Technical Appendix E1

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION AND LIQUEFACTION EVALUATION PROPOSED DORADO LOGISTICS CENTER

NEC of Perris Boulevard and Modular Way
Moreno Valley, California
for
Trammell Crow Company

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

October 3, 2012

Trammell Crow Company
3501 Jamboree Road, Suite 230
Newport Beach, California 92660

Attention: Mr. David Drake
Senior Vice President

Project No.: **12G189-1**

Subject: **Geotechnical Investigation and Liquefaction Evaluation**
Proposed Dorado Logistics Center
NEC Perris Boulevard and Modular Way
Moreno Valley, California

Gentlemen:

In accordance with your request, we have conducted a geotechnical investigation and liquefaction evaluation 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,

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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

- Extensive demolition will be required in the western portion of the site. The existing building and canopy will require demolition and removal. In general, the existing pavements are in good condition, however, they will not be reused with the proposed development. Debris resulting from demolition should be disposed of in accordance with all applicable federal, state and local specifications and regulations. Alternatively, concrete demolition debris may be pulverized to a maximum 2 inch particle size for later use as structural fill.
- Initial site preparation should include stripping of any surficial vegetation from the site. At the time of our investigation, ground surface cover within the eastern half of the site consisted of sparse grass and weed growth.
- Fill soils were encountered at the boring locations surrounding the existing buildings. The fill soils encountered extend to depths of 2½ to 9± feet. In addition, based on information obtained from the previous soils report, these fill soils may have been placed during grading operations for the current development.
- The near surface soils encountered on the eastern-half of the site consist of potentially collapsible and compressible native alluvial soils and are not considered to be suitable to support the foundation loads of the new structure.
- Remedial grading is recommended to be performed within the new building pad area to remove the near surface soils disturbed during demolition, and the low strength alluvium. The existing soils located within the developed portion of the site (western-half) should be removed to a depth of 3 feet below existing grade and to a depth of 3 feet below pad grade. The eastern-half of the site, beginning with the existing detention basin, should be overexcavated to a depth of 5 feet below existing grade and to a depth of 5 feet below proposed pad grade. In addition, the soils within the proposed foundation influence zones should be overexcavated to a depth of 3 feet below proposed foundation bearing grade.
- After the overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated, moisture conditioned, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.
- The new parking area subgrade soils are recommended to be scarified to a depth of 12± inches, thoroughly moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Liquefaction

- Based on the results of laboratory testing and the liquefaction analysis, the on-site soils are not considered to be susceptible to liquefaction. Therefore, liquefaction is not considered to be a design concern for this project.

Building Foundations
- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,500 psf maximum allowable soil bearing pressure.
- Reinforcement consisting of at least four (4) No. 5 rebars (2 top and 2 bottom) in strip footings due to the presence of potentially expansive soils. Additional reinforcement may be necessary for structural considerations.
- Laboratory testing indicates that the on-site soils possess concentrations of soluble sulfates classified as moderate with respect to potential for concrete attack. In accordance with ACI Publication 318 requirements, it is recommended that all concrete which contacts the on-site soils incorporate the following characteristics:
    - Cement Type II (Two)
    - Minimum compressive Strength (f'_c) = 4,000 lbs/in^2
    - Maximum Water/Cement Ratio: 0.50

Building Floor Slab
- Conventional Slab-on-Grade, 6 inches thick.
- Reinforcement consisting of No. 4 bars at 24 inches on-center, in both directions, due to the presence of expansive soils. The actual floor slab reinforcement to be determined by the structural engineer.

Flatwork and Sidewalks
- Minimum slab thickness: 5 inches
- Minimum slab reinforcement: No. 3 bars at 18 inches on center, in both directions.
- Slab edges to be thickened to 12 inches where adjacent to landscape areas. The thickened edge should contain longitudinal reinforcement consisting of least two (2) No. 4 Bars.
- Presaturation of subgrade soils to at least 120% of optimum to a depth of at least 18 inches prior to concrete placement.

Pavements

ASPHALT PAVEMENTS (R = 20)					
Materials	**Thickness (inches)**				
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Light Truck Traffic (TI = 6.0)	Moderate Truck Traffic (TI = 7.0)	Heavy Truck Traffic (TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	5	8	10	12	14
Compacted Subgrade (90% minimum compaction)	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Light Truck Traffic (TI =6.0)	Moderate Truck Traffic (TI = 7.0)	Heavy Truck Traffic (TI = 8.0)
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. 12P308, dated August 27, 2012. 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. Based on the location of the subject site, this investigation also included a site-specific liquefaction evaluation. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the northeast corner of Perris Boulevard and Modular Way in Moreno Valley, California. The site is bounded to the north by a vacant lot and an auto towing facility, to the east by Kitching Street, to the south by Modular Way, and to the west by Perris Boulevard. The general location of the site is illustrated on the Site Location Map included as Plate 1 in Appendix A of this report.

The overall site is a rectangular-shaped parcel, 51.7± acres in size. The western-half of the site is currently developed as a manufacturing facility for the El Dorado Stone company. A metal frame structure, 275± feet by 450± feet in size, is located in the northwest region of the site. The metal frame structure appears to be used for distribution of the El Dorado Stone products. A two-story tilt-up office building, 75± feet by 120± feet in size, is located in the southwest region of the site. Both structures are assumed to be supported on shallow foundations and slab-on-grade floors. Ground surface cover surrounding the existing buildings generally consists of Portland cement concrete pavements. Three (3) isolated areas of crushed aggregate base are located in the central region of the overall site. These areas of crushed aggregate base appear to be used for storage of stone veneer and/or automobile parking. Numerous pallets of stone veneer are located throughout the western half of the site. The eastern half of the site is developed with a detention basin, measuring 420± feet by 615± feet in size, located on the western portion of the eastern half. The detention basin ranges in depth from 7 to 8± feet. The remainder of the eastern half is vacant and undeveloped. Ground surface cover consists of sparse native grass and weed growth.

Topographic information was obtained from an ALTA survey prepared by Albert A. Webb Associates. Within the western half of the site, site topography slopes to the east, at a gradient of 1 to 1.5± percent, toward the detention basin. The eastern half of the site, slopes gently to the southeast, at a gradient of less than 1 percent. With the exception of the detention basin, there is 10± feet of elevation differential across the entire site.

3.2 Proposed Development

Based on a conceptual site plan provided by the client, the site will be developed with one (1) new industrial building, 1,130,000± ft^2 in size. Loading docks will be constructed in a cross-dock configuration on the north and south sides of the building. The new structure will be surrounded by Portland cement concrete pavements in the loading dock/trailer truck courtyard areas and asphaltic concrete pavements in the automobile parking and drive lane areas. Several landscape planters and concrete flatwork are expected to be constructed throughout the site.

Detailed structural information has not been provided. It is assumed that the new building will be a single story structure of concrete tilt-up construction, presumable supported on a shallow foundation system and a slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 120 kips and 3 to 5 kips per linear foot, respectively.

No significant amounts of below grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. Based on the site topography, cuts and fills of 3 to 5± feet are expected to be necessary to achieve the proposed site grades.

3.3 Previous Studies

We were provided with a previous geotechnical report prepared by LOR Geotechnical Group, Inc., entitled Preliminary Geotechnical Investigation, 56± Acre Parcel, SE of Perris Boulevard and Edwin Road, Moreno Valley, California; dated May 24, 1999. This reports presented remedial grading recommendations for the existing buildings at the subject site. The report recommended the removal of soils on the order of 10 to 15 feet below the building areas. This report also recommended the removal of soils within all improvements outside of the building areas to a depth of 5± feet. The excavations were recommended to be backfilled with engineered compacted fill.

It should be noted that we have not been provided with any documentation indicating that the existing development was prepared in accordance with the LOR geotechnical report.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of eleven (11) borings advanced to depths of 5 to 50± feet below currently existing site grades. All of the borings were logged during drilling by a member of our staff.

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

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

4.2 Geotechnical Conditions

Pavements

Existing pavements were encountered at the ground surface at Boring Nos. B-1, B-2, and B-3. These pavements generally consist of 5 to 7± inches of Portland cement concrete with no discernible underlying aggregate base.

Aggregate Base

Boring Nos. B-4, B-5, and B-6 were excavated through a surficial layer of aggregate base. The aggregate base was approximately 2 to 3± inches thick.

Artificial Fill

Artificial fill soils were encountered beneath the existing pavements and aggregate base materials at Boring Nos. B-1 through B-6. These fill soils extend to depths of 2½ to 9± feet below existing grade. The fill materials generally consist of medium stiff to very stiff, mottled, sandy clays and medium dense sandy silts. The fill soils possess variable strengths and a disturbed appearance, resulting in their classification as fill.

Alluvium

Native alluvial soils were encountered beneath the fill materials at Boring Nos. B-1 through B-6, and at the ground surface at Boring Nos. B-7 through B-11. The native alluvial soils encountered generally consist of interbeded layers of stiff to hard clayey silts, sandy clays and loose to medium dense sandy silts, silty sands, and clayey sands. These alluvial soils generally possess moderate to extensive calcareous deposits and some cementation. As discussed in a subsequent section of this report, some of these alluvial soils also possess a medium expansive potential. Native alluvial soils extend to the maximum depth explored of 50± feet.

Groundwater

No free water was encountered during the drilling of Boring Nos. B-1 through B-6 and Boring Nos. B-8 through B-11. These borings caved at depths ranging from 2 to 20± feet and did not contain any free water at the time of completion.

Free water was observed at a depth of 25 feet within Boring No. B-7 at the time of boring completion. Based on this water level reading, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth of approximately 25± feet at the time of our subsurface exploration.

As part of our research, we reviewed historic groundwater levels obtained from the State Water Resources Control Board website, www.geotracker.waterboards.ca.gov. No data was available in the vicinity of the site. The nearest monitoring well was located approximately 1½± miles north of the site. Water level readings within this environmental monitoring well indicate groundwater levels of 48 to 54± 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. The field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring Logs and are periodically referenced throughout this report.

In-situ Density and Moisture Content

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

Consolidation

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

Maximum Dry Density and Optimum Moisture Content

One representative bulk sample has been tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. 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. The results of this test are plotted on Plate C-9 in Appendix C of this report.

Direct Shear

A direct shear test was performed on a remolded soil sample to determine its shear strength parameters. The 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 test are presented on Plate C-10.

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 discussed further in a subsequent section of this report:

Sample Identification	**Soluble Sulfates (%)**	**Sulfate Classification**
B-1 @ 0 to 5 feet	0.011	Negligible
B-4 @ 0 to 5 feet	0.104	Moderate
B-7 @ 0 to 5 feet	0.033	Negligible

Expansion Index

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

Sample Identification	**Expansion Index**	**Expansive Potential**
B-4 @ 0 to 5 feet	26	Low
B-7 @ 0 to 5 feet	55	Medium

Grain Size Analysis

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

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on several representative samples of soil. This test is used to determine the Liquid Limit and Plastic Limit of the soil. The Plasticity Index is the difference between the two limits. Plasticity Index is a general indicator of the expansive potential of the soil, with higher numbers indicating higher expansive potential. Soils with a PI greater than 25 are considered to have a high plasticity, and a high expansion potential. The results of the Atterberg Limits testing are presented on the test boring logs.

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

6.1 Seismic Design Considerations

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

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

Based on standards in place at the time of this report, the proposed development must be designed in accordance with the requirements of the 2010 California Building Code (CBC).

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 2010 CBC Seismic Design Parameters have been generated using Earthquake Ground Motion Parameters, a 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 2010 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:

2010 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.500
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.600
Site Class	---	D
Short-Period Site Coefficient at 0.2 sec Period	F_a	1.0
Long-Period Site Coefficient at 1.0 sec Period	F_v	1.5
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.500
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.900
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.000
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.600

Liquefaction

The California Geological Survey (CGS) has not yet conducted detailed seismic hazards mapping in the area of the subject site. The general liquefaction susceptibility of the site was determined by research of the Riverside County GIS website. Research of the Riverside County GIS website indicates that the subject site is located within a zone of moderate liquefaction susceptibility (deep groundwater). Therefore, one of the borings at the site was extended to a depth of 50± feet.

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 grain size 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). Clayey (cohesive) soils or soils which possess clay particles ($d<0.005$mm) in excess of 20 percent (Seed and Idriss, 1982) 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 originally developed by Seed, et al. (Seed and Idriss 1971). 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). The current version of a generally accepted baseline chart (Youd and Idriss, 1997) is used to determine CRR as a function of the corrected SPT N-value $(N_1)_{60}$. 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.

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-7, which was advanced to a depth of 50± feet. Boring No. B-7 was performed within the footprint of proposed building. The liquefaction potential was analyzed utilizing a design peak ground acceleration (PGA) of 0.40g for a magnitude 6.95 seismic event. The design PGA was obtained in accordance with the 2010 CBC and ASCE 7-05 and is equal to S_{DS} divided by 2.5. The analysis was performed using a historic groundwater depth of 22 feet, which is expected to be representative of a conservative, historic groundwater elevation at the subject site.

If liquefiable soils are identified, the potential settlements that could occur as a result of liquefaction are determined using the procedure developed by Tokimatsu and Seed (1987). This procedure uses the induced cyclic stress ratio, the corrected N-value and the earthquake magnitude 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 results of the liquefaction analysis generally indicate that the subsurface soils at the subject site are not susceptible to liquefaction. These soils possess factors of safety against liquefaction in excess of 1.3 and are not considered susceptible to liquefaction. In addition, the subsurface profiles at boring locations generally consist of very stiff sandy clays. These clayey soils are considered non-liquefiable due to their fine grained, cohesive, characteristics and the results of the Atterberg limits testing with respect to the requirements of Special Publication 117A. Based on laboratory results and subsequent analysis, liquefaction is not considered to be a design concern for this project.

6.2 Geotechnical Design Considerations

General

Artificial fill soils were encountered at the ground surface within the developed portion of the site (western-half). These fill soils extend to depths of 2½± to 9± feet below existing grade and appear to have been placed during construction of the existing buildings. Based on the conditions encountered at the boring locations, subsequent laboratory testing, and on the relatively recent age of construction, it is expected that these soils were placed as compacted fill. Therefore, these existing soils are expected to be suitable for support of the intended structure. However, these areas will require limited remedial grading as a result of the expected demolition activities.

The eastern half of the site encountered native alluvial soils at the ground surface. These alluvial soils consist of stiff to very stiff clayey silts, medium dense fine sandy clays, and possess extensive calcareous deposits, some cementation, and are potentially collapsible when exposed to moisture infiltration. These soils also possess a potential for moderate consolidation when exposed to load increases in the range of those that will be exerted by the foundations of the new structure. Furthermore, these soils exhibit a medium expansive potential.

Therefore, remedial grading is considered warranted within the proposed building area in order to remove and replace the near surface soils as compacted structural fill, as they are not considered suitable for support of the proposed structure.

Settlement

The recommended remedial grading will remove the disturbed existing fill soils as well as some of the native alluvium, and replace these materials as compacted structural fill. The native soils and existing fill soils that will remain in place below the recommended depth of overexcavation will not be subject to significant load increases from the foundations of the new structure. Provided that the recommended remedial grading is completed, the post-construction static settlement of the proposed structure is expected to be within tolerable limits.

Expansion

The near surface soils at this site generally consist of silty clays, clayey silts and sandy clays. Laboratory testing indicates that these materials have a low to medium expansion potential (EI = 26 and EI = 55). Based on the presence of expansive soils, special care should be taken to properly moisture condition and maintain adequate moisture content within all subgrade soils as well as newly placed fill soils. The foundation and floor slab design recommendations contained within this report are made in consideration of the expansion index test results. It is recommended that additional expansion index testing be conducted at the completion of rough grading to verify the expansion potential of the as-graded building pad.

Soluble Sulfates

The results of the soluble sulfate testing indicate that the selected samples of the on-site soils contain levels of soluble sulfates that are classified as having a moderate potential to attack concrete, in accordance with the American Concrete Institute (ACI) Publication 318-05 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, it is recommended that a sulfate-resistant concrete mix design be utilized for the foundations and floor slabs at this site. In accordance with the ACI 318 requirements, it is recommended that this concrete incorporate the following characteristics:

- Cement Type: II (Two)
- Minimum Compressive Strength (f'_c) = 4,000 lbs/in^2
- Maximum Water/Cement Ratio: 0.50

It is recommended that additional sulfate testing be performed at the completion of rough grading to verify the concentrations which are present in the actual building pad subgrade soils.

Shrinkage/Subsidence

Removal and recompaction of the near surface soils is estimated to result in an average shrinkage of 12 to 16 percent. 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

Detailed grading and foundation plans were not available at the time of this report. It is therefore recommended that we be provided with copies of the preliminary 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

Initial site preparation should include stripping of any vegetation and organic debris. These materials should be disposed of off-site. The actual extent of site stripping should be determined during grading by a representative of the geotechnical engineer, based on the organic content of the encountered materials.

Extensive demolition of the existing structures and surrounding improvements will be required at this site. Demolition of the existing structures should include all foundations, floor slabs, and any associated utilities. The existing pavements at the site are not expected to be reused with the proposed development.

All remnants of the previous structure, including foundations, floor slabs, and debris resulting from demolition activities should be properly disposed of off-site. Concrete and asphalt debris may be re-used within the compacted fills, provided they are pulverized and the maximum particle size is less than 2 inches.

The aggregate base material may be utilized as the base course for flexible pavements provided that the material complies with the appropriate specifications for crushed miscellaneous base

(CMB) contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction. Alternatively, the base material may be utilized as structural fill.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the western-half of the site in order to remove the near surface fill soils disturbed during demolition of the existing buildings and foundations. Within this portion of the site, the existing fill soils should be overexcavated to a depth of 3 feet below existing grade and 3 feet below proposed pad grade, whichever is greater.

In addition, remedial grading is required within the eastern-half of the site, beginning with the detention basin, in order to remove the near-surface, low-strength native soils. Within this portion of the site, it is recommended that the existing soils be overexcavated to a depth of 5 feet below existing grade and 5 feet below proposed pad grade.

Where not encompassed within the general building pad overexcavations, additional overexcavation should be performed within the influence zones of the new foundations, extending to a depth of 3 feet below proposed bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building perimeters. If the proposed structures incorporate any exterior columns (such as for a canopy or overhang) the 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 unsuitable fill materials or 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, moisture treated to 2 to 4 percent above optimum moisture content, and compacted. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining 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.

The foundation areas for non-retaining site walls should be overexcavated to a depth of 1 foot below proposed foundation bearing grade. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Parking Areas

Based on economic considerations, overexcavation of the existing fill 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 at least 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed parking 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 undocumented fill soils or collapsible native alluvium in the parking areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking area should be graded in a manner similar to that described for the building area.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2010 CBC and the grading code of the city of Moreno Valley.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

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

Utility Trench Backfill

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

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

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of silty clays, clayey silts, sandy clays, and sandy silts. Some of these materials may be subject to minor caving within shallow excavations. Where caving does occur, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 1.5h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

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

Expansive Soils

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

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

Groundwater

Boring No B-7 encountered ground water at a depth of 25± feet below ground surface. Based on the anticipated depth to groundwater, it is not expected that the groundwater will affect excavations for the foundations or utilities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by structural fill soils used to replace a portion of the existing fill soils and low strength, potentially collapsible, native soils. The new structural fill soils are expected to extend to a depth of at least 3 feet below foundation bearing grade. Based on this subsurface profile, the proposed structure may be supported on a shallow foundation system.

Building 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: Four (4) No. 5 rebars (2 top and 2 bottom), due to the presence of medium expansive soils.
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 24 inches below adjacent grade.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. Additional rigidity may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Within the new building area, soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill or competent native alluvial soils, 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 at least 2 to 4 percent of 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 movements (settlement and/or heave) of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.5 and 0.75 inches, respectively. Differential movements are expected to occur over a 50-foot span, thereby resulting in an angular distortion of less than 0.003 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: 225 lbs/ft^3
- Friction Coefficient: 0.25

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against suitable compacted structural fill. The maximum allowable passive pressure is 2000 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 floor of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.

- Minimum slab reinforcement: Reinforcement consisting of No. 4 bars at 24 inches on-center, in both directions, due to the presence of expansive soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed loading.

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

- Moisture condition the floor slab subgrade soils to 2 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

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

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

6.7 Exterior Flatwork Design and Construction

Subgrades which will support new exterior slabs-on-grade for patios, sidewalks and driveways should be prepared in accordance with the recommendations contained in the ***Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, exterior flatwork will be underlain by compacted fill soils extending to at least 12 inches below proposed grade. Based on geotechnical considerations, exterior slabs on grade may be designed as follows:

- Minimum slab thickness: 5 inches

- Slab edges to be thickened to 12 inches where adjacent to landscape areas. The thickened edge should contain longitudinal reinforcement consisting of least two (2) No. 4 Bars.

- Minimum slab reinforcement: No. 3 bars at 18 inches on center, in both directions.
- Presaturation of subgrade soils to at least 120% of optimum to a depth of at least 18 inches 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.
- Control joints should be provided at a maximum spacing of 8 feet on center in two directions for slabs and at 4 feet on center for sidewalks. Control joints are intended to direct cracking. **Minor to moderate cracking and/or movement of exterior concrete slabs on grade should be expected.** If such cracking is not considered acceptable, the flatwork areas should be overexcavated to a depth of 24 inches below finished grade, to allow for placement of a new layer of very low expansive (EI < 20) structural fill.
- Expansion or felt joints should be used at the interface of exterior slabs on grade and any fixed structures to permit relative movement.

6.8 Retaining Wall Design and Construction

Although not indicated on the site plan, the proposed development may require some small retaining walls (less than 3 to 5± feet in height) to facilitate the new site grades.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters for two different types of wall backfill: on-site soils and imported aggregate base material. The on-site soils generally consist of clayey silts, sandy clays, clayey sands, and sandy silts. **The expansive clayey soils should not be used for retaining wall backfill.** Based on the results of direct shear testing, these on-site soils possess a friction angle of 22 degrees.

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 base of the retaining wall upwards at an angle of approximately 60 degrees from the heel of the retaining wall.

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type	
		Imported Aggregate Base	On-Site Soils
Internal Friction Angle (ϕ)		38°	22°
Unit Weight		130 lbs/ft^3	120 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	30 lbs/ft^3	55 lbs/ft^3
	Active Condition (2h:1v backfill)	44 lbs/ft^3	Not Recommended*
	At-Rest Condition (level backfill)	50 lbs/ft^3	75 lbs/ft^3

*It is recommended that the on-site soils not be used for wall backfill material for an active 2h:1v backfill condition.

Regardless of the backfill type, the walls should be designed using a soil-footing coefficient of friction of 0.25 and an equivalent passive pressure of 225 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 addition to the lateral earth pressures presented in the previous section, the 2010 CBC requires that for structures assigned to Seismic Design Categories D through F, retaining walls should be designed for lateral earth pressures due to earthquake motion. The recommended seismic pressure distribution is triangular in shape, with a maximum magnitude of 23H lbs/ft^2, where H is the overall height of the wall. The maximum pressure should be assumed to occur at the top of the wall, decreasing to 0 at the base of the wall. The seismic pressure distribution is based on the Mononobe-Okabe equation, utilizing a peak ground acceleration of 0.40g. This peak site acceleration is equal to $S_{DS}/2.5$, in accordance with the 2010 CBC.

Retaining Wall Foundation Design

The foundation subgrade soils for the new retaining should be prepared in accordance with the grading recommendations presented in Section 6.3 of this report. The foundations should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

It is recommended that a minimum 1-foot thick layer of free-draining granular material (less than 5 percent passing the No. 200 sieve) should be placed against the face of the retaining walls. This material should be approved by the geotechnical engineer. The layer of free draining granular material should be separated from the backfill soils using an approved geotextile. If the layer of free-draining material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557-91). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

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

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should include a one cubic foot gravel pocket surrounded by a suitable geotextile at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system.

6.9 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The on-site soils generally consist of silty clays, clayey silts, sandy clays, and sandy silts.

These soils are considered to possess fair pavement support characteristics with an estimated R-values of 20. The subsequent pavement design is based upon an assumed R-value of 20. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that 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

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

ASPHALT PAVEMENTS (R = 20)					
Materials	**Thickness (inches)**				
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Light Truck Traffic (TI = 6.0)	Moderate Truck Traffic (TI = 7.0)	Heavy Truck Traffic (TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	5	8	10	12	14
Compacted Subgrade (90% minimum compaction)	12	12	12	12	12

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

Portland Cement Concrete

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

PORTLAND CEMENT CONCRETE PAVEMENTS				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Light Truck Traffic (TI =6.0)	Moderate Truck Traffic (TI = 7.0)	Heavy Truck Traffic (TI = 8.0)
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. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness.

7.0 GENERAL COMMENTS

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

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

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

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

8.0 REFERENCES

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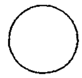
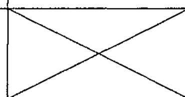
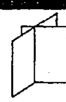
APPENDIX A

SOURCE: RIVERSIDE COUNTY
THOMAS GUIDE, 2009

SITE LOCATION MAP	
PROPOSED COMMERCIAL/INDUSTRIAL BUILDING	
MORENO VALLEY, CALIFORNIA	
SCALE: 1" = 2400'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: DM CHKD: JAS	
SCG PROJECT 12G189-1	
PLATE 1	

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 GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
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.: 12G189
PROJECT: Proposed Dorado Logistics Center
LOCATION: Moreno Valley, California

DRILLING DATE: 9/4/12
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

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

SURFACE ELEVATION: 1471 feet MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					7± inches Portland cement concrete, no discernible Aggregate base							
		12	3.5		FILL: Dark Gray Brown fine Sandy Clay, little Silt, mottled, medium stiff to stiff-moist to very moist	104	20					EI = 26 @ 0 to 5'
		13	4.5+		FILL: Mottled Dark Gray Brown fine to medium Sandy Clay and Silty Clay, stiff-moist to very moist	108	18					
5		66			ALLUVIUM: Brown Silty fine to medium Sand, abundant calcareous deposits, dense-moist	109	11					
		19	4.5+		Brown Clayey Silt, little fine Sand, calcareous deposits, stiff-moist to very moist	102	20					
10		27	4.5+		@ 9 feet, Gray Brown Clayey Silt	106	17					
15		21			Light Brown Clayey Silt to Silty Clay, little fine Sand, slightly porous, calcareous deposits, stiff-damp to moist		16					
20		20			Light Orange Brown Silty fine Sand to fine Sandy Silt, trace Clay, medium dense-damp to moist		13					
25		19			Light Gray Brown fine Sandy Silt, trace Clay, slightly porous, trace calcareous deposits, medium dense-moist to very moist		19					
30		16					24					
					Boring Terminated at 30'							

TBL 12G189.GPJ SOCALGEO.GDT 10/3/12

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL

A California Corporation

BORING NO. B-2

JOB NO.: 12G189
PROJECT: Proposed Dorado Logistics Center
LOCATION: Moreno Valley, California

DRILLING DATE: 9/4/12
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

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

SURFACE ELEVATION: 1469 feet MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					7± inches Portland cement concrete, no discernible Aggregate base							
		16	4.5+		FILL: Dark Brown fine Sandy Clay, little Silt, mottled, stiff-moist	114	11					
		46	4.5+		FILL: Brown fine to medium Sandy Clay, mottled, little Silt, hard-damp to moist	123	9					
5		21			FILL: Dark Gray Brown fine Sandy Silt, trace Clay, mottled, medium dense-damp to moist	123	12					
		22			ALLUVIUM: Brown Silty fine Sand to fine Sandy Silt, slightly porous, medium dense-damp to moist	115	11					
10		11	4.5		Gray Brown Silty Clay, trace calcareous veining, slightly porous, medium stiff-very moist	86	32					
		12			Light Gray Clayey Silt, trace fine Sand, calcareous deposits, slightly porous, stiff-moist to very moist		26					
15												
20		30	4.5+		Brown to Red Brown Silty Clay to Clayey Silt, trace fine Sand, very stiff-damp	120	8					
		10			Light Gray Brown Clayey Silt, trace fine Sand, stiff-moist to very moist		25					
25		19	4.5+			92	23					

Boring Terminated at 26'

TBL 12G189.GPJ SOCALGEO.GDT 10/3/12

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 12G189	DRILLING DATE: 9/4/12	WATER DEPTH:
PROJECT: Proposed Dorado Logistics Center	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 2 feet
LOCATION: Moreno Valley, California	LOGGED BY: Brett Isen	READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS							
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: 1467.5 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					5± inches Portland cement concrete, no discernible Aggregate base							
	X	15			FILL: Dark Gray fine Sandy Clay, mottled, stiff to very stiff-damp to moist		12					
	X	15			ALLUVIUM: Brown fine Sandy Clay, little Silt, trace calcareous veining, stiff to very stiff-damp to moist		14					
5												
					Boring Terminated at 5'							

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 12G189
PROJECT: Proposed Dorado Logistics Center
LOCATION: Moreno Valley, California

DRILLING DATE: 9/4/12
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1466.5 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					2 to 3± inches Aggregate base							
	X	18	4.5+		FILL: Dark Gray Brown fine Sandy Clay, mottled, stiff-damp to moist	109	16					EI = 55 @ 0 to 5'
	X	29	4.5+		ALLUVIUM: Gray Brown fine Sandy Clay, little calcareous deposits, very stiff-damp to moist	110	15					
5	X	14	4.5+		Gray Brown Clayey Silt to Silty Clay, little fine Sand, calcareous deposits, slightly porous, stiff-moist	100	17					
	X	12			Light Gray Brown to Light Brown fine Sandy Silt, abundant calcareous deposits, loose-very moist to wet	56	55					
10	X	11				67	35					
					Red Brown Silty fine Sand, trace calcareous nodules, medium dense-damp to moist							
15	X	15					10					
					Gray Brown Clayey Silt, little fine Sand, calcareous nodules, stiff-damp							
20	X	21					12					
					Boring Terminated at 20'							

TBL 12G189.GPJ SOCALGEO.GDT 10/3/12

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 12G189
PROJECT: Proposed Dorado Logistics Center
LOCATION: Moreno Valley, California
DRILLING DATE: 9/4/12
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen
WATER DEPTH:
CAVE DEPTH: 9 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1470 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					2 to 3± inches Aggregate base							
	X	17			FILL: Gray Brown fine Sandy Clay, little Silt, stiff-damp to very moist		12					
	X	20					20					
5												
	X	25	4.5+		FILL: Dark Gray fine Sandy Clay, very stiff-damp		13					
	X	14	4.5+				15					
10					ALLUVIUM: Brown Silty Clay, trace to little fine Sand, abundant calcareous nodules, stiff-damp							
					Light Brown Clayey Silt, trace fine Sand, abundant calcareous deposits, medium stiff-very moist to wet							
	X	12					33					
15												
					Boring Terminated at 20'							

TBL 12G189.GPJ SOCALGEO.GDT 10/3/12

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 12G189
PROJECT: Proposed Dorado Logistics Center
LOCATION: Moreno Valley, California

DRILLING DATE: 9/4/12
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

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

SURFACE ELEVATION: 1467.5 feet MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					2 to 3± inches Aggregate base							
	X	10			FILL: Dark Gray Brown fine Sandy Clay, mottled, stiff-very moist		26					
5	X	21			ALLUVIUM: Light Gray Brown fine Sandy Clay, trace calcareous nodules, medium dense-moist to very moist		22					
	X	16			Light Gray Brown fine Sandy Silt, trace to little Clay, abundant calcareous deposits, medium dense-moist to very moist		22					
10	X	13					33					
15	X	19			Red Brown Clayey Silt, trace fine Sand, very stiff-damp		10					
20	X	16			Brown Silty fine Sand, trace medium to coarse Sand, medium dense-moist		12					
					Boring Terminated at 20'							

TBL 12G189.GPJ SOCALGEO.GDT 10/3/12

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

A California Corporation

BORING NO. B-7

JOB NO.: 12G189	DRILLING DATE: 9/4/12	WATER DEPTH: 25 feet
PROJECT: Proposed Dorado Logistics Center	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 27 feet
LOCATION: Moreno Valley, California	LOGGED BY: Brett Isen	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1464 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
		18	4.5+		ALLUVIUM: Light Gray Brown Clayey Silt, trace fine root fibers, stiff-damp to moist	74	14					
		37			ALLUVIUM: Light Gray fine Sandy Silt, abundant calcareous deposits, medium dense to dense -damp to moist	83	14					
5		52				86	13					
		39	4.5+		Brown to Light Brown Clayey Silt, trace to little fine Sand, abundant calcareous deposits, slightly porous, very stiff-damp to moist	105	13					
10		36	4.5+			101	8					
		13			Red Brown fine Sandy Clay, little Silt, trace to little calcareous deposits, stiff- damp to moist		11					
15												
		13			Gray Brown Clayey fine Sand, trace Silt, trace calcareous deposits, medium dense-moist		13			32		
20												
		15			Interbedded layers of Gray Brown Clayey fine Sand to fine Sandy Silt, medium dense-very moist to wet		17	27	14	39		
25												
		22					17			44		
30												
		18					18	32	14	46		

TBL 12G189.GPJ SOCALGEO.GDT 10/3/12

SOUTHERN CALIFORNIA GEOTECHNICAL

A California Corporation

BORING NO. B-7

JOB NO.: 12G189
PROJECT: Proposed Dorado Logistics Center
LOCATION: Moreno Valley, California

DRILLING DATE: 9/4/12
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

WATER DEPTH: 25 feet
CAVE DEPTH: 27 feet
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 (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Interbedded layers of Gray Brown Clayey fine Sand to fine Sandy Silt, medium dense-very moist to wet							
40	X	37					14			31		
					Brown Clayey fine Sand, calcareous deposits, very stiff-very moist							
45	X	18					18	34	15	43		
					Gray Silty Clay, calcareous nodules, very stiff-very moist							
50	X	20	2.75				23			70		
					Boring Terminated at 50'							

TBL 12G189.GPJ SOCALGEO.GDT 10/3/12

TEST BORING LOG

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 12G189
PROJECT: Proposed Dorado Logistics Center
LOCATION: Moreno Valley, California

DRILLING DATE: 9/6/12
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

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

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: 1465 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray fine Sandy Clay, trace calcareous deposits, very stiff-damp							
	X	19					12					
	X	15					11					
5					Light Gray Clayey Silt, trace fine Sand, trace calcareous deposits, medium stiff-moist							
	X	5					17					
					Light Gray fine Sandy Clay, abundant calcareous deposits, very stiff-damp to moist							
	X	37					14					
10												
					Red Brown fine to medium Sandy Clay, trace to little Silt, trace calcareous deposits, very stiff-dry to damp							
	X	19					7					
15					Boring Terminated at 15'							

TBL 12G189.GPJ SOCALGEO.GDT 10/3/12

BORING NO. B-9

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 12G189
PROJECT: Proposed Dorado Logistics Center
LOCATION: Moreno Valley, California

DRILLING DATE: 9/6/12
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

WATER DEPTH:
CAVE DEPTH: 2 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: 1464.5 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	16			ALLUVIUM: Light Gray fine Sandy Clay, abundant calcareous deposits, very stiff-damp to moist		14					
	X	11			Light Gray Clayey Silt, abundant calcareous nodules, stiff-damp to moist		13					
5												
					Boring Terminated at 5'							

TBL 12G189.GPJ SOCALGEO.GDT 10/3/12

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-10

JOB NO.: 12G189
PROJECT: Proposed Dorado Logistics Center
LOCATION: Moreno Valley, California
DRILLING DATE: 9/6/12
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen
WATER DEPTH:
CAVE DEPTH: 2 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1464 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	18			ALLUVIUM: Light Gray fine Sandy Clay, trace calcareous nodules, trace fine root fibers, very stiff-damp to moist		13					
5	X	28			Light Gray Clayey Silt, abundant calcareous nodules, very stiff-damp to moist		13					
					Boring Terminated at 5'							

TBL 12G189.GPJ SOCALGEO.GDT 10/3/12

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-11

JOB NO.: 12G189
PROJECT: Proposed Dorado Logistics Center
LOCATION: Moreno Valley, California
DRILLING DATE: 9/6/12
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen
WATER DEPTH:
CAVE DEPTH: 15 feet
READING TAKEN: At Completion

SURFACE ELEVATION: 1465 feet MSL

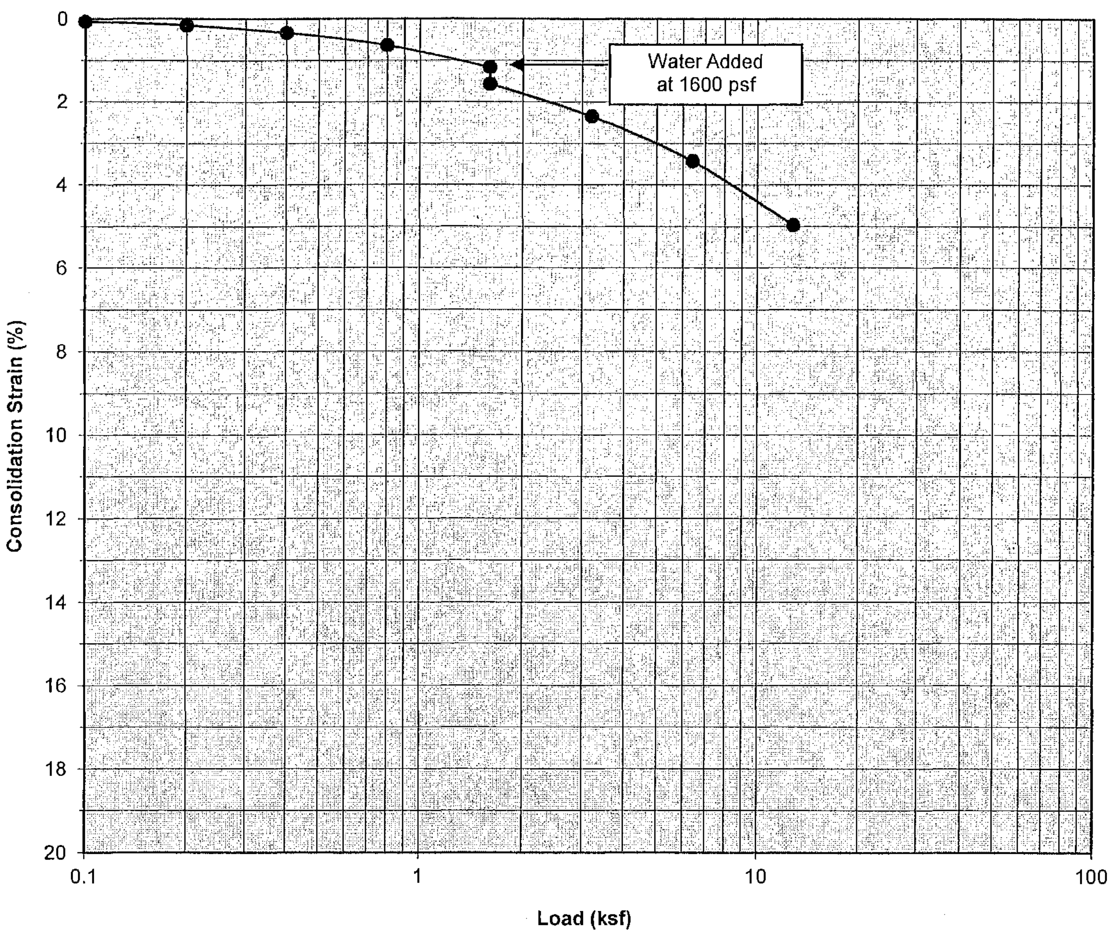
FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray fine Sandy Clay, trace calcareous nodules, hard-damp to moist							
	X	61	3.5			78	14					
					Light Brown Silty Clay to Clayey Silt, abundant calcareous deposits, stiff-moist							
	X	16	4.5+			73	17					
					Light Brown Clayey Silt, abundant calcareous deposits, stiff-moist to very moist							
5	X	21	4.5+			72	21					
	X	9	4.5+			67	19					
					Brown fine Sandy Clay, abundant calcareous deposits, porous, hard-damp							
	X	50	4.5+			104	9					
10												
					Red Brown Clayey fine Sand, medium dense-damp to moist							
	X	21				112	15					
15												
					Gray Brown Silty fine Sand, trace Clay, medium dense-moist							
	X	16					11					
20												
					Gray Brown Clayey fine Sand, medium dense-moist to very moist							
	X	13					16					
25					Boring Terminated at 25'							

TBL 12G189.GPJ SOCALGEO.GDT 10/3/12

TEST BORING LOG

APPENDIX C

Consolidation/Collapse Test Results



Classification: ALLUVIUM: Gray Brown fine Sandy Clay

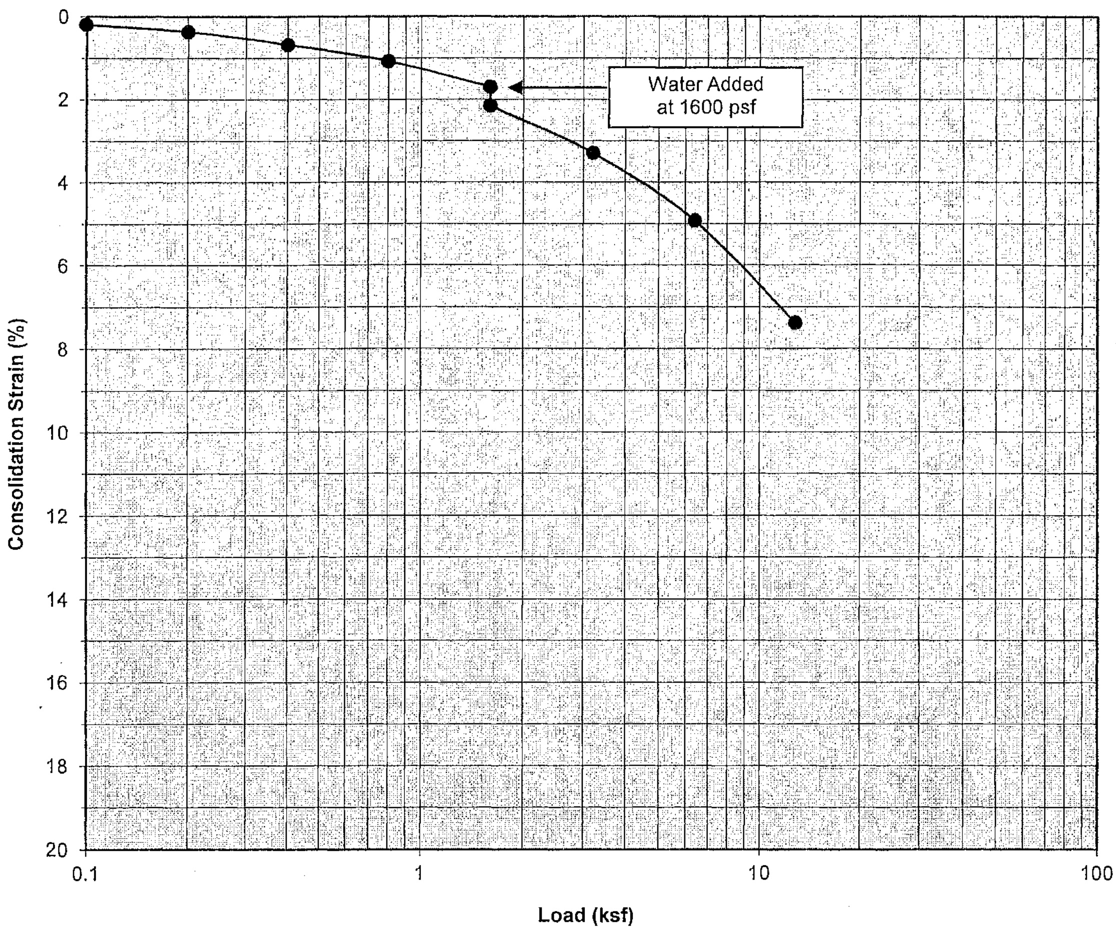
Boring Number:	B-4	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	3 to 4	Initial Dry Density (pcf)	109.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.40

Proposed Dorado Logistics Center
Moreno Valley, California
Project No. 12G189

PLATE C- 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: ALLUVIUM: Gray Brown Clayey Silt to Silty Clay, little fine Sand

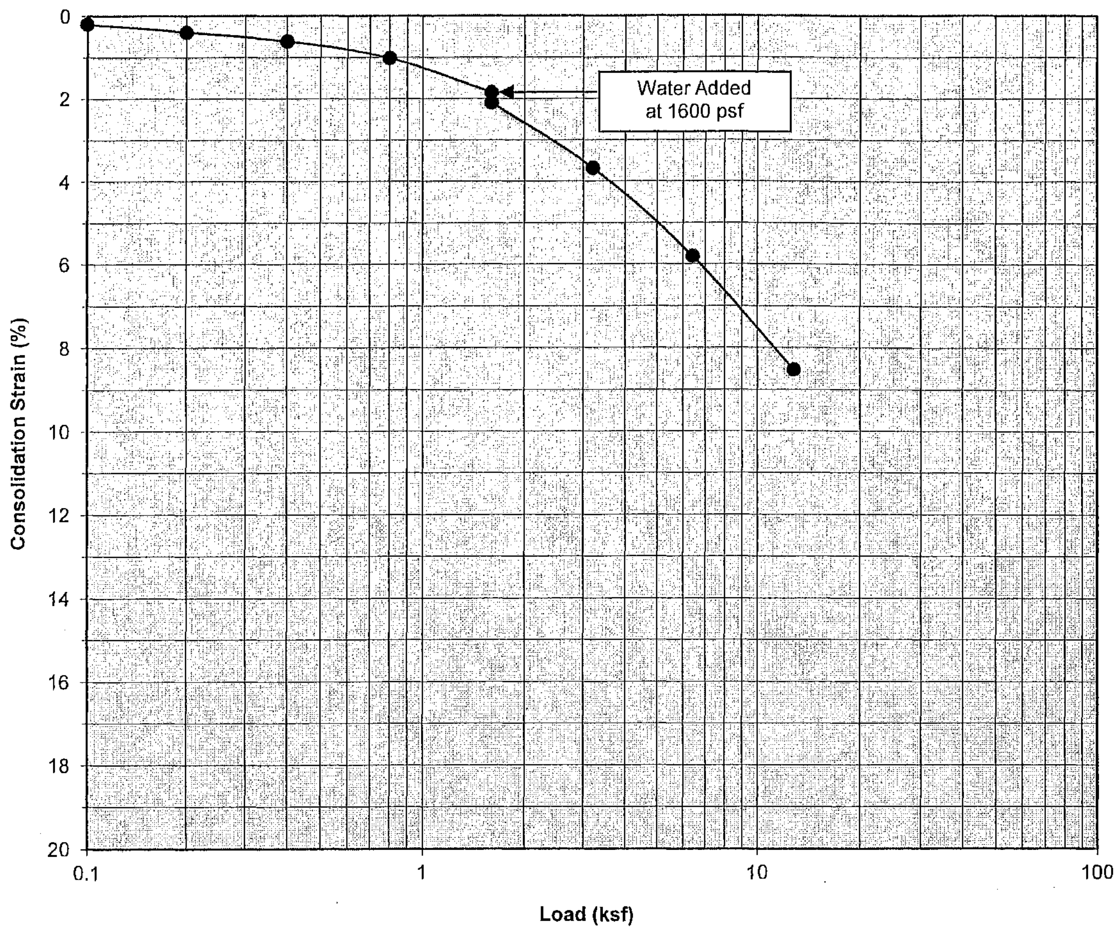
Boring Number:	B-4	Initial Moisture Content (%)	18
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	5 to 6	Initial Dry Density (pcf)	101.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.46

Proposed Dorado Logistics Center
Moreno Valley, California
Project No. 12G189

PLATE C- 2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



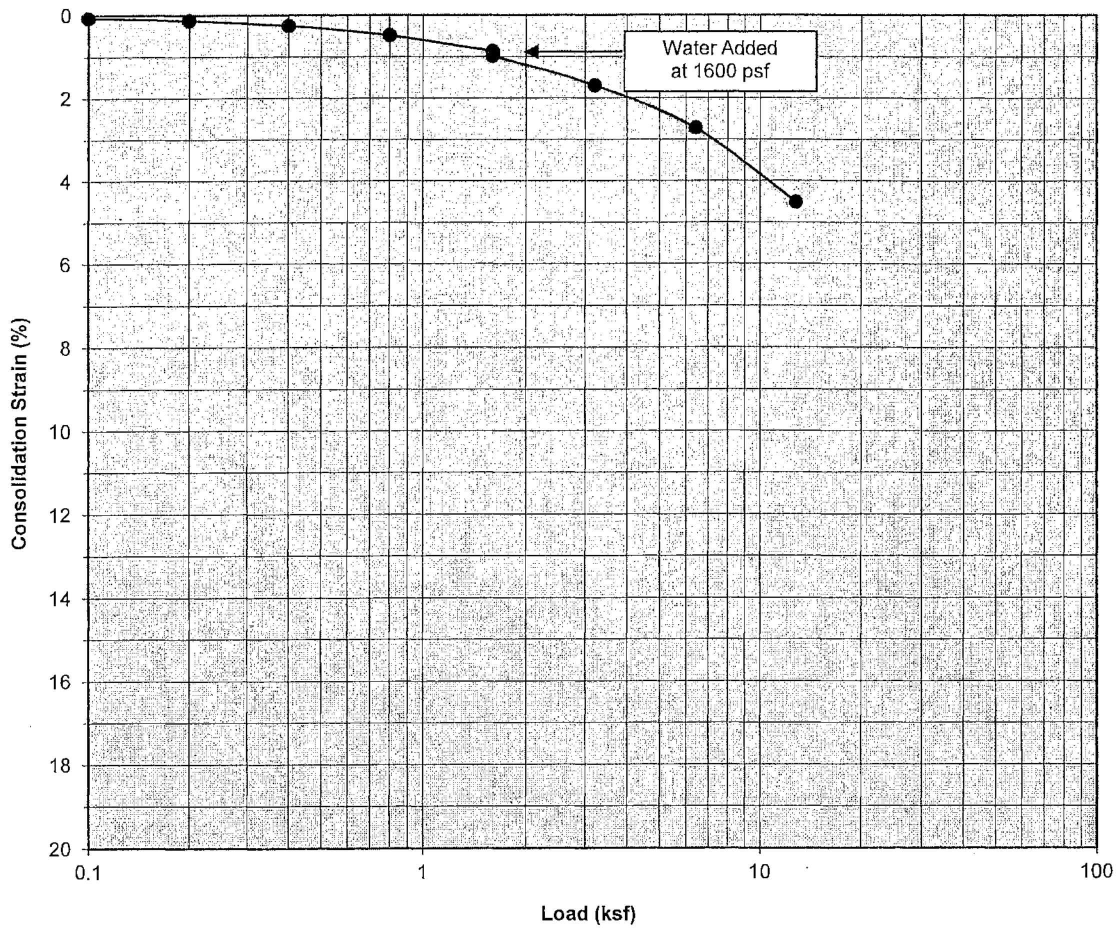
Classification: ALLUVIUM: Light Gray Brown to Light Brown Silty fine Sand

Boring Number:	B-4	Initial Moisture Content (%)	56
Sample Number:	---	Final Moisture Content (%)	76
Depth (ft)	7 to 8	Initial Dry Density (pcf)	56.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	61.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.25

Proposed Dorado Logistics Center
Moreno Valley, California
Project No. 12G189
PLATE C- 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

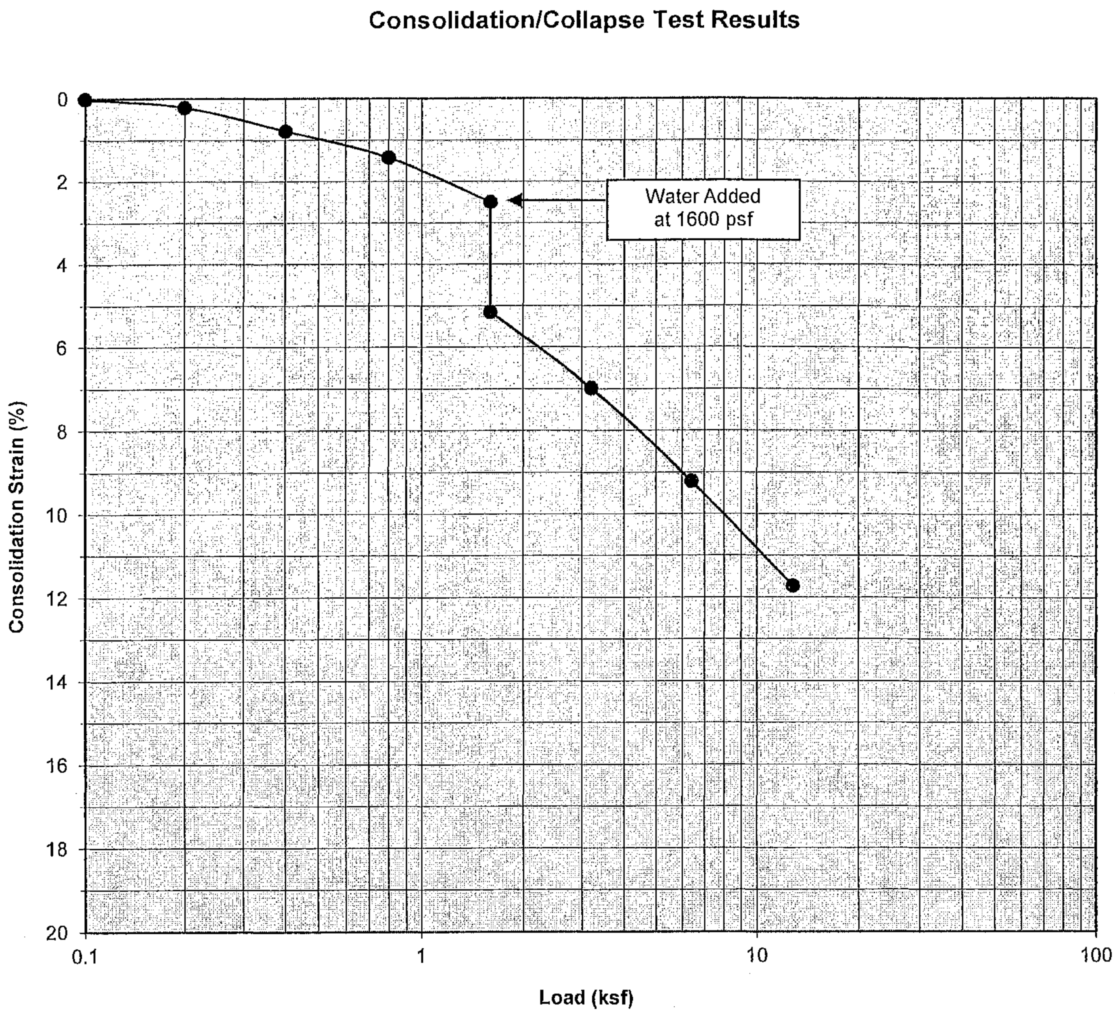


Classification: ALLUVIUM: Light Gray Brown to Light Brown fine Sandy Silt

Boring Number:	B-4	Initial Moisture Content (%)	35
Sample Number:	---	Final Moisture Content (%)	39
Depth (ft)	9 to 10	Initial Dry Density (pcf)	67.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	70.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.12

Proposed Dorado Logistics Center
Moreno Valley, California
Project No. 12G189
PLATE C- 4

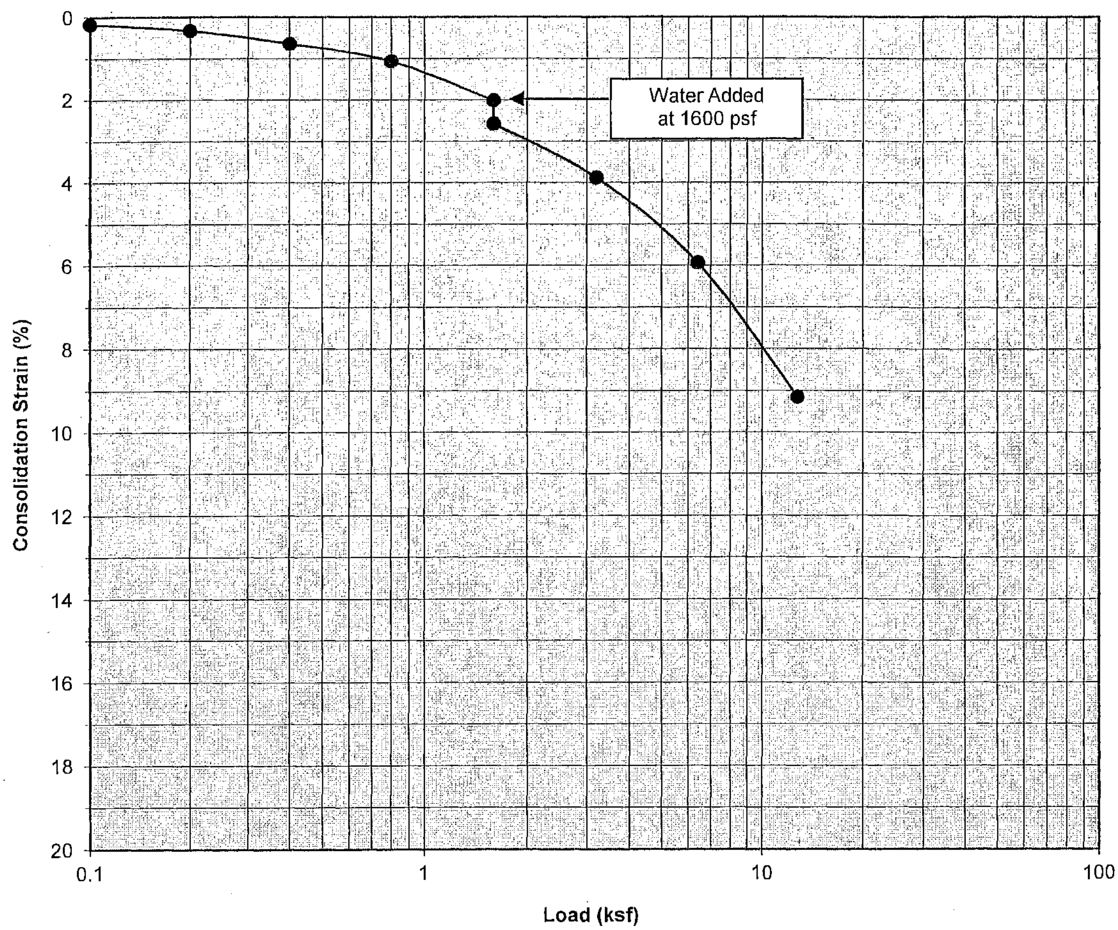
SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: ALLUVIUM: Light Gray fine Sandy Clay, trace calcareous nodules

Boring Number:	B-11	Initial Moisture Content (%)	14
Sample Number:	---	Final Moisture Content (%)	34
Depth (ft)	1 to 2	Initial Dry Density (pcf)	78.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	88.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.65

Consolidation/Collapse Test Results



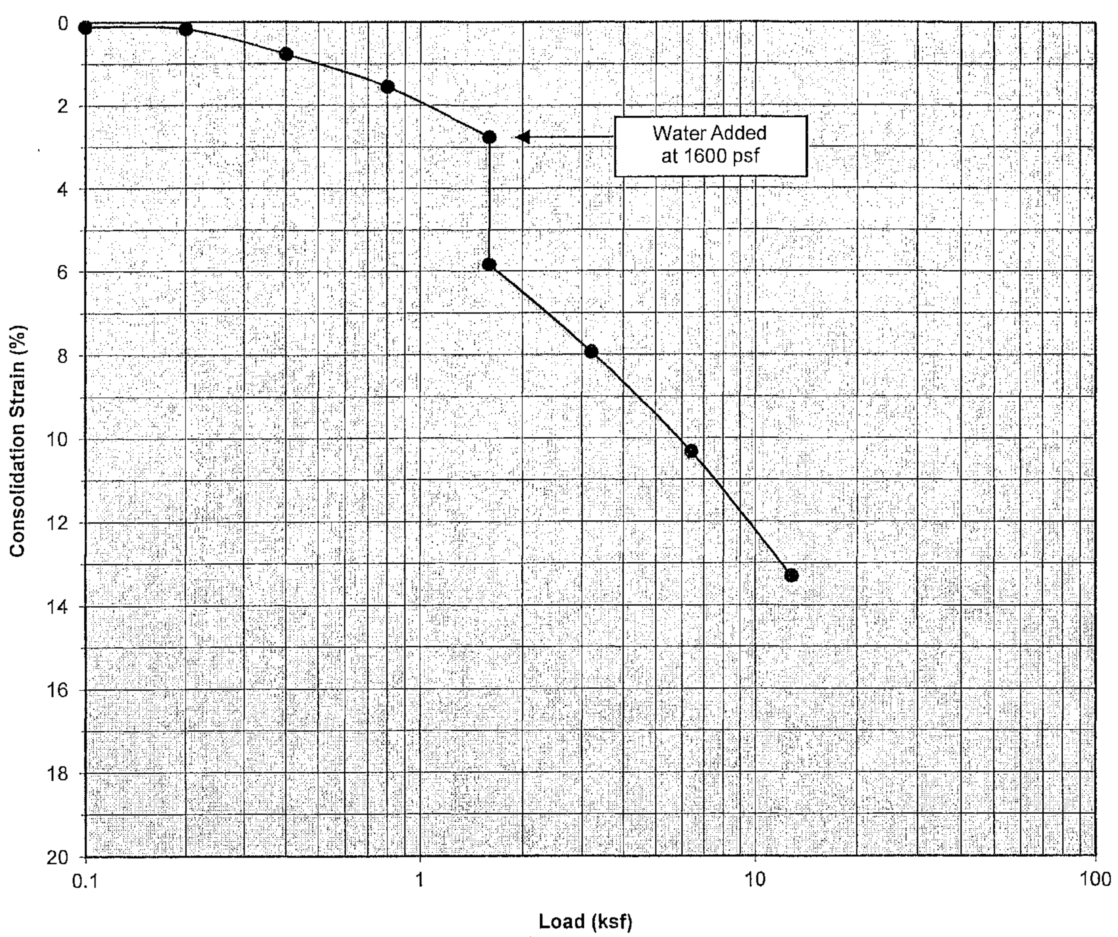
Classification: ALLUVIUM: Light Gray Silty Clay to Clayey Silt

Boring Number:	B-11	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	40
Depth (ft)	3 to 4	Initial Dry Density (pcf)	72.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	80.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.56

Proposed Dorado Logistics Center
Moreno Valley, California
Project No. 12G189
PLATE C- 6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: ALLUVIUM: Light Brown Clayey Silt, abundant calcareous deposits

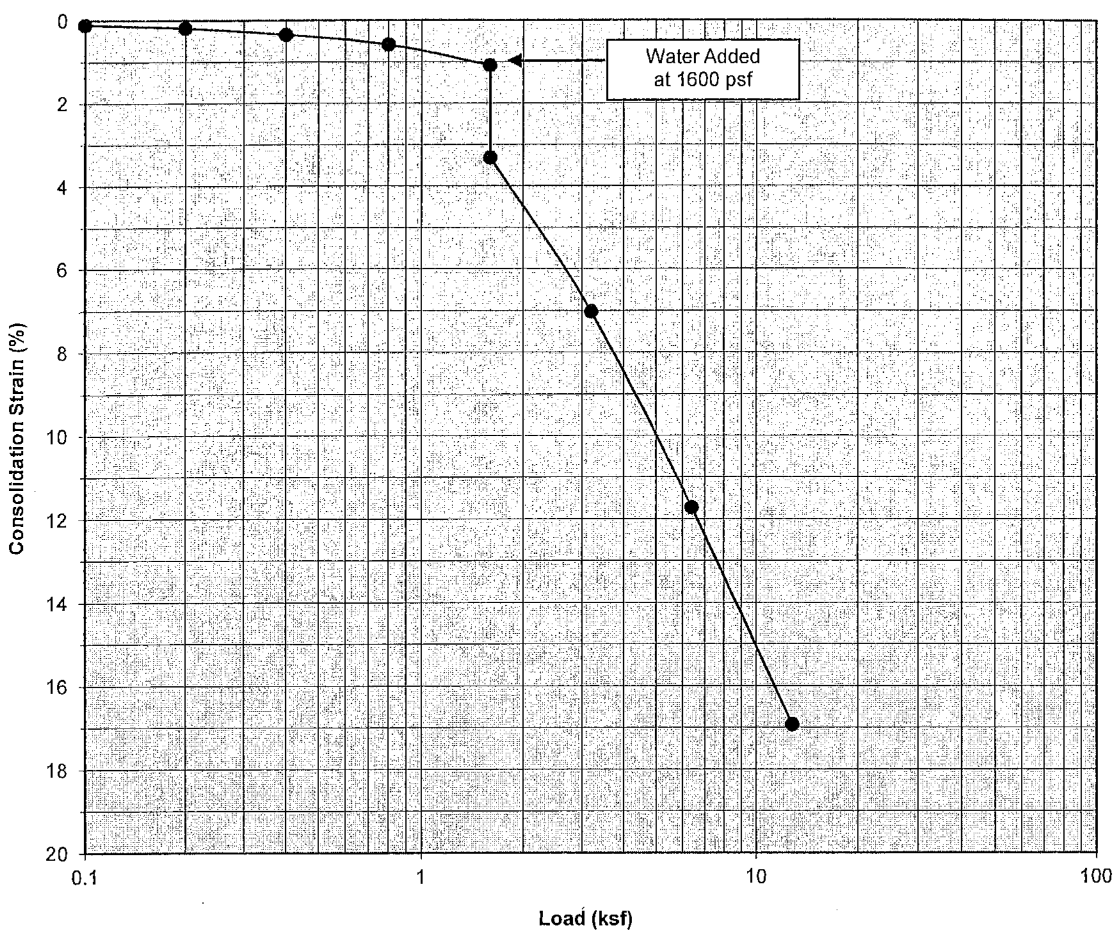
Boring Number:	B-11	Initial Moisture Content (%)	21
Sample Number:	---	Final Moisture Content (%)	38
Depth (ft)	5 to 6	Initial Dry Density (pcf)	72.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	83.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	3.08

Proposed Dorado Logistics Center
Moreno Valley, California
Project No. 12G189

PLATE C- 7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

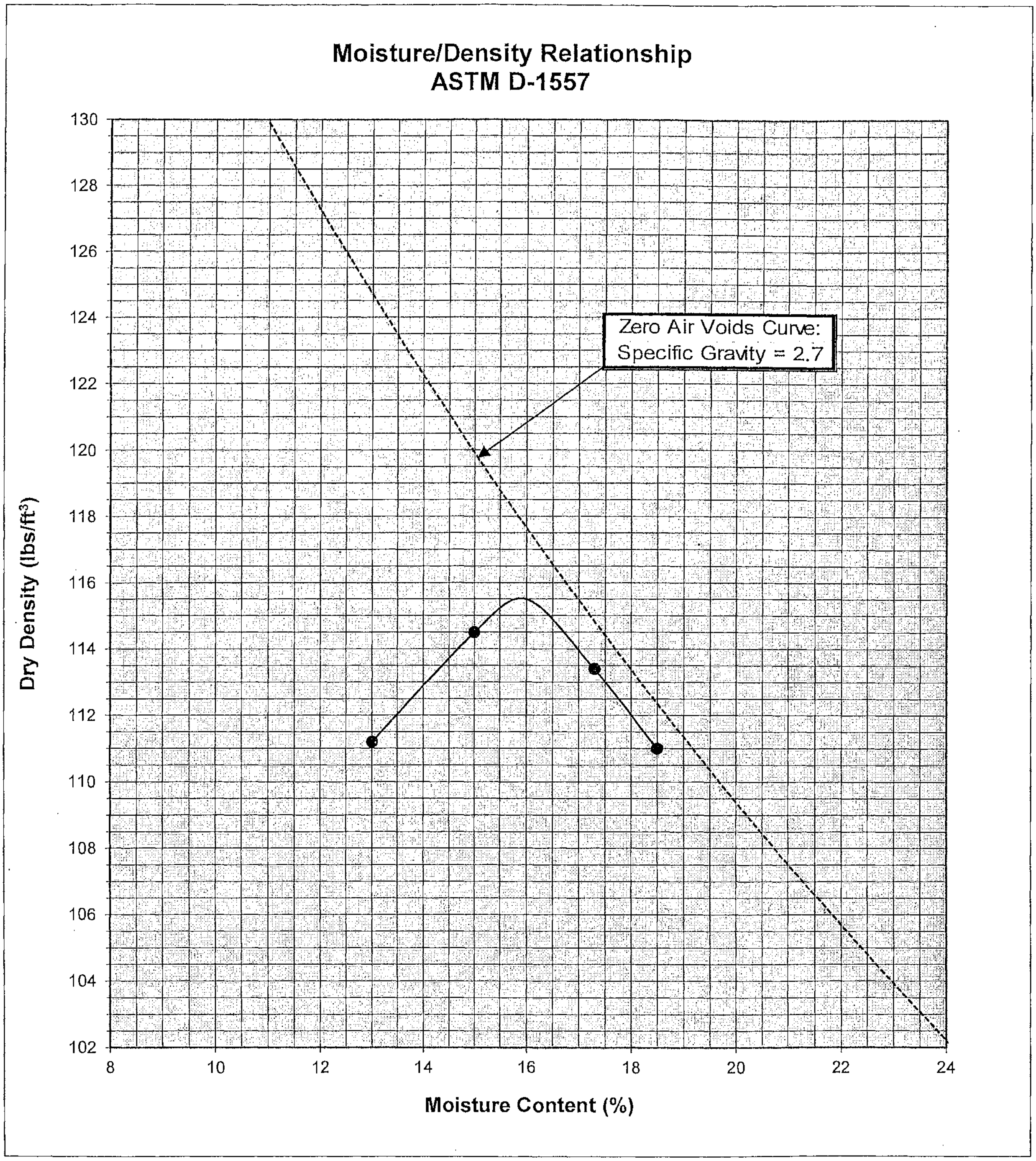


Classification: ALLUVIUM: Light Brown Clayey Silt, abundant calcareous deposits

Boring Number:	B-11	Initial Moisture Content (%)	19
Sample Number:	---	Final Moisture Content (%)	37
Depth (ft)	7 to 8	Initial Dry Density (pcf)	67.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	80.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.23

Proposed Dorado Logistics Center
Moreno Valley, California
Project No. 12G189
PLATE C- 8

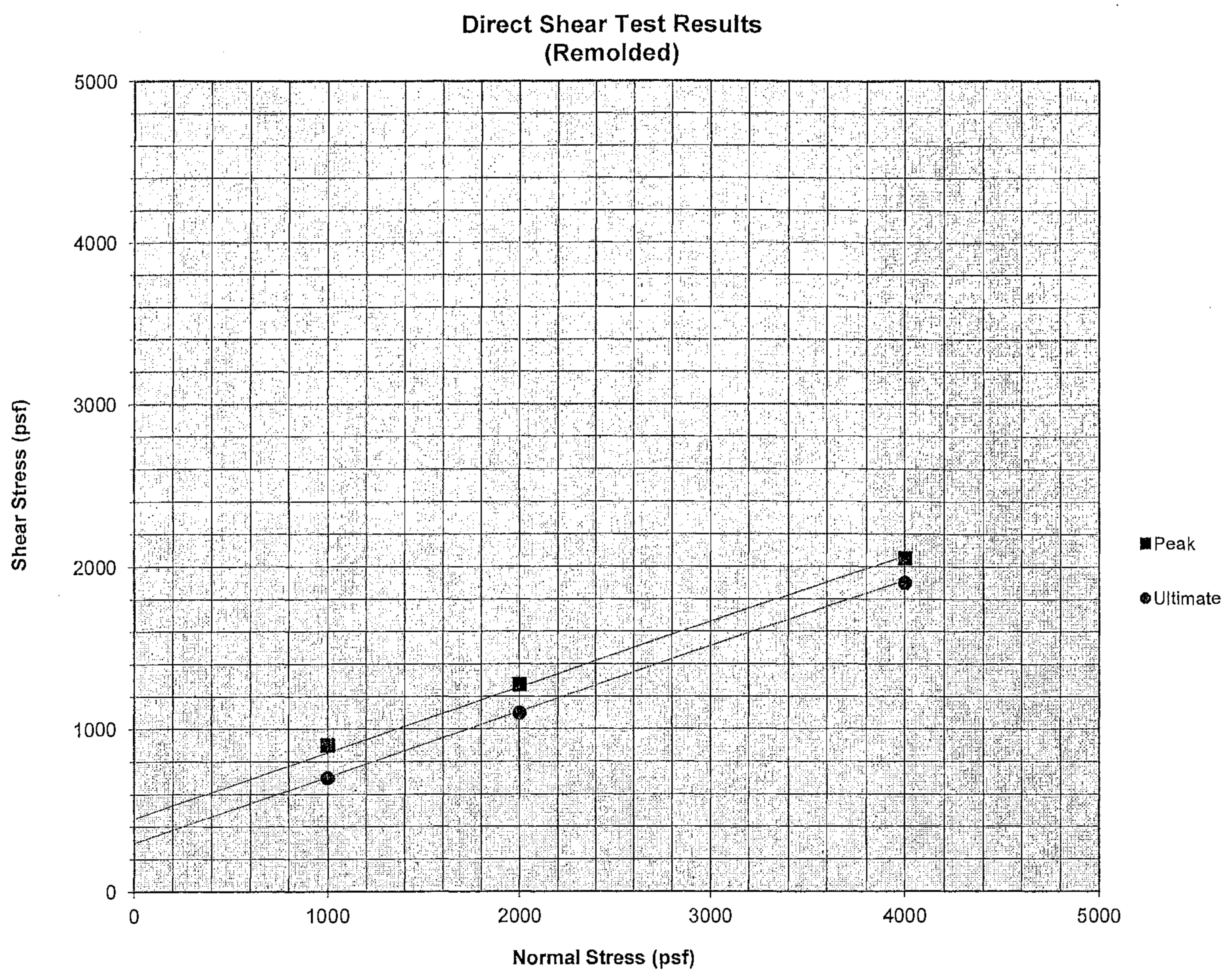
SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-4 @ 0 to 5'
Optimum Moisture (%)	16
Maximum Dry Density (pcf)	115.5
Soil Classification	Brown fine Sandy Clay

Proposed Dorado Logistics Center
Moreno Valley, California
Project No. 12G189
PLATE C-9

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Sample Description: B-4 @ 0 to 5'

Classification: Brown fine Sandy Clay

Sample Data	
Remolded Moisture Content	16.0
Final Moisture Content	24.0
Remolded Dry Density	104.0
Percent Compaction	90.0
Final Dry Density	--
Specimen Diameter (in)	2.4
Specimen Thickness (in)	1.0

Test Results

	Peak	Ultimate
ϕ (°)	22.0	22.0
C (psf)	450	300

Proposed Dorado Logistics Center
Moreno Valley, California
Project No. 12G189

PLATE C-10

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.

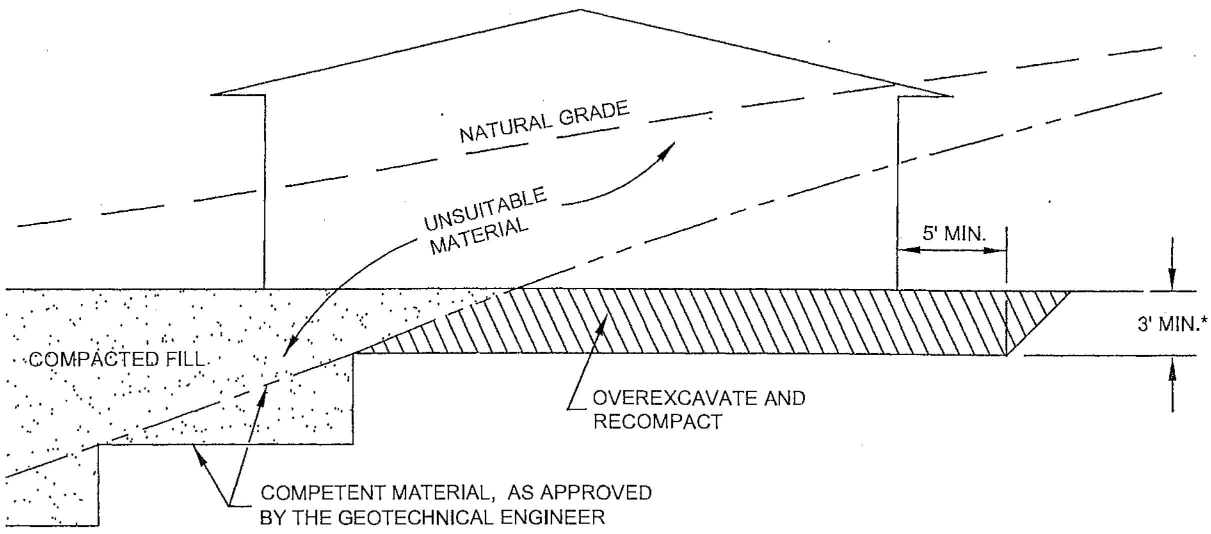
Subdrains

- 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.

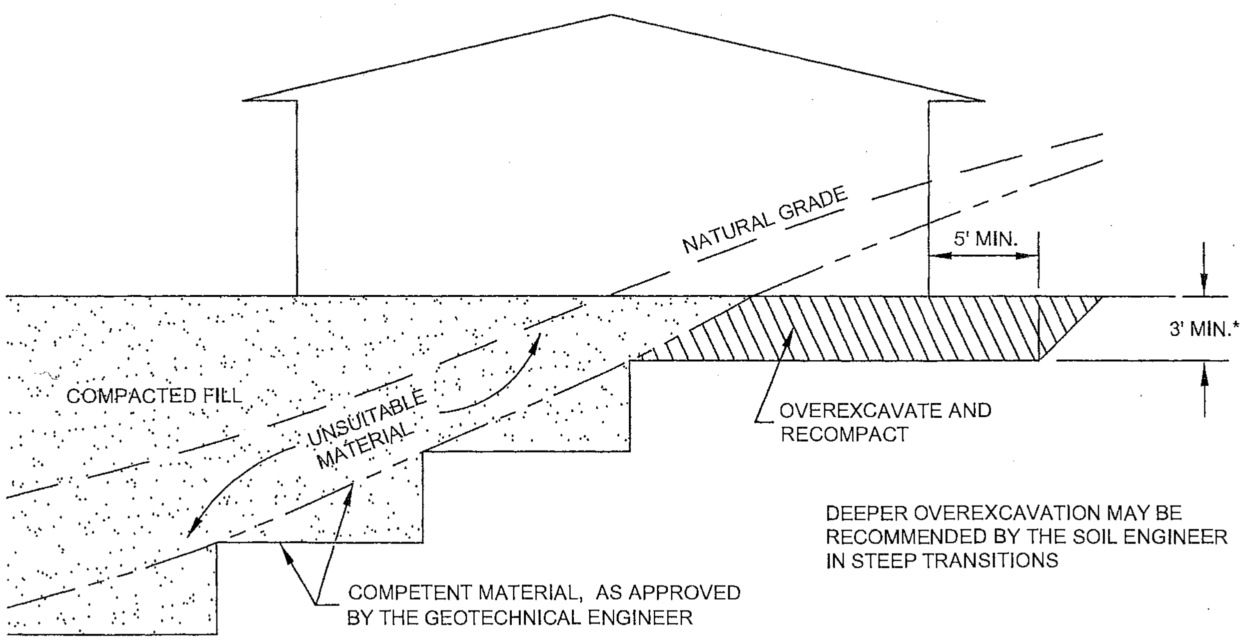
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- 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)



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

TRANSITION LOT DETAIL

GRADING GUIDE SPECIFICATIONS

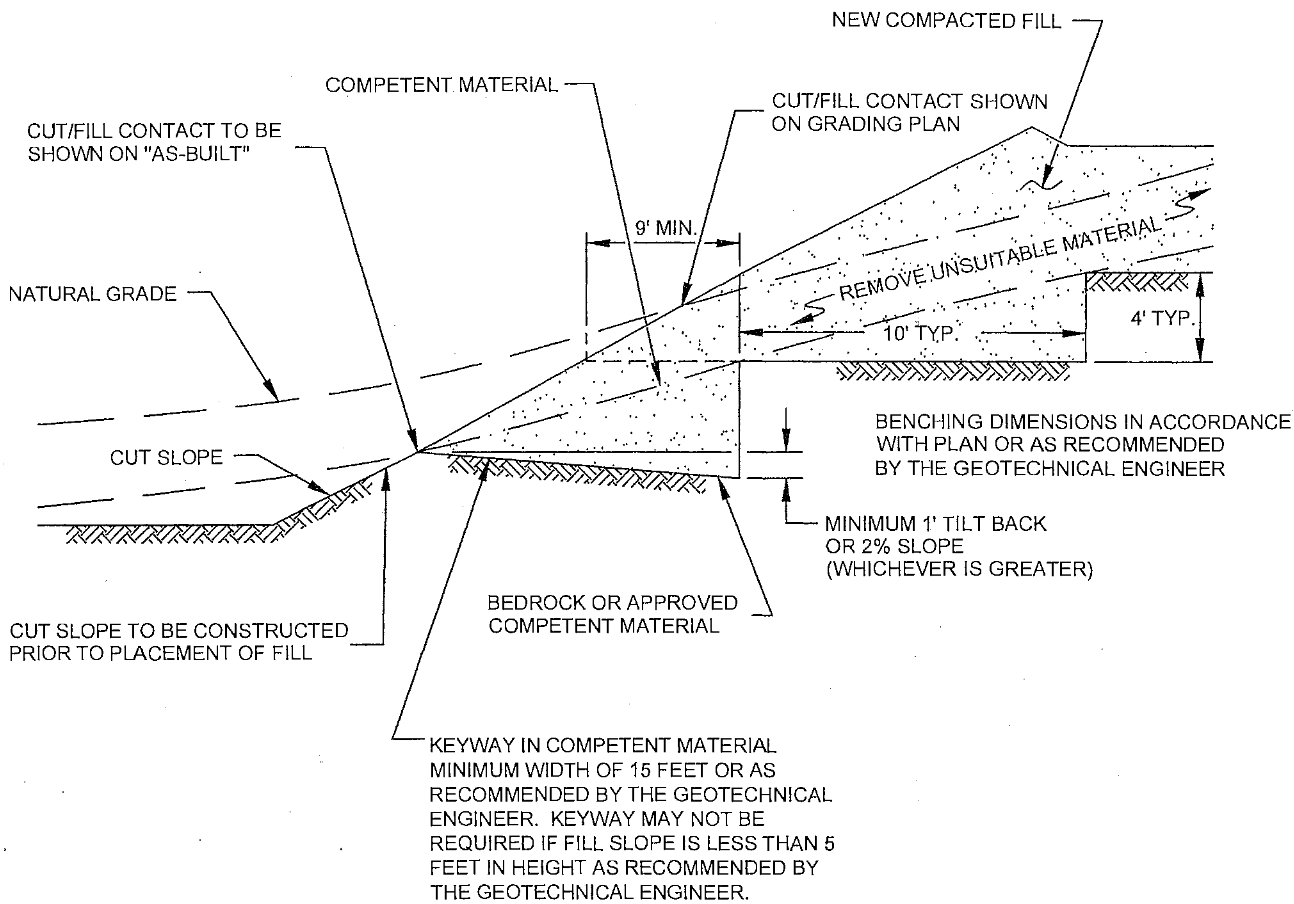
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

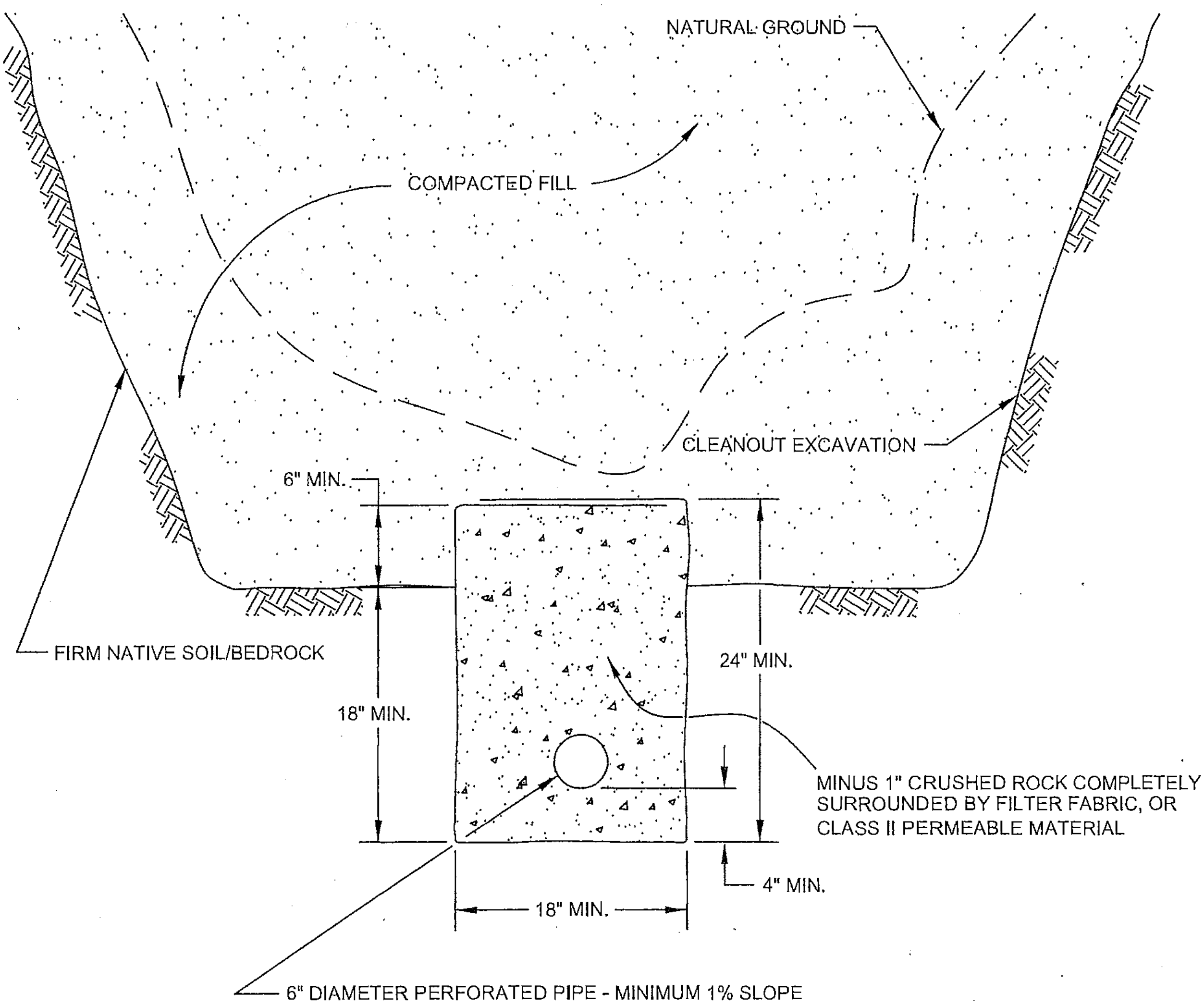
PLATE D-1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



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



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

GRADING GUIDE SPECIFICATIONS

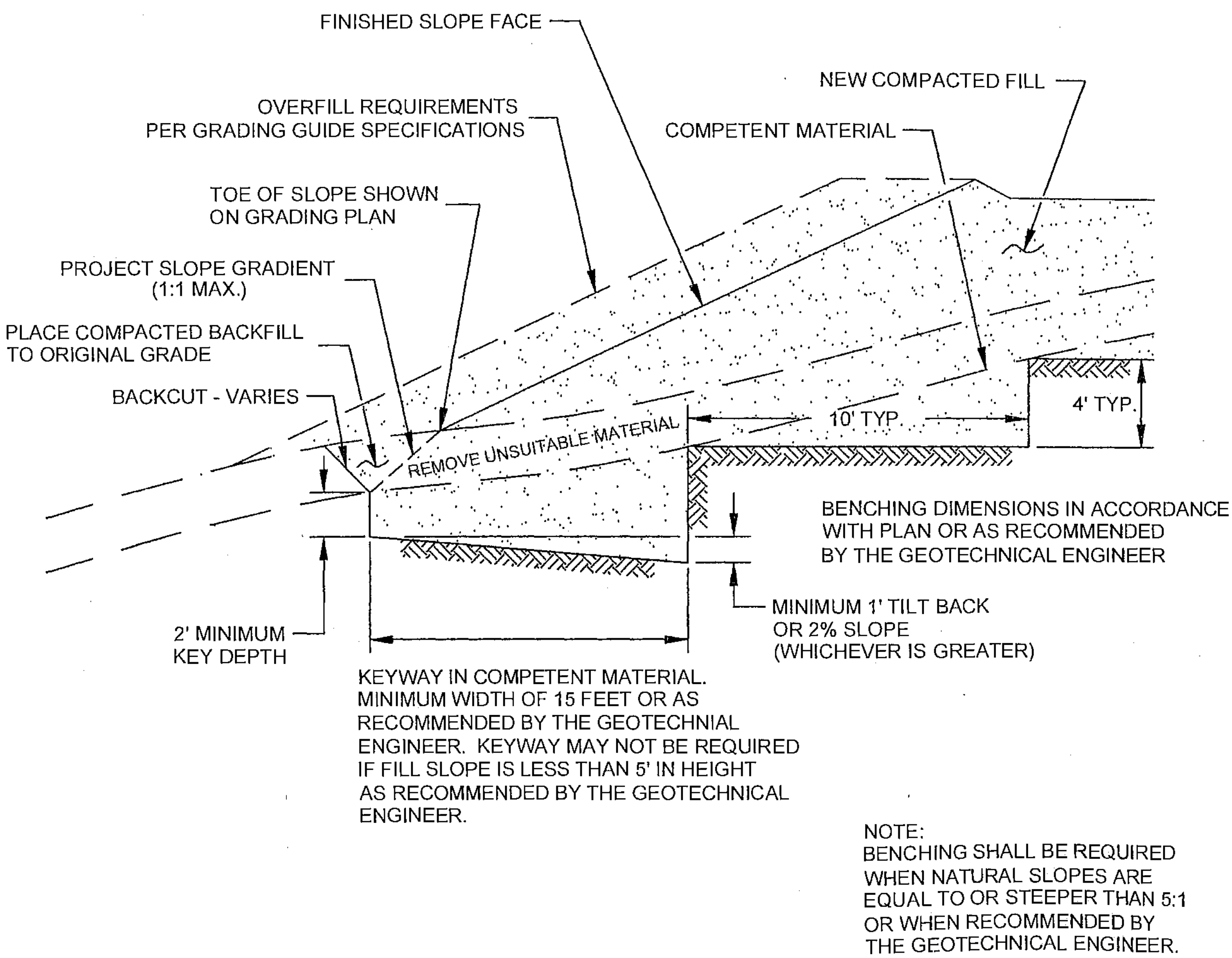
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

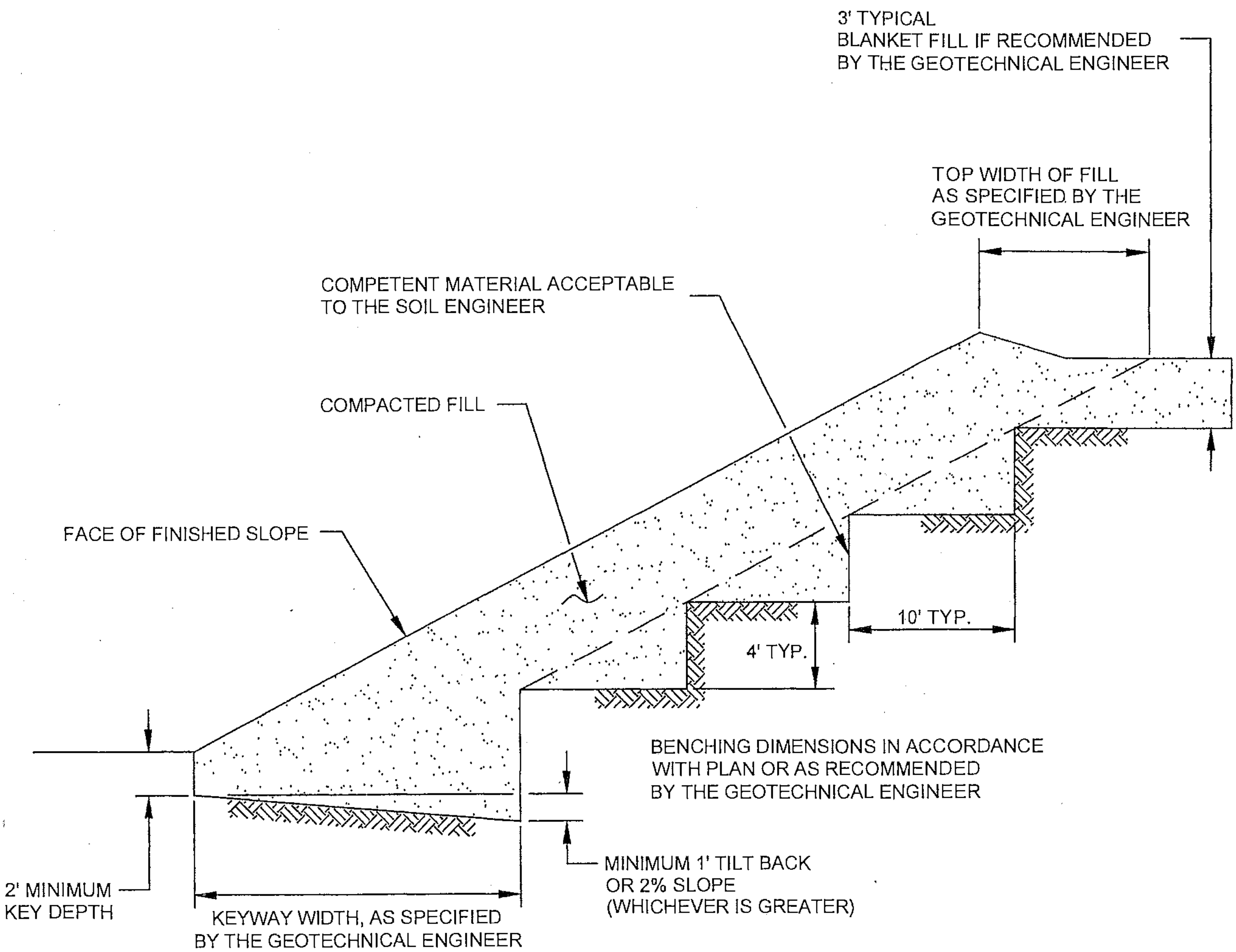
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

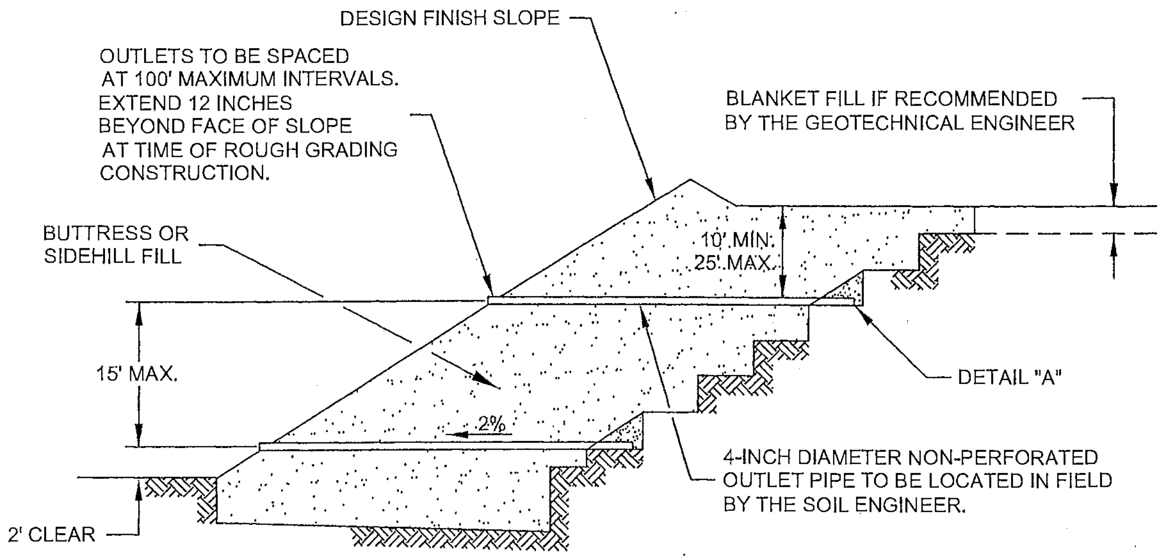
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



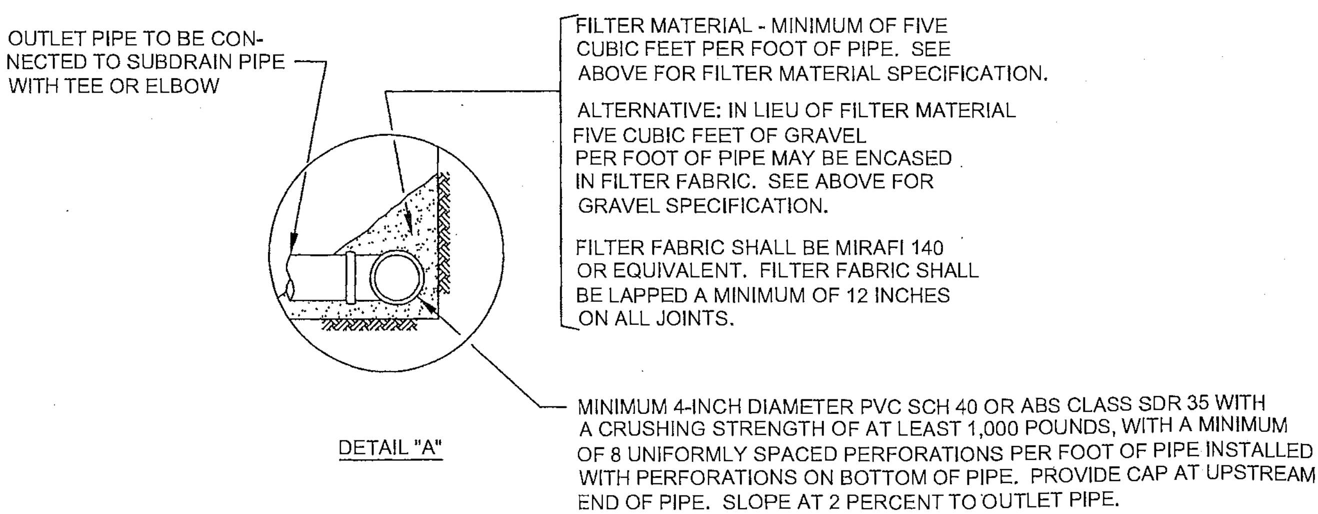
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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



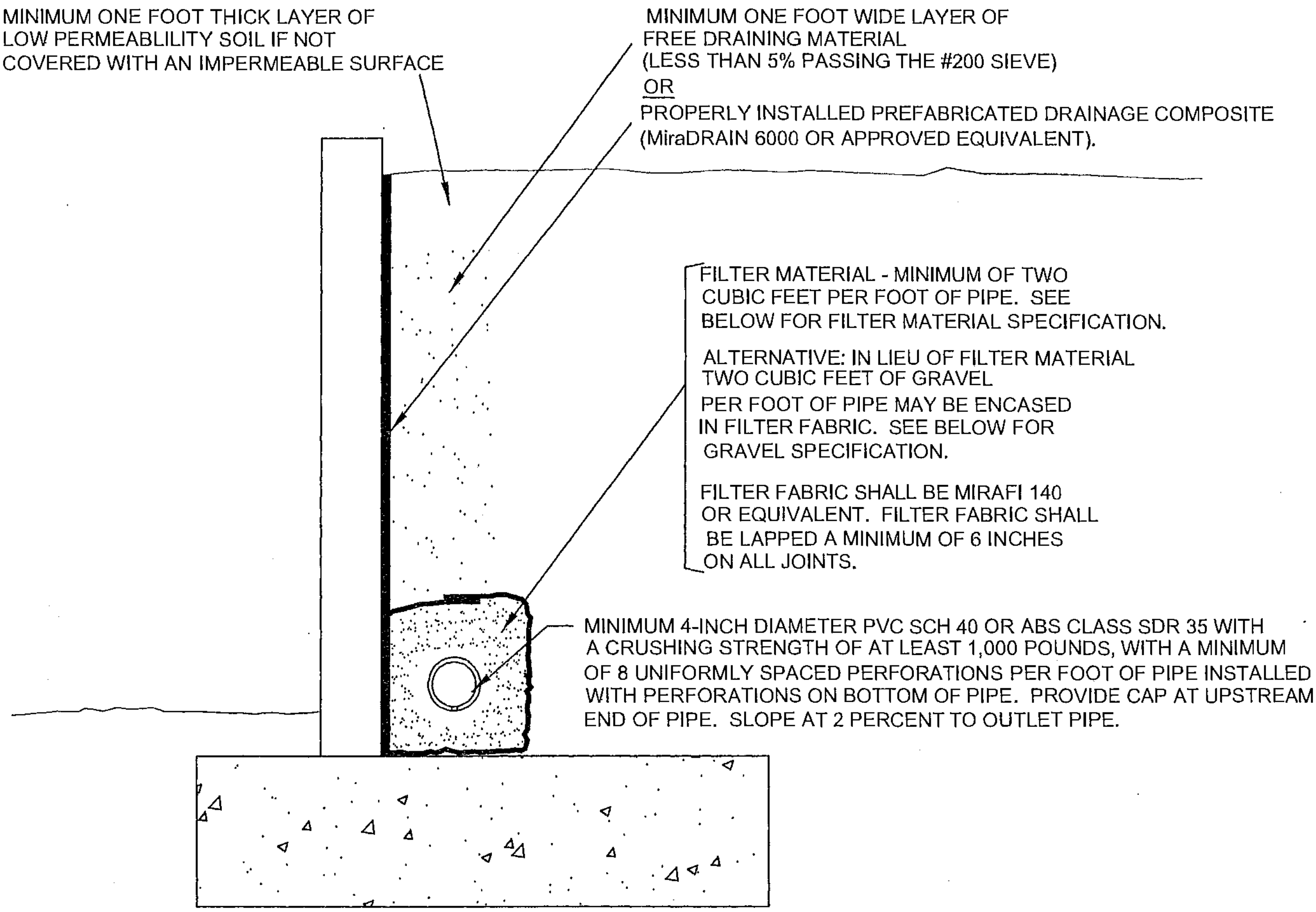
DETAIL "A"

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

SLOPE FILL SUBDRAINS

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-6	



"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

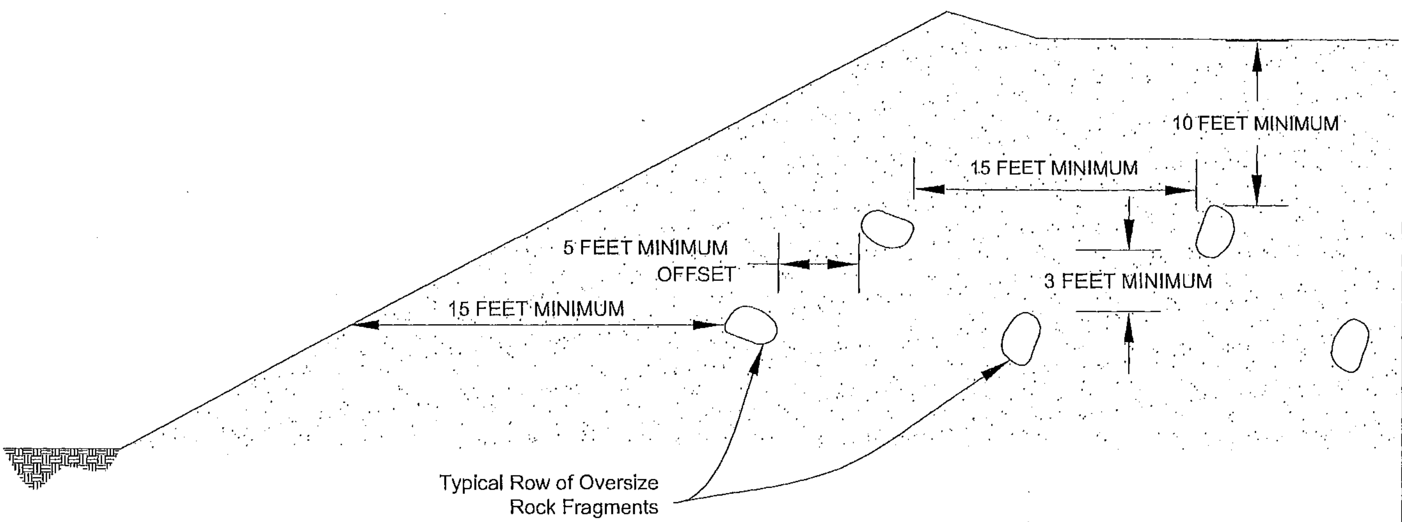
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

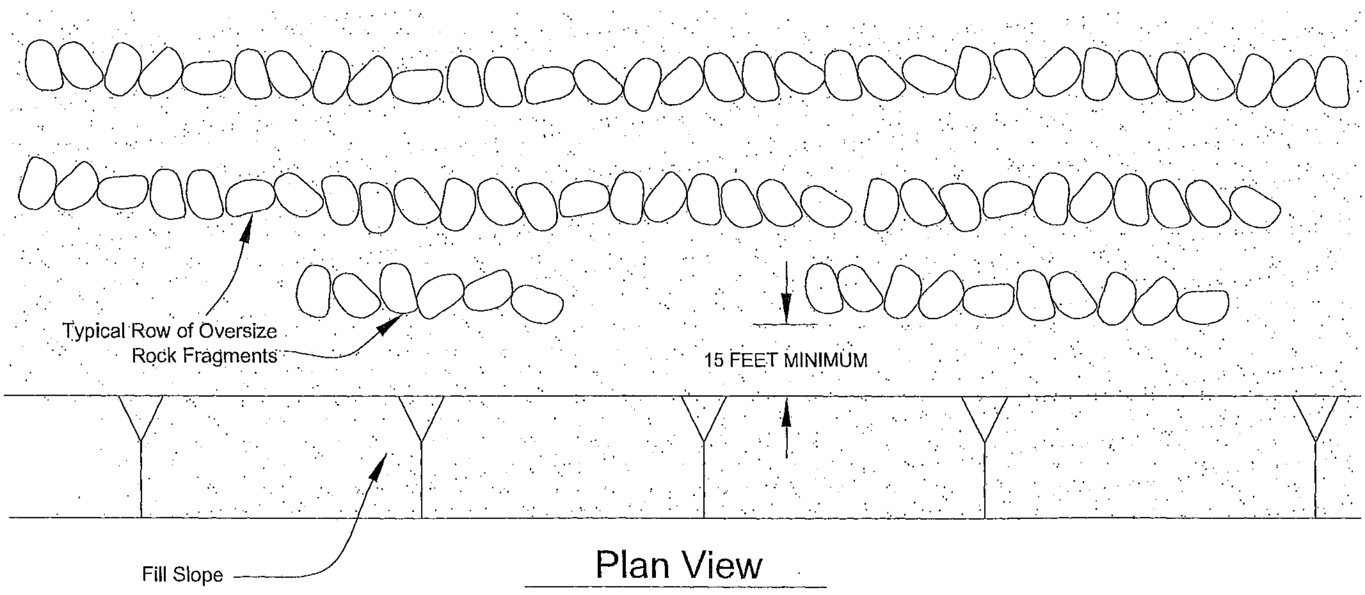
PLATE D-7

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

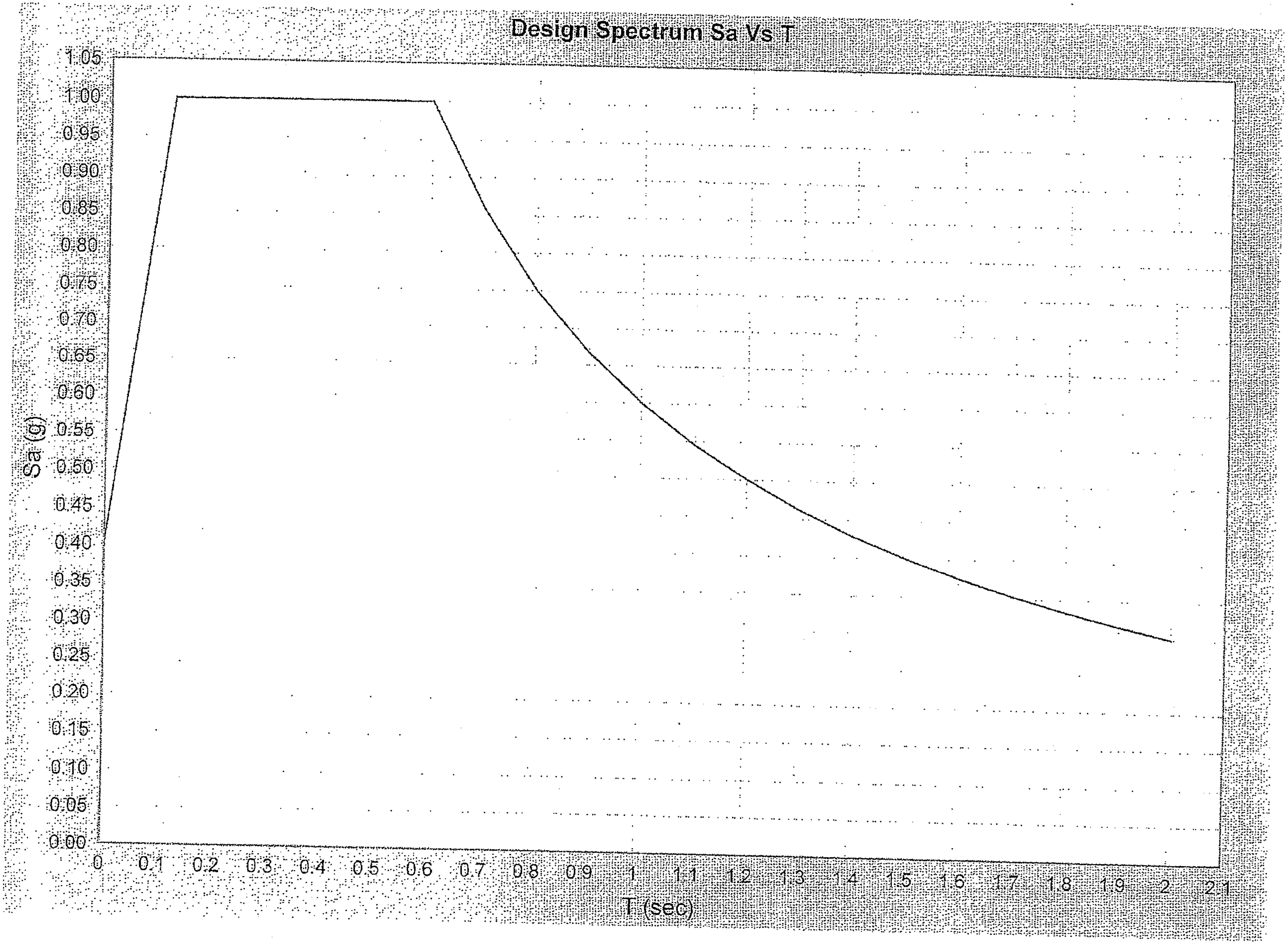
DRAWN: PM
CHKD: GKM

PLATE D-8

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E



Design Spectrum Sa Vs T
Sa (g)
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2
2.1
T (sec)

Conterminous 48 States
2009 International Building Code
Latitude = 33.870317
Longitude = -117.22202100000001
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.01 deg grid spacing

Period (sec)	Sa (g)
0.2	1.500 (Ss, Site Class B)
1.0	0.600 (S1, Site Class B)

Conterminous 48 States
2009 International Building Code
Latitude = 33.870317
Longitude = -117.22202100000001
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	1.500 (SMs, Site Class D)
1.0	0.900 (SM1, Site Class D)

Conterminous 48 States
2009 International Building Code
Latitude = 33.870317
Longitude = -117.22202100000001
Design Spectral Response Accelerations SDs and SD1
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	1.000 (SDs, Site Class D)
1.0	0.600 (SD1, Site Class D)

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Dorado Logistics Center	Design Acceleration	0.4 (g)
Project Location	Moreno Valley, California	Design Magnitude	6.95
Project Number	12G189	Historic High Depth to Groundwater	22 (ft)
Engineer	PM	Current Depth to Groundwater	25 (ft)

Boring No. B-7

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	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)	Cyclic Stress Ratio to Cause Liquefaction (M=7.5)	Cyclic Stress Ratio to Cause Liquefaction (M=6.95)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)				(6)	(7)	(8)	(9)		
	0	22	11		120		1.3	1.23	0.75	0.0	0.0	1320	1320	1320	0.97	0.05	0.06	0.25	N/A	Above Water Table
24.5	22	27	24.5	15	120	39	1.3	0.82	0.95	15.3	23.3	2940	2784	2940	0.94	0.26	0.31	0.26	N/A	Non-Liquefiable: PI>12
29.5	27	32	29.5	22	120	44	1.3	0.78	0.95	21.3	30.5	3540	3072	3259	0.93	INDET	INDET	0.28	N/A	Non-Liquefiable
34.5	32	37	34.5	18	120	46	1.3	0.75	1	17.6	26.1	4140	3360	3547	0.89	0.30	0.37	0.29	N/A	Non-Liquefiable: PI>12
39.5	37	42	39.5	37	120	31	1.3	0.72	1	34.7	45.1	4740	3648	3835	0.85	INDET	INDET	0.29	N/A	Non-Liquefiable
44.5	42	47	44.5	18	120	43	1.3	0.70	1	16.3	24.6	5340	3936	4123	0.81	0.28	0.33	0.29	N/A	Non-Liquefiable: PI>12
49.5	47	50	48.5	20	120	70	1.3	0.68	1	17.6	26.1	5820	4166	4354	0.78	0.30	0.37	0.28	1.30	Non-Liquefiable

Notes:

* Assumed

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

(2) Overburden Correction, Lao and Whitman, 1986, $C_N = (2.0 \text{ ksf} / p'_o)^{1/2}$

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

(4) N-value corrected for energy, rod length, and overburden

(5) N-value corrected for fines content per Eq. 5 (Youd and Idriss, 1997). Allows use of base curve, Fig 2 (Youd and Idriss, 1997)

(6) Calculated by Eq. 2 (Youd and Idriss, 1997), gives same results as Fig 40 of Seed and Idriss, ASCE, September 1971

(7) Per Figure 2, base curve (Youd and Idriss, 1997) using$(N_1)_{60CS}$. Curve also presented as Fig 7.1 (SCEC, 1997). INDET indicates that the $(N_1)_{60}$ plots to the right of the vertical portion of the base curve, and the Cyclic Stress Ratio required to induce liquefaction is indeterminant. The layer is non-liquefiable.

(8) Corrected for Magnitude Weighting using revised Idriss factors (Fig 12, Youd and Idriss (1997) and Fig 7.2, SCEC (1997))

(9) Per Seed and Idriss, ASCE, September 1971 $\frac{\tau_{av}}{\sigma_o'} = \frac{0.65(\sigma_o)}{\sigma_o'} * \frac{a_{max}}{g} * r_d$

(10) Per SCEC (1997), thefollowing guidelines apply to the factor of safety against liquefaction:

Consequence of Liquefaction	$(N_1)_{60}$ (clean sand)	Factor of Safety
Settlement	<= 15	1.1
	>= 30	1.0
Surface Manifestation	<= 15	1.2
	>= 30	1.0
Lateral Spread	<= 15	1.3
	>= 30	1.0

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Dorado Logistics Center
Project Location	Moreno Valley, California
Project Number	12G189
Engineer	PM

Boring No. B-7

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	N_{CORR}	$(N_1)_{60\text{-CORR}}$	Liquefaction Factor of Safety	Cyclic Stress Ratio Induced by Design Earthquake (M=6.95)	Magnitude Weighting Factor	Cyclic Stress Ratio Induced by Design Earthquake (M=7.5)			Volmetric Strain (%)	Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)		
0	0	22	11	0.0	0	0.0	N/A	0.25	1.21	0.21			0.0	0.00	Above Water Table
24.5	22	27	24.5	15.3	3	18.3	N/A	0.26	1.21	0.21			0.0	0.00	Non-Liquefiable: PI>12
29.5	27	32	29.5	21.3	4	25.3	N/A	0.28	1.21	0.23			0.0	0.00	Non-Liquefiable
34.5	32	37	34.5	17.6	4	21.6	N/A	0.29	1.21	0.24			0.0	0.00	Non-Liquefiable: PI>12
39.5	37	42	39.5	34.7	2	36.7	N/A	0.29	1.21	0.24			0.0	0.00	Non-Liquefiable
44.5	42	47	44.5	16.3	3	19.3	N/A	0.29	1.21	0.24			0.0	0.00	Non-Liquefiable: PI>12
49.5	47	50	48.5	17.6	5	22.6	1.30	0.28	1.21	0.23			0.0	0.00	Non-Liquefiable

Total Deformation (in)	0.00

Notes:

- * Assumed
- (1) N_{60} calculated previously for the individual layer
- (2) Correction for fines content per Table 7.2 (SCEC 97)
- (3) Corrected N_{60}
- (4) Factor of Safety against Liquefaction, calculated previously for the individual layer
- (5) Earthquake induced cyclic shear stress ratio calculated previously for the individual layer
- (6) Factor to convert M=6.95 shear stress ratio to M=7.5 shear stress ratio, Seed, et al., 1983
- (7) Corrected for Magnitude Weighting using revised Idriss factors (Fig 12, Youd and Idriss (1997) and Fig 7.2, SCEC (1997))
- (8) Voumetric Strain Induced in a Liquefiable Layer, Tokimatsu and Seed, ASCE August 1987
  (Strain N/A if Factor of Safety against Liquefaction > 1.2)