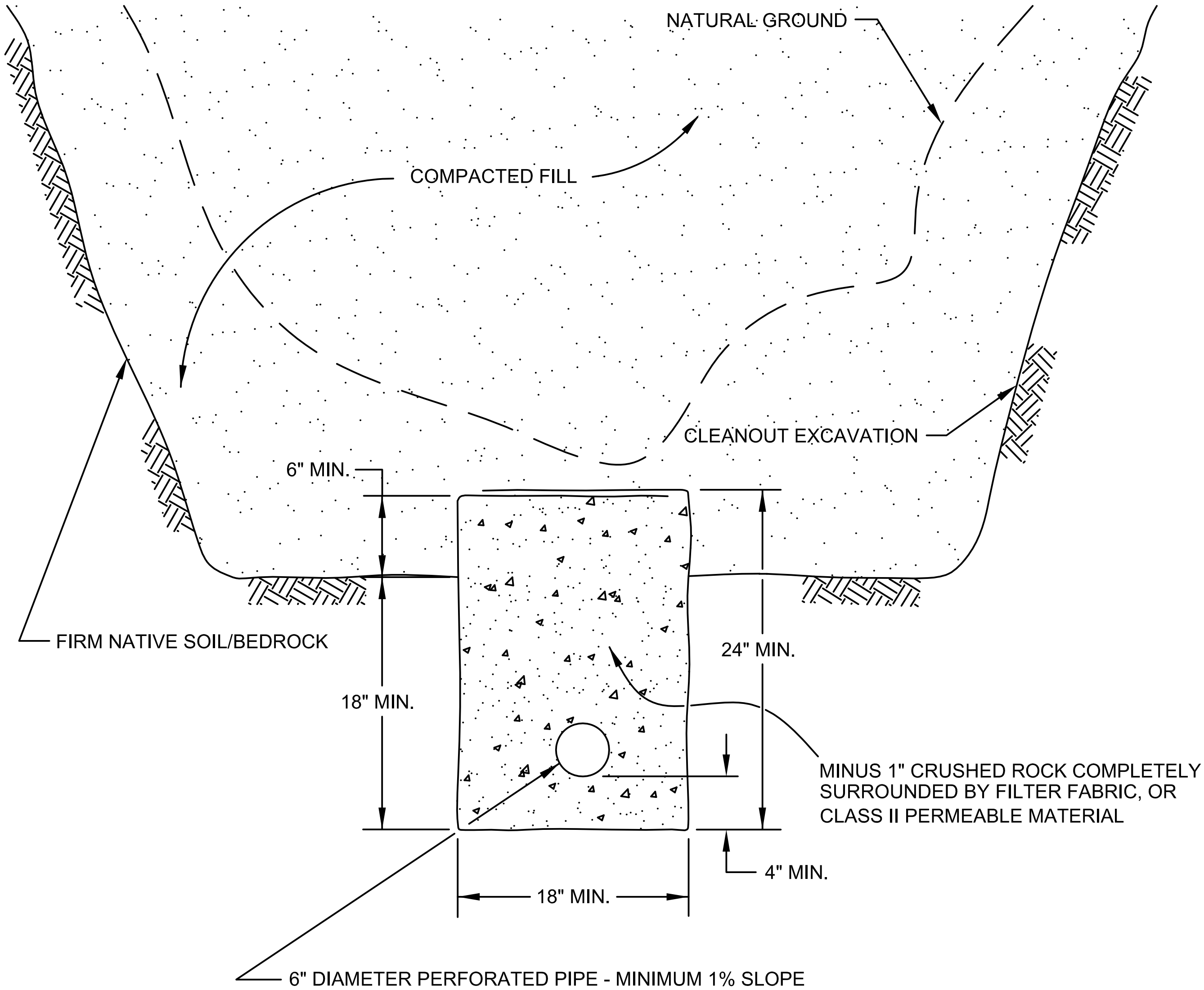
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SOUTHERN CALIFORNIA GEOTECHNICAL



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

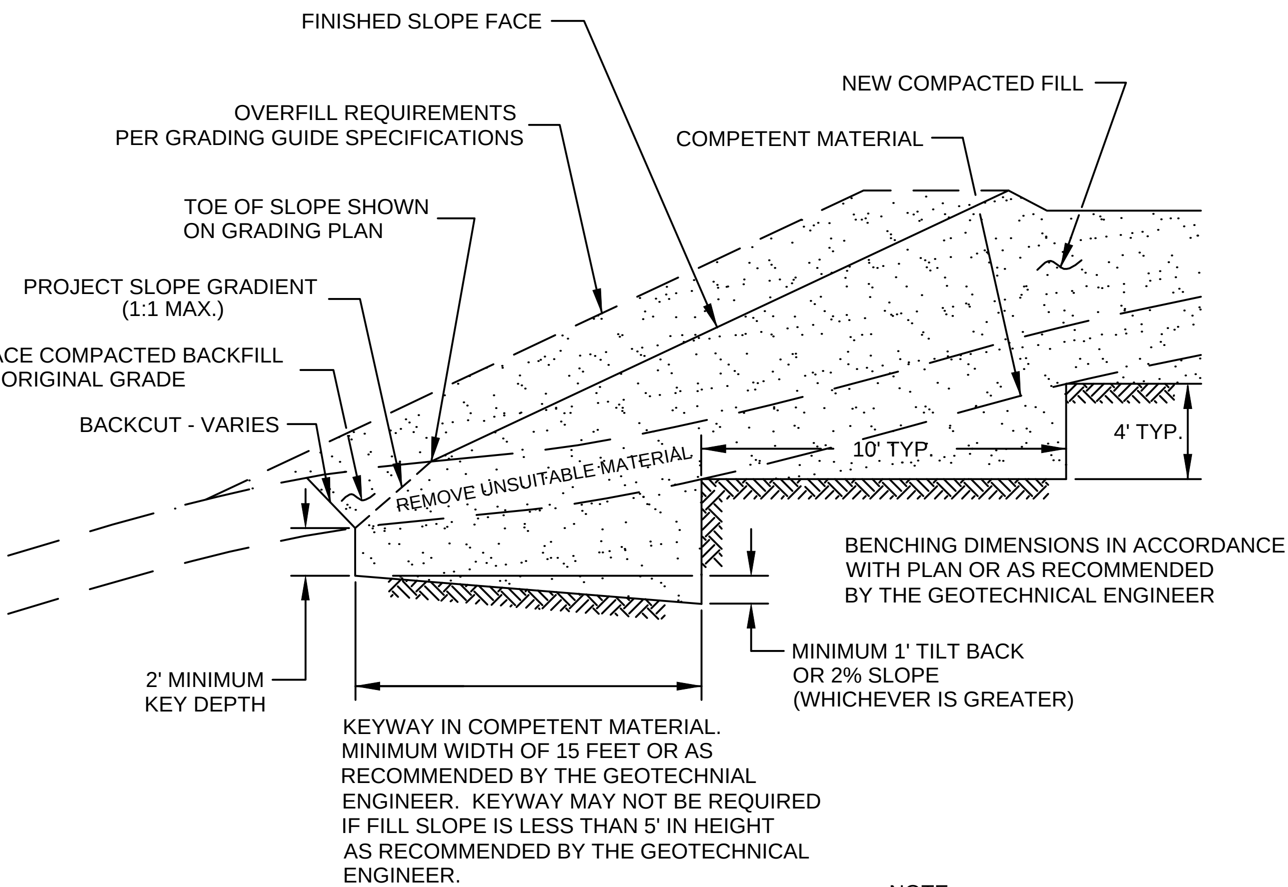
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

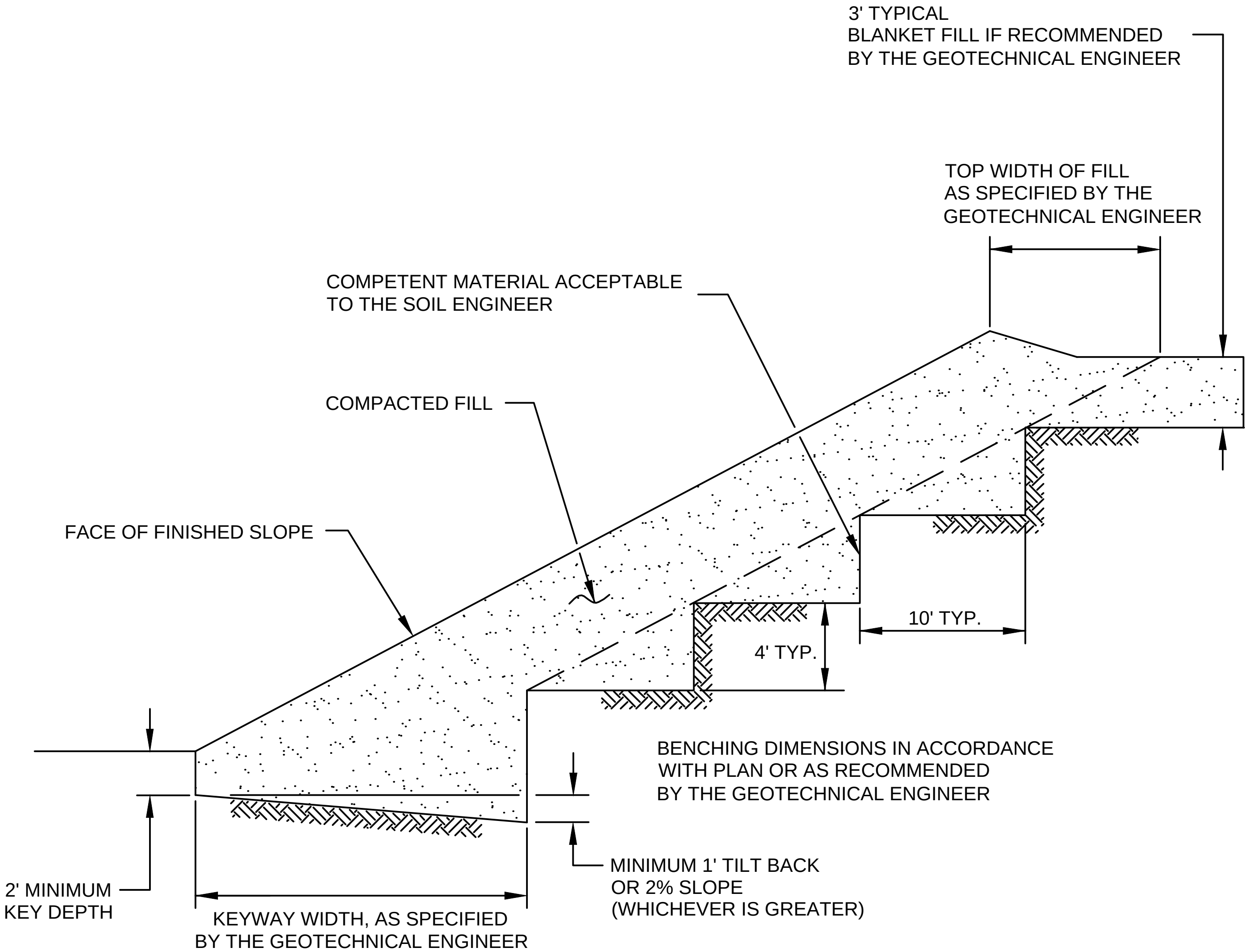


NOTE:
BENCHING SHALL BE REQUIRED WHEN NATURAL SLOPES ARE EQUAL TO OR STEEPER THAN 5:1 OR WHEN RECOMMENDED BY THE GEOTECHNICAL ENGINEER.

FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

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



STABILIZATION FILL DETAIL

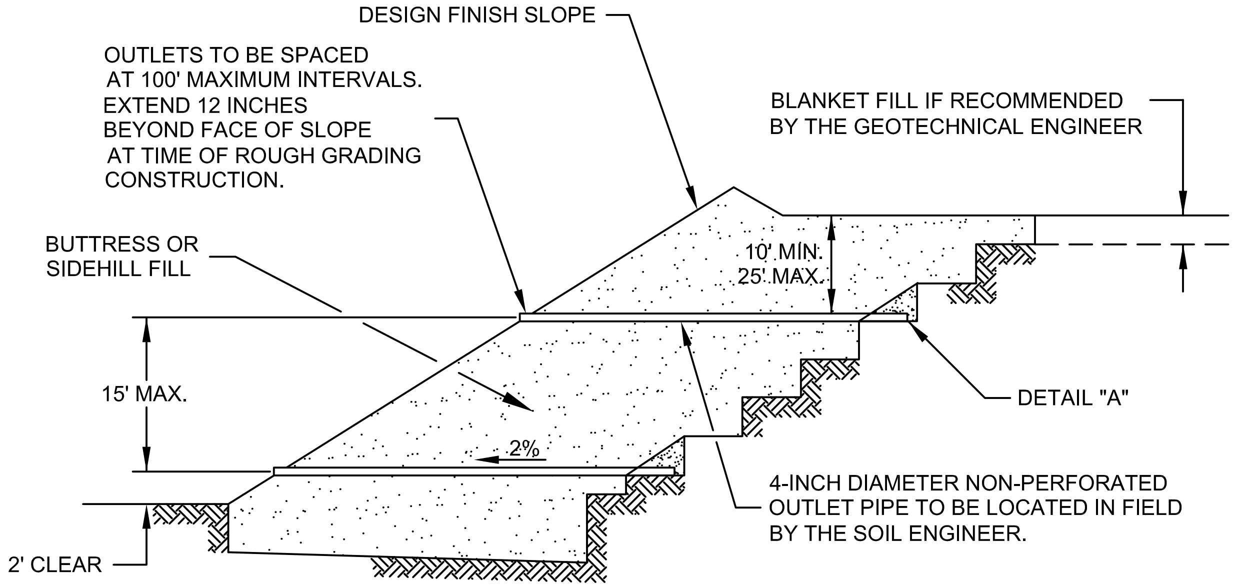
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

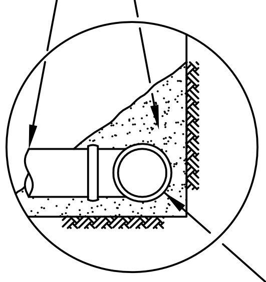
SAND EQUIVALENT = MINIMUM OF 50

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

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

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

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



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

<u>DETAIL "A"</u>

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

SLOPE FILL SUBDRAINS

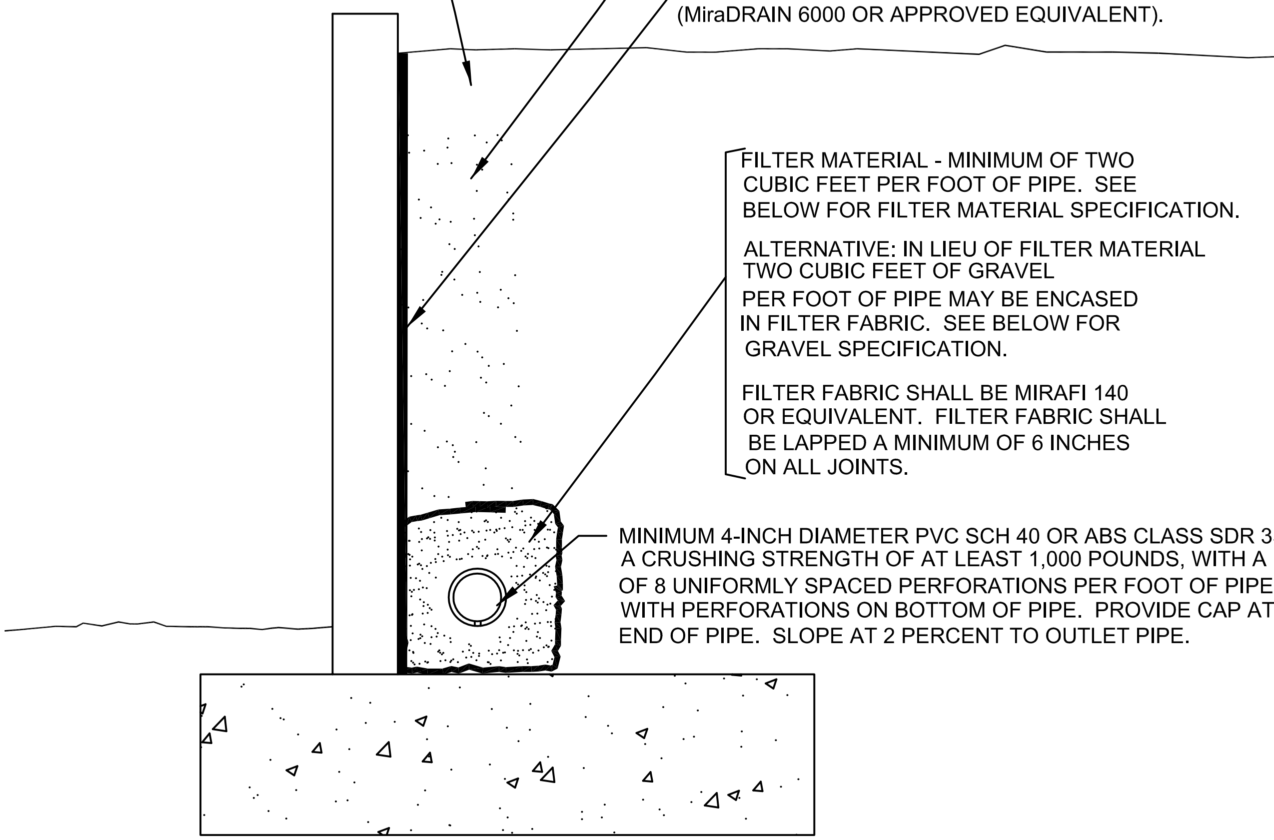
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

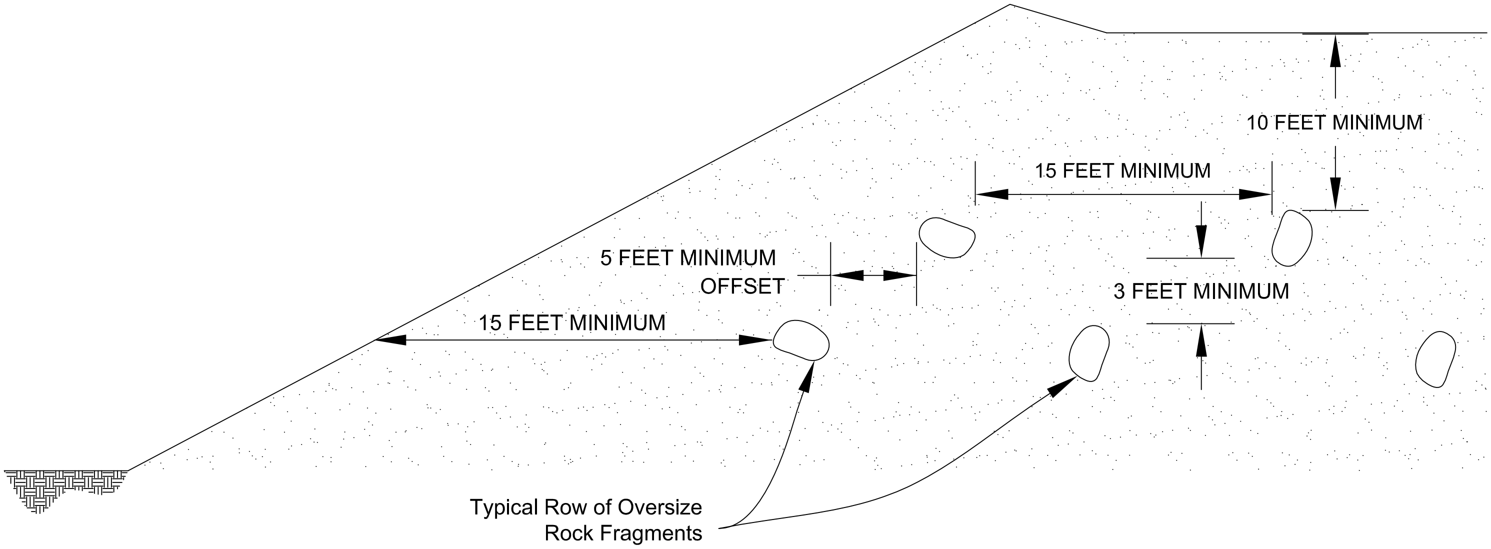
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

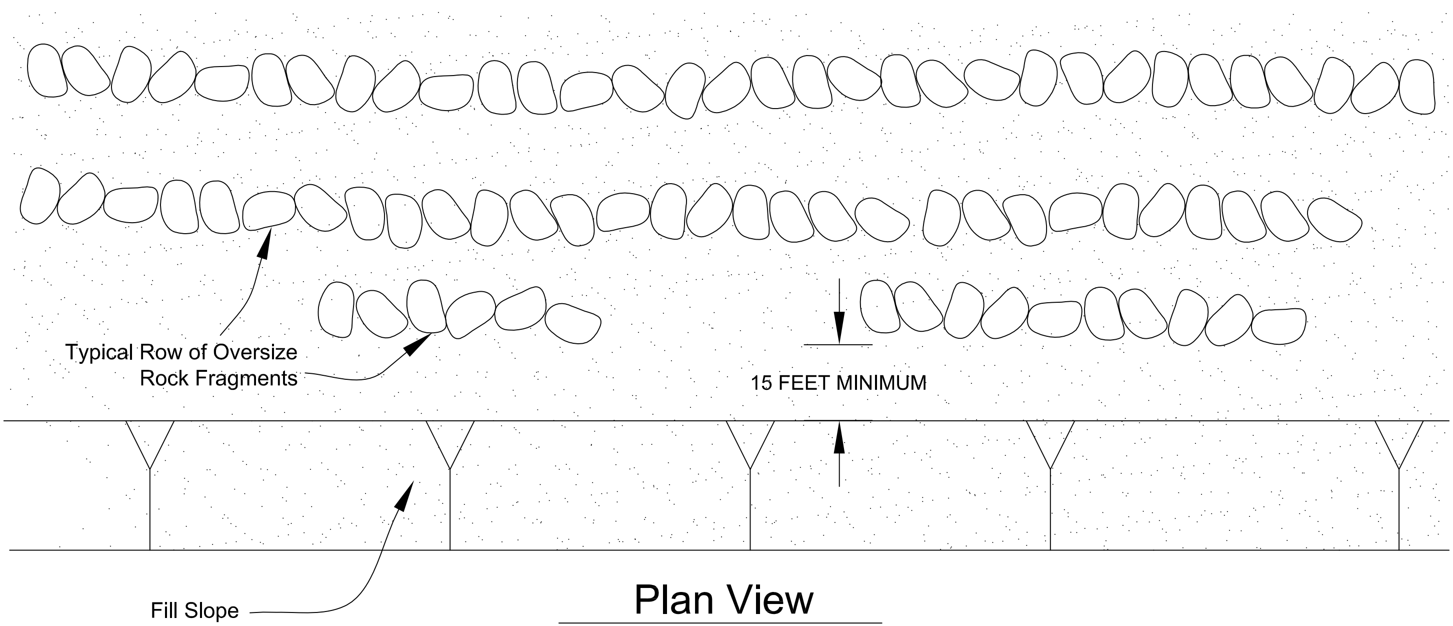
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



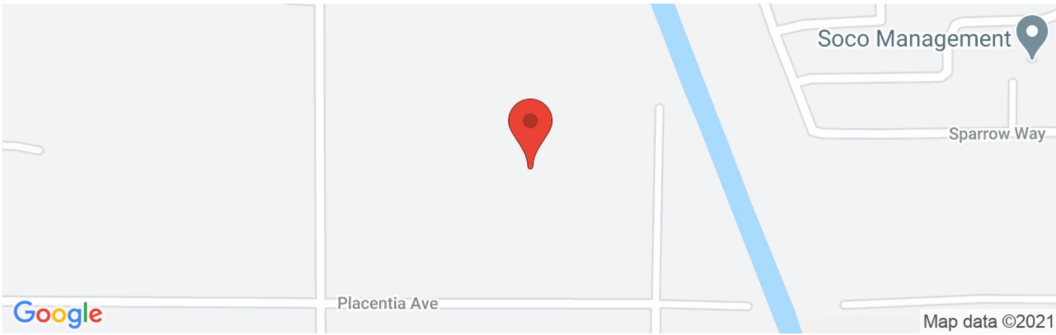
Plan View

PLACEMENT OF OVERSIZED MATERIAL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: PM CHKD: GKM	
PLATE D-8	

APPENDIX E

SEAOC | OSHPD

Latitude, Longitude: 33.824379, -117.210255



Date	4/9/2021, 2:16:54 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.572	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.5	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.55	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.525	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.636	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.572	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.627	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.932	Mapped value of the risk coefficient at short periods
C_{R1}	0.911	Mapped value of the risk coefficient at a period of 1 s

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

SEISMIC DESIGN PARAMETERS - 2019 CBC

PROPOSED WAREHOUSE

PERRIS, CALIFORNIA

DRAWN: MD
CHKD: RGT

SCG PROJECT
21G147-3

PLATE E-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Proposed Warehouse	MCE_G Design Acceleration	0.550 (g)
Project Location	Perris	Design Magnitude	6.97
Project Number	21G147-1	Historic High Depth to Groundwater	25 (ft)
Engineer	RF	Depth to Groundwater at Time of Drilling	29 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-1		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.97)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
12.5	0	25	12.5		120		1.3	1.05	1.1	1.31	0.75	0.0	0.0	1500	1500	1500	0.96	1.02	1.02	0.06	0.06	N/A	N/A	Above Water Table
29.5	25	32	28.5	23	120	66	1.3	1.05	1.3	0.86	0.95	33.5	39.1	3420	3202	3420	0.88	1.23	0.88	2.00	2.00	0.34	5.92	Non-Liquefiable
34.5	32	37	34.5	34	120		1.3	1.05	1.3	0.88	1	52.9	52.9	4140	3547	3797	0.85	1.23	0.85	2.00	2.00	0.36	5.63	Non-Liquefiable
39.5	37	42	39.5	30	120		1.3	1.05	1.3	0.84	1	44.5	44.5	4740	3835	4085	0.82	1.23	0.82	2.00	2.00	0.36	5.50	Non-Liquefiable
44.5	42	47	44.5	24	120	34	1.3	1.05	1.3	0.80	1	34.3	39.7	5340	4123	4373	0.80	1.23	0.8	2.00	1.96	0.37	5.33	Non-Liquefiable
49.5	47	50	48.5	32	120		1.3	1.05	1.3	0.82	1	46.4	46.4	5820	4354	4603	0.77	1.23	0.78	2.00	1.92	0.37	5.21	Non-Liquefiable

Notes:

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

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Warehouse
Project Location	Perris
Project Number	21G147-1
Engineer	RF

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60-CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
12.5	0	25	12.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	25.00		0.000		0.00	Above Water Table
29.5	25	32	28.5	33.5	5.6	39.1	5.92	0.01	-0.74	0.00	7.00		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	52.9	0.0	52.9	5.63	0.00	-1.82	0.00	5.00		0.000		0.00	Non-Liquefiable
39.5	37	42	39.5	44.5	0.0	44.5	5.50	0.00	-1.15	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	34.3	5.5	39.7	5.33	0.01	-0.78	0.00	5.00		0.000		0.00	Non-Liquefiable
49.5	47	50	48.5	46.4	0.0	46.4	5.21	0.00	-1.30	0.00	3.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				0.00	

Notes:

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

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

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

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

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

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

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

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

LIQUEFACTION EVALUATION

Project Name	Proposed Warehouse		MCE_G Design Acceleration	0.550 (g)
Project Location	Perris		Design Magnitude	6.97
Project Number	21G147-1		Historic High Depth to Groundwater	25 (ft)
Engineer	RF		Depth to Groundwater at Time of Drilling	29 (ft)
			Borehole Diameter	6 (in)
Boring No.	B-5			

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.97)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
12.5	0	25	12.5		120		1.3	1.05	1.1	1.31	0.75	0.0	0.0	1500	1500	1500	0.96	1.02	1.02	0.06	0.06	N/A	N/A	Above Water Table
29.5	25	32	28.5	33	120		1.3	1.05	1.3	0.89	0.95	49.5	49.5	3420	3202	3420	0.88	1.23	0.88	2.00	2.00	0.34	5.92	Non-Liquefiable
34.5	32	37	34.5	16	120	26	1.3	1.05	1.21	0.80	1	21.0	26.2	4140	3547	3797	0.85	1.15	0.91	0.32	0.33	0.36	0.94	**Liquefiable**
39.5	37	42	39.5	28	120	44	1.3	1.05	1.3	0.85	1	42.0	47.6	4740	3835	4085	0.82	1.23	0.82	2.00	2.00	0.36	5.50	Non-Liquefiable
44.5	42	47	44.5	39	120		1.3	1.05	1.3	0.87	1	60.4	60.4	5340	4123	4373	0.80	1.23	0.8	2.00	1.96	0.37	5.33	Non-Liquefiable
49.5	47	50	48.5	97	120		1.3	1.05	1.3	1.35	1	232.6	232.6	5820	4354	4603	0.77	1.23	0.78	2.00	1.92	0.37	5.21	Non-Liquefiable

Notes:

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

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Warehouse
Project Location	Perris
Project Number	21G147-1
Engineer	RF

Boring No. B-5

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
12.5	0	25	12.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	25.00		0.000		0.00	Above Water Table
29.5	25	32	28.5	49.5	0.0	49.5	5.92	0.00	-1.55	0.00	7.00		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	21.0	5.1	26.2	0.94	0.08	0.16	0.04	5.00		0.009		0.54	**Liquefiable**
39.5	37	42	39.5	42.0	5.6	47.6	5.50	0.00	-1.40	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	60.4	0.0	60.4	5.33	0.00	-2.46	0.00	5.00		0.000		0.00	Non-Liquefiable
49.5	47	50	48.5	232.6	0.0	232.6	5.21	0.00	-19.68	0.00	3.00		0.000		0.00	Non-Liquefiable

Total Deformation (in) 0.54

Notes:

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

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

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

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

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

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

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

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

LIQUEFACTION EVALUATION

Project Name	Proposed Warehouse	MCE_G Design Acceleration	0.550 (g)
Project Location	Perris	Design Magnitude	6.97
Project Number	21G147-3	Historic High Depth to Groundwater	25 (ft)
Engineer	PM	Depth to Groundwater at Time of Drilling	30 (ft)
		Borehole Diameter	6 (in)

Boring No. B-9

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.97)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
12.5	0	25	12.5		120		1.3	1.05	1.1	1.31	0.75	0.0	0.0	1500	1500	1500	0.96	1.02	1.02	0.06	0.06	N/A	N/A	Above Water Table
24.5	25	27	26	47	120	35	1.3	1.05	1.3	0.97	0.95	76.6	82.1	3120	3058	3120	0.90	1.23	0.89	2.00	2.00	0.33	6.11	Non-Liquefiable
29.5	27	32	29.5	16	120	47	1.3	1.05	1.204	0.82	0.95	20.4	26.0	3540	3259	3540	0.88	1.15	0.93	0.32	0.34	0.34	0.98	**Liquefiable**
34.5	32	37	34.5	17	120	44	1.3	1.05	1.227	0.80	1	22.7	28.3	4140	3547	3859	0.85	1.17	0.9	0.40	0.42	0.36	1.18	**Liquefiable**
39.5	37	42	39.5	31	120	20	1.3	1.05	1.3	0.86	1	47.0	51.5	4740	3835	4147	0.82	1.23	0.82	2.00	2.00	0.36	5.50	Non-Liquefiable
44.5	42	47	44.5	51	120	17	1.3	1.05	1.3	0.96	1	87.1	91.0	5340	4123	4435	0.80	1.23	0.8	2.00	1.96	0.37	5.33	Non-Liquefiable
49.5	47	50	48.5	34	120	40	1.3	1.05	1.3	0.85	1	51.3	56.9	5820	4354	4666	0.77	1.23	0.78	2.00	1.92	0.37	5.21	Non-Liquefiable

Notes:

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

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Warehouse
Project Location	Perris
Project Number	21G147-3
Engineer	PM

Boring No. B-9

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
12.5	0	25	12.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	25.00		0.000		0.00	Above Water Table
24.5	25	27	26	76.6	5.5	82.1	6.11	0.00	-4.39	0.00	2.00		0.000		0.00	Non-Liquefiable
29.5	27	32	29.5	20.4	5.6	26.0	0.98	0.08	0.17	0.04	5.00		0.008		0.50	**Liquefiable**
34.5	32	37	34.5	22.7	5.6	28.3	1.18	0.06	0.02	0.02	5.00		0.005		0.31	**Liquefiable**
39.5	37	42	39.5	47.0	4.5	51.5	5.50	0.00	-1.71	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	87.1	3.9	91.0	5.33	0.00	-5.21	0.00	5.00		0.000		0.00	Non-Liquefiable
49.5	47	50	48.5	51.3	5.6	56.9	5.21	0.00	-2.16	0.00	3.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				0.80	

Notes:

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

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

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

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

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

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

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

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

APPENDIX G

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 21G147-1
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/26/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: 29 feet
CAVE DEPTH: 33 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	14			ALLUVIUM: Brown fine Sandy Silt, little Clay, trace fine root fibers, slightly porous, medium dense-damp		9					
	X	34	4.5		Gray Brown Silty Clay, abundant calcareous nodules and veining, very stiff to hard-very moist		26					
5												
	X	26	4.5		@ 6 feet, trace fine Sand		18					
	X	32	4.5		Red Brown fine Sandy Clay, trace medium Sand, little to some iron oxide staining, hard-damp to moist		10					
10												
					Red Brown Silty fine to medium Sand, trace Clay, medium dense to very dense-damp to moist							
	X	16					6					
15												
	X	54					8					
20												
	X	30					5					
25												
					Brown fine Sandy Silt, trace Clay, little iron oxide staining, medium dense-very moist to wet							
	X	23			(water level)		16			66		
30												
					Brown Silty fine to medium Sand, trace coarse Sand, trace fine Gravel, dense-wet							
	X	34					10					

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 21G147-1	DRILLING DATE: 3/26/21	WATER DEPTH: 29 feet
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 33 feet
LOCATION: Perris, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Brown Silty fine to medium Sand, trace coarse Sand, trace fine Gravel, dense-wet							
					Brown Silty fine to coarse Sand, dense-wet							
	X	30			@ 38½ feet, Brown fine Sandy Clay lense		12					
40												
					Red Brown, Clayey fine to medium Sand, medium dense to dense-wet							
	X	24	4.5				11			34		
45												
	X	32	2.5				15					
50												
					Boring Terminated at 50'							

TBL 21G147-1.GPJ SOCALGEO.GDT 4/16/21

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 21G147-1
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/26/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ALLUVIUM: Gray Brown, Silty Clay, little fine Sand, very stiff-damp to moist							
	X	28	3.0			102	12					EI = 56 0 to 5 feet
	X	79/10"	4.5		@ 3 feet, trace medium to coarse Sand, moderately cemented, some calcareous nodules and veining, hard	110	10					
5	X	50/5"			Light Red Brown Silty fine Sand, little Clay, trace medium to coarse Sand, slightly cemented, trace calcareous nodules and veining, very dense-damp		4					Disturbed
	X	93/11"	4.5		Light Red Brown fine Sandy Clay, little to some Silt, trace calcareous nodueles and veining, hard-damp	126	8					
10	X	99/11"	4.5			129	9					
					Red Brown Silty Clay, trace fine to coarse Sand, hard-moist							
15	X	81/11"	4.5			120	11					
					Red Brown Clayey fine to medium Sand, trace fine Gravel, slightly cemented, very dense-damp to moist							
20	X	88/10"					8					

Boring Terminated at 20'

TBL 21G147-1.GPJ SOCALGEO.GDT 4/16/21

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 21G147-1
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/26/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	28			ALLUVIUM: Gray Brown fine Sandy Silt, little to some calcareous nodules and veining, medium dense-moist	100	13					
	X	42	4.5		Gray Brown Silty Clay, little fine Sand, some calcareous nodules and veining, very stiff-moist	112	13					
5	X	84/11"			Light Red Brown Silty fine to coarse Sand, trace to little Clay, little calcareous nodules and veining, very dense-moist	118	9					
	X	37	4.5		Light Red Brown Silty Clay, little fine Sand, little calcareous nodules and veining, very stiff-damp	105	9					
10	X	71			Light Red Brown Silty fine Sand, trace Clay, dense-damp	121	7					
15	X	39			Brown Clayey fine to medium Sand, some Silt, dense-damp		8					

Boring Terminated at 15'

TBL 21G147-1.GPJ SOCALGEO.GDT 4/16/21

TEST BORING LOG **PLATE B-3**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 21G147-1
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/26/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 15 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	16		ALLUVIUM: Dark Gray Brown Silty fine Sand, little Clay, little to some calcareous nodules and veining, medium dense-damp to moist	98	8					
	X	87/11"	4.5	Brown Clayey Silt, little fine Sand, some calcareous nodules and veining, hard-damp	115	12					
5	X	81/10"		Gray Brown Silty fine Sand, little Clay, abundant to extensive calcareous nodules and veining, very dense-damp	104	9					
	X	36	4.5	Brown fine Sandy Clay, little Silt, little calcareous nodules and veining, very stiff-damp	120	9					
	X	41		Gray Brown Silty fine Sand, little Clay, medium dense-moist	104	9					
10				Gray Brown Clayey fine Sand, trace medium to coarse Sand, medium dense-damp							
	X	40		Gray Brown Clayey fine to medium Sand, trace coarse Sand, dense-moist		13					
15											
	X	65	4.0	Red Brown fine Sandy Clay, little Silt, hard-damp		12					
20				Boring Terminated at 20'							

TBL 21G147-1.GPJ SOCALGEO.GDT 4/16/21

SOUTHERN CALIFORNIA GEOTECHNICAL
SoCalGeo
A California Corporation

BORING NO. B-5

JOB NO.: 21G147-1
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/26/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: 27 feet
CAVE DEPTH: 29 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
0				ALLUVIUM: Gray Brown Silty Clay, little fine Sand, little calcareous noduels and veining, very stiff to hard-damp							
	X	24	2.5			10					
	X	32				4					
5				Light Red Brown fine Sandy Clay, little Silt, moderatly cemented, little calcareous nodules and veining, hard-damp to moist							
	X	68	4.5			7					
	X	44	4.5			13					
10											
				Brown Clayey fine Sand, little Silt, trace calcareous nodules and veining, dense-damp							
	X	34				7					
15											
				Red Brown fine Sandy Clay, some Silt, trace calcareous nodules and veining, hard-damp							
	X	84/11"	4.5			8					
20											
				Red Brown Clayey fine Sand, little Silt, little medium Sand, trace calcareous nodules and veining, little iron oxide staining, medium dense-damp							
	X	26				7			22		
25											
	X	33		@ 28½ feet, wet		11					
30											
				Red Brown Clayey fine to medium Sand, trace Silt, trace coarse Sand, little iron oxide staining, medium dense-wet							
	X	16				12			26		

TBL 21G147-1.GPJ SOCALGEO.GDT 4/16/21

TEST BORING LOG

PLATE B-5a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 21G147-1
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/26/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: 27 feet
CAVE DEPTH: 29 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Red Brown Clayey fine to medium Sand, trace Silt, trace coarse Sand, little iron oxide staining, medium dense-wet							
	X	28			@ 38½ feet, little calcareous nodules and veining		20			44		
40												
					Gray Brown fine to coarse Sand, trace Silt, dense-wet							
	X	39					14					
45					Red Brown Silty fine Sand, trace calcareous nodules and veining, dense-wet		14					
					Gray Brown fine to coarse Sand, trace Silt, very dense-wet							
	X	81/10"					29					
50												

Boring Terminated at 50'

TBL 21G147-1.GPJ SOCALGEO.GDT 4/16/21

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 21G147-1
PROJECT: Proposed Warehouse
LOCATION: Perris, California

DRILLING DATE: 3/26/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 8 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ALLUVIUM: Brown Silty fine Sand, little calcareous nodules and veining, medium dense-moist							
	X	18					10					
					Brown Clayey fine Sand, some Silt, very dense-damp							
	X	50					7					
5												
					Gray Brown fine Sandy Clay, little Silt, trace calcareous nodules and veining, dense to very dense-damp							
	X	66	4.5				8					
	X	45	4.5				8					
10												
					Boring Terminated at 10'							

TBL 21G147-1.GPJ SOCALGEO.GDT 4/16/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-7

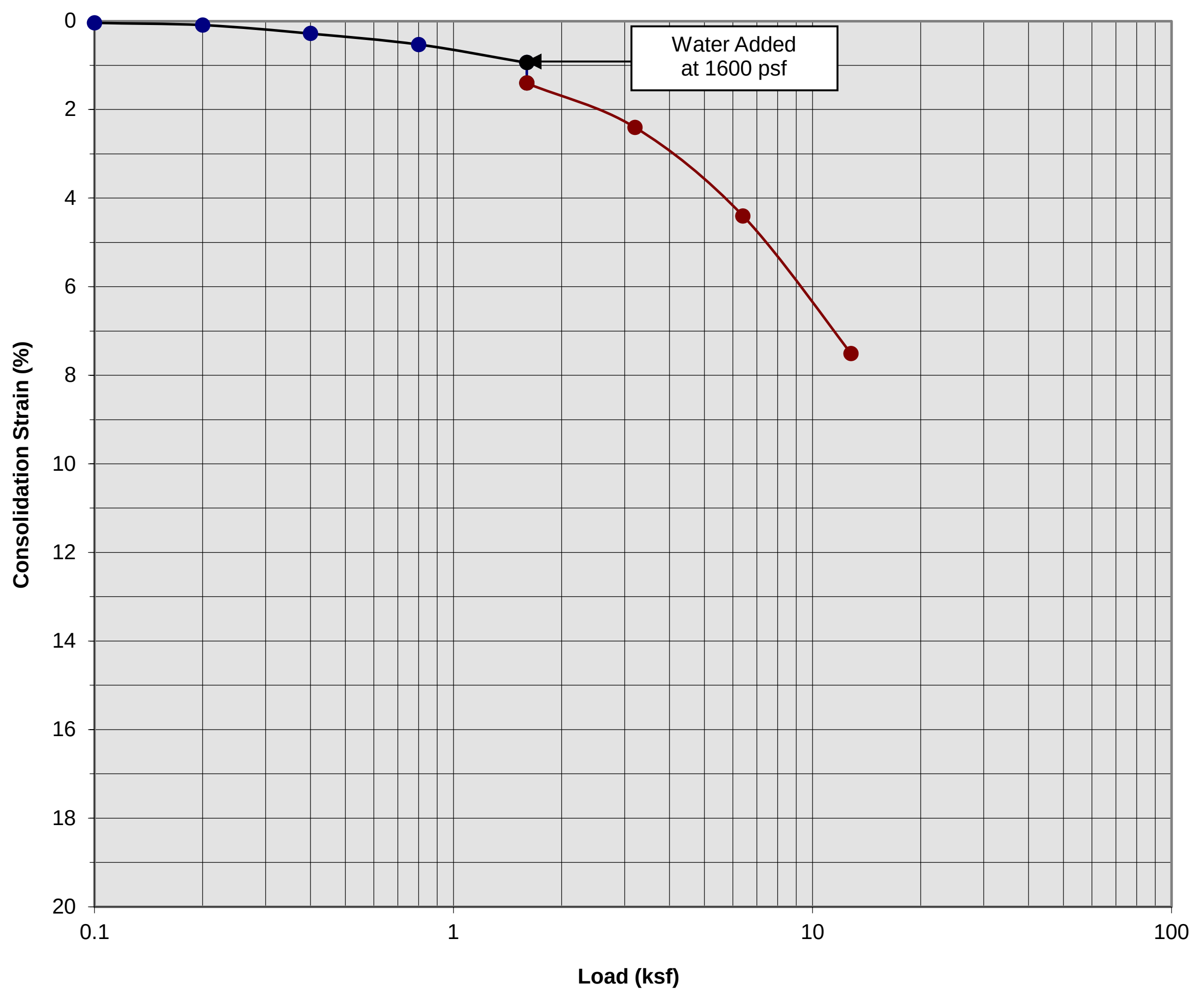
JOB NO.: 21G147-1
PROJECT: Proposed Warehouse
LOCATION: Perris, California
DRILLING DATE: 3/26/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward
WATER DEPTH: Dry
CAVE DEPTH: --
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	17	4.5		ALLUVIUM: Dark Gray Brown Silty Clay, little fine to coarse Sand, little calcareous nodules and veining, slightly cemented, very stiff-damp		11					
5	X	17			Light Brown fine to coarse Sand, trace Silt, medium dense-dry to damp		2					
	X	22			Gray Brown Silty fine Sand, little Clay, little calcareous nodules and veining, little iron oxide staining, medium dense-moist		10					
10	X	40			Red Brown Clayey fine Sand, little iron oxide staining, trace calcareous nodules and veining, dense-damp to moist		8					

Boring Terminated at 10'

TBL 21G147-1.GPJ SOCALGEO.GDT 4/16/21

Consolidation/Collapse Test Results



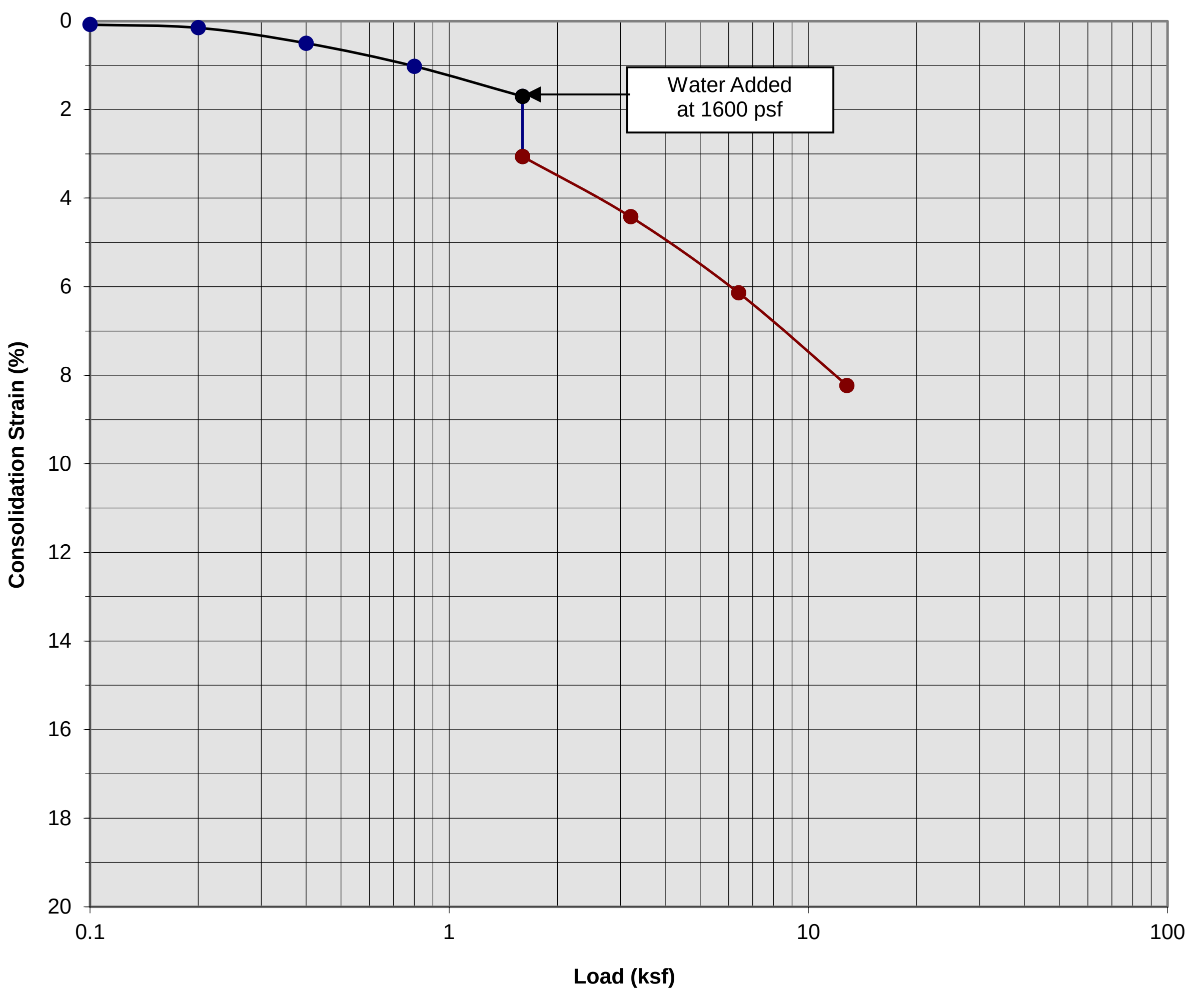
Classification: Gray Brown Silty Clay, little fine Sand

Boring Number:	B-3	Initial Moisture Content (%)	13
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	3 to 4	Initial Dry Density (pcf)	112.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	121.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.46

Proposed Warehouse
Perris, CA
Project No. 21G147-1
PLATE C- 1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



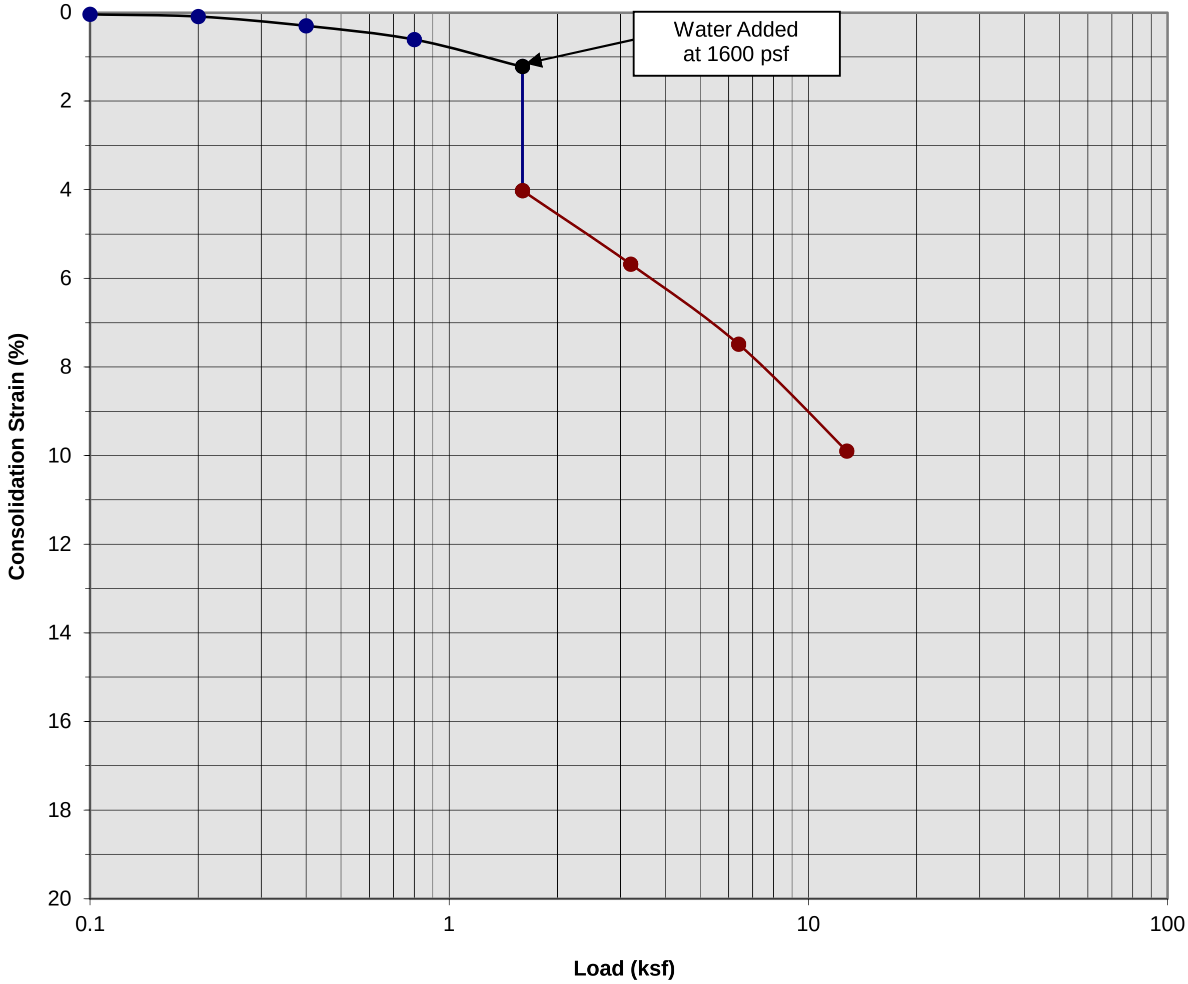
Classification: Light Red Brown Silty fine to coarse Sand, trace to little Clay

Boring Number:	B-3	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	5 to 6	Initial Dry Density (pcf)	118.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	128.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.36

Proposed Warehouse
Perris, CA
Project No. 21G147-1
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

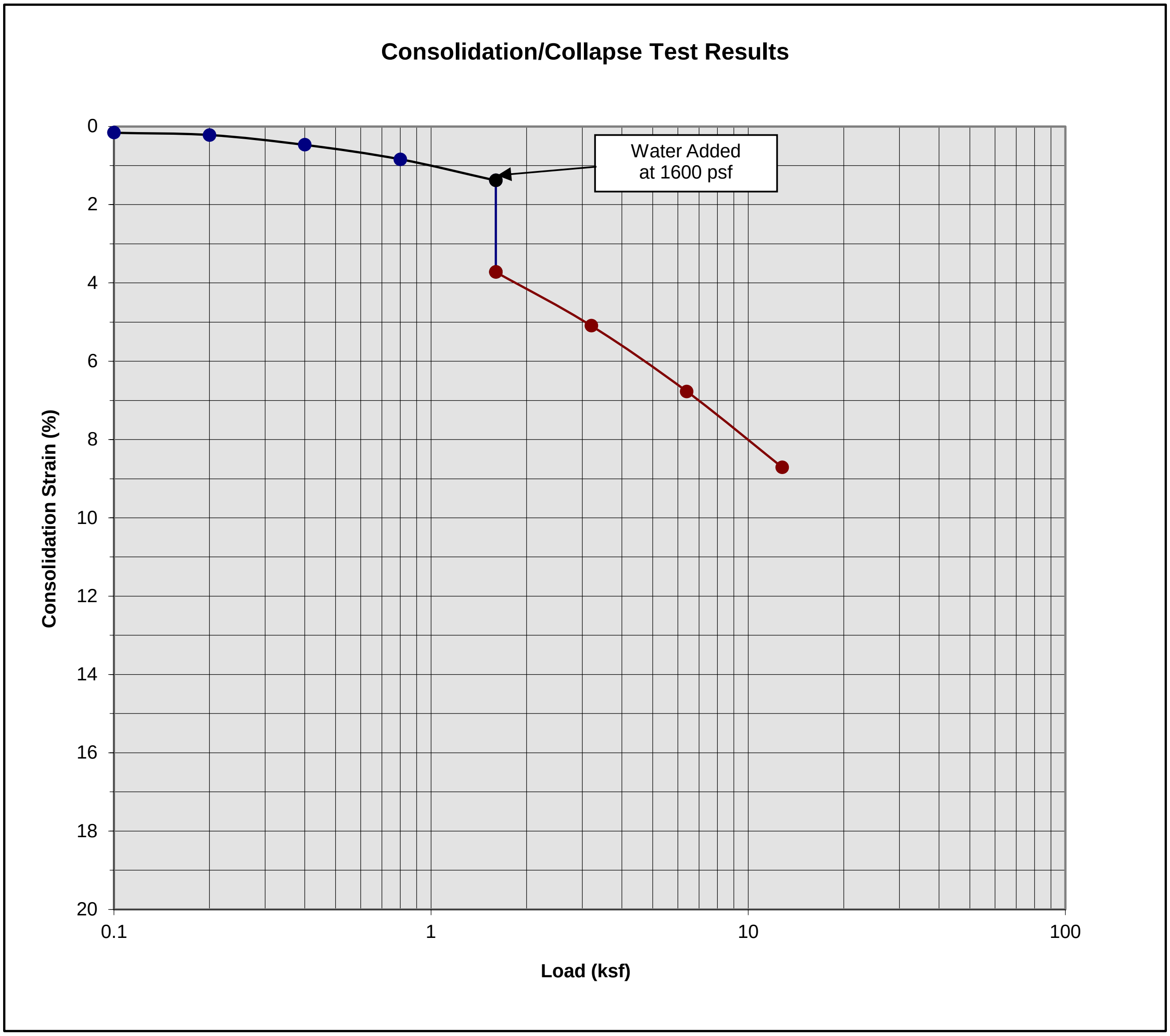


Classification: Light Red Brown Silty Clay, little fine Sand

Boring Number:	B-3	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	7 to 8	Initial Dry Density (pcf)	105.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	116.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.80

Proposed Warehouse
Perris, CA
Project No. 21G147-1
PLATE C- 3

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

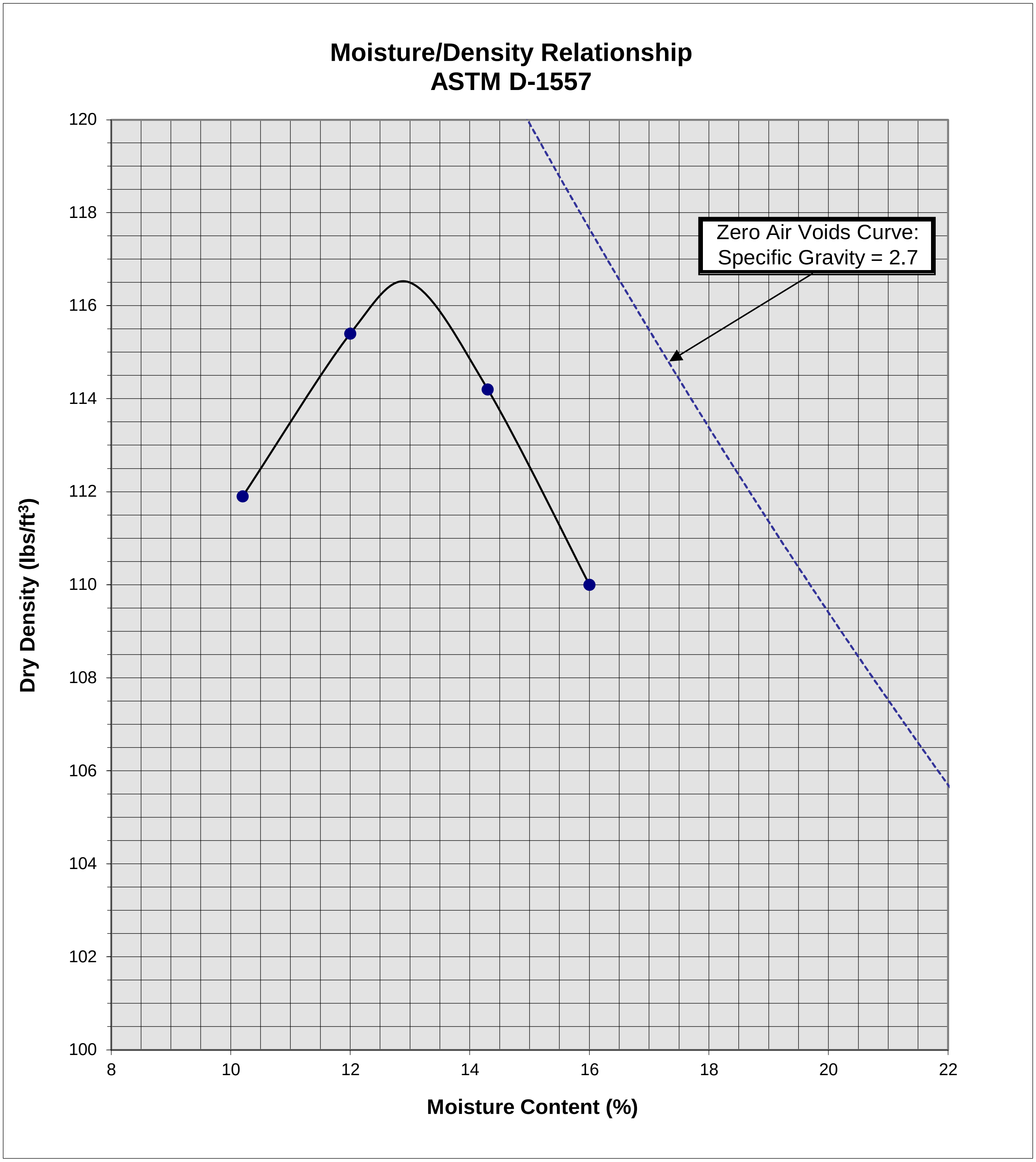


Classification: Light Red Brown Silty fine Sand, trace Clay

Boring Number:	B-3	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	9 to 10	Initial Dry Density (pcf)	121.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	132.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.34

Proposed Warehouse
Perris, CA
Project No. 21G147-1
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-3 @ 0-5'
Optimum Moisture (%)	13
Maximum Dry Density (pcf)	116.5
Soil Classification	Gray Brown Silty Clay, some fine Sand

Proposed Warehouse
Perris, California
Project No. 21G147-1
PLATE C-5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Appendix 4: Historical Site Conditions

Phase I Environmental Site Assessment or Other Information on Past Site Use

Not included.