GEOTECHNICAL INVESTIGATION
PROPOSED WAREHOUSE

13131 Los Angeles Street
Irwindale, California
for
Duke Realty

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

November 19, 2018

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Duke Realty
200 Spectrum Center Drive, Suite 1600
Irvine, California 92618

Attention: Mr. Michael Weber
Assistant Development Services Manager

Project No.: **18M192-1**

Subject: **Geotechnical Investigation**
Proposed Warehouse
13131 Los Angeles Street
Irwindale, California

Gentlemen:

In accordance with your request, we have performed 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.

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1.0 EXECUTIVE SUMMARY

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

Site Preparation

- Demolition of the existing structures and pavements will be necessary in order to facilitate the construction of the proposed development. Demolition should include all foundations, floor slabs, utilities and any other subsurface improvements that will not remain in place with the new development. Debris resultant from demolition should be disposed of offsite. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2 inch particle size, well mixed with the on-site soils, and incorporated into new structural fills or it may be crushed and made into CMB, if desired.
- Initial site stripping should include the removal of any surficial vegetation. Based on conditions encountered at the time of the subsurface exploration, stripping of a few trees and some vegetation will be necessary along the perimeter of the site. Site stripping should remove any tree root masses in their entirety. These materials should be disposed of offsite.
- The near surface soils encountered at the boring and trench locations generally comprise dense to very dense native alluvial soils consisting of well graded sands and sandy gravels with significant cobble and boulder content.
- Remedial grading is recommended within the proposed building area, in order to provide uniform support conditions for the new foundations and the floor slab of the proposed structure and to remove any soils disturbed during demolition. We recommend that the proposed building pad area be overexcavated to a depth of at least 3 feet below existing grade and to a depth of at least 3 feet below proposed pad grade. The overexcavation should also extend to a sufficient depth to remove all of the artificial fill materials within the building pad area. Overexcavation within the foundation areas is recommended to extend to a depth of at least 2 feet below proposed foundation bearing grade.
- As discussed above, the native alluvial soils possess significant amounts of oversized materials, including cobbles and boulders. Where grading will require excavation into these materials, consideration should be given to using selective grading techniques to remove the cobbles and/or boulders from these soils prior to reuse as fill. Recommendations regarding selective grading and handling of oversized materials are provided in Section 6.3 and Appendix D of this report.
- After overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated. The resulting soils should be scarified and thoroughly flooded to achieve a moisture content of 0 to 4 percent above optimum moisture, to a depth of at least 24 inches. The overexcavation subgrade soils should then be recompacted under the observation of the geotechnical engineer. The previously excavated soils may then be replaced as structural fill, compacted to 90 percent of the ASTM D-1557 maximum dry density.
- The new parking area subgrade soils are recommended to be scarified to a depth of 12± inches, moisture conditioned to 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Building Foundations

- Spread footing foundations, supported in newly placed structural fill soils.
- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2.
- Reinforcement consisting of at least two (2) No. 5 rebars (1 top and 1 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

Building Floor Slabs

- Conventional Slab-on-Grade, at least 6 inches thick.
- Modulus of Subgrade Reaction: k = 200 psi/in.
- Reinforcement is not expected to be necessary for geotechnical considerations.
- The actual thickness and reinforcement of the floor slabs should be determined by the structural engineer.

Pavements

ASPHALT PAVEMENTS (R = 60)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	3	3	3	3	4
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 5.0 & 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	6	7	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 18P442, dated November 2, 2018. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located on the north side of Los Angeles Street, at the street address of 13131 Los Angeles Street in Irwindale, California. The site is bounded to the north by Rivergrade Road, to the east and west by commercial/industrial developments, and to the south by Los Angeles Street. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in Appendix A of this report.

The site is an irregular-shaped parcel, approximately 24.9 acres in size. The site was previously operated by Hanson Structural Precast, which is no longer operational. The site is developed with two (2) industrial buildings, which possess footprint areas of 3,800 and 9,768± ft² in size, located in the southern area of the site. One of the buildings, in the southwestern area of the site is a two-story structure of brick and mortar construction. The second building, located in the south-central area of the site, is of concrete tilt-up construction with two (2) external canopy structures located on the north and east sides of the building. All of these structures are assumed to be supported on conventional shallow foundations and we assume that the buildings possess concrete slab-on-grade floors.

Temporary structures, including three office trailers, are present in the southwestern area of the site, and several small wooden shacks are also present in various locations throughout the site.

Cranes on rails, hoppers, conveyors, and various other manufacturing equipment are present in the central area of the site. A below grade hopper, approximately 15 feet deep, is located in the central area of the site.

The ground surface cover throughout the majority of the site generally consists of crushed aggregate base. Ground surface cover surrounding the buildings in the southern area of the site consists of asphaltic concrete pavements. The pavements are in poor condition with areas of moderate to severe cracking throughout. Portland cement concrete (PCC) casting beds and slabs are also present throughout the site. Ground surface cover in the remainder of the site consists of sparse native grass and weed growth along the property lines and in the northeastern corner of the site. There are a several trees located near the existing buildings, along the eastern property line, and in the north-central area of the site.

Topographic information for the site was obtained from an ALTA survey provided by Thienes Engineering. This survey provides limited topographic information using spot elevations, which are mostly concentrated along the property lines. Based on this survey information, the site topography ranges in elevations from 363± feet mean sea level (msl) in the northern area of the site to a minimum elevation of 343± feet msl in along the southern property line. The site topography appears to slope gently downward toward the south at a gradient of approximately 1± percent.

3.2 Proposed Development

Based on the site plan prepared by RGA, the site will be developed with one (1) new warehouse. The warehouse will be 528,710± ft^2 in size located in the south-central area of the site. Loading docks will be constructed along the north and western sides of the building. The building will be surrounded by asphaltic concrete pavements in the automobile parking and drive areas, and Portland cement concrete pavements in the loading dock and truck traffic areas. Areas of landscaped planters and concrete flatwork are also expected throughout the site. All of the existing buildings and manufacturing equipment will be demolished to facilitate the new construction.

Detailed structural information is not currently available. It is assumed that the new building will be of concrete tilt-up construction, typically supported on conventional shallow foundation systems, with a slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 80 to 100 kips and 3 to 7 kips per linear foot, respectively.

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

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of three (3) borings advanced to a depth of 50± feet below existing site grades. In addition, a total of fifteen (15) trenches were excavated at the site to depths of 5 to 10± feet below existing site grades. In addition to the borings and trenches, the existing concrete slabs were cored in eight (8) different locations in order to determine the thickness of the existing slab section. All of the borings and trenches were logged during drilling and excavation by a member of our staff. The field investigation was performed on April 30 2018, as indicated in the boring and trench logs included in Appendix B of this report.

The borings were advanced with large truck mounted Becker Hammer drilling rig. The trenches were excavated using a rubber tire backhoe with a 36-inch-wide bucket. Representative bulk and soil samples were taken during drilling and excavation. Standard Penetration Test (SPT)samples were taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. This sampler was 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 approximate locations of the borings and trenches are indicated on the Exploration Location Plan, included as Plate 2 in Appendix A of this report. The Boring and Trench Logs, which illustrate the conditions encountered at the boring and trench locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Pavements and Ground Surface Cover

Asphaltic concrete pavements were encountered at the ground surface at Trench No. T-1. At this location, the pavement section consists of 2± inches of asphaltic concrete with no discernable layer of aggregate base.

Boring No. B-1 encountered a surficial layer of pea gravel at the ground surface approximately 1 inch thick, underlain by 6± inches of crushed aggregate base.

Boring Nos. B-2 and B-3 and all of the Trenches, except Trench No. T-1 and T-8, were drilled/excavated in areas covered with a layer of aggregate base. The base materials encountered at the boring and trench locations possesses very high strengths and appears to consist of cement treated base. At these locations, the base layer measures 8 to 18± inches thick.

Alluvium

Native alluvial soils were encountered at the ground surface at Trench No. T-8, and beneath the pavement or aggregate base at all of the boring and trench locations, extending to the maximum depth explored of 50± feet. The alluvium generally consists of dense to very dense gravelly fine to coarse sands and fine to coarse sandy gravels and fine to medium sands with occasional to extensive cobbles and occasional boulders. Boring No. B-2 encountered a layer of silty fine sand with trace amounts of medium to coarse sands at depths of 27 to 29½± feet below existing site grades.

Groundwater

Free water was not encountered during excavation of any of the trenches or drilling of the borings. Based on the lack of any water within the trenches or borings, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 50± feet at the time of the subsurface exploration.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the historic groundwater depths in this area is CGS Open File Report 98-13, the Seismic Hazard Evaluation of the Baldwin Park Quadrangle which indicates that the historic high groundwater level for the site is 35 feet below the ground surface.

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 Trench Logs and are periodically referenced throughout this report.

Moisture Content

The moisture content has been determined for selected representative samples. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring and Trench Logs.

Maximum Dry Density and Optimum Moisture Content

Representative bulk samples have been tested for their maximum dry densities and optimum moisture contents. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plates C-1 and C-2 in Appendix C of this report. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Soluble Sulfates

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

Sample Identification	Soluble Sulfates (%)	Severity
B-1 @ 0 to 5 feet	0.004	Not Applicable (S0)
B-3 @ 0 to 5 feet	0.006	Not Applicable (S0)

Corrosivity Testing

Representative bulk samples of the near-surface soils were submitted to a subcontracted analytical laboratory for determination of electrical resistivity, pH, and chloride concentrations.

The resistivity of the soils is a measure of their potential to attack buried metal improvements such as utility lines. The results of the resistivity and pH testing are presented below:

Sample Identification	**Resistivity** (ohm-cm)	**pH**	**Chlorides** (mg/kg)
B-1 @ 0 to 5 feet	6,000	8.3	16
B-3 @ 0 to 5 feet	6,800	8.1	12

Direct Shear

Direct shear testing was performed on one selected soil sample to determine its shear strength parameters. This test was performed in accordance with ASTM D-3080. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Three samples of the same soil are prepared by remolding them to 90± percent compaction and near optimum moisture. Each of the three samples are then loaded with different normal loads and the resulting shear strength is determined for that particular normal load. The shearing of the samples is performed at a rate slow enough to permit the dissipation of excess pore water pressure. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The results of the direct shear tests are presented on Plate C-3.

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

6.1 Seismic Design Considerations

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

Faulting and Seismicity

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

Seismic Design Parameters

The 2016 California Building Code (CBC) was adopted by all municipalities within Southern California on January 1, 2017. The 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.

The 2016 CBC Seismic Design Parameters have been generated using U.S. Seismic Design Maps, a web-based software application developed by the United States Geological Survey. This software application, available at the USGS web site, calculates seismic design parameters in accordance with the 2016 CBC, utilizing a database of deterministic site accelerations at 0.01 degree intervals. The table below is a compilation of the data provided by the USGS application. A copy of the output generated from this program is included in Appendix E of this report. A copy of the Design Response Spectrum, as generated by the USGS application is also included in Appendix E. Based on this output, the following parameters may be utilized for the subject site:

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter		**Value**
Mapped Spectral Acceleration at 0.2 sec Period	S_S	2.239
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.745
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	2.239
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.118
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.493
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.745

Ground Motion Parameters

For the liquefaction evaluation, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2016 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-10. 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 7-10. The web-based software application U.S. Seismic Design Maps (described in the previous section) was used to determine PGA_M, which is 0.787g. A portion of the program output is included as Plate E-2 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 7.71, based on the peak ground acceleration and soil classification D.

Liquefaction

Research of the Seismic Hazards Zones Map for the Baldwin Park Quadrangle, published by the California Geological Survey (CGS) indicates that the site subject site is located within a liquefaction hazard zone. Based on this mapping, 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). 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 No. B-1, which was advanced to a depth of 50± feet. The liquefaction potential was analyzed at the boring location utilizing a PGA_M of 0.787g related to a 7.74 magnitude seismic event. The liquefaction evaluation was performed using the reported historic high groundwater depth of 35 feet.

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

Conclusions and Recommendations

The liquefaction evaluation indicates that none of the soils below the historic high groundwater table are subject to liquefaction during the design seismic event. Based on the results of this analysis, no design considerations related to liquefaction are considered warranted for this project.

6.2 Geotechnical Design Considerations

General

The site is generally underlain by dense to very dense well-graded sands and sandy gravels. The soils encountered at the boring and trench locations generally possess significant over-sized material including extensive cobble content and occasional boulders throughout the depths explored. Some remedial grading is considered warranted in order within the proposed building area to provide uniform support characteristics beneath the proposed slabs and foundations, and to help facilitate construction activities by removing some of the over-sized materials.

Demolition of the existing pavements and structures is also expected to cause significant disturbance to the near surface soils. Any soils disturbed during demolition should also be removed prior to the placement of structural fill soils. The excavated soils may be moisture conditioned and recompacted as structural fill.

Los Angeles County Section 111 Statement

Based on the results of our geotechnical analysis, the proposed development will be safe with regard to landslides, settlement and/or slippage. In addition, the proposed development will not adversely affect the geologic stability of the adjacent properties. This finding is in accordance with Section 111 of the Los Angeles County Building Code.

Settlement

The native alluvium is dense to very dense and possesses relatively high strengths. The near surface 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.

Corrosion Potential

The results of the electrical resistivity and pH testing indicate that a sample of the on-site soils has resistivity values ranging from 6,000 to 6,800 ohm-cm, and pH values ranging from 8.1 to 8.3. 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 not considered to be corrosive to ductile iron pipe.

Therefore, polyethylene protection is expected to be required for cast iron or ductile iron pipes. It should be noted that SCG does not practice in the field of corrosion engineering, and therefore, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.

Expansion

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

Soluble Sulfates

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

Shrinkage/Subsidence

Removal and recompaction of the near surface fill soils is estimated to result in an average shrinkage of 0 to 8 percent. It should be noted that the potential shrinkage estimate is based on correlated density relationships using hammer blow counts recorded during sampling 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. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

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

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

Grading and Foundation Plan Review

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

6.3 Site Grading Recommendations

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

Site Stripping and Demolition

Demolition of the existing structures and pavements will be necessary in order to facilitate the construction of the proposed development. Demolition should include all foundations, floor slabs, equipment such as the below grade hopper and above grade cranes, utilities and any other subsurface improvements that will not remain in place with the new development. Debris resultant from demolition should be disposed of offsite. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2 inch particle size, well mixed with the on-site soils, and incorporated into new structural fills or it may be crushed and made into CMB, if desired.

Initial site stripping should include removal of any surficial vegetation. Based on conditions encountered at the time of the subsurface exploration, stripping of some trees will be necessary along the perimeter of the site. Site stripping should remove any root masses in their entirety. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building pad area in order to remove a portion of the near-surface alluvium, and all soils disturbed during demolition. Based on conditions encountered at the trench locations, the existing soils within the proposed building area are recommended to be overexcavated to a depth of at least 3 feet below existing grade and to a depth of at least 3 feet below proposed building pad subgrade elevations, whichever is greater. 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 at least 2 foot below proposed bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building perimeter, and to an extent equal to the depth of fill below the new foundations. If the proposed structure will incorporate 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 structure. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if loose, porous, or low density native soils are encountered at the base of the overexcavation.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, and thoroughly flooded to raise the moisture content of the underlying soils to at least 0 to 4 percent above optimum moisture content, extending to a depth of at least 24 inches. The moisture conditioning of the overexcavation subgrade soils should be verified by the geotechnical engineer. The subgrade soils should then be 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.

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

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

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

The grading recommendations presented above for the proposed parking area 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 the existing fill 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 removed soils replaced as compacted structural fill.

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

The existing soils within the areas of any proposed retaining and site walls should be overexcavated to a depth of 2 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 subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils, as discussed for the building area. The previously excavated soils may then be replaced as compacted structural fill.

<u>Fill Placement</u>

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 0 to 4 percent above the optimum moisture content, and compacted.

- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2016 CBC and the grading code of the city of Irwindale and county of Los Angeles.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Selective Grading and Oversized Material Placement

The native alluvial soils possess significant cobble and/or boulder content. It is expected that large scrapers (Caterpillar 657 or equivalent) will be adequate to move the cobble containing soils as well as some of the soils containing smaller boulders. It may be necessary to move such larger boulders individually, and place them as oversized materials in accordance with the Grading Guide Specifications, in Appendix D of this report.

Since the proposed grading will require excavation of cobble and boulder containing soils, it may be desirable to selectively grade the proposed building pad area. The presence of particles greater than 3 inches in diameter within the upper 1 to 3 feet of the building pad subgrade will impact the utility and foundation excavations. Depending on the depths of fills required within the proposed parking areas, it may be feasible to sort the on-site soils, placing the materials greater than 3 inches in diameter within the lower depths of the fills, and limiting the upper 1 to 3 feet of soils to materials less than 3 inches in size. Oversized materials could also be placed within the lower depths of the recommended overexcavations. In order to achieve this grading, it would likely be necessary to use rock buckets and/or rock sieves to separate the oversized materials from the remaining soil. Although such selective grading will facilitate further construction activities, it is not considered mandatory and a suitable subgrade could be achieved without such extensive sorting. However, in any case, it is recommended that all materials greater than 6 inches in size be excluded from the upper 1 foot of the surface of any compacted fills.

The placement of any oversized materials should be performed in accordance with the Grading Guide Specifications included in Appendix D of this report. If disposal of oversized materials is required, rock blankets or windrows should be used and such areas should be observed during construction and placement by a representative of the geotechnical engineer.

Imported Structural Fill

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

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. It is recommended that materials in excess of 6 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 the city of Irwindale and the county of Los Angeles. 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 at this site generally consist of well grades sands and sandy gravels. These materials may be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. Temporary excavation slopes should be no steeper than 2h:1v. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Groundwater

The static groundwater table at this site is considered to exist at a depth in excess of 50± feet. Therefore, groundwater is not expected to impact 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 new structural fill soils used to replace a portion of the near-surface alluvium. These structural fill soils are expected to extend to depths of at least 2 feet below proposed foundation bearing grade, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, the proposed 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: Two (2) No. 5 rebars (1 top and 1 bottom).

- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.

- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressure presented above may be increased by one-third when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on geotechnical considerations; additional reinforcement may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

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

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

Estimated Foundation Settlements

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

Lateral Load Resistance

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

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

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

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floors of the new buildings may be constructed as conventional slabs-on-grade supported on newly placed structural fill soils, extending to a depth of at least 3 feet below the proposed pad grade. Based on geotechnical considerations, the floor slabs may be designed as follows:

- Minimum slab thickness: 6 inches.

- Modulus of Subgrade Reaction: k = 200 psi/in

- Minimum slab reinforcement: Reinforcement is not expected to be required for geotechnical conditions. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

- Slab underlayment: If moisture sensitive floor coverings will be used the minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area wherever 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. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview.

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

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

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

6.7 Retaining Wall Design and Construction

Although not indicated on the site plan, some retaining walls may be required to facilitate the new site grades. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

Based on the conditions encountered at the boring and trench locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near surface soils generally consist of well graded sands and sandy gravels. Based on their classifications, the near surface soils are expected to possess a friction angle of at least 32 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density.

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

RETAINING WALL DESIGN PARAMETERS

<table>
<tr><th colspan="2" rowspan="2">Design Parameter</th><th>Soil Type</th></tr>
<tr><th>On-Site Soils</th></tr>
<tr><td colspan="2">Internal Friction Angle (ϕ)</td><td>32°</td></tr>
<tr><td colspan="2">Unit Weight</td><td>130 lbs/ft^3</td></tr>
<tr><td rowspan="3">Equivalent Fluid Pressure:</td><td>Active Condition (level backfill)</td><td>40 lbs/ft^3</td></tr>
<tr><td>Active Condition (2h:1v backfill)</td><td>61 lbs/ft^3</td></tr>
<tr><td>At-Rest Condition (level backfill)</td><td>61 lbs/ft^3</td></tr>
</table>

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

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

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

Seismic Lateral Earth Pressures

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

Retaining Wall Foundation Design

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

Backfill Material

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

It is recommended that a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls be used. If the drainage composite material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The drainage composite should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557-91). 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, reliable, drainage system will be necessary in conjunction with the appropriate backfill material. This drainage system should consist of a 4-inch diameter perforated pipe surrounded by 3 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 connected to a sump pump system.

6.8 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The on-site soils generally consist of well graded sands and sandy gravels. Based on their classification, these materials are expected to possess good to excellent pavement support characteristics, with R-values in the range of 60 to 70. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon an assumed R-value of 60. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

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

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

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

ASPHALT PAVEMENTS (R=60)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	3	3	3	3	4
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 Portland cement 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				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 5.0 & 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	6	7	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

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

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.

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.

Sadigh, K., Chang, C. –Y., Egan, J. A., Makdisi. F., Youngs, R. R., "*Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data*", Seismological Research Letters, Seismological Society of America, Volume 68, Number 1, January/ February 1997, pp. 180-189.

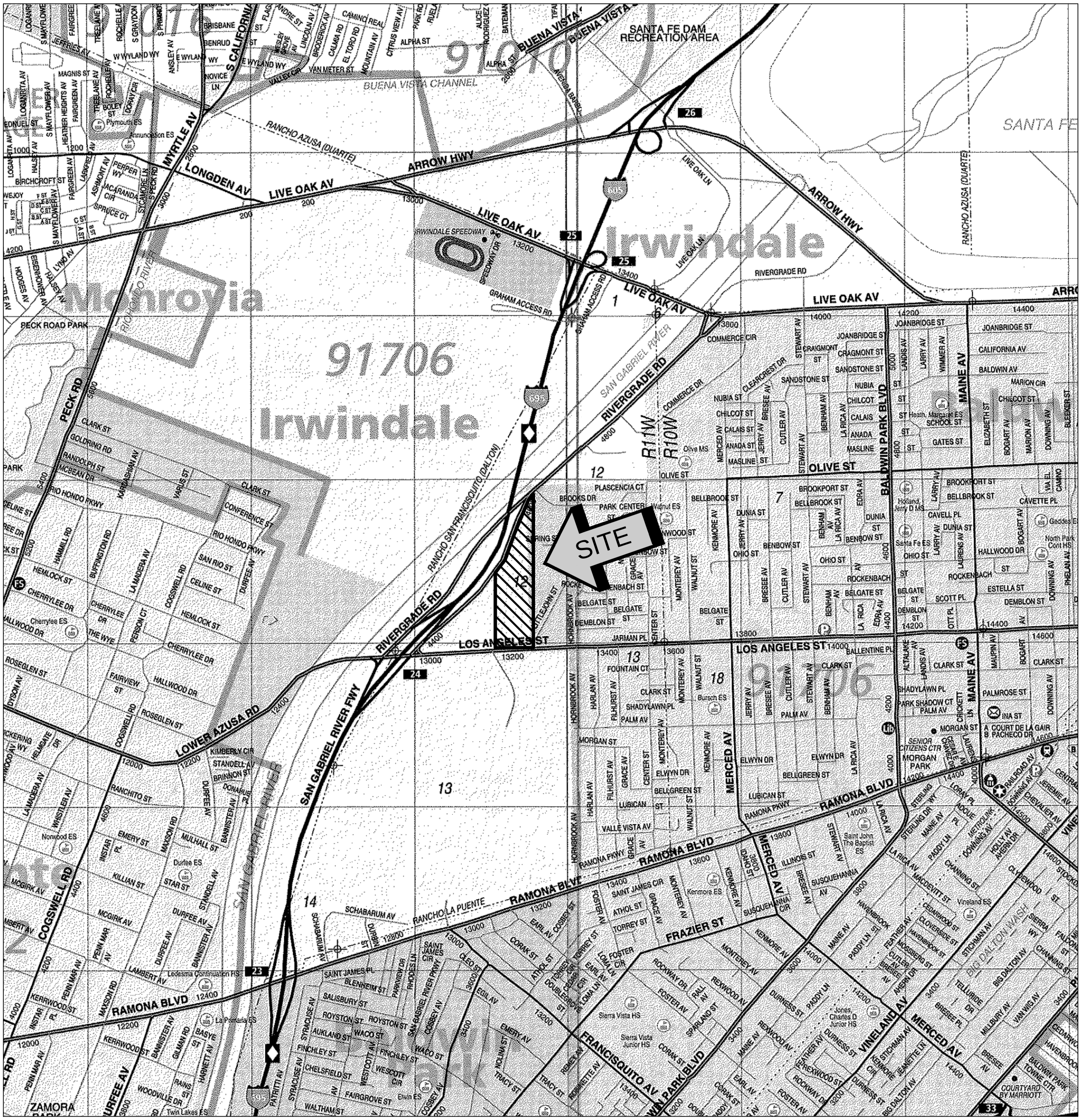
Southern California Earthquake Center (SCEC), University of Southern California, "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California," Committee formed 1997.

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.

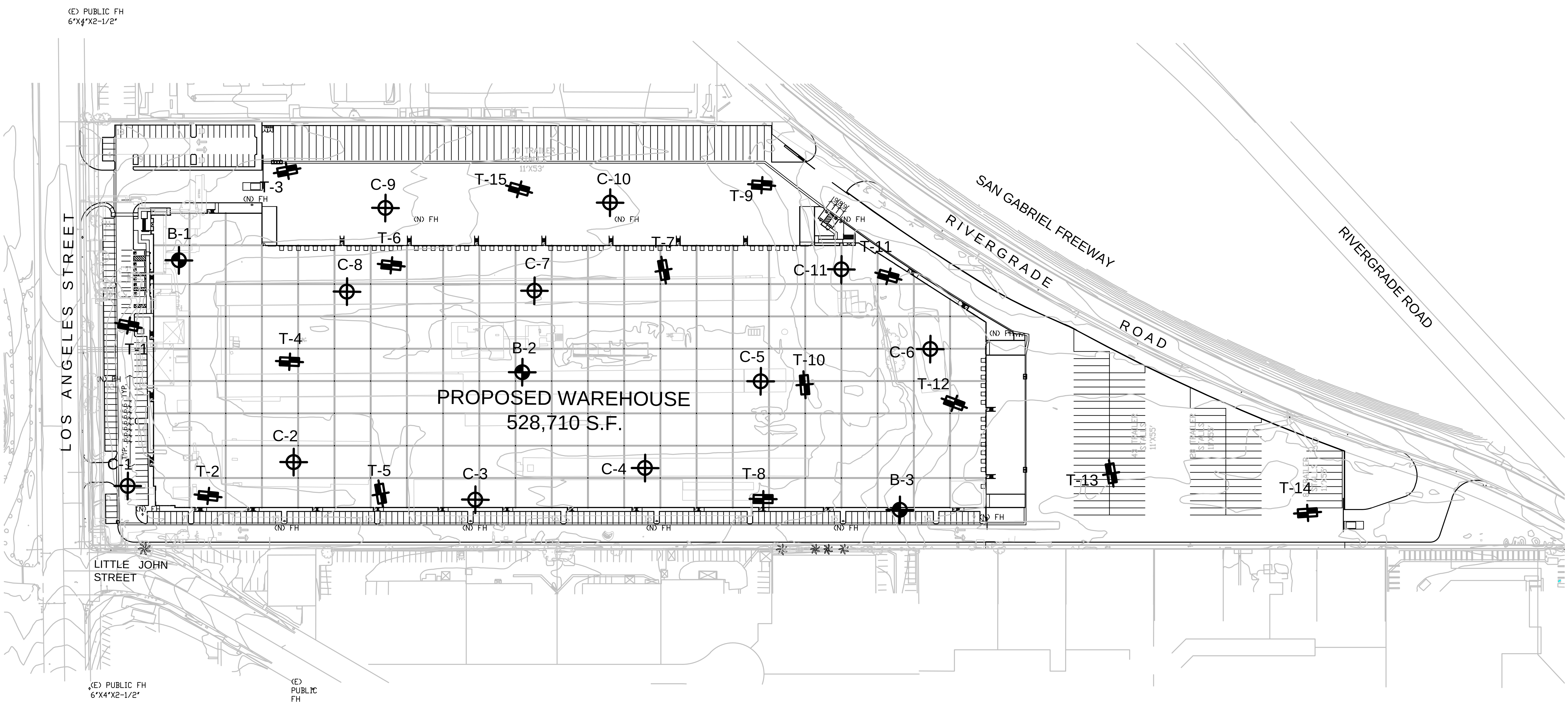
Youd, T. L. and Idriss, I. M. (Editors), "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," Salt Lake City, UT, January 5-6 1996, NCEER Technical Report NCEER-97-0022, Buffalo, NY.

APPENDIX A



SOURCE: LOS ANGELES COUNTY
THOMAS GUIDE, 2013

SITE LOCATION MAP	
PROPOSED WAREHOUSE	
IRWINDALE, CALIFORNIA	
SCALE: 1" = 2400'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: PML CHKD: RGT	
SCG PROJECT 18M192-1	
PLATE 1	



GEOTECHNICAL LEGEND

- APPROXIMATE BORING LOCATION
- APPROXIMATE CORE LOCATION
- APPROXIMATE TRENCH LOCATION

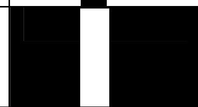
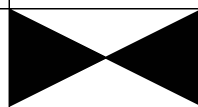
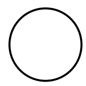
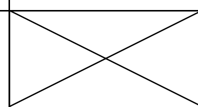
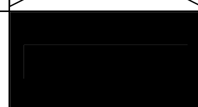
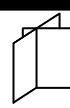
NORTH
SoCalGeo

SITE PLAN PROVIDED BY RGA.

EXPLORATION LOCATION PLAN	
PROPOSED WAREHOUSE	
IRWINDALE, CALIFORNIA	
SCALE: 1" = 150'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: PM CHKD: RGT	
SCG PROJECT 18M192-1	
PLATE 2	

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

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

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

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

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

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

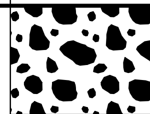
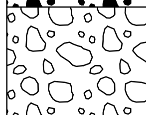
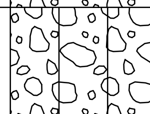
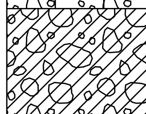
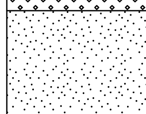
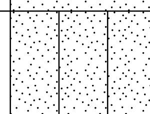
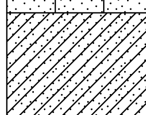
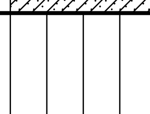
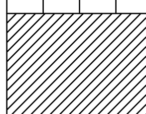
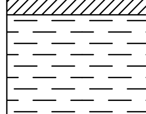
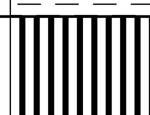
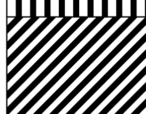
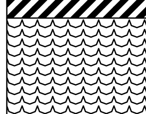
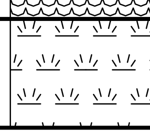
LIQUID LIMIT: The moisture content above which a soil behaves as a liquid.

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

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

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

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (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

BORING NO. B-1

JOB NO.: 18G145
PROJECT: Proposed Warehouse
LOCATION: Irwindale, California

DRILLING DATE: 4/30/18
DRILLING METHOD: Becker Hammer
LOGGED BY: Anthony Luna

WATER DEPTH: Dry
CAVE DEPTH: 39 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG		DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					1± inch Gravel, 6± inches Aggregate base							
	X	40			ALLUVIUM: Light Gray fine to medium Sand, little coarse Sand, little fine Gravel, dense-dry to damp		2					
	X	52			Light Gray Gravelly fine to coarse Sand, occasional Cobbles, very dense-dry		1					
5												
	X	69			Light Gray to Gray Brown to Brown fine to coarse Sandy Gravel to Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense to very dense-dry to damp		2					
	X	36					3					
10												
	X	71					2					
15												
	X	50/5"					2					
20												
	X	63					2					
25												
	X	69/11"			@ 28½ to 30 feet, moist		8					
30												
	X	50/2"					3					

TBL 18G145.GPJ SOCALGEO.GDT 5/23/18

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 18G145	DRILLING DATE: 4/30/18	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Becker Hammer	CAVE DEPTH: 39 feet
LOCATION: Irwindale, California	LOGGED BY: Anthony Luna	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Light Gray to Gray Brown to Brown fine to coarse Sandy Gravel to Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense to very dense-dry to damp							
	X	81					2					
40												
	X	50/4"					2					
45												
	X	50/5"					2					
50					Boring Terminated at 50'							

TBL 18G145.GPJ SOCALGEO.GDT 5/23/18

SOUTHERN CALIFORNIA GEOTECHNICAL
SoCalGeo
A California Corporation

BORING NO. B-2

JOB NO.: 18G145	DRILLING DATE: 4/30/18	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Becker Hammer	CAVE DEPTH: 34 feet
LOCATION: Irwindale, California	LOGGED BY: Anthony Luna	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					10± inches Aggregate base							
	X	50/3"			ALLUVIUM: Dark Gray Gravelly fine to coarse Sand to fine to coarse Sandy Gravel, very dense-moist		11					
	X	50/4"			Gray to Gray Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense to very dense-dry to damp		4					
5												
	X	37					2					
	X	50/4"					3					
10												
	X	50/4"					2					
15												
	X	50/4"					2					
20												
					Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, trace to little Silt, dense-damp							
	X	41					5					
25												
					Dark Brown Silty fine Sand, trace medium to coarse Sand, very dense-very moist							
	X	62/11"					18					
30					Gray Brown to Brown Gravelly fine to coarse Sand, occasional Cobbles, very dense-damp		4					
					Gray Brown fine to coarse Sandy Gravely to Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, very dense-dry to damp							
	X	50/4"					3					

BORING NO. B-2

JOB NO.: 18G145	DRILLING DATE: 4/30/18	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Becker Hammer	CAVE DEPTH: 34 feet
LOCATION: Irwindale, California	LOGGED BY: Anthony Luna	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Gray Brown fine to coarse Sandy Gravely to Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, very dense-dry to damp							
	X	50/3"					3					
40												
	X	50/4"					2					
45												
	X	50/3"					2					
50					Boring Terminated at 50'							

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 18G145
PROJECT: Proposed Warehouse
LOCATION: Irwindale, California

DRILLING DATE: 4/30/18
DRILLING METHOD: Becker Hammer
LOGGED BY: Anthony Luna

WATER DEPTH: Dry
CAVE DEPTH: 15 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
					8± inches Aggregate base							
	X	69			ALLUVIUM: Gray Brown Gravelly fine to coarse Sand, very dense-dry to damp		2					
	X	31			Light Gray Brown Gravelly fine to coarse Sand to fine to coarse Sandy Gravel, extensive Cobbles, occasional Boulders, dense to very dense-dry to damp		1					
5												
	X	50/6"					1					
	X	51					1					
10												
	X	50/5"					1					
15												
	X	50/4"					1					
20												
	X	50/5"					1					
25												
	X	50/3"					1					
30												
	X	50/2"					4					

TBL 18G145.GPJ SOCALGEO.GDT 5/23/18

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 18G145	DRILLING DATE: 4/30/18	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Becker Hammer	CAVE DEPTH: 15 feet
LOCATION: Irwindale, California	LOGGED BY: Anthony Luna	READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Light Gray Brown Gravelly fine to coarse Sand to fine to coarse Sandy Gravel, extensive Cobbles, occasional Boulders, dense to very dense-dry to damp							
	X	50/5"			@ 38½ to 40 feet, damp to moist		7					
40												
	X	50/5"					4					
45												
	X	50/5"			@ 48½ to 50 feet, moist		3					
50												
					Boring Terminated at 50'							

TBL 18G145.GPJ SOCALGEO.GDT 5/23/18

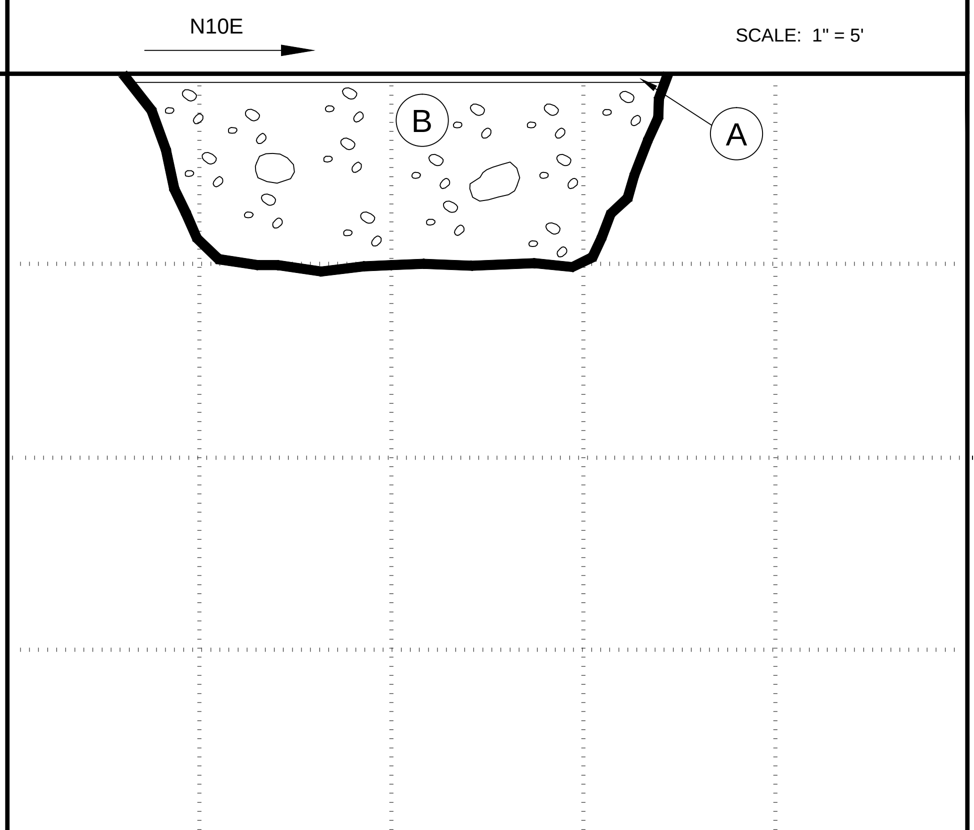
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-1

JOB NO.: 18G145-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	LOGGED BY: Jason Hiskey	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N10E	READINGS TAKEN: At Completion
DATE: 4-30-2018	ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: ASPHALTIC CONCRETE (AC): 2 inches thick
	b		2	B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry to damp
5	b		2	
				Trench Terminated @ 5 feet
10				
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-4

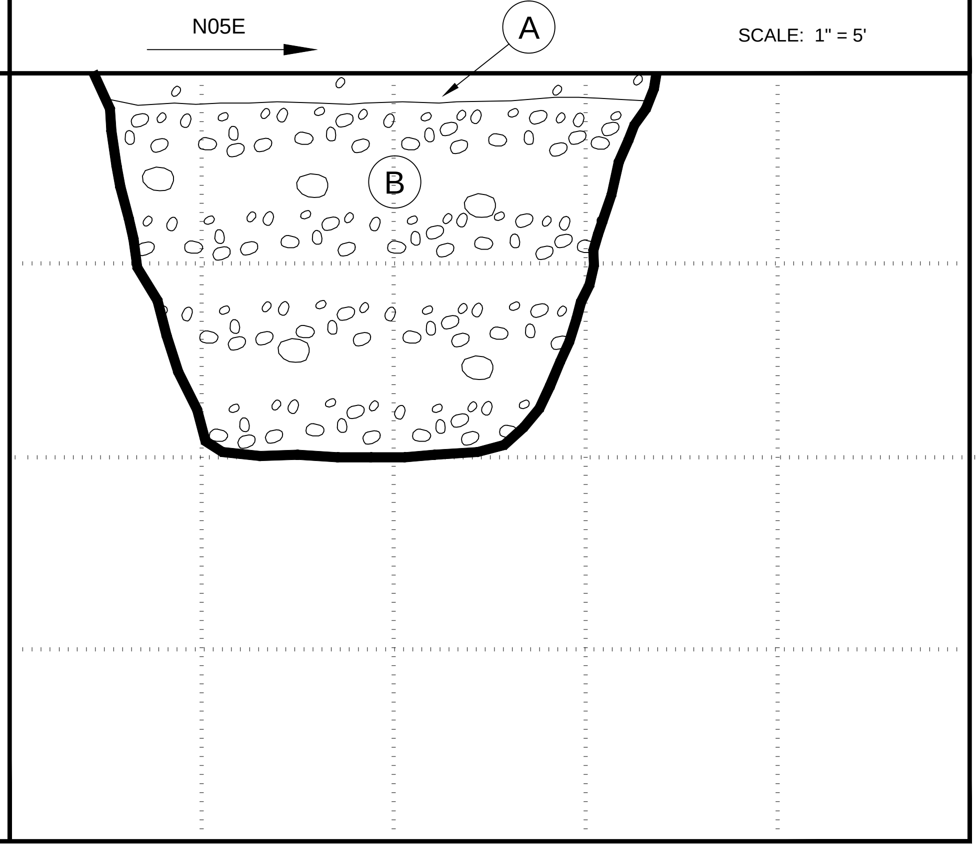
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-2

JOB NO.: 18G145-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	LOGGED BY: Jason Hiskey	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N05E	READINGS TAKEN: At Completion
DATE: 4-30-2018	ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		2	A: AGGREGATE BASE (AB): 8 inches thick (Cement Treated) B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry to damp
5	b		2	
	b		2	
10	b		2	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-5

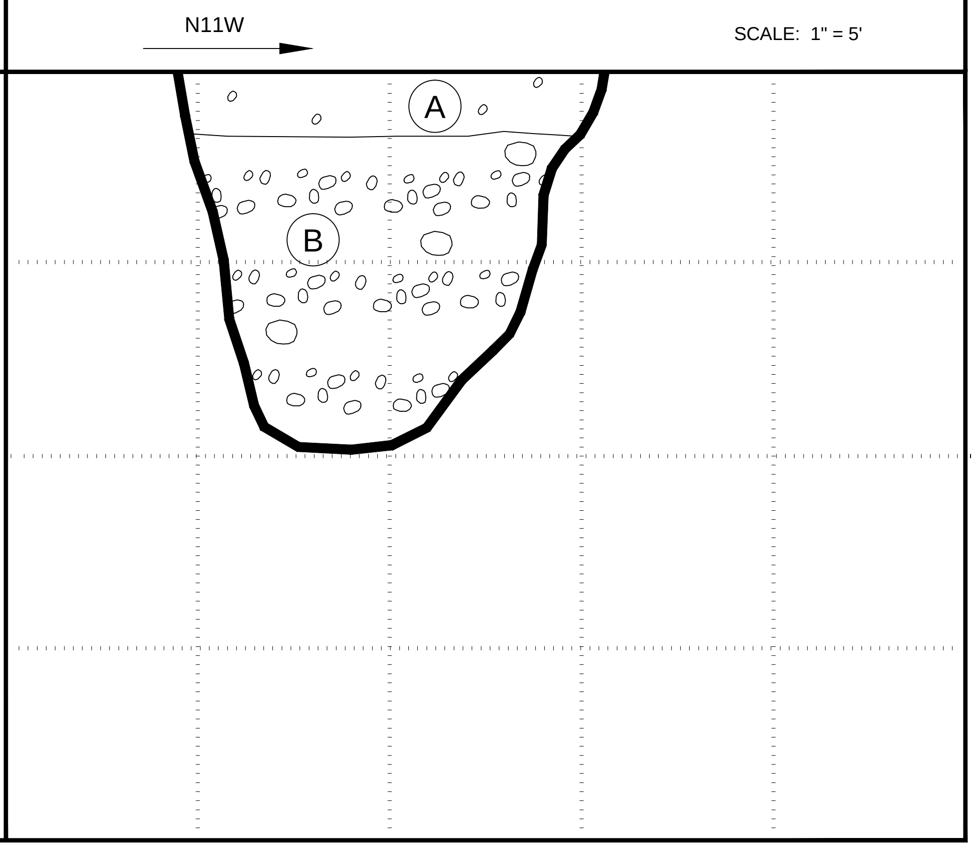
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-3

JOB NO.: 18G145-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	LOGGED BY: Jason Hiskey	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N11W	READINGS TAKEN: At Completion
DATE: 5-1-2018	ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: AGGREGATE BASE (AB): 18 inches thick (Cement Treated)
	b		2	B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry to damp
	b		1	
5				
	b		1	
10	b		1	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-6

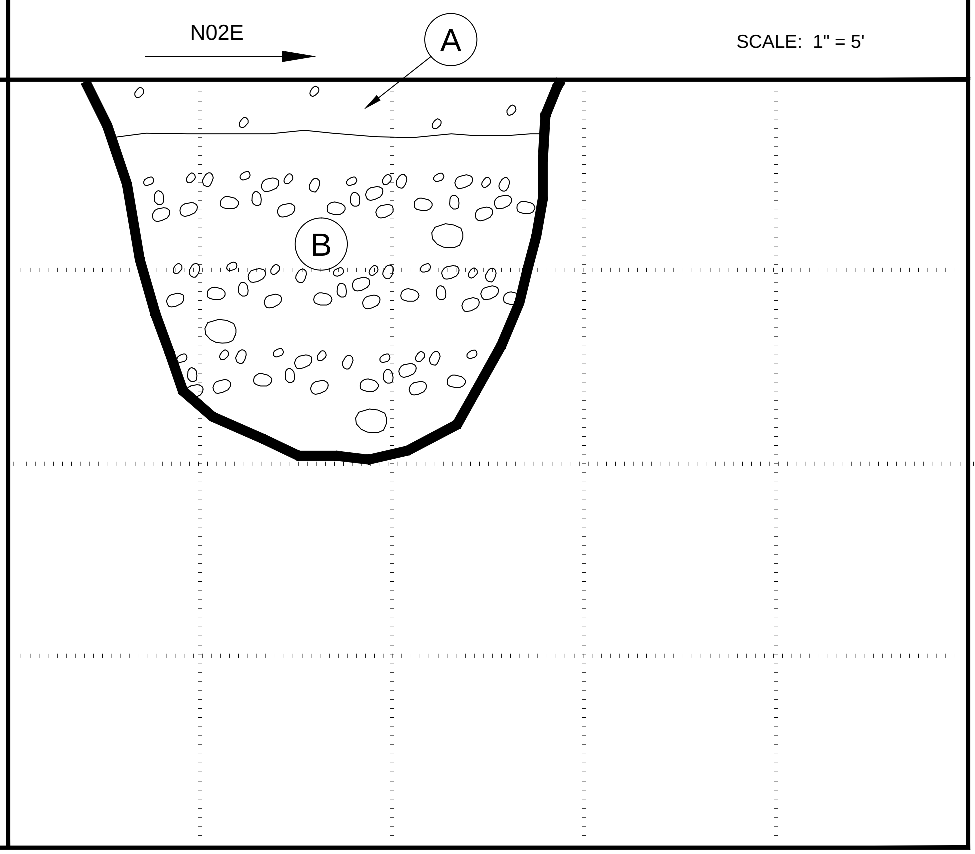
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-4

JOB NO.: 18G145-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	LOGGED BY: Jason Hiskey	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N02E	READINGS TAKEN: At Completion
DATE: 4-30-2018	ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: AGGREGATE BASE (AB): 18 inches thick (Cement Treated)
	b		2	B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry to damp
5	b		2	
	b		2	
10	b		1	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-7

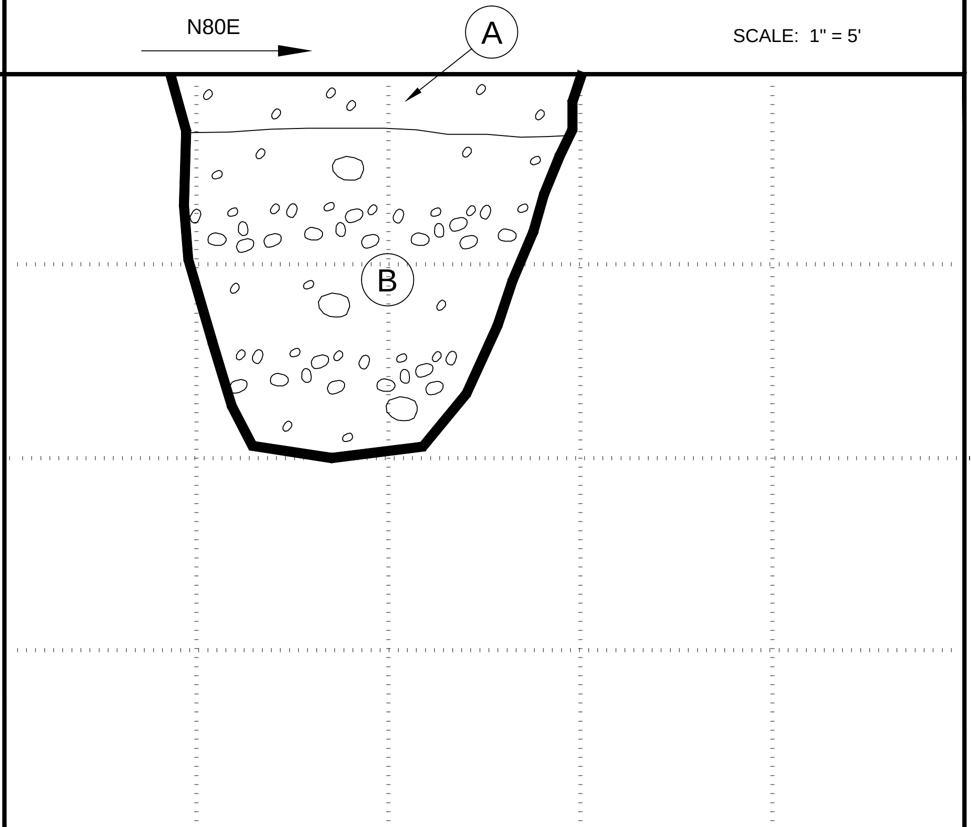
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-5

JOB NO.: 18G145-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	LOGGED BY: Jason Hiskey	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N80E	READINGS TAKEN: At Completion
DATE: 5-1-2018	ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: AGGREGATE BASE (AB): 18 inches thick (Cement Treated)
	b		2	B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry to damp
	b		1	
5				
	b		2	
10	b		2	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-8

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-6

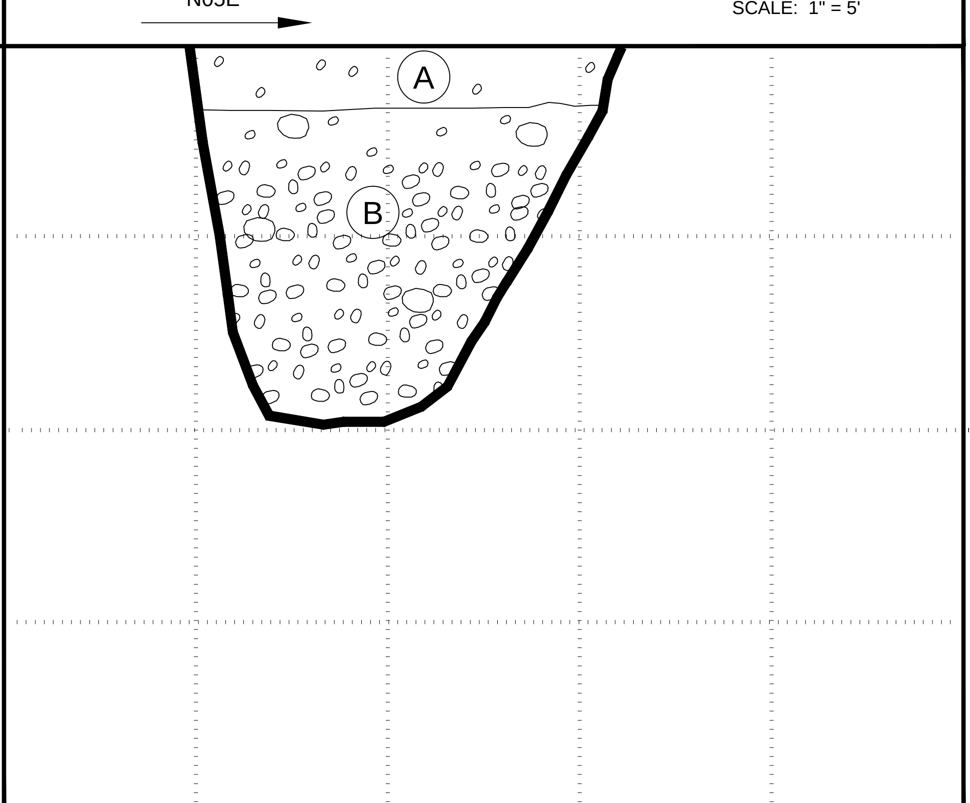
JOB NO.: 18G145-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	LOGGED BY: Jason Hiskey	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N05E	READINGS TAKEN: At Completion
DATE: 5-1-2018	ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: AGGREGATE BASE (AB): 18 inches thick (Cement Treated)
	b		1	B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry
5	b		1	
	b		1	
10	b		1	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION

N05E

SCALE: 1" = 5'



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

TRENCH LOG

PLATE B-9

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-7

JOB NO.: 18G145-1
PROJECT: Proposed Warehouse
LOCATION: Irwindale, CA
DATE: 5-1-2018

EQUIPMENT USED: Backhoe
LOGGED BY: Jason Hiskey
ORIENTATION: N80E
ELEVATION:

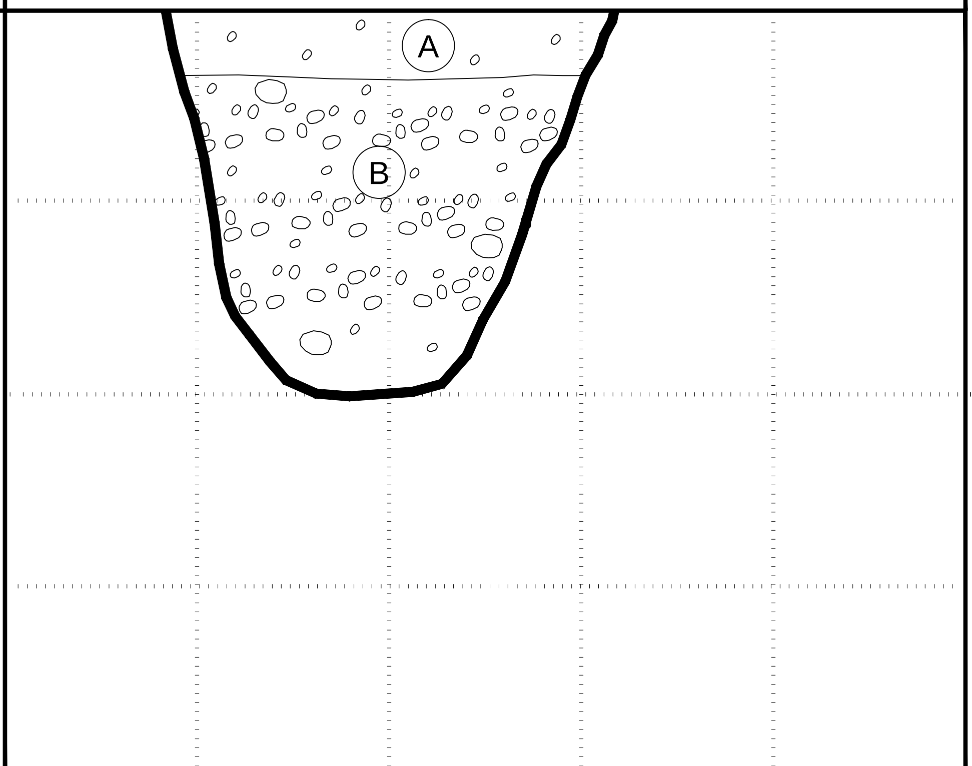
WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: AGGREGATE BASE (AB): 18 inches thick (Cement Treated)
	b		1	B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry to damp
5	b		1	
	b		2	
10	b		1	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION

N80E

SCALE: 1" = 5'



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

TRENCH LOG

PLATE B-10

SOUTHERN CALIFORNIA GEOTECHNICAL

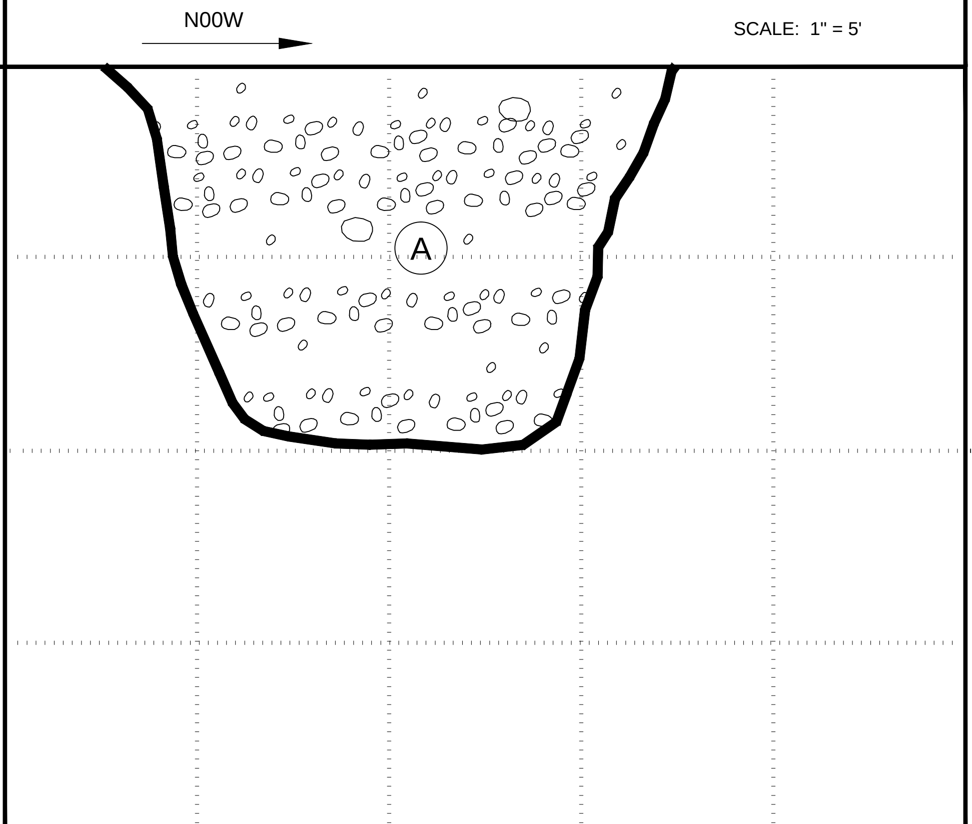
TRENCH NO.
T-8

JOB NO.: 18G145-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	LOGGED BY: Jason Hiskey	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N00W	READINGS TAKEN: At Completion
DATE: 4-30-2018	ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		2	A: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry to damp
	b		2	
5				
	b		4	
10	b		2	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION

SCALE: 1" = 5'



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

TRENCH LOG

PLATE B-11

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-9

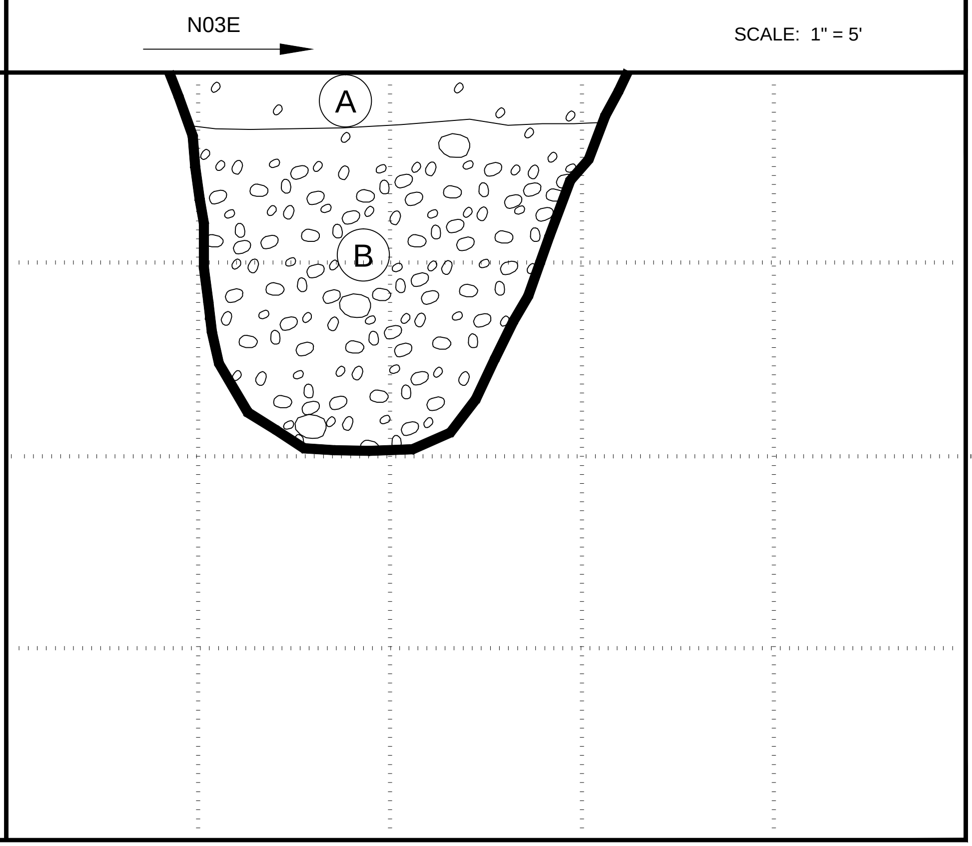
JOB NO.: 18G145-1
PROJECT: Proposed Warehouse
LOCATION: Irwindale, CA
DATE: 5-1-2018

EQUIPMENT USED: Backhoe
LOGGED BY: Jason Hiskey
ORIENTATION: N03E
ELEVATION:

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

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		1	A: AGGREGATE BASE (AB): 12 inches thick (Cement Treated) B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry
	b		1	
5				
	b		1	
10	b		1	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-12

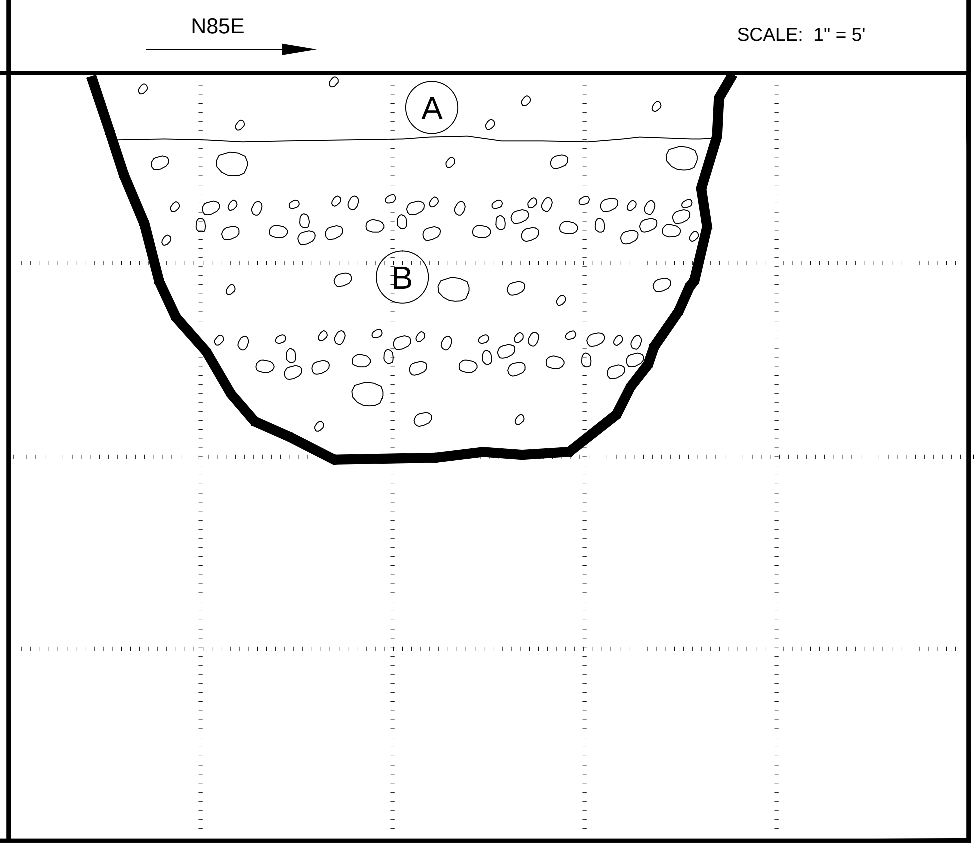
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-10

JOB NO.: 18G145-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	LOGGED BY: Jason Hiskey	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N85E	READINGS TAKEN: At Completion
DATE: 4-30-2018	ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: AGGREGATE BASE (AB): 18 inches thick (Cement Treated)
	b		3	B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry to damp
5	b		2	
	b		2	
10	b		3	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-13

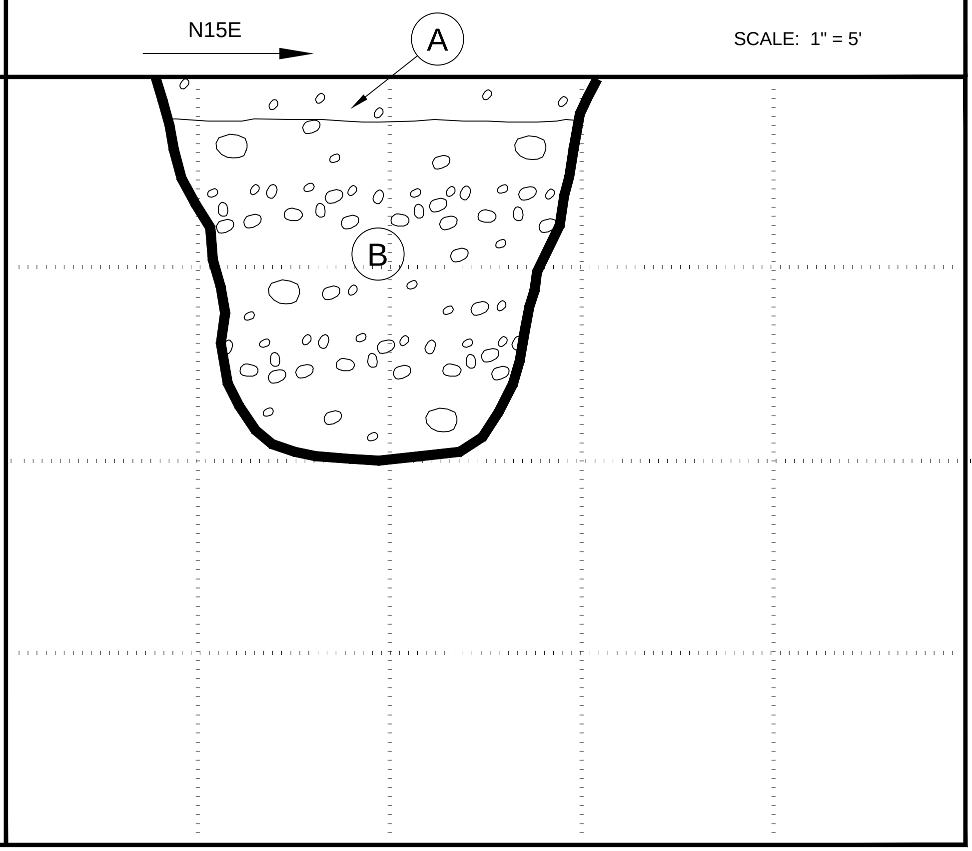
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-11

JOB NO.: 18G145-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	LOGGED BY: Jason Hiskey	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N15E	READINGS TAKEN: At Completion
DATE: 5-1-2018	ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		2	A: AGGREGATE BASE (AB): 12 inches thick (Cement Treated) B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry to damp
	b		2	
5				
	b		2	
10	b		1	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-14

SOUTHERN CALIFORNIA GEOTECHNICAL

**TRENCH NO.
T-12**

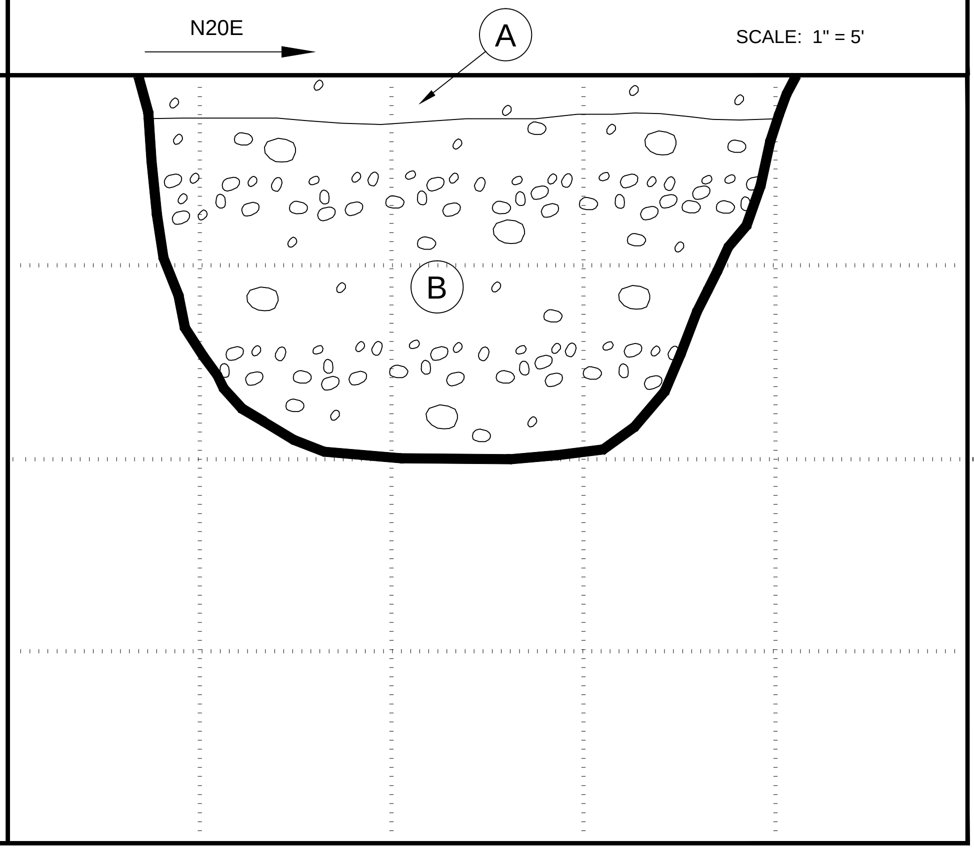
JOB NO.: 18G145-1
PROJECT: Proposed Warehouse
LOCATION: Irwindale, CA
DATE: 4-30-2018

EQUIPMENT USED: Backhoe
LOGGED BY: Jason Hiskey
ORIENTATION: N20E
ELEVATION:

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

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: AGGREGATE BASE (AB): 12 inches thick (Cement Treated)
	b		3	B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry to damp
	b		2	
5				
	b		3	
10	b		5	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-15

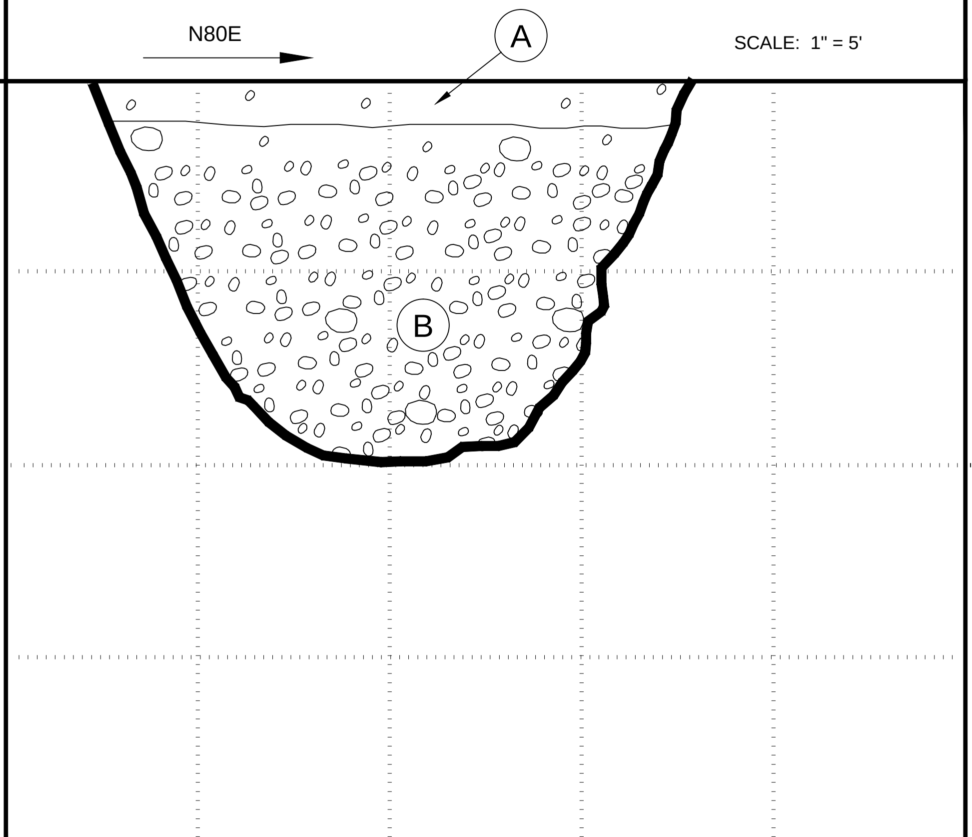
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-13

JOB NO.: 18G145-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	LOGGED BY: Jason Hiskey	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N80E	READINGS TAKEN: At Completion
DATE: 4-30-2018	ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: AGGREGATE BASE (AB): 10 inches thick (Cement Treated)
	b		3	B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry to damp
	b		2	
5				
	b		1	
10	b		2	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-16

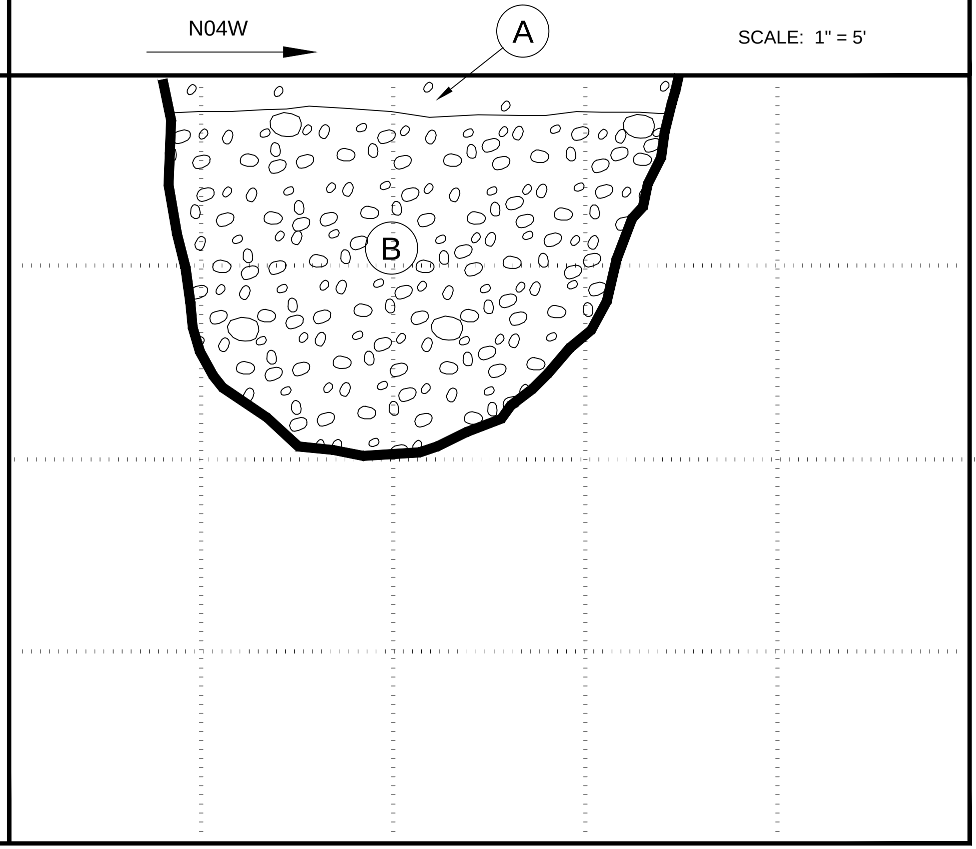
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-14

JOB NO.: 18G145-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	LOGGED BY: Jason Hiskey	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N04W	READINGS TAKEN: At Completion
DATE: 4-30-2018	ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: AGGREGATE BASE (AB): 12 inches thick (Cement Treated)
	b		2	B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry to damp
5	b		2	
	b		1	
10	b		2	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-17

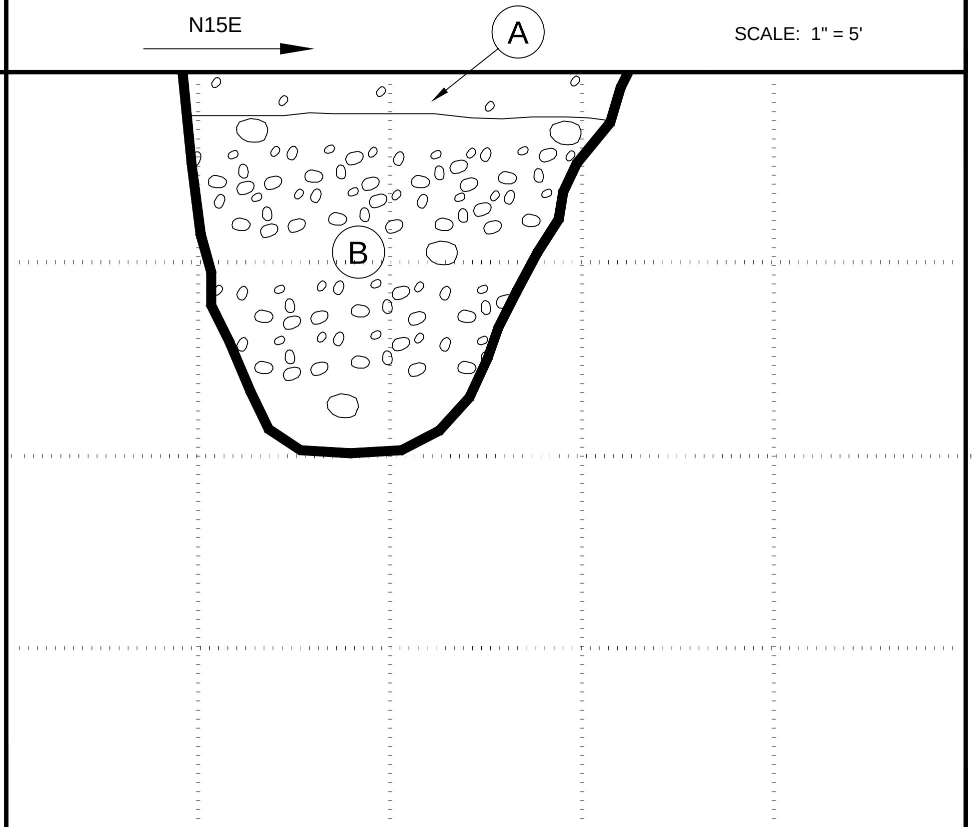
SOUTHERN CALIFORNIA GEOTECHNICAL

**TRENCH NO.
T-15**

JOB NO.: 18G145-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	LOGGED BY: Jason Hiskey	SEEPAGE DEPTH: Dry
LOCATION: Irwindale, CA	ORIENTATION: N15E	READINGS TAKEN: At Completion
DATE: 5-1-2018	ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		1	A: AGGREGATE BASE (AB): 12 inches thick (Cement Treated) B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry
	b		1	
5				
	b		1	
10	b		1	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION

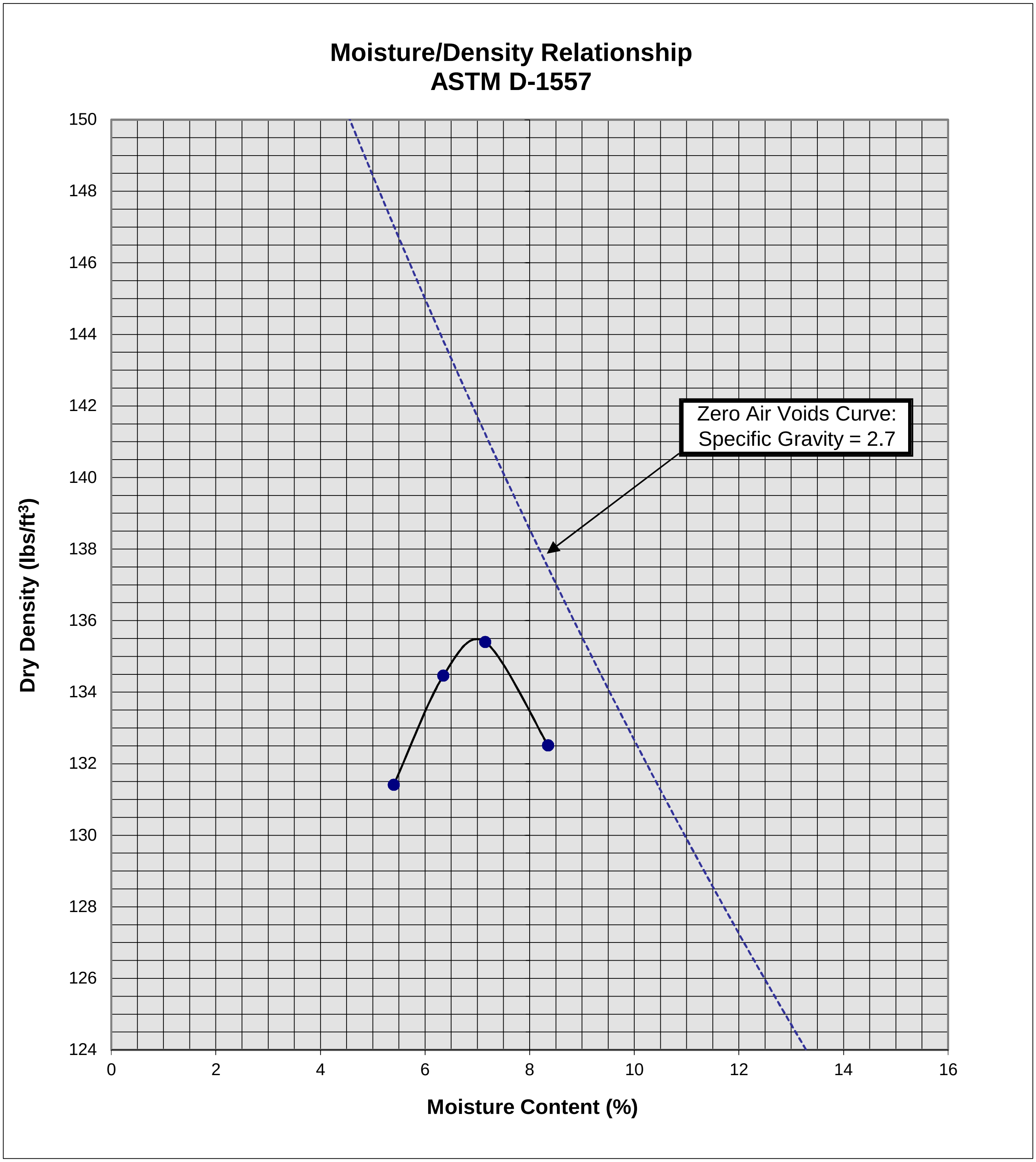


KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-18

APPENDIX C

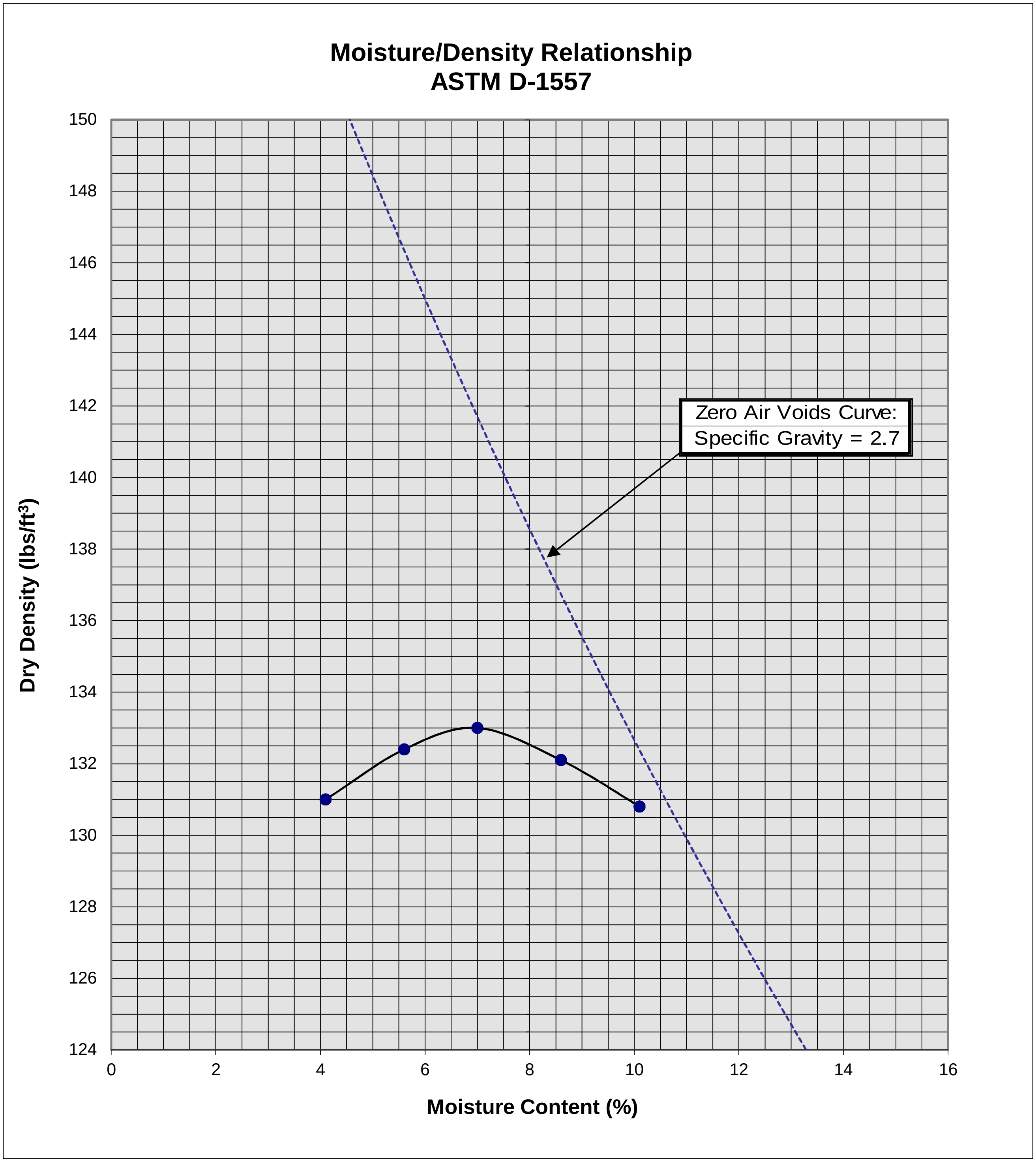


Soil ID Number	B-1 @ 0 to 5'
Optimum Moisture (%)	5.5
Maximum Dry Density (pcf)	138
Soil Classification	Brown fine to coarse Sand, some fine Gravel, some Cobbles

Note: Maximum Density and Optimum Moisture are based on 15% rock correction.

Proposed Warehouse
Irwindale, California
Project No. 18G145
PLATE C-1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



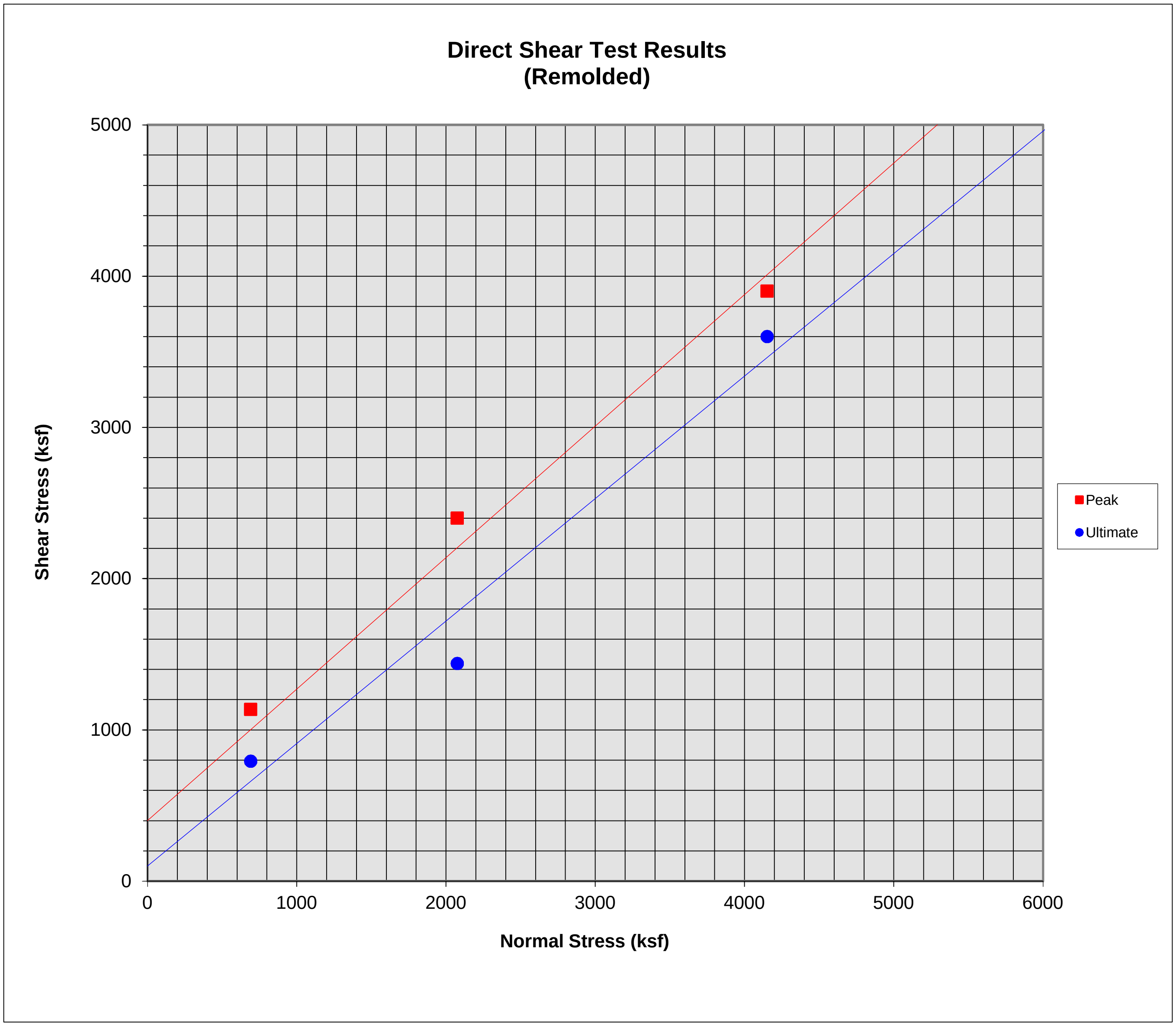
Soil ID Number	B-3 @ 0 to 5'
Optimum Moisture (%)	4.5
Maximum Dry Density (pcf)	141
Soil Classification	Gray Brown Gravelly fine to coarse Sand, some Cobbles

Note: Maximum Density and Optimum Moisture are based on 35% rock correction.

Proposed Warehouse
Irwindale, California
Project No. 18G145

PLATE C-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Sample Description: B-1 @ 0 to 5 feet

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

Sample Data

Remolded Moisture Content	7.0
Final Moisture Content	14.0
Remolded Dry Density	122.0
Percent Compaction	90.0
Final Dry Density	---
Specimen Diameter (in)	
Specimen Thickness (in)	2.4

Test Results

	Peak	Ultimate
ϕ (°)	41	39
C (psf)	400	100

Proposed Commercial/Industrial Building
Irwindale, California
Project No. 18G145
PLATE C-3

SoCalGeo 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.

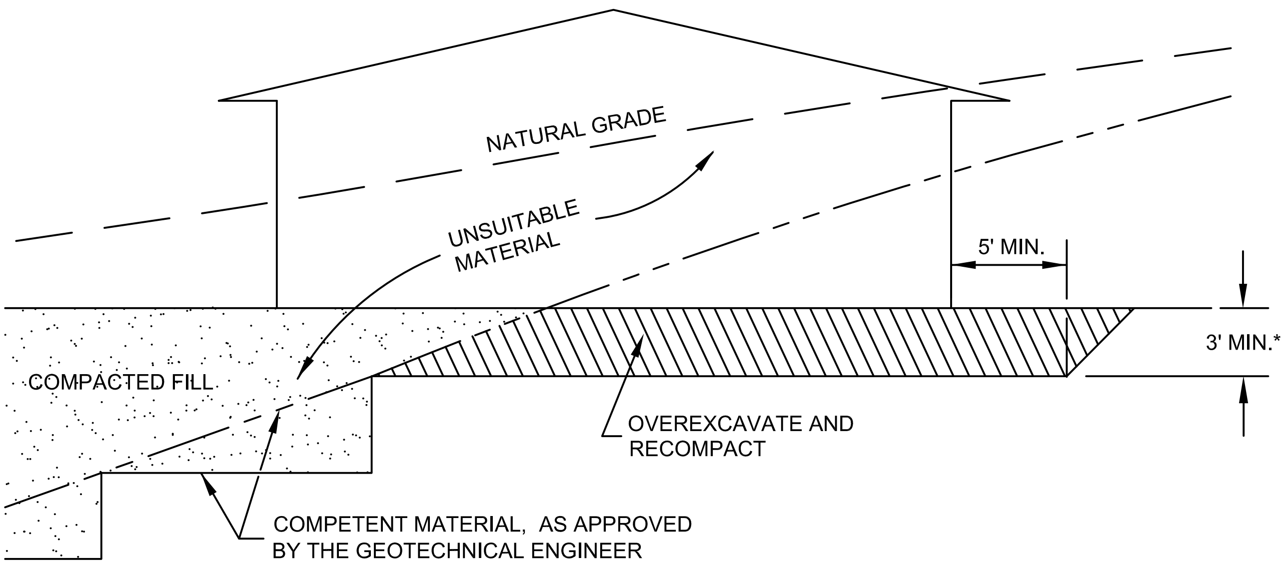
Subdrains

- 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.

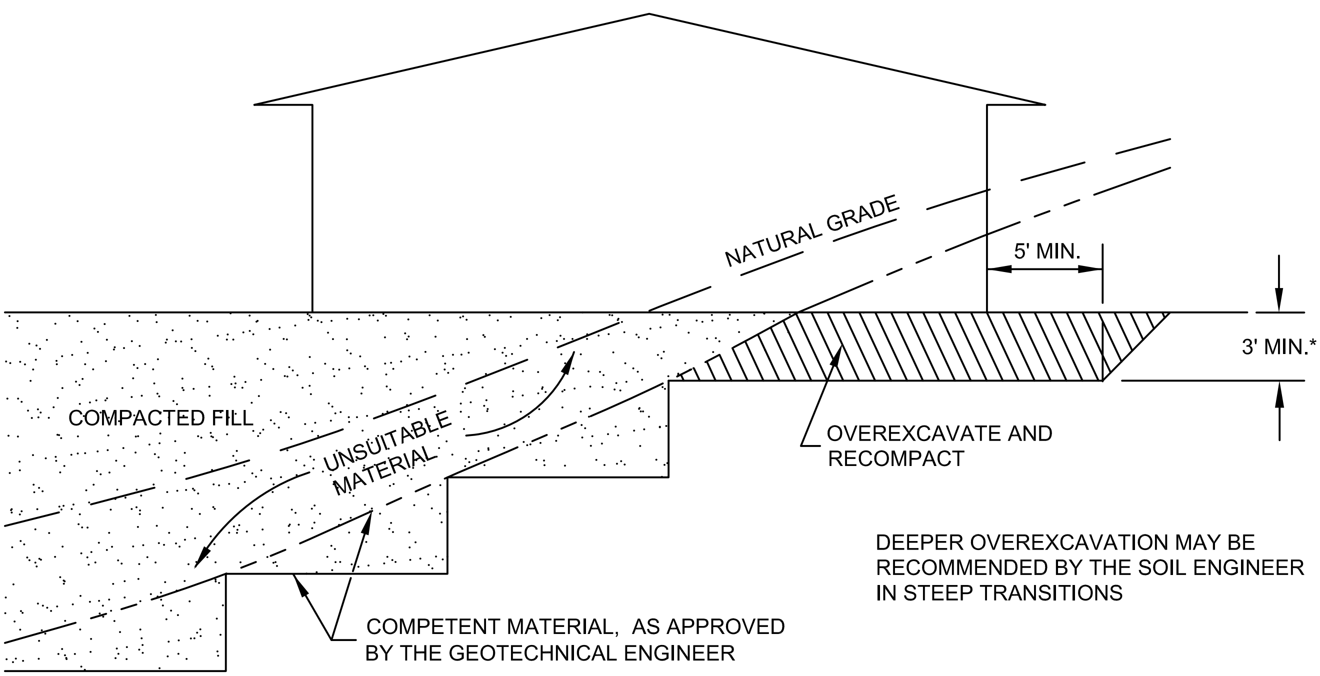
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- 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)



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

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

TRANSITION LOT DETAIL

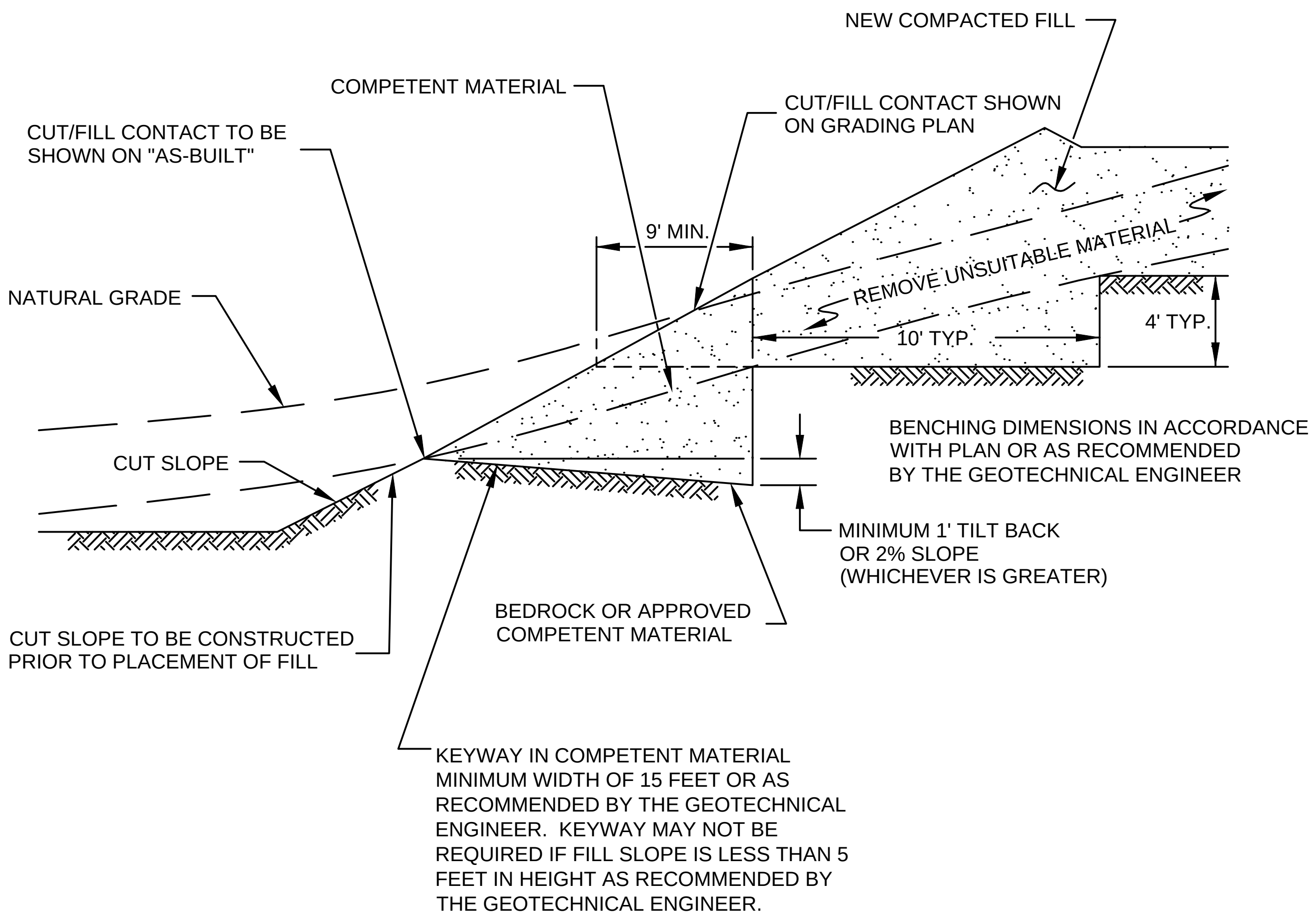
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

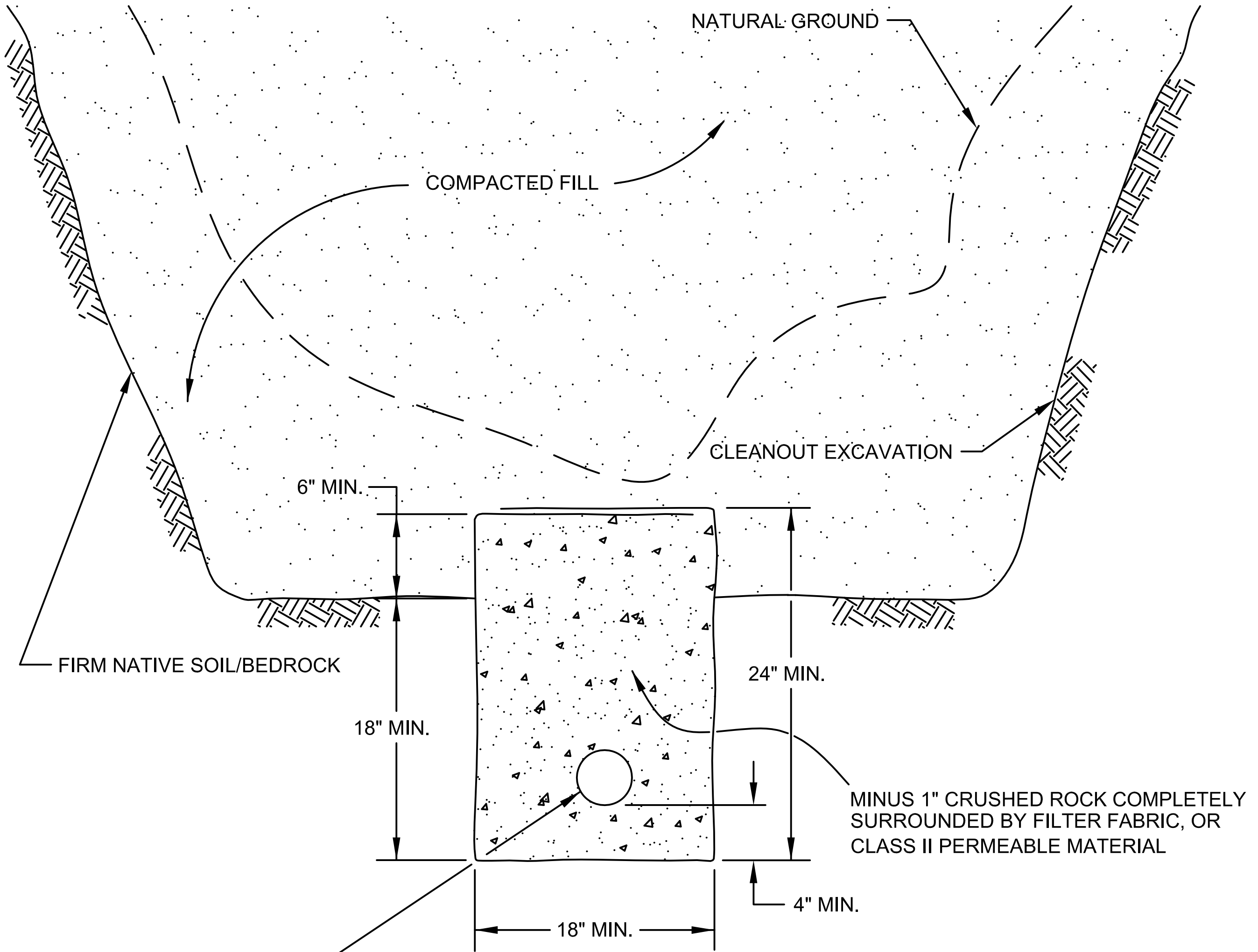
GRADING GUIDE SPECIFICATIONS

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

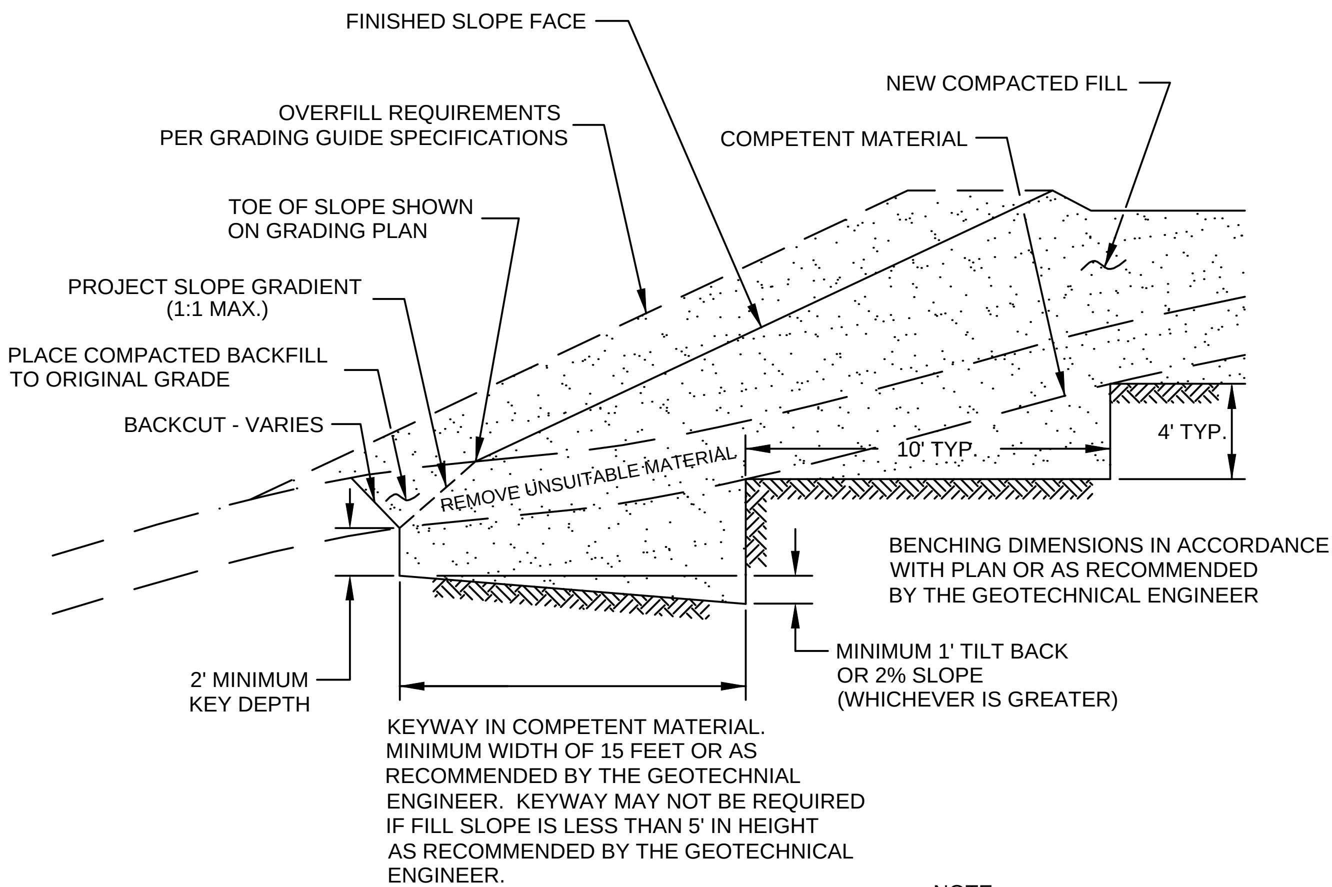
NOT TO SCALE

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

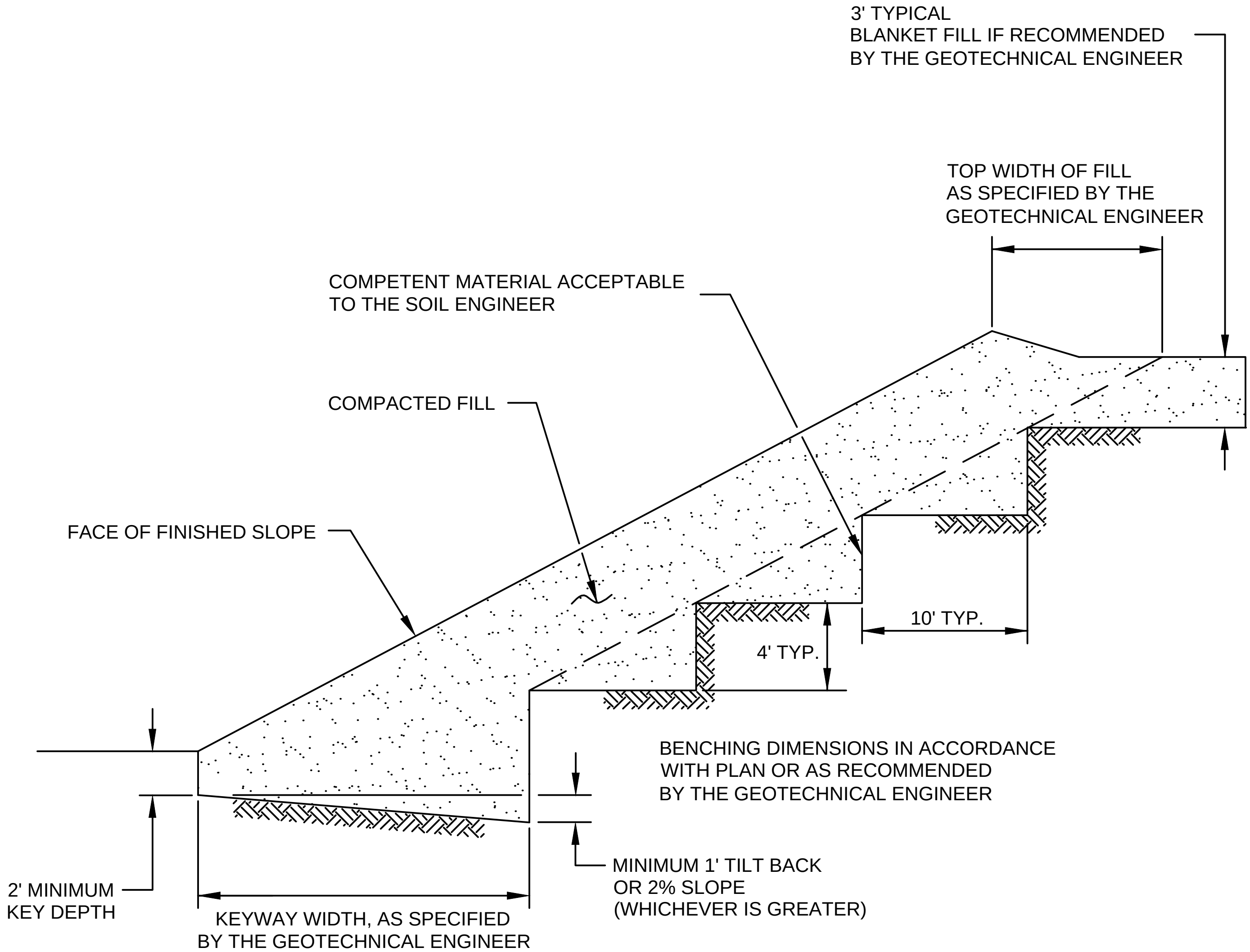
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo
SOUTHERN
CALIFORNIA
GEOTECHNICAL



STABILIZATION FILL DETAIL

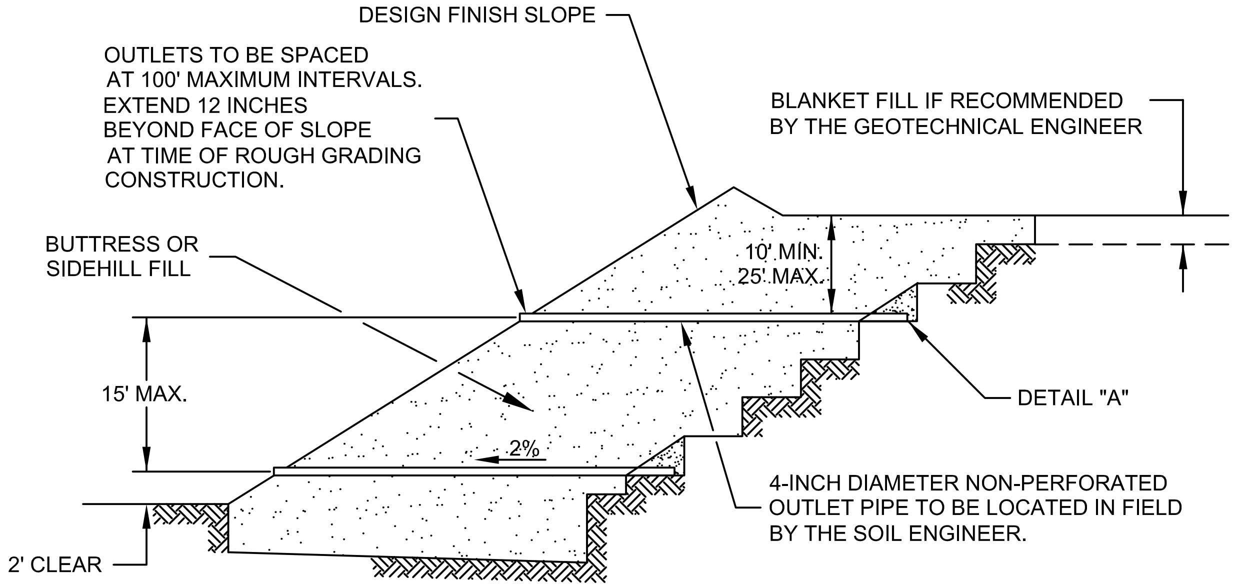
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

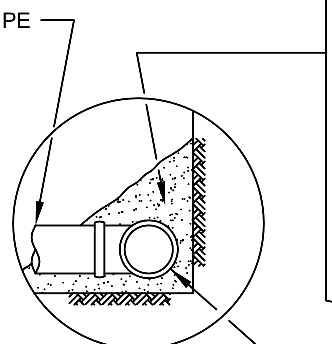
SAND EQUIVALENT = MINIMUM OF 50

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

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

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

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



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

DETAIL "A"

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

SLOPE FILL SUBDRAINS

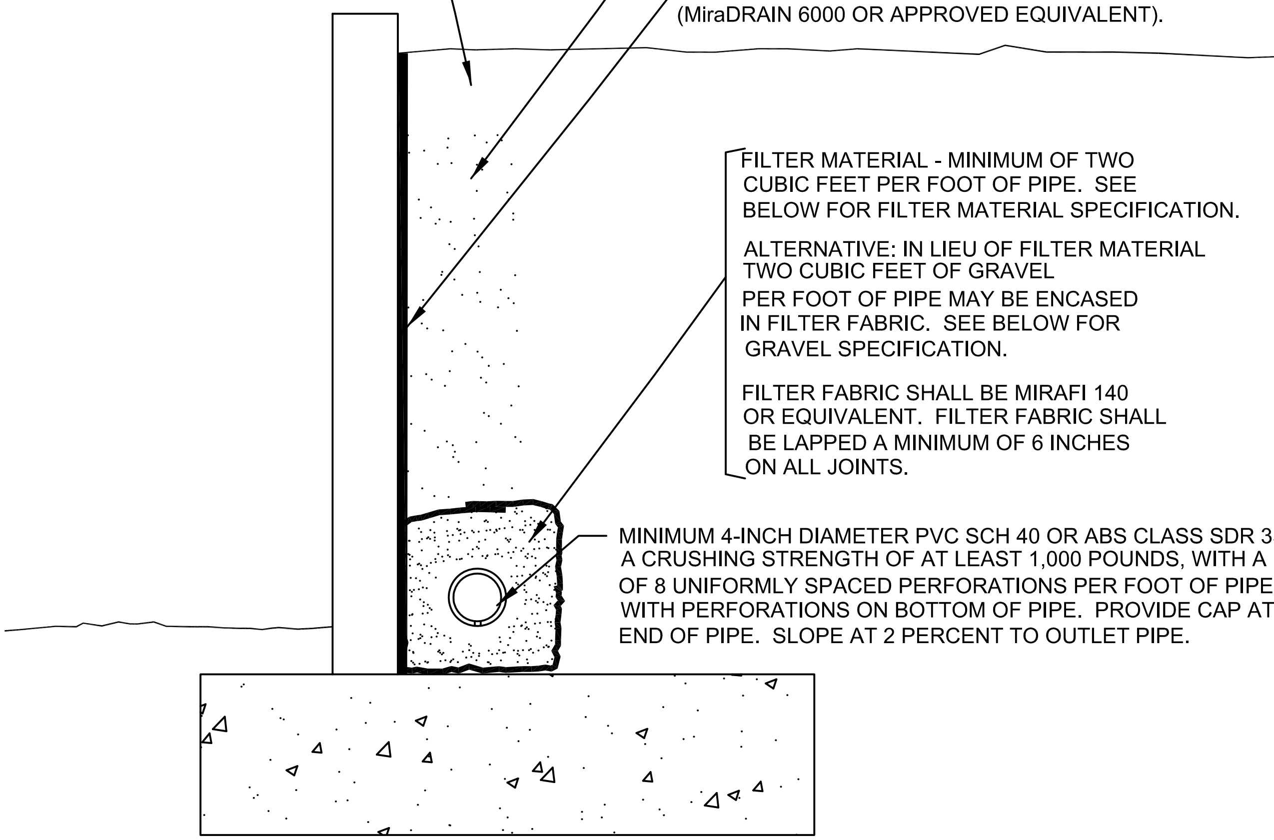
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

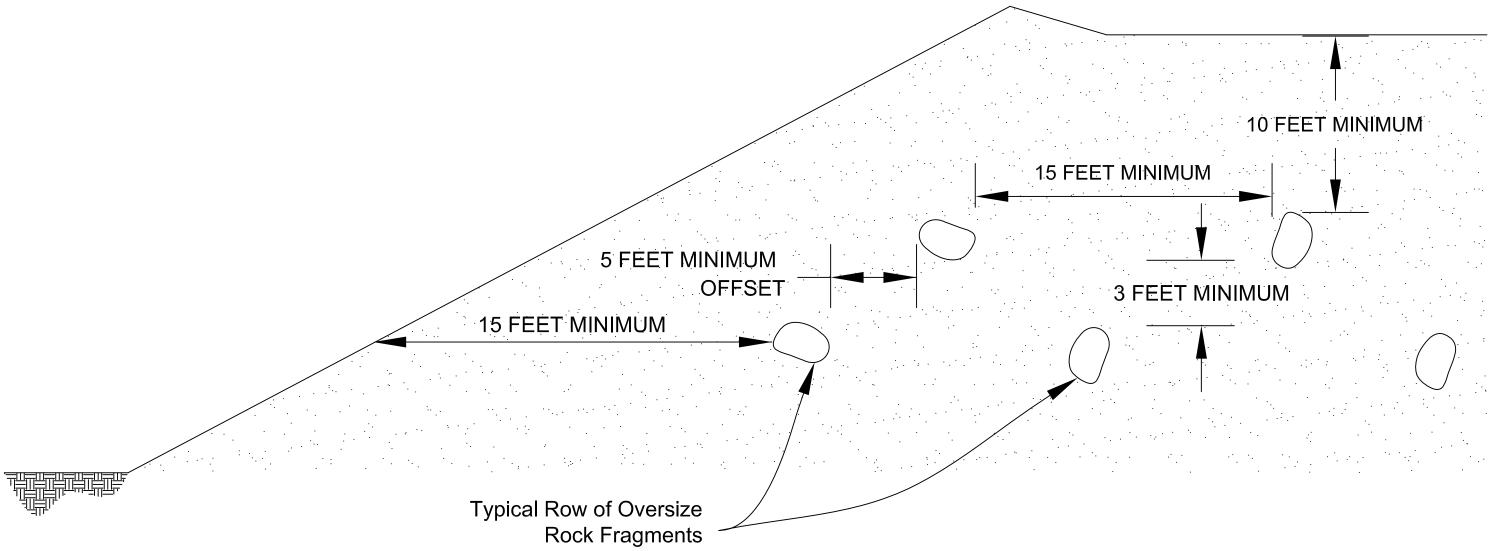
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

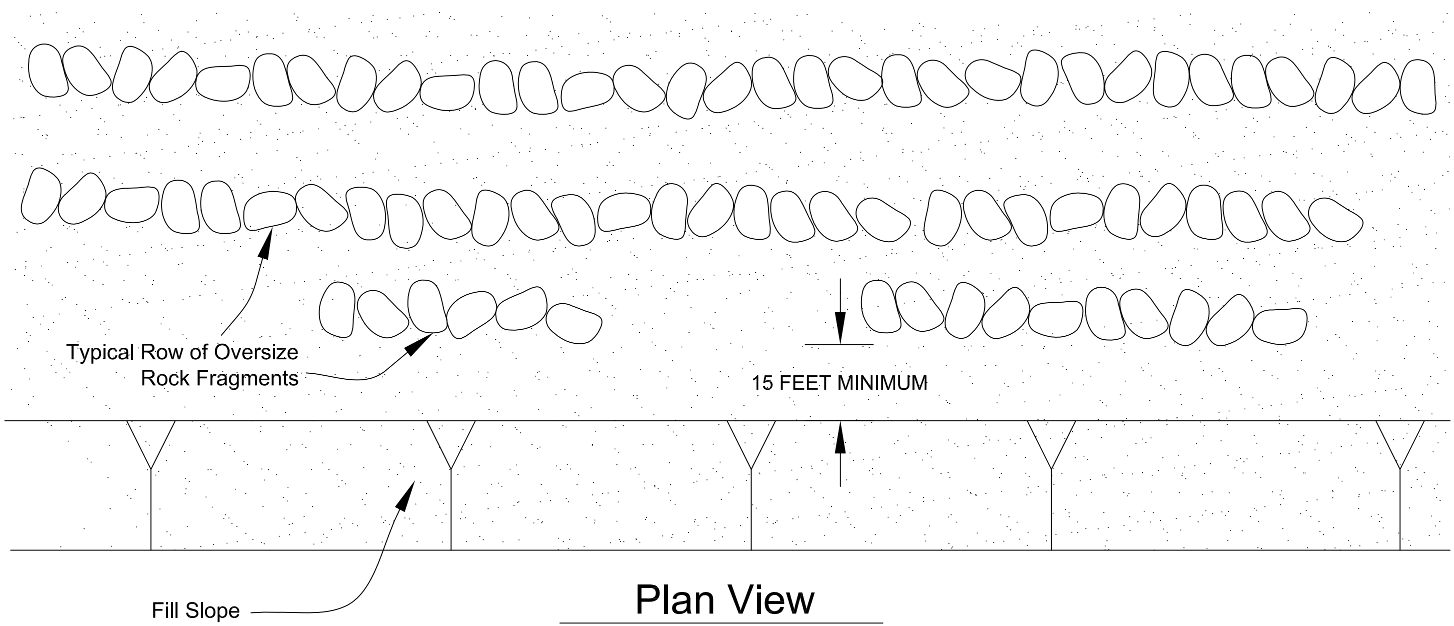
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

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

APPENDIX E

USGS Design Maps Summary Report

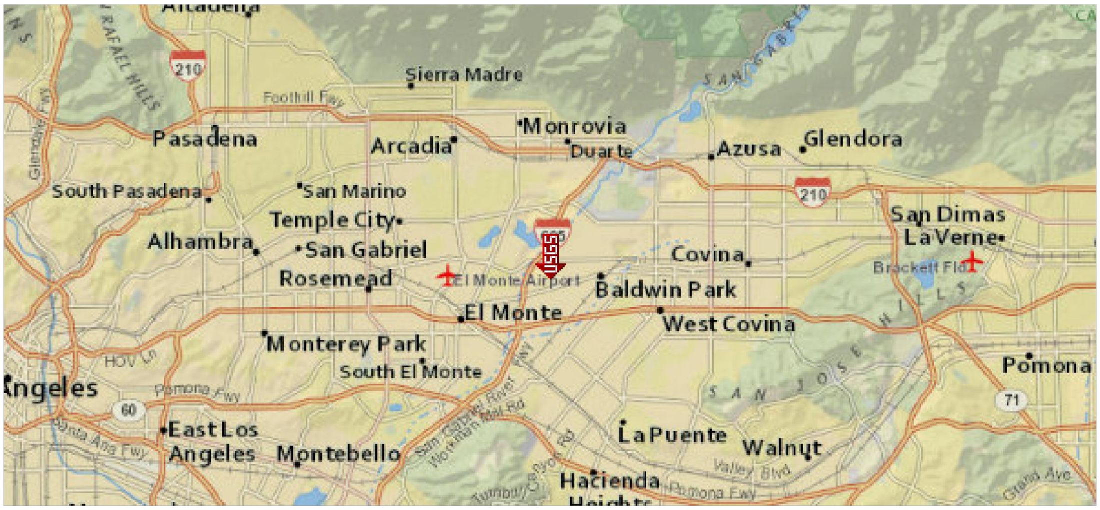
User–Specified Input

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 34.09403°N, 117.98557°W

Site Soil Classification Site Class D – "Stiff Soil"

Risk Category I/II/III

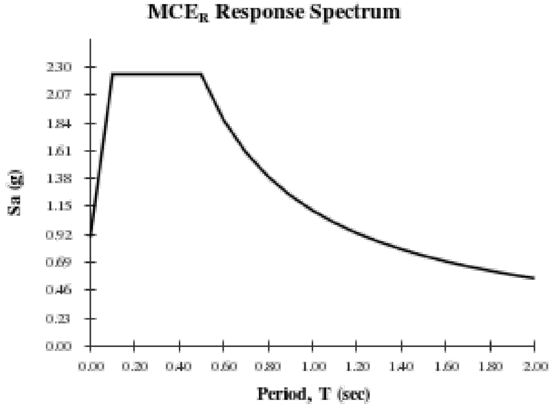


USGS–Provided Output

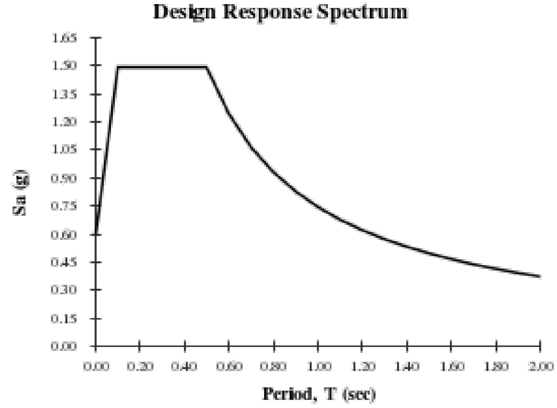
S_S =	2.239 g	S_{MS} =	2.239 g	S_{DS} =	1.493 g
S_1 =	0.745 g	S_{M1} =	1.118 g	S_{D1} =	0.745 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



SOURCE: U.S. GEOLOGICAL SURVEY (USGS)
<http://geohazards.usgs.gov/designmaps/us/application.php>

SEISMIC DESIGN PARAMETERS	
PROPOSED WAREHOUSE	
IRWINDALE, CALIFORNIA	
DRAWN: AL CHKD: RGT	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
SCG PROJECT 18M192-1	
PLATE E-1	

Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4]	PGA = 0.787
Equation (11.8–1):	$PGA_M = F_{PGA}PGA = 1.000 \times 0.787 = 0.787$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.787 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5]	C_{RS} = 0.999
From Figure 22-18 [6]	C_{R1} = 1.017

SOURCE: U.S. GEOLOGICAL SURVEY (USGS)
<http://geohazards.usgs.gov/designmaps/us/application.php>

MCE PEAK GROUND ACCELERATION
PROPOSED WAREHOUSE
IRWINDALE, CALIFORNIA

DRAWN: AL
CHKD: RGT
SCG PROJECT
18M192-1
PLATE E-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Proposed Warehouse	MCE_G Design Acceleration	0.787 (g)
	Irwindale, CA	Design Magnitude	7.71
Project Number	18G145	Historic High Depth to Groundwater	35 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-1		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.71)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
15.5	0	35	17.5		120		1.3	1.05	1.1	1.01	0.85	0.0	0.0	2100	2100	2100	0.96	0.99	1	0.06	N/A	N/A	N/A	Above Water Table
39.5	35	37	36	50	120		1.3	1.05	1.3	0.94	1	83.7	83.7	4320	4258	4320	0.90	0.92	0.79	2.00	1.45	0.47	3.12	Non-Liquefiable
39.5	37	42	39.5	81	120		1.3	1.05	1.3	1.20	1	172.1	172.1	4740	4459	4740	0.88	0.92	0.78	2.00	1.43	0.48	2.97	Non-Liquefiable
44.5	42	47	44.5	50	120		1.3	1.05	1.3	0.92	1	81.6	81.6	5340	4747	5340	0.86	0.92	0.76	2.00	1.40	0.50	2.80	Non-Liquefiable
49.5	47	50	48.5	50	120		1.3	1.05	1.3	0.91	1	80.7	80.7	5820	4978	5820	0.85	0.92	0.75	2.00	1.37	0.51	2.70	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	Irwindale, CA
Project Number	18G145
Engineer	DWN

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60-CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter 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)			
15.5	0	35	17.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	35.00		0.000		0.00	Above Water Table
39.5	35	37	36	83.7	0.0	83.7	3.12	0.00	-4.54	0.00	2.00		0.000		0.00	Non-Liquefiable
39.5	37	42	39.5	172.1	0.0	172.1	2.97	0.00	-13.29	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	81.6	0.0	81.6	2.80	0.00	-4.34	0.00	5.00		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	0.787 (g)
	Irwindale, CA	Design Magnitude	7.71
Project Number	18G145	Historic High Depth to Groundwater	35 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-2		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.71)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
15.5	0	35	17.5		120		1.3	1.05	1.1	1.01	0.85	0.0	0.0	2100	2100	2100	0.96	0.99	1	0.06	N/A	N/A	N/A	Above Water Table
39.5	35	37	36	50	120		1.3	1.05	1.3	0.94	1	83.7	83.7	4320	4258	4320	0.90	0.92	0.79	2.00	1.45	0.47	3.12	Non-Liquefiable
39.5	37	42	39.5	50	120		1.3	1.05	1.3	0.93	1	82.8	82.8	4740	4459	4740	0.88	0.92	0.78	2.00	1.43	0.48	2.97	Non-Liquefiable
44.5	42	47	44.5	50	120		1.3	1.05	1.3	0.92	1	81.6	81.6	5340	4747	5340	0.86	0.92	0.76	2.00	1.40	0.50	2.80	Non-Liquefiable
49.5	47	50	48.5	50	120		1.3	1.05	1.3	0.91	1	80.7	80.7	5820	4978	5820	0.85	0.92	0.75	2.00	1.37	0.51	2.70	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	Irwindale, CA
Project Number	18G145
Engineer	DWN

Boring No. B-2

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)			
15.5	0	35	17.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	35.00		0.000		0.00	Above Water Table
39.5	35	37	36	83.7	0.0	83.7	3.12	0.00	-4.54	0.00	2.00		0.000		0.00	Non-Liquefiable
39.5	37	42	39.5	82.8	0.0	82.8	2.97	0.00	-4.46	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	81.6	0.0	81.6	2.80	0.00	-4.34	0.00	5.00		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	0.787 (g)
	Irwindale, CA	Design Magnitude	7.71
Project Number	18G145	Historic High Depth to Groundwater	35 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-3		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.71)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
15.5	0	35	17.5		120		1.3	1.05	1.1	1.01	0.85	0.0	0.0	2100	2100	2100	0.96	0.99	1	0.06	N/A	N/A	N/A	Above Water Table
39.5	35	37	36	50	120		1.3	1.05	1.3	0.94	1	83.7	83.7	4320	4258	4320	0.90	0.92	0.79	2.00	1.45	0.47	3.12	Non-Liquefiable
39.5	37	42	39.5	50	120		1.3	1.05	1.3	0.93	1	82.8	82.8	4740	4459	4740	0.88	0.92	0.78	2.00	1.43	0.48	2.97	Non-Liquefiable
44.5	42	47	44.5	50	120		1.3	1.05	1.3	0.92	1	81.6	81.6	5340	4747	5340	0.86	0.92	0.76	2.00	1.40	0.50	2.80	Non-Liquefiable
49.5	47	50	48.5	50	120		1.3	1.05	1.3	0.91	1	80.7	80.7	5820	4978	5820	0.85	0.92	0.75	2.00	1.37	0.51	2.70	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	Irwindale, CA
Project Number	18G145
Engineer	DWN

Boring No. B-3

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	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)			
15.5	0	35	17.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	35.00		0.000		0.00	Above Water Table
39.5	35	37	36	83.7	0.0	83.7	3.12	0.00	-4.54	0.00	2.00		0.000		0.00	Non-Liquefiable
39.5	37	42	39.5	82.8	0.0	82.8	2.97	0.00	-4.46	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	81.6	0.0	81.6	2.80	0.00	-4.34	0.00	5.00		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)