GEOTECHNICAL INVESTIGATION AND LIQUEFACTION EVALUATION
PROPOSED ORANGE SHOW WAREHOUSE

Orange Show Road, East of Waterman Avenue
San Bernardino, California
for
Hillwood

August 27, 2013

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Hillwood Investment Properties
901 Via Piemonte, Suite 175
Ontario, California 91764

Attention: Mr. John Schaefer

Project No.: **13G157-1**

Subject: **Geotechnical Investigation and Liquefaction Evaluation**
Proposed Orange Show Warehouse
Orange Show Road, east of Waterman Avenue
San Bernardino, 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,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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1.0 EXECUTIVE SUMMARY

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

Site Preparation

- Initial site preparation should include stripping of any surficial vegetation. The surficial vegetation including grass and weed growth, trees and any organic soils should be properly disposed of off-site. Demolition of the vacant lumber facility will be necessary in the southeastern portion of the site as well as an equipment pad with four above ground storage tanks and a small wooden structure in the northern half of the site. This will include demolition of the existing buildings, structures, improvements and removal of the resultant debris off-site. Any existing pavements should be stripped and removed from the site. Alternatively, concrete and asphalt debris may be crushed and reused on site within compacted fills or made into miscellaneous base.
- Artificial fill materials were encountered in the eastern portion of the site in the upper 3 to 10½± feet below the ground surface. The fill materials encountered at the boring and trench locations generally consist of loose to medium dense sandy soils and buried debris consisting of concrete tiles, concrete fragments, and slab fragments. The fill materials encountered at the site are considered to be undocumented fill, and are not suitable for the support of the proposed structures.
- The near-surface native alluvial soils at this site generally consist of very loose to loose fine sands, silty fine sands and fine sandy silts. These very loose to loose soils extend to depths of 10 to 12± feet and possess variable strengths and unfavorable consolidation and collapse characteristics within the upper 4± feet.
- Remedial grading is recommended to be performed within the proposed building area in order to remove the artificial fill materials and the upper portion of the alluvial soils. The existing soils within the proposed building area should be overexcavated to a depth of at least 5 feet below existing grade and to a depth of at least 5 feet below the proposed building pad subgrade elevation. The depth of overexcavation should also be sufficient to remove any existing fill soils.
- The proposed foundation influence zones should be overexcavated to a depth of 4 feet below proposed foundation bearing grade.
- Following evaluation of the subgrade by the geotechnical engineer, the exposed subgrade soils should be scarified, moisture conditioned to 2 to 4 percent above optimum, and recompacted. The resulting soils may be replaced as compacted structural fill.

Liquefaction

- Our site-specific liquefaction evaluation indicates that some of the on-site soils are subject to liquefaction during the design seismic event.
- The liquefaction analysis indicates total dynamic settlements of 2.8 to 4.4± inches at the site. The liquefaction-induced differential settlements within the building area are expected to be on the order of 1.6± inches. Assuming that this settlement occurs across a distance of 100± feet, a maximum angular distortion of about of 0.002 inches per inch would result.

- Standard practice dictates that the proposed building can be supported on a shallow foundation system, with the understanding that some cosmetic distress could occur due to liquefaction. Such distress will be typical of buildings of this type, in this area, in the event of a large earthquake.

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 3,000 lbs/ft^2 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 liquefiable soils. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade, 5 inches thick.
- Reinforcement is not considered necessary, for geotechnical considerations. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed slab loading.

Pavements

ASPHALT PAVEMENTS (R = 50)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	3	4	5	5	7
Compacted Subgrade	12	12	12	12	12

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

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 11P404R, dated June 12, 2013. 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 on the south side of Orange Show Road, approximately 400 feet east of the intersection of Orange Show Road and Waterman Avenue in San Bernardino, California. The site is bounded to the north by Orange Show Road, to the west by a water treatment facility, to the southwest by a BNSF railroad easement, to the southeast by the Santa Ana River, and to the east by single family residences. The general location of the site is illustrated on the Site Location Map, included as Plate 1 in Appendix A of this report.

The subject site consists of ten irregular-shaped parcels, which total approximately 56.7± acres in size. The northern half of the subject site is generally vacant and undeveloped. An unpaved road traverses the northern half of the subject site in a north-south direction. A small wood-framed structure is located in the northeastern parcel. Based on a conversation with a Utiliquest technician, this structure houses one of two pump stations located at the subject site. An equipment pad with four (4) above ground storage tanks, approximately 15± feet in height, is located in the southwest area of the northern half of the site. The northern half of the subject site appears to have been recently tilled. Ground surface cover within the northern half of the site generally consists of exposed soil with sparse native grass and weed growth. An area approximately 100 feet wide by 200 feet long located on the south side of the northeast parcel is surrounded by a soil berm. The soil berm is 3± feet in height and has an inclination of approximately 2h: 1v.

The southwestern most parcel is currently vacant and undeveloped. The ground surface cover within this portion of the site generally consists of native grasses and weeds with areas of exposed soil.

A former Home Lumber (HL) facility is located within the southeastern parcels of the site. Seven (7) structures from this abandoned facility still remain. One (1) large canopy/storage structure, approximately 9,625± ft^2 in size, is located in the southwestern region of the HL facility. This structure has wooden siding and is supported by steel columns. Several masonry block walls, approximately 3 to 4± feet in height, are located to the north of this canopy building. One (1) small masonry block structure is located in the southern portion of the HL facility. Based on conversations with a Utiliquest technician, this masonry block structure houses the second of two pump stations located at the subject site. Two (2) two-story wood frame buildings with footprint areas of 1,800± ft^2 and 2,500± ft^2 are located in the northern portion of the HL facility. These buildings are connected by a wooden canopy. Another canopy structure located east of the two-story buildings houses a conveyor belt system. This structure is approximately 4,200 ft^2 in size with steel columns and a concrete floor. A two-story hopper structure is located 50± feet north of the conveyor belt system. The foundation systems for these structures are unknown, but we presume that all of these structures are supported by conventional shallow foundation systems. The remnants of a former structure or equipment pad currently consisting of a concrete slab with three steel beams extending upward vertically, are located approximately 75± feet south of the conveyor belt canopy structure. Ground surface cover within the HL facility generally consists of asphaltic concrete pavements, aggregate base, and open graded gravel. The asphaltic concrete pavements are in very poor condition with severe cracking throughout.

Topographical information for the subject site was obtained from an ALTA survey provided by the project civil engineer. This plan indicates that the site grades range from elevations of 1031.0± feet mean sea level (msl) in the northeast portion of the site to an elevation of 1012.0± feet msl in the western portion of the site. With the exception of minor localized variations in topography in the southwestern region of the site, site topography slopes downward to the west at a gradient of approximately 1± percent.

3.2 Proposed Development

A preliminary site plan for the proposed development, prepared by HPA, was provided to our office by the client. Based on the preliminary site plan, the site will be developed with one (1) new warehouse. The new warehouse will be located in the northern to central area of the site and will be approximately 1,199,310± ft^2 in size. Truck loading docks will be constructed on the north and south sides of the building. The building will be surrounded by asphaltic concrete pavements for automobile parking and drive lanes, Portland cement concrete pavements in the loading dock areas, landscaped planters, and decorative concrete flatwork.

Detailed structural information has not been provided. It is assumed that the proposed structure will be of concrete tilt-up construction, typically supported on a conventional shallow foundation system and a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 3 to 6 kips per linear foot, respectively.

No significant amounts of below grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. Based on the assumed relatively level site topography, cuts and fills of less than 3 to 6± feet are expected to be necessary to achieve the proposed building pad grade.

3.3 Aerial Photograph Review

As a part of our research for this project, we reviewed readily available historical aerial photographs from the internet. Historical aerial photographs were obtained from Historicaerials.com and Google Earth. These photographs were used to characterize past conditions and usage of the subject property. Aerial photographs from the following years were available for review: 1938, 1959, 1968, 1980, 1994, 2002, 2006, 2009. Each of the reviewed photographs are readily available on the internet from the aforementioned sources. A brief summary of the aerial photograph review is presented below.

1938:
The subject site is vacant and undeveloped at the time of this photograph. The northwest bank of the Santa Ana River appears to encroach on the southeastern portion of the subject site. East Orange Show Road and Waterman Avenue are both visible from this photograph and both roads

appear to be unpaved. The existing railroad easement is located along the southwestern is also present.

<u>1959:</u>
The subject site is being utilized for agricultural purposes at the time this photograph was taken. Row crops appear to be throughout the subject site. Both Waterman Avenue and East Orange Show Road appear to be paved.

<u>1968:</u>
The subject site is vacant and undeveloped at the time this photograph was taken.

<u>1980:</u>
The subject site is generally unchanged from the previous photograph with the exception of the triangular shaped parcel, located in the southwest region of the site, which appears to have been possibly graded. The Santa Ana River no longer encroaches on the southeastern portion of the subject site. A row of large trees appear to be present along the current southeastern property line, which was formerly covered by the Santa Ana River.

<u>1994:</u>
The southeastern region of the site is developed with a Lumber facility at the time of this photograph. Several dirt access roads traverse the subject site in this photograph.

<u>2002:</u>
The northern expansion of the lumber facility appears at the time of this photograph. The triangular shaped portion of the site, located adjacent to the BNSF railroad easement along the southwestern property line, contains stock piles of what appear to be soil and miscellaneous debris. Four (4) above ground storage tanks appear on this photograph to the north of the triangular portion of the site.

<u>2006:</u>
Stockpiles are no longer present in the triangular-shaped parcel in the southwest area of the site. A structure is now visible north of the lumber facility expansion on this photograph.

<u>6/5/2009:</u>
The subject site is generally unchanged from the previous photograph.

<u>6/19/2009:</u>
A large rectangular shaped excavation to the northeast of the lumber facility appears in this photograph. The excavation is approximately 300± feet by 275± feet and slopes downward to the center of the excavation which is approximately 125± feet by 140± feet. A small masonry block structure is being constructed near the southern corner of the site in this photograph.

<u>11/15/2009:</u>
The northern half of the subject site and the triangular parcel in the southwest portion of the site is vacant and undeveloped. The lumber facility appears to be nonoperational and all lumber materials are gone in this photograph. The aforementioned excavation was backfilled prior to this photograph. This photograph resembles the current site conditions encountered during the geotechnical investigation.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of twelve (12) borings advanced to depths of 5 to 51½± feet below existing site grades. The two 50± foot deep borings were performed as part of the liquefaction evaluation. 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.

In addition to the twelve borings, five (5) test pits were excavated at the subject site. The test pits were excavated within the eastern portion of the site to depths of 7 to 12± feet using a backhoe with rubber tires, equipped with a 36-inch wide bucket. Smaller bucket sizes were utilized where dense concrete fill materials were encountered. All of the test pits were logged by a member of our staff.

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

4.2 Geotechnical Conditions

Pavements

Asphaltic concrete pavements were encountered at the ground surface at Boring No. B-7 and Trench No. T-1. These pavements consisted of 2 to 5± inches of asphaltic concrete with no discernible underlying layer of aggregate base.

Aggregate base was encountered at the ground surface at Trench Nos. T-2, T-3, and T-5. The thickness of aggregate base varied from 4 to 6 inches at these locations.

Artificial Fill

Artificial fill soils were encountered at the ground surface at Boring Nos. B-8, B-9, B-10 and B-12 extending to depths of 3 to 8½± feet below existing site grades. The fill soils generally consist of loose fine sandy silts, loose to medium dense silty fine to medium sands, and medium dense silty fine sands to fine sandy silts. The fill soils possess a disturbed appearance and artificial debris including wire, plastic, brick, and asphaltic concrete fragments, resulting in their classification as artificial fill. Materials classified as possible fill were encountered at Boring No. B-12 between depths of 8½ and 12± feet. These materials possess a slightly disturbed appearance, but lack obvious indicators of fill, such as artificial debris.

Artificial fill materials were also encountered at all of the test pit locations, extending to depths of 3 to 10½± feet. The trenches encountered fill soils similar to those encountered at the boring locations. However, the trenches also encountered fill materials consisting primarily of buried debris such as concrete tiles, concrete slab fragments, and concrete debris resembling boulders with lengths or diameters exceeding 2 to 3± feet. The depths of the buried concrete debris fills encountered were 7± feet and 8± feet at Trench Nos. T-3 and T-5, respectively.

Alluvium

Native alluvial soils were encountered at the ground surface, beneath the fill soils, and/or beneath the pavements at all of the boring and trench locations. The near surface alluvial soils at the boring and trench locations generally consist of very loose to loose silty fine sands, fine to coarse sands, and fine sandy silts extending to depths of 10 to 12± feet below existing site grades. At greater depths, the underlying native alluvial soils generally consist of interbedded layers of medium dense fine to coarse sands, silty fine sands, and fine sandy silts extending to the maximum depth explored of 51½± feet below existing site grades. Occasional medium stiff to stiff clayey silt to silty clay strata were encountered at depths ranging from 17 to 48± feet at several boring locations.

Groundwater

Free water was encountered during drilling of Boring Nos. B-3 and B-10 at depths of approximately 40 and 38± feet, respectively, below the existing site grades. Due to caving conditions within the open boreholes, delayed readings could not be taken within the open boreholes. Based on the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depths of 38 to 40± feet at the time of the subsurface exploration. Additional research (USGS Bulletin 1898, Matti and Carson, 1991) indicates that the minimum historic depth to groundwater at the site is 10± feet.

5.0 LABORATORY TESTING

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

Classification

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

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-11 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample has been tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plate C-12 in Appendix C of this report. This test is generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

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-11 @ 0 to 5 feet	7	Very Low

Soluble Sulfates

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

Sample Identification	Soluble Sulfates (%)	ACI Classification
B-2 @ 0 to 5 feet	0.016	Negligible
B-11 @ 0 to 5 feet	0.001	Negligible

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.

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on representative soil samples. 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 structure should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. 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 latest edition 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	1.820
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.631
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}	1.820
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.946
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.213
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.631

*The 2010 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site coefficients are to be determined in accordance with Section 11.4.7 of ASCE 7.05. However, Section 20.3.1 of ASCE 7.05 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site class is determined using the standard procedures. **If the proposed structure has a fundamental period greater than 0.5 seconds, SCG should be contacted to revise these seismic design parameters.**

Ground Motion Parameters

For the purposes of the liquefaction analysis performed for this study, we determined a design acceleration in accordance with the 2010 CBC, Section 1803. In accordance with the CBC, the design acceleration was calculated as $S_{DS}/2.5$. For the subject site, this equates to a design acceleration of 0.45. The associated earthquake magnitude was obtained from the 2002 Interactive Deaggregation program available on the USGS website. This program indicates that the predominant earthquake in the vicinity of the subject site associated with the peak ground acceleration has a deaggregated modal magnitude of (M_W) of 6.75.

Liquefaction

Liquefaction is the loss of the 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<0.005mm) 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 California Geological Survey (CGS) has not yet conducted detailed seismic hazards mapping in the area of the subject site. The general liquefaction susceptibility of the site was determined by research of the San Bernardino County Official Land Use Plan, General Plan, Geologic Hazard Overlay. Map FH30C for the San Bernardino South Quadrangle indicates that the subject site is located within a zone of high liquefaction susceptibility. Therefore, the scope of this geotechnical investigation was expanded to include a site-specific liquefaction evaluation.

As part of the liquefaction evaluation, Boring Nos. B-3 and B-10 were extended to a depths of 50± feet. These borings encountered free water at a depths of 38 and 40± feet during drilling. The historic high groundwater depth was obtained from the document entitled Liquefaction Susceptibility in the San Bernardino Valley and Vicinity, Southern California-A Regional Evaluation, USGS Bulletin 1898 (Matti and Carson), which indicates a historic high groundwater depth at the subject site of 10± feet. Therefore, the historic high groundwater table was considered to be 10± feet for the liquefaction evaluation.

The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report. The liquefaction analysis was performed for Boring Nos. B-3 and B-10. The liquefaction potential of the site was analyzed utilizing a maximum peak ground acceleration (PGA) of 0.45g for a magnitude 6.75 seismic event.

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 have identified potentially liquefiable soils at the site. The potentially liquefiable strata are located at various depths between 10 and 40½± feet. Soils which are located above the historic groundwater table, or possess factors of safety in excess of 1.3, or possess plastic indices (PI) greater than 12 with water contents less than 85 percent of the liquid limit are considered non-liquefiable. Settlement analyses were conducted for each of the potentially liquefiable strata.

Based on the settlement analysis (also tabulated on the spreadsheets in Appendix F) total dynamic (liquefaction induced) settlements of 2.8 and 4.4± inches could be expected at Boring Nos. B-3 and B-10, respectively. The associated differential settlement would therefore be on the order of 1.6± inches. The estimated differential settlement could be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of about 0.002 inches per inch. This settlement is considered to be within the structural tolerances of a typical building supported on a shallow foundation system. However, it should be noted that minor to moderate repairs, including repair of damaged drywall and stucco, etc., could be required after the occurrence of liquefaction-induced settlements.

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

6.2 Geotechnical Design Considerations

General

Artificial fill materials were encountered in the eastern portion of the site in the upper 3 to 10½± feet below the ground surface. The fill materials encountered at the boring and trench locations generally consist of loose to medium dense fine to coarse sands, silty sands, and debris consisting of concrete tiles, fragments and slabs. Other debris was encountered in various quantities throughout the sandy fill soils, consisting of plastic, asphalt fragments, brick fragments, metallic fragments and wires. The fill materials encountered at the site are considered to be undocumented fill, and are not suitable for the support of the proposed structure.

The near surface native alluvial soils encountered within the upper 10± feet at all of the boring locations, possess very loose to loose relative densities. These materials possess and unfavorable consolidation and collapse characteristics, particularly in the upper 4± feet below the ground surface. These soils are not considered suitable for the support of the proposed structure in their current state, since they could be subject to excessive post-construction settlements. Therefore, remedial grading is considered warranted within the proposed building area in order to remove the artificial fill in its entirety and a portion of the near-surface alluvial soils and replace these materials as compacted structural fill.

During our review of readily available historic aerial photographs, we observed an excavation located in the northeastern portion of the subject site. The area of this excavation is indicated on the Boring and Trench Location Plan, enclosed as Plate 2 in Appendix B, of this report. Based on our review of several aerial photographs from the year 2009, this excavation was performed and backfilled in the year 2009. The shape of the excavation was rectangular with dimensions of approximately 300 by 275± feet, including the slopes into the excavation. The depth of the excavation is not known. Based on e-mail correspondence with a representative of the client, we understand that the purpose of this excavation was to detain water discharged during the construction of a well. Since the depth of the excavation is unknown, there is a potential for deeper fills to be present in this former excavation area. However, based on the apparent slope widths as observed on the photograph, and an assumed slope inclination of 2h:1v, the depth of the excavation may be on the order of 40± feet.

Our review of aerial photographs from the years of 1938 to 1968 indicates that the northwestern bank of the Santa Ana River encroached into the southeast portion of the present day subject site. Since this area of the site was subsequently graded to a relatively flat ground surface, artificial fill materials are expected to be present throughout the previously flooded area. This area is also indicated on the Boring and Trench Location Plan.

Trench Nos. T-3 and T-5 encountered fill materials which consisted primarily of concrete tiles and large concrete fragments. These materials were 7 to 8± feet in thickness and very difficult to excavate with a backhoe. **In order to perform the recommended remedial grading, it may be necessary to utilize an excavator in areas where the fill materials primarily consist of concrete debris.**

Settlement

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

Expansion

The near surface soils have been determined to possess a very low expansion potential (EI = 7). All imported fill soils should have very low expansive characteristics. In addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintaining 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.

Soluble Sulfates

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

Shrinkage/Subsidence

Removal and recompaction of the near surface native soils is estimated to result in an average shrinkage of 12 to 15 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 dependant 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

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 grading and foundation plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

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

Site Stripping and Demolition

Initial site stripping should include removal of any surficial vegetation. This should include any weeds, grasses, shrubs, and trees. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Demolition of the existing structures and surrounding improvements will be required at this site. Demolition of the structures should include all foundations, floor slabs, and any associated utilities. Any excavations associated with demolition should be backfilled with compacted fill soils.

All remnants of the previous structures, including foundations, floor slabs, and debris resulting from demolition activities should be properly disposed of off-site. Alternatively, 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.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building area in order to remove the upper portion of the alluvial soils and all of the artificial fill materials in their entirety. Based on conditions encountered at the boring locations, the existing soils within the proposed building area are recommended to be overexcavated to a depth of at least 5 feet below the proposed building pad subgrade elevation and to a depth of at least 5 feet below existing grade, whichever is greater. The depth of the overexcavation should also extend to a depth sufficient to remove all artificial fill soils or any soils disturbed during demolition. Artificial fill materials extended to depths 3 to 10½± feet at the boring and trench locations.

Where not encompassed within the general building pad overexcavation, additional overexcavation should be performed within the influence zones of the new foundations, to provide for a new layer of compacted structural fill extending to a depth of 4 feet below proposed bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building perimeter, and to an extent equal to the depth of fill below the new foundations. If the proposed structure incorporates any exterior columns (such as for a canopy or overhang) the 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. **Trench No. T-1 encountered a very moist clayey silt to silty clay stratum between depths of 5½ to 7± feet.** If very moist silt or clay layers are encountered at the base of the overexcavation, some subgrade stabilization may be required. Scarification and air drying of these materials may be sufficient to obtain a stable subgrade. However, if highly unstable soils are identified, and if the construction schedule does not allow for delays associated with drying, the silty clay/clayey silt stratum may be removed in its entirety. The depth of removal should be evaluated by the geotechnical engineer of record to verify the suitability to serve as the structural fill subgrade. Some localized areas of deeper excavation may be required if additional fill materials or loose, porous, or low density native soils are encountered at the base of the overexcavation.

Trench Nos. T-3 and T-5 encountered fill materials which consisted primarily of concrete tiles and large concrete fragments. These materials were 7 to 8± feet in thickness and very difficult to excavate with a backhoe. **In order to perform the recommended remedial grading, it may be necessary to utilize an excavator in areas where the fill materials primarily consist of concrete debris.**

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 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

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

The existing soils within the areas of any proposed retaining walls should be overexcavated to a depth of 3 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pad. Additional overexcavation should be performed to remove any undocumented fill soils. 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 and Drive Areas

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

The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. Any such materials should be removed to a level of firm and unyielding soil. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to at least 2 to 4 percent above the optimum moisture content, 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 San Bernardino.
- 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 very low expansive (EI < 20), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. 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). It is recommended that materials in excess of 3 inches in size not be used for utility trench backfill. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by City of San Bernardino. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

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

6.4 Construction Considerations

Excavation Considerations

The near surface soils generally consist of sands, silty sands and sandy silts. These materials will likely be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 2h:1v. 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.

Trench Nos. T-3 and T-5 encountered fill materials which consisted primarily of concrete tiles and large concrete fragments. These materials were 7 to 8± feet in thickness and very difficult to excavate with a backhoe. **In order to perform the recommended remedial grading, it may be necessary to utilize an excavator in areas where the fill materials primarily consist of concrete debris.**

Moisture Sensitive Subgrade Soils

The near surface soils possess appreciable silt 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.

Groundwater

The static groundwater table at this site is considered to exist at a depth of approximately 38 to 40± feet. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by structural fill soils used to replace existing low strength, collapsible, near surface soils alluvial soils and all of the undocumented fill materials. These new structural fill soils are expected to extend to depths of at least 4 feet below proposed foundation bearing grade, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, the proposed structure may be supported on shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 3,000 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) in strip footings, due to the presence of potentially liquefiable soils.
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

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

Foundation Construction

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

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

Estimated Foundation Settlements

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

Lateral Load Resistance

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

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

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

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floor of the new structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, extending to a depth of at least 5 feet below proposed finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 5 inches.

- Minimum slab reinforcement: Not required for soil conditions. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

- Slab underlayment: If moisture sensitive floor coverings will be used, the minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of floor slab where the floor coverings will be placed. 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 will not be used, 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 slab should be completed by the structural engineer to verify adequate thickness and reinforcement. Additional rigidity may be necessary for structural considerations.

6.7 Retaining Wall Design and Construction

Retaining walls are expected to be necessary within truck dock areas. Additionally, although not indicated on the site plan, some small (less than 3 to 5± feet in height) retaining walls may be required to facilitate the new site grades. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

Based on the 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 select granular material. The near surface, on-site soils generally consist of fine to coarse sands, silty fine sands and occasional fine sandy silts. Based on the composition of the on-site soils, these soils have been assigned a friction angle of 30 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 heel of the retaining wall upwards at an angle of approximately 60° from horizontal.

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type: Imported Aggregate Base	Soil Type: On-Site Sands and Silty Sands
Internal Friction Angle (ϕ)		38°	30°
Unit Weight		130 lbs/ft^3	125 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	30 lbs/ft^3	41 lbs/ft^3
	Active Condition (2h:1v backfill)	44 lbs/ft^3	63 lbs/ft^3
	At-Rest Condition (level backfill)	50 lbs/ft^3	67 lbs/ft^3

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

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

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

Seismic Lateral Earth Pressures

In 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 0.45g. This peak site acceleration is equal to S_{DS}/2.5, in accordance with the 2010 CBC.

Retaining Wall Foundation Design

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

Backfill Material

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

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

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

Subsurface Drainage

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

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.

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

6.8 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The on-site soils generally consist of silty sands, sands and sandy silts. These soils are considered to possess good pavement support characteristics with estimated R-values of 45 to 60. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon an assumed R-value of 50. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

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

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

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

ASPHALT PAVEMENTS (R=50)					
Materials	Thickness (inches)				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	3	4	5	5	7
Compacted Subgrade	12	12	12	12	12

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)			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	6	7	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. 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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Youd, T. L. and Idriss, I. M. (Editors), "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," Salt Lake City, UT, January 5-6 1996, NCEER Technical Report NCEER-97-0022, Buffalo, NY.

APPENDIX A

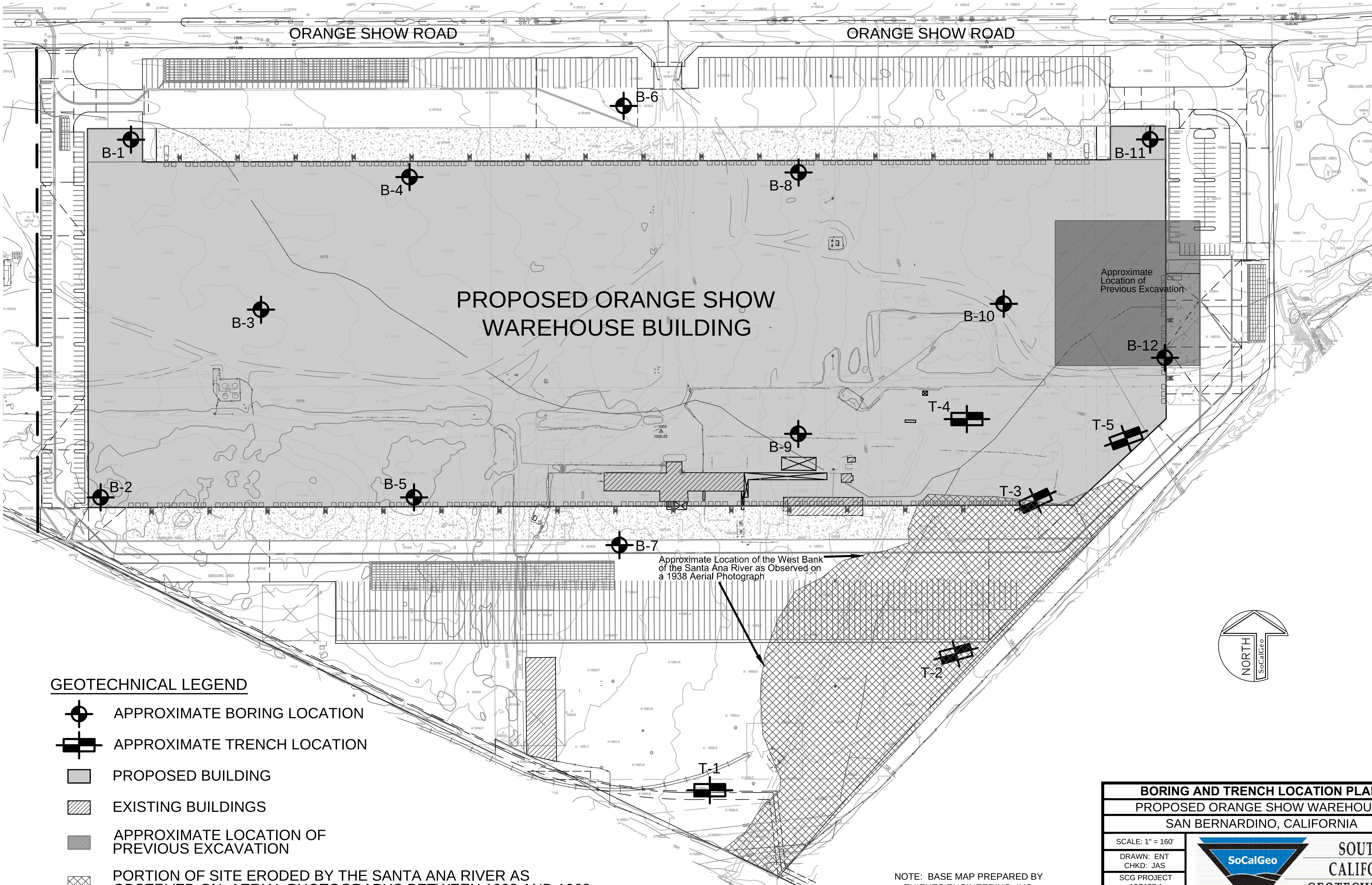
SOURCE: SAN BERNARDINO COUNTY
THOMAS GUIDE, 2009

SITE LOCATION MAP

PROPOSED ORANGE SHOW WAREHOUSE

SAN BERNARDINO, CALIFORNIA

SCALE: 1" = 2400'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: ENT CHKD: JAS	
SCG PROJECT 13G157-1	
PLATE 1	



GEOTECHNICAL LEGEND

- APPROXIMATE BORING LOCATION
- APPROXIMATE TRENCH LOCATION
- PROPOSED BUILDING
- EXISTING BUILDINGS
- APPROXIMATE LOCATION OF PREVIOUS EXCAVATION
- PORTION OF SITE ERODED BY THE SANTA ANA RIVER AS OBSERVED ON AERIAL PHOTOGRAPHS BETWEEN 1938 AND 1968

NOTE: BASE MAP PREPARED BY THIENES ENGINEERING, INC.

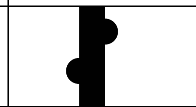
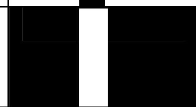
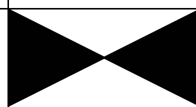
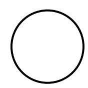
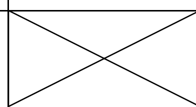
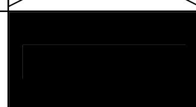
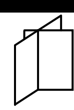
BORING AND TRENCH LOCATION PLAN
PROPOSED ORANGE SHOW WAREHOUSE
SAN BERNARDINO, CALIFORNIA

SCALE: 1" = 160'
DRAWN: ENT
CHKD: JAS
SCG PROJECT 13G157-1
PLATE 2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

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

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

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

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

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

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

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

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

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

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 13G157
PROJECT: Orange Show Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/24/13
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

WATER DEPTH: Dry
CAVE DEPTH: 18 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray Brown Silty fine Sand to fine Sandy Silt, trace Iron oxide staining, loose-dry to damp							
	X	4					3					
	X	4					7					
5					Brown fine Sand, trace medium to coarse Sand, trace to little Silt, loose-damp to moist							
	X	6					4					
	X	5					11					
10												
					Gray Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, medium dense-dry							
15	X	22					1					
					Dark Gray to Dark Gray Brown fine Sandy Silt, trace medium Sand, medium dense-moist							
20	X	11					15					
					Gray to Dark Gray Silty Clay, with thinly interbedded lenses of Silty fine Sand, trace Iron oxide staining, stiff-very moist							
25	X	15					27					
	X	16			Blue Gray to Gray Brown fine Sandy Silt, medium dense-very moist		20					
					Dark Gray Silt, trace calcareous nodules, very stiff-very moist		39					
30												
					Boring Terminated at 30'							

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 13G157
PROJECT: Orange Show Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/24/13
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

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

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray Brown Silty fine Sand to fine Sandy Silt, trace fine root fibers, loose to medium dense-dry to damp							
	X	15				85	2					
					Brown Silty fine to medium Sand, porous, loose-dry to damp							
	X	11				107	2					
5					Light Gray Brown to Brown fine Sand, trace medium to coarse Sand, trace fine Gravel, loose-dry to damp							
	X	14				97	1					
	X	12				93	3					
					Gray Brown fine Sand, trace to little Silt, loose-damp							
	X	14				94	7					
10												
					Brown fine to coarse Gravel, trace fine Gravel, medium dense-dry to damp							
15	X	23				101	2					
	X	25					2					
20												

Boring Terminated at 20'

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 13G157	DRILLING DATE: 7/24/13	WATER DEPTH: 40 feet
PROJECT: Orange Show Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 25 feet
LOCATION: San Bernardino, 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: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray fine Sandy Silt, porous, loose-dry							
		13				98	2					
					Gray Silty fine Sand, slightly porous, loose-dry							
		7			Gray Brown fine to medium Sand, trace fine Gravel, loose-dry	100	1					
5		9			Light Gray fine to coarse Sand, loose-dry	99	1					
		9				94	2					
		12			Gray Brown fine Sand, trace to little Silt, loose-damp	92	3					
10												
					Brown fine to medium Sand, little coarse Sand, trace fine Gravel, medium dense-dry to damp							
15		17					2			4		
					Dark Gray Brown Silty fine Sand, medium dense-moist							
20		11					14			26		
					Dark Gray fine Sandy Clay, trace Iron oxide staining, medium dense-very moist							
25		13					22	44	26	77		
					Dark Gray Silty fine Sand, trace medium Sand, medium dense-very moist							
30		25					23			37		

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 13G157
PROJECT: Orange Show Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/24/13
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

WATER DEPTH: 40 feet
CAVE DEPTH: 25 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
	X	17			Dark Gray Silty fine Sand, trace medium Sand, medium dense-very moist to wet		23			49		
					Gray fine to coarse Sand, trace fine Sand, medium dense-very moist to wet		17			7		
40	X	21			Dark Gray Clayey Silt, medium dense-wet		18				6	
							24	38	24	72		
45	X	29			@ 45 feet, occasional thin interbedded Silt lenses		28			80		
					Blue Gray fine Clayey Silt, little fine Sand, trace Organics, medium dense-wet							
50	X	17					39	46	34	88		

Boring Terminated at 51½'

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 13G157
PROJECT: Orange Show Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/24/13
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

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

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray Brown Silty fine Sand, trace calcareous veining, medium dense-dry							
	X	14				92	1					
	X	5			Brown Silty fine to medium Sand, loose-damp	106	3					
5	X	7			Light Gray fine to coarse Sand, loose-dry	104	1					
	X	14			Gray Brown fine Sand, loose-dry to damp	98	1					
	X	12				90	4					
10												
					Orange Brown fine Sand, trace coarse Sand, trace fine Gravel, medium dense-damp							
15	X	16					4					
					Gray Brown fine to coarse Sand, trace fine Gravel, medium dense-dry to damp							
	X	27					2					
20												

Boring Terminated at 20'

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

BORING NO. B-5

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 13G157
PROJECT: Orange Show Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/24/13
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray Brown Silty fine Sand, trace fine root fibers, trace calcareous veining, slightly porous, loose-dry							
		8					1					
		6					1					
5												
		6					2					
					Light Gray Brown Silty fine to medium Sand, trace coarse Sand, loose-damp							
		6					6					
10												
					Gray fine to coarse Sand, medium dense-dry							
15		17					1					
					Dark Gray Silty Clay to Clayey Silt, trace Iron oxide staining, trace calcareous nodules, medium stiff-very moist							
20		7					39					
		9				93	29					
					Gray thinly interbedded lenses of Silty Clay and Silty fine Sand, fine Sandy Silt, stiff to medium dense-very moist to wet							
25		13					32					
		14					14					
30					Boring Terminated at 30'							

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 13G157
PROJECT: Orange Show Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/24/13
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

WATER DEPTH: Dry
CAVE DEPTH: 3 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	9			ALLUVIUM: Light Gray Brown fine Sandy Silt, trace fine root fibers, trace calcareous veining, loose-damp		3					
	X	11			Light Brown Silty fine Sand, medium dense-damp		3					
5					Boring Terminated at 5'							

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-7

JOB NO.: 13G157	DRILLING DATE: 7/25/13	WATER DEPTH: Dry
PROJECT: Orange Show Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 5 feet
LOCATION: San Bernardino, 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: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					5± inches Asphaltic concrete, no discernible Aggregate base							
	X	9			ALLUVIUM: Gray Brown fine Sand, trace medium Sand, trace Silt, loose-damp		4					
	X	6					6					
5												
					Boring Terminated at 5'							

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 13G157
PROJECT: Orange Show Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/24/13
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

WATER DEPTH: Dry
CAVE DEPTH: 17 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	8			FILL: Light Gray fine Sandy Silt to Silty fine Sand, loose-dry to damp		2					
5	X	6			FILL: Light Gray Silty fine to medium Sand, trace Plastic debris, loose-dry to damp		2					
	X	6			ALLUVIUM: Light Gray Brown fine to coarse Sand, loose-dry		1					
10	X	4			Dark Brown Silty fine Sand, trace medium to coarse Sand, very loose to loose-moist		12					
15	X	9			Gray Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, loose-moist		12					
20	X	31			Orange Brown fine to medium Sand, trace coarse Sand, dense-damp		3					
25	X	14			Light Gray fine to coarse Sand, trace fine Gravel, medium dense-damp		2					
					Blue Gray to Dark Gray Silty Clay, trace fine Sand, stiff-very moist		34					
	X	35			Dark Gray Silty fine Sand, trace medium Sand, medium dense-moist		14					
30					Dark Gray fine to medium Sand, trace coarse Sand, trace fine Gravel, medium dense-damp		5					

Boring Terminated at 30'

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-9

JOB NO.: 13G157
PROJECT: Orange Show Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/25/13
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

WATER DEPTH: Dry
CAVE DEPTH: 14 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	21		FILL: Light Gray Brown fine Sandy Silt to Silty fine Sand, trace Iron oxide staining, slightly porous, medium dense-damp to moist	99	9					
	10		FILL: Light Brown fine to medium Sand, trace coarse Sand, medium dense-very moist	89	17					
5	10		ALLUVIUM: Light Gray fine Sand, medium dense-dry to damp	83	2					
	8		@ 7 to 8 feet, moist	86	8					
10	18		Gray fine to coarse Sand, little fine Gravel, medium dense-damp	112	5					
15	20		Light Gray fine to medium Sand, trace coarse Sand, trace fine Gravel, loose to medium dense-dry to damp		2					
	4	1.25	Dark Gray Silty Clay to Clayey Silt, trace calcareous nodules, trace to little fine Sand, soft-very moist		36					
20	14	1.75			29					
					11					
25	10		Dark Gray Silty fine Sand, trace medium Sand, medium dense-very moist	100	21					

Boring Terminated at 25'

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-10

JOB NO.: 13G157
PROJECT: Orange Show Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/24/13
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

WATER DEPTH: 38 feet
CAVE DEPTH: 21 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					FILL: Gray Brown fine Sand, trace to little Silt, medium dense-dry to damp							
	X	24					2					
					ALLUVIUM: Gray fine to medium Sand, loose-dry to damp							
	X	9					2					
5												
	X	8			@ 6 to 11 feet, trace fine to coarse Gravel		2					
	X	4					2					
10	■	12				100	1					
					Light Gray fine to medium Sand, little coarse Sand, little fine Gravel, medium dense-dry							
15	X	23					1			6		
					Dark Gray Clayey Silt to Silty Clay, trace fine Sand, medium stiff-very moist							
20	X	8	2.75				31	38	13	88		
					Gray Brown Silty Clay, stiff-very moist							
25	X	15	3.0				39			94		
					Gray Brown to Brown Silty fine Sand, medium dense-moist		13			28		
30	X	12	2.25		Dark Gray Clayey Silt, little fine Sand, with interbedded layers of Silt, trace medium Sand, medium dense-very moist		34	48	28	79		
					Gray to Dark Gray fine to medium Sand, dense to very dense-moist to wet							

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-10

JOB NO.: 13G157
PROJECT: Orange Show Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/24/13
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

WATER DEPTH: 38 feet
CAVE DEPTH: 21 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
	X	44			Gray to Dark Gray fine to medium Sand, dense to very dense-moist to wet		5					
40	X	67			@ 41 feet, 2" thick lense of Silty fine Sand		16					
					Dark Gray fine to coarse Sand, trace fine Gravel, dense to very dense-very moist to wet							
45	X	50					14					
					Blue Gray to Dark Gray Silty fine to medium Sand, dense-wet							
50	X	35					19			21		
					Boring Terminated at 51½'							

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-11

JOB NO.: 13G157
PROJECT: Orange Show Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/25/13
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Brett Isen

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

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (SURFACE ELEVATION: --- MSL)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray Brown Silty fine Sand to fine Sandy Silt, trace fine root fibers, loose-dry to damp							
	X	9				100	1					
	X	6				89	2					
5	X	9				90	2					
	X	8				93	2					
					Dark Gray Brown Silty fine Sand, trace Iron oxide staining, loose-damp							
	X	9				102	8					
10					Gray fine Sand, loose-damp							
					Gray fine to coarse Sand, trace fine Gravel, medium dense-dry							
15	X	31				97	1					
					Dark Gray fine Sandy Silt, some Clay, trace calcareous nodules, slightly porous, stiff-very moist							
	X	19				101	18					
20					Boring Terminated at 20'							

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-12

JOB NO.: 13G157	DRILLING DATE: 7/25/13	WATER DEPTH: Dry
PROJECT: Orange Show Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 12 feet
LOCATION: San Bernardino, California	LOGGED BY: Brett Isen	READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					FILL: Light Brown Silty fine to medium Sand, trace fine Gravel, trace Asphaltic concrete fragments, medium dense-dry to damp							
		36				116	3					
					FILL: Brown Silty fine to corase Sand, trace fine Gravel, Asphaltic concrete fragments, medium dense-dry to damp							
		30				117	2					
5		14				110	2					
					FILL: Gray Brown Silty fine Sand, trace fine Gravel, trace Iron oxide staining, trace Metallic fragments, loose-damp							
		12				103	7					
					POSSIBLE FILL: Light Gray to Brown fine Sand, loose-damp							
		5				92	3					
10												
					ALLUVIUM: Gray Brown to Dark Brown fine Sand, trace Silt, medium dense-damp							
		15					8					
15												
					Dark Gray fine Sandy Silt with thinly interbedded layers of Clayey Silt, stiff to medium dense-very moist							
		11					27					
20												
					Gray Brown Silty fine Sand, trace Iron oxide staining, medium dense-moist							
		12					14					
25			2.0		Gray Brown to Brown Silty Clay to Clayey Silt, trace Iron oxide staining, stiff-very moist		31					
					Dark Gray Brown Silty fine Sand, medium dense-very moist							
		29					17					
30												
					Boring Terminated at 30'							

TBL 13G157.GPJ SOCALGEO.GDT 8/27/13

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-1

JOB NO.: 13G157-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Orange Show Warehouse	LOGGED BY: Daryl Kas	SEEPAGE DEPTH: Dry
LOCATION: San Bernardino, CA	ORIENTATION: N 90 W	READINGS TAKEN: At Completion
DATE: 7-31-2013	TOP OF TRENCH ELEVATION: 1022 msl	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)
	b		1
	b		4
	b		4
	b		2
5			
	b		18
10			
15			

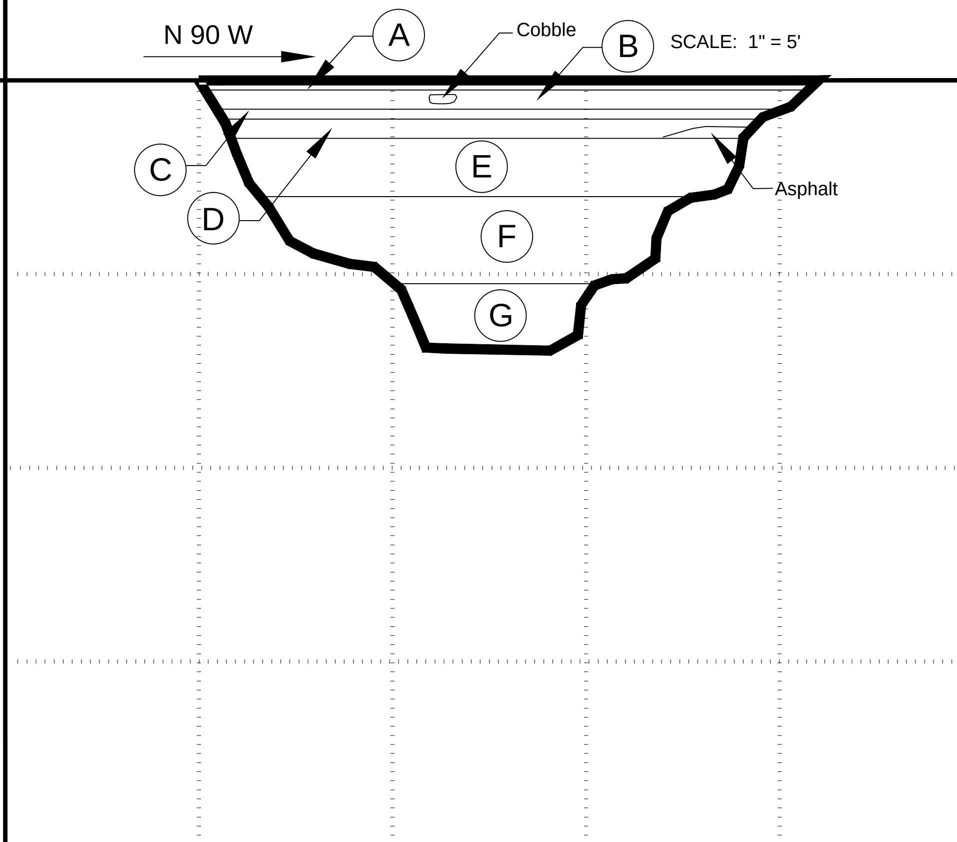
EARTH MATERIALS DESCRIPTION

A: PAVEMENT: 2 inches Asphaltic concrete, no discernable Aggregate base
B: FILL: Gray Brown Silty fine Sand, loose - damp
C: FILL: Light Brown fine to coarse Sand, trace fine to coarse Gravel, occasional Cobbles, loose to medium dense - damp
D: FILL: Dark Gray Brown fine to coarse Sand, medium dense - damp
E: FILL: Gray Brown Silty fine Sand to fine Sandy Silt, mottled, medium dense - damp
F: ALLUVIUM: Light Gray Brown fine to coarse Sand, loose - dry to damp

G: ALLUVIUM: Dark Gray Brown Silty Clay to Clayey Silt, moderately porous, stiff - very moist

Trench Terminated @ 7 '

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-13

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-2

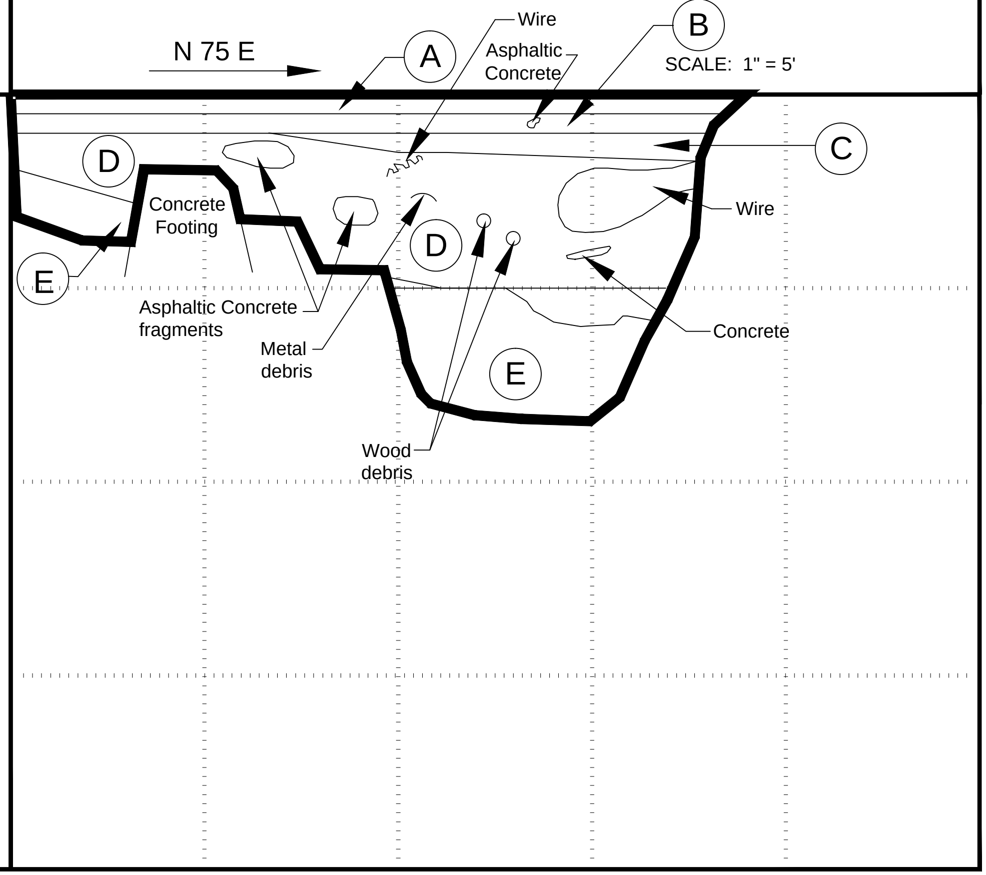
JOB NO.: 13G157-1
PROJECT: Proposed Orange Show Warehouse
LOCATION: San Bernardino, CA
DATE: 7-31-2013

EQUIPMENT USED: Backhoe
LOGGED BY: Daryl Kas
ORIENTATION: N 75 E
TOP OF TRENCH ELEVATION: 1022½ msl

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

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		3	A: AGGREGATE BASE: 4 to 6 inches
	b		3	B: FILL: Brown Silty fine to medium Sand, trace fine to coarse Gravel, trace artificial debris including Asphaltic concrete fragments, medium dense - dry to damp
	b		3	C: FILL: Gray/Brown, fine to medium Sand, trace to little Silt, plastic bag, loose - dry to damp
5				D: FILL: Brown Silty fine to coarse Sand, trace to little fine to coarse Gravel, some debris including Concrete fragments, Asphalt, Metal, Wires, Fabric, Plastic, Tile, Wood, loose - damp to moist
	b		7	
	b		1	E: ALLUVIUM: Light Gray fine to coarse Sand, trace fine to coarse Gravel, loose - dry to moist
				Trench Terminated @ 8½'
10				
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-14

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-3

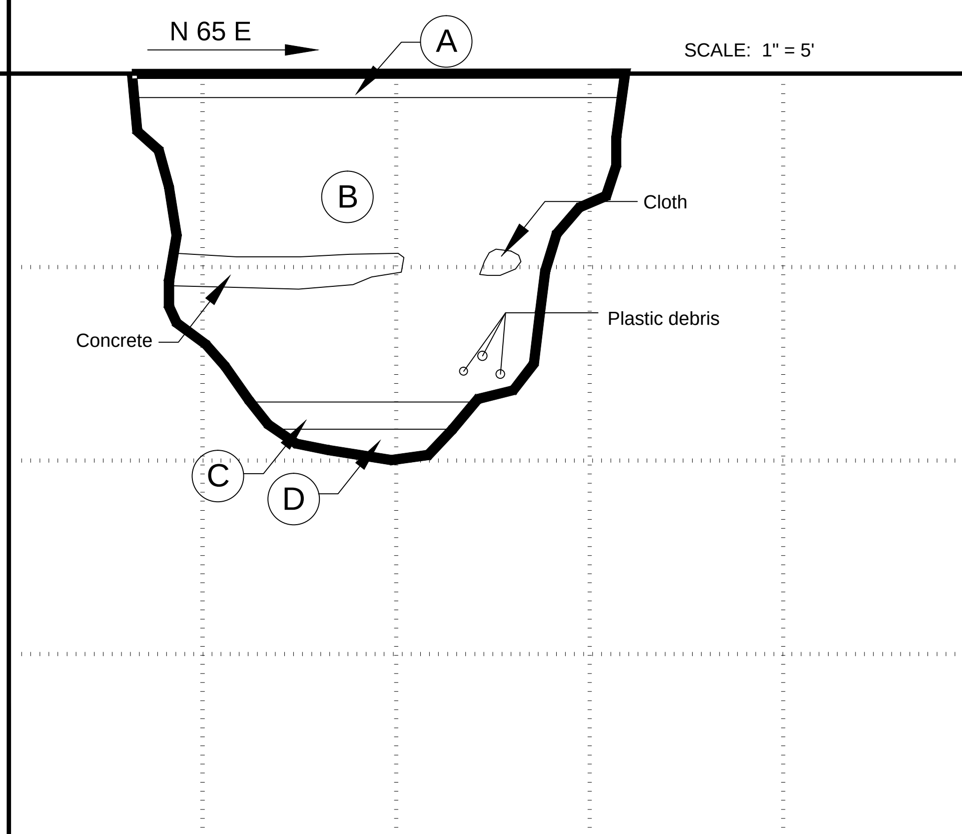
JOB NO.: 13G157-1
PROJECT: Proposed Orange Show Warehouse
LOCATION: San Bernardino, CA
DATE: 7-31-2013

EQUIPMENT USED: Backhoe
LOGGED BY: Daryl Kas
ORIENTATION: N 65 E
TOP OF TRENCH ELEVATION: 1023½ msl

WATER DEPTH:
SEEPAGE DEPTH:
READINGS TAKEN:

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: AGGREGATE BASE: 6 inches B: FILL: Red/Brown concrete tile fragments with fine to coarse Sand throughout, dense - damp
	b		4	
5				
				C: ALLUVIUM: Light Gray Brown fine to coarse Sand, trace fine to coarse Gravel, loose to medium dense - moist D: ALLUVIUM: Dark Gray Brown fine Sandy Silt, trace Iron oxide staining, loose - moist to very moist
	b		11	
10	b		16	Trench Terminated @ 10 '
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

PLATE B-15

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-4

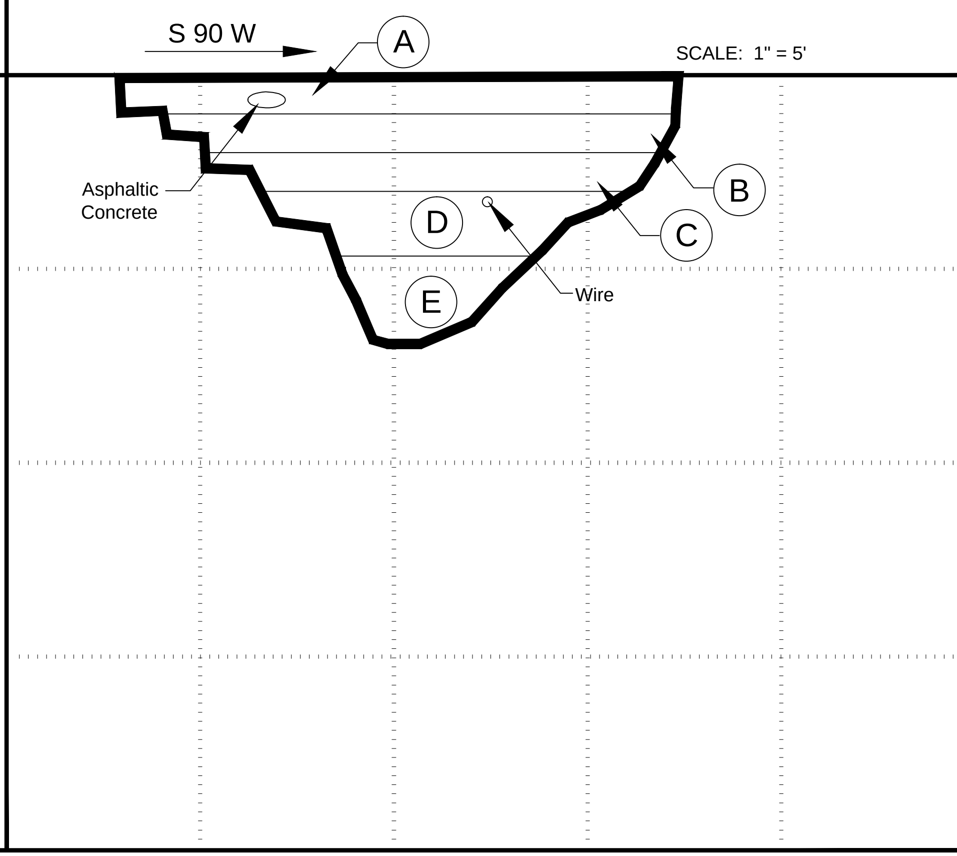
JOB NO.: 13G157-1
PROJECT: Proposed Orange Show Warehouse
LOCATION: San Bernardino, CA
DATE: 7-31-2013

EQUIPMENT USED: Backhoe
LOGGED BY: Daryl Kas
ORIENTATION: S 90 W
TOP OF TRENCH ELEVATION: 1025 msl

WATER DEPTH:
SEEPAGE DEPTH:
READINGS TAKEN:

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		4	A: FILL: Light Gray Silty Sand, little to some fine to coarse Gravel, Plastic debris, loose - damp
	b		5	B: FILL: Light Gray Brown Silty Sand, loose to medium dense - damp
	b		2	C: FILL: Light Gray fine to medium Sand, fine to coarse Gravel, loose to medium dense - dry to damp
	b		3	
	b		6	D: FILL: Dark Gray Brown Silty Sand, trace Metallic debris, loose to medium dense - damp
5				
				E: ALLUVIUM: Light Gray fine Sand, loose to medium dense - damp
	b		5	Trench Terminated @ 7 '
10				
15				

GRAPHIC REPRESENTATION



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

TRENCH LOG

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-5

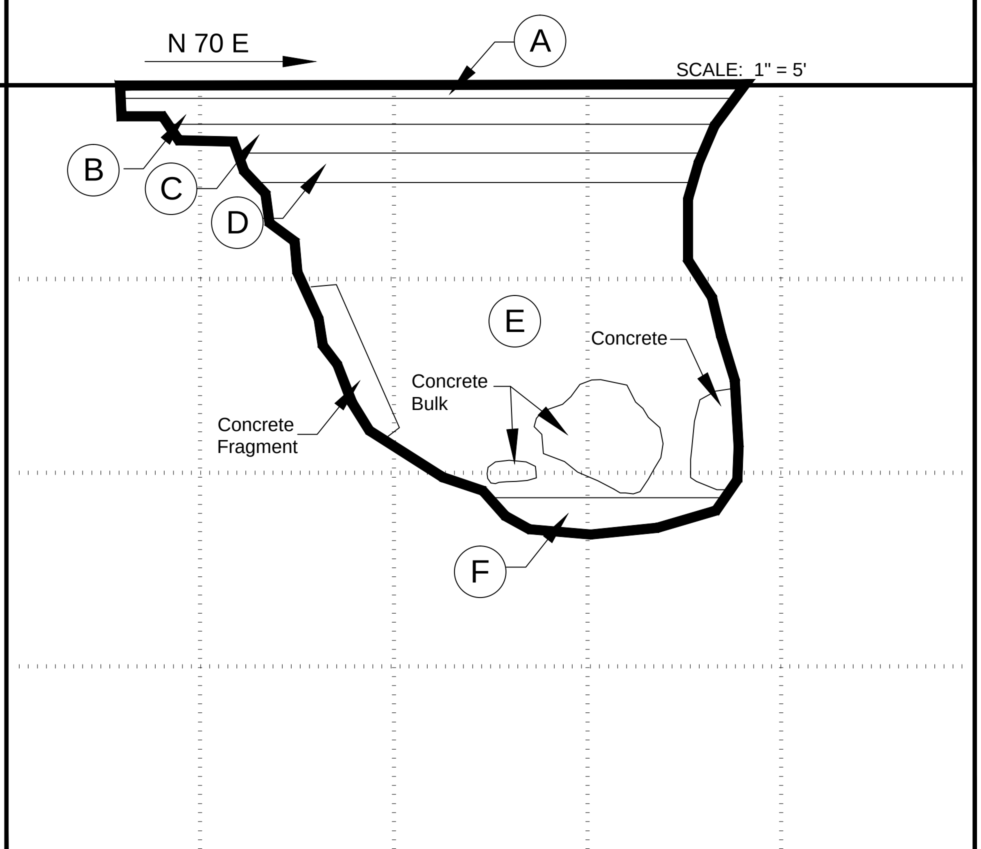
JOB NO.: 13G157-1
PROJECT: Proposed Orange Show Warehouse
LOCATION: San Bernardino, CA
DATE: 7-31-2013

EQUIPMENT USED: Backhoe
LOGGED BY: Daryl Kas
ORIENTATION: N 70 E
TOP OF TRENCH ELEVATION: 1024½ msl

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

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: AGGREGATE BASE: 4 inches
	b		1	B: FILL: Brown fine to coarse Sand, trace to little fine to coarse Gravel, medium dense - dry
	b		2	C: FILL: Gray Brown Silty fine Sand, medium dense - dry to damp
				D: CONCRETE SLAB: 6 inches
5	b		12	E: FILL: Red Brown, abundant Concrete tile fragments and several Concrete fragments with lengths or diameters in excess of 2-3 feet, with fine to coarse Sand, some debris including Asphalt fragments, Plastic, Paper, dense - moist
10	b		2	F: ALLUVIUM: Light Brown fine to coarse Sand, medium dense - dry to damp
				Trench Terminated @ 12 '
15				

GRAPHIC REPRESENTATION



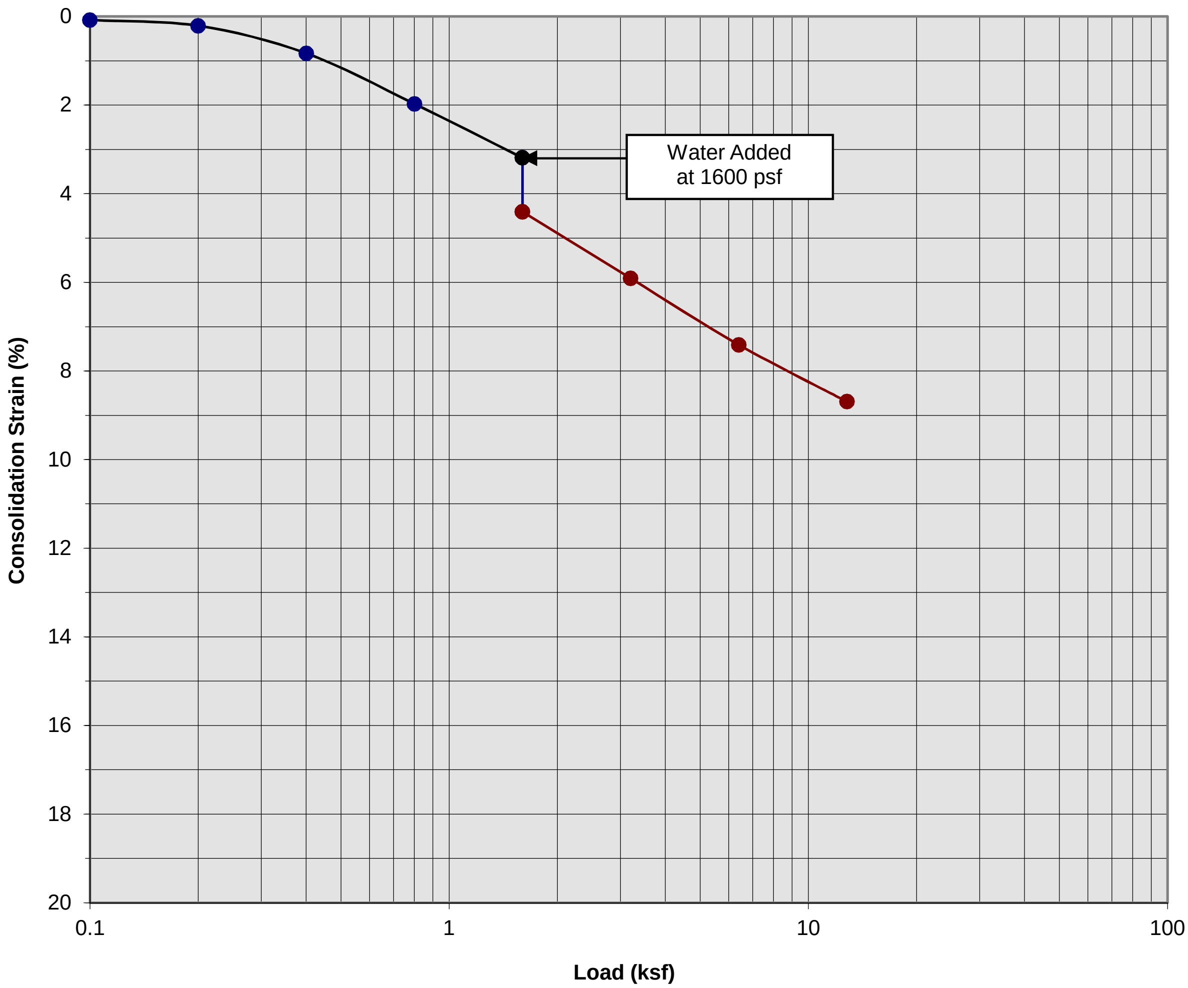
KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER (RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-17

APPENDIX C

Consolidation/Collapse Test Results



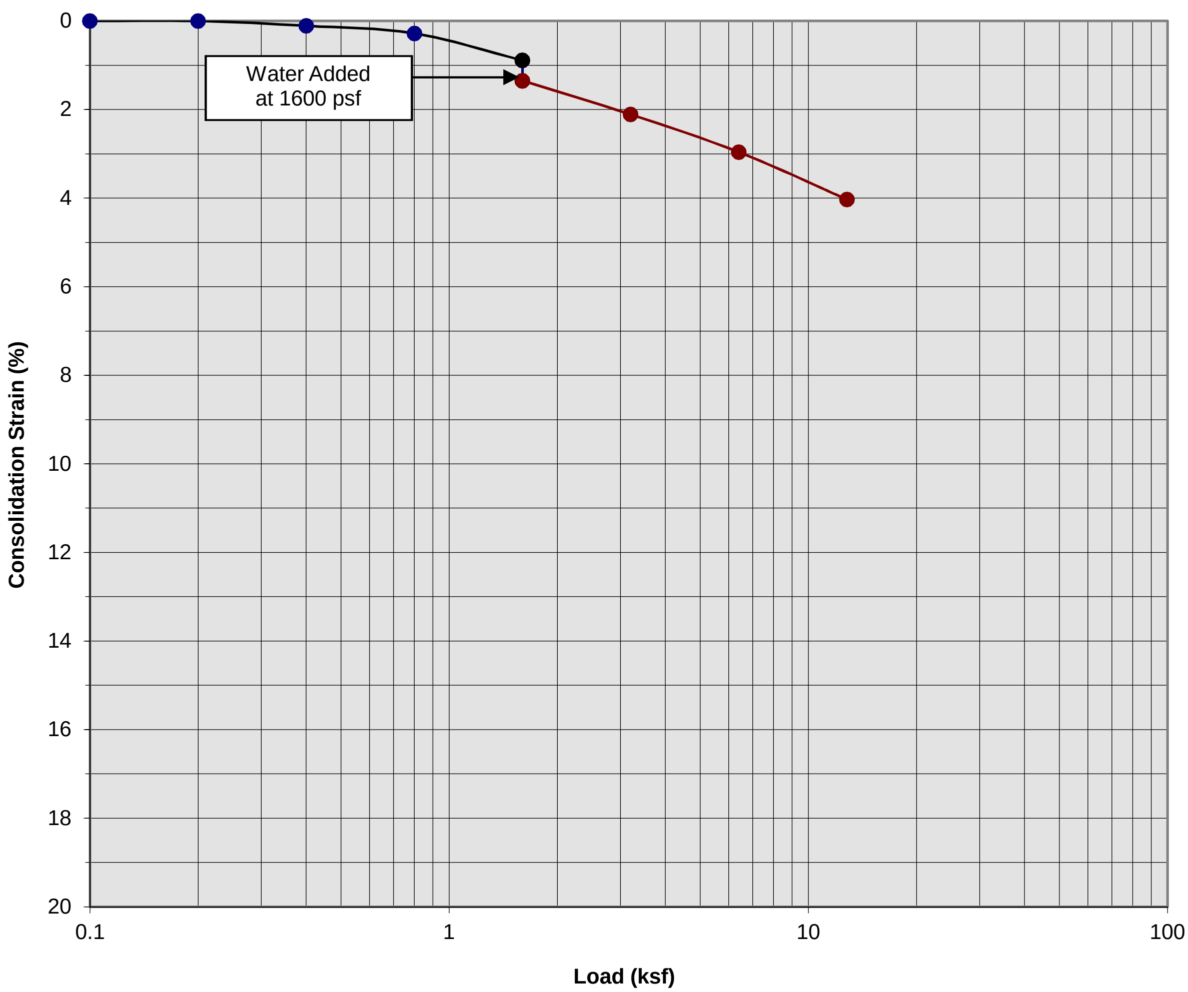
Classification: Gray Brown fine to medium Sand

Boring Number:	B-3	Initial Moisture Content (%)	1
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	3 to 4	Initial Dry Density (pcf)	100.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.22

Orange Show Warehouse
San Bernardino, California
Project No. 13G157
PLATE C- 1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

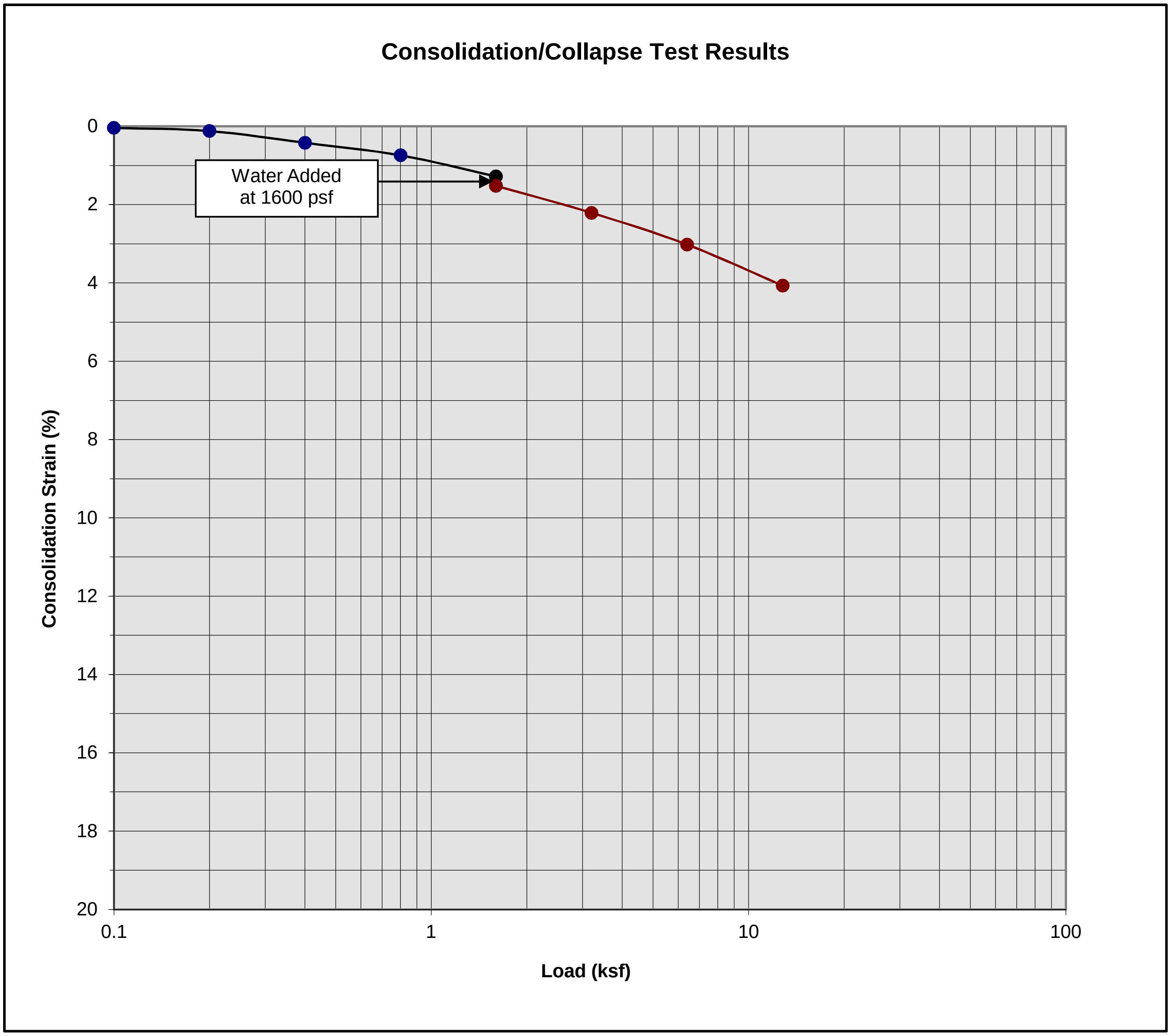


Classification: Light Gray fine to coarse Sand

Boring Number:	B-3	Initial Moisture Content (%)	1
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	5 to 6	Initial Dry Density (pcf)	99.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	103.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.46

Orange Show Warehouse
San Bernardino, California
Project No. 13G157
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

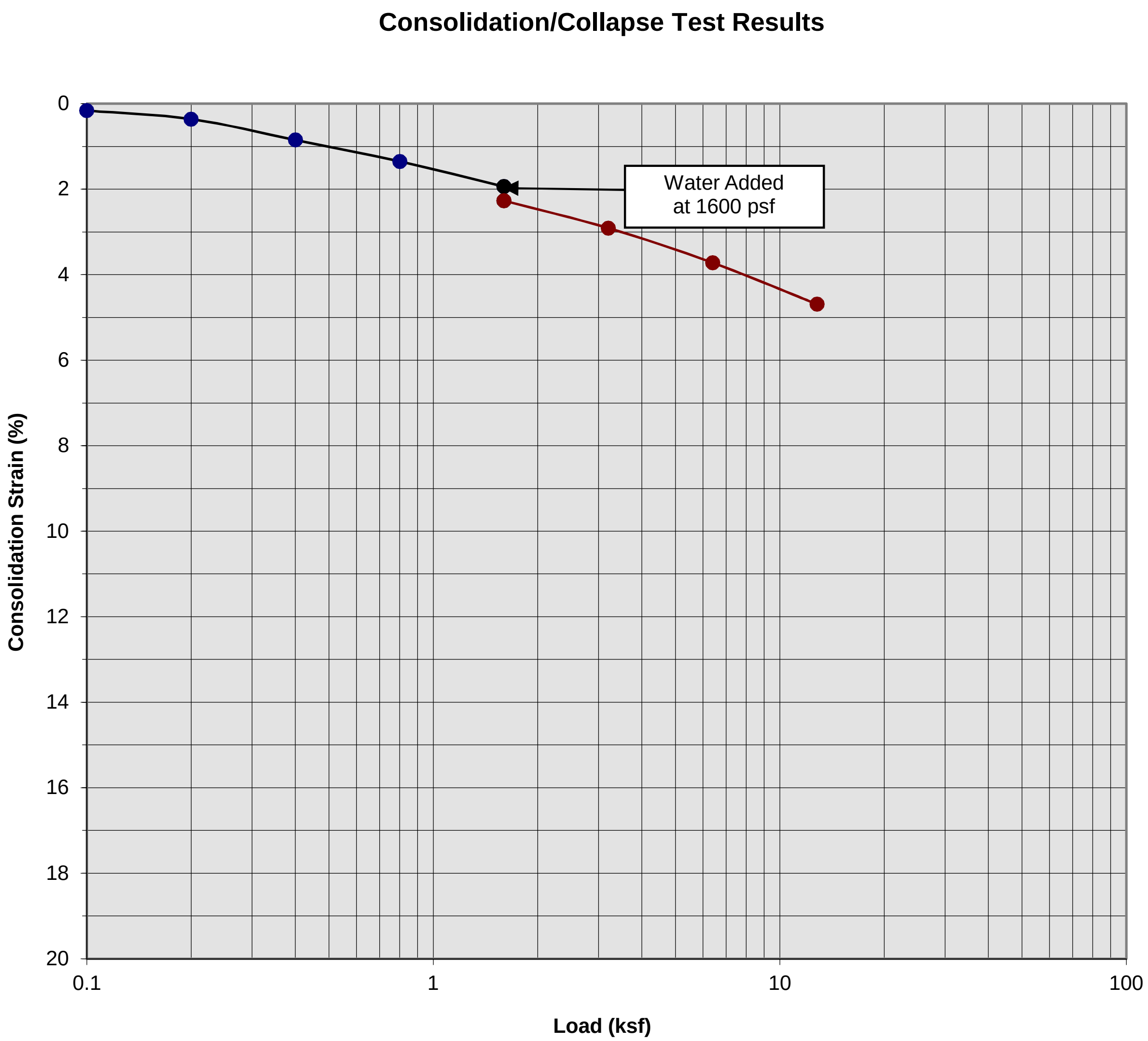


Classification: Light Gray fine to coarse Sand

Boring Number:	B-3	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	26
Depth (ft)	7 to 8	Initial Dry Density (pcf)	93.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	95.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.24

Orange Show Warehouse
San Bernardino, California
Project No. 13G157
PLATE C- 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



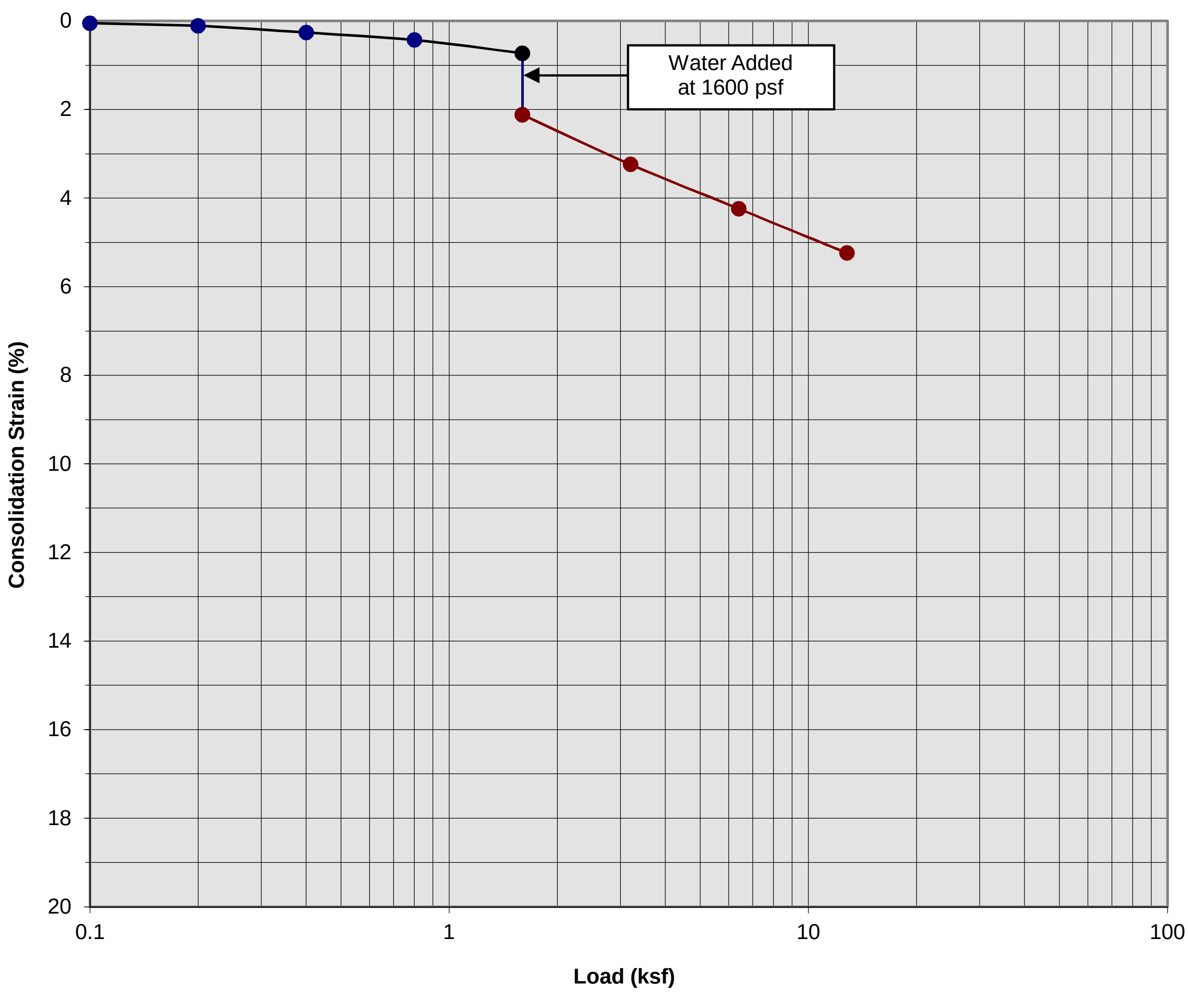
Classification: Gray Brown fine Sand, trace to little Silt

Boring Number:	B-3	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	24
Depth (ft)	9 to 10	Initial Dry Density (pcf)	93.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	98.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.33

Orange Show Warehouse
San Bernardino, California
Project No. 13G157
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

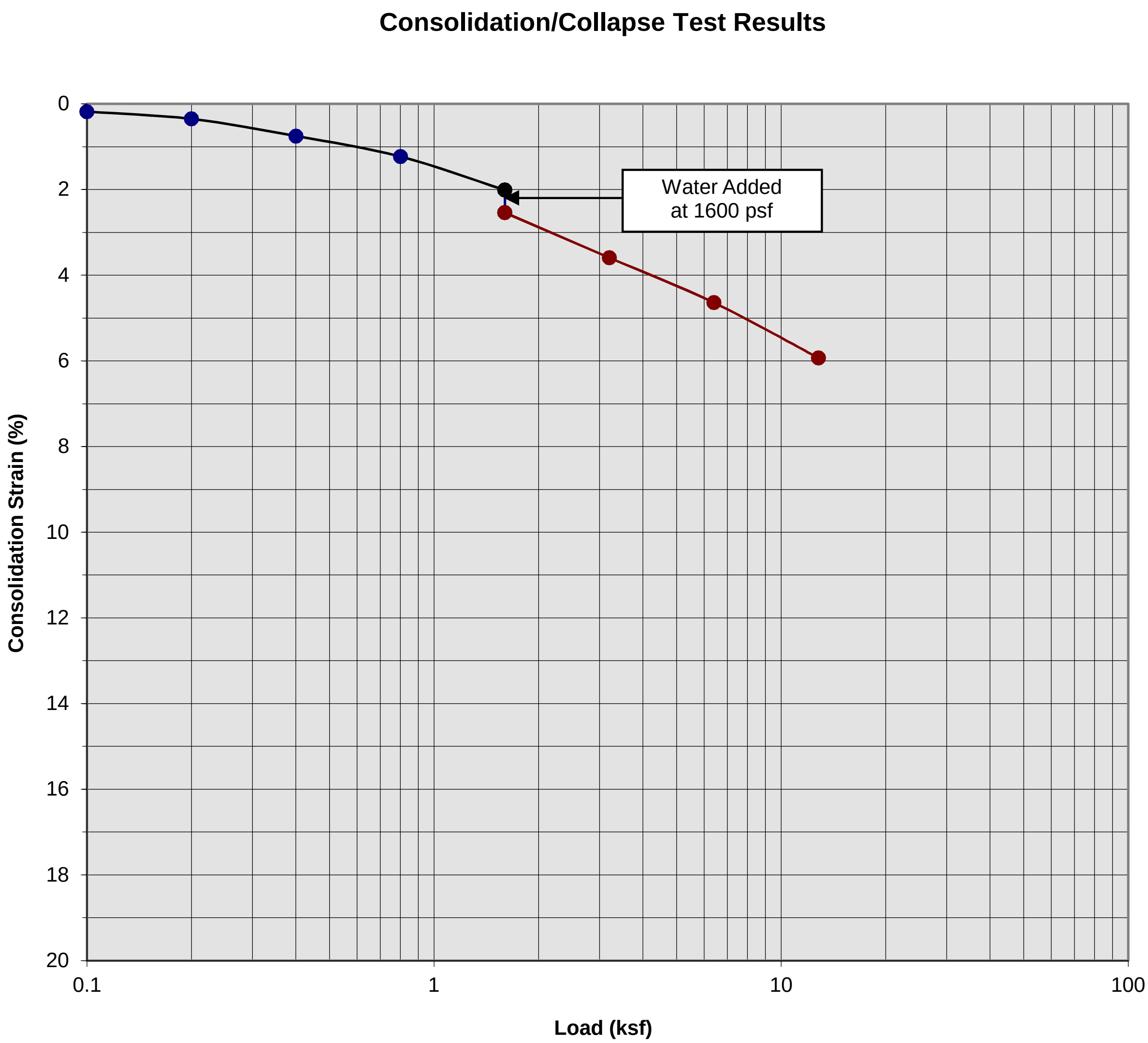


Classification: Light Brown fine to medium Sand, trace coarse Sand

Boring Number:	B-9	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	3 to 4	Initial Dry Density (pcf)	88.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	93.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.39

Orange Show Warehouse
San Bernardino, California
Project No. 13G157
PLATE C- 5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



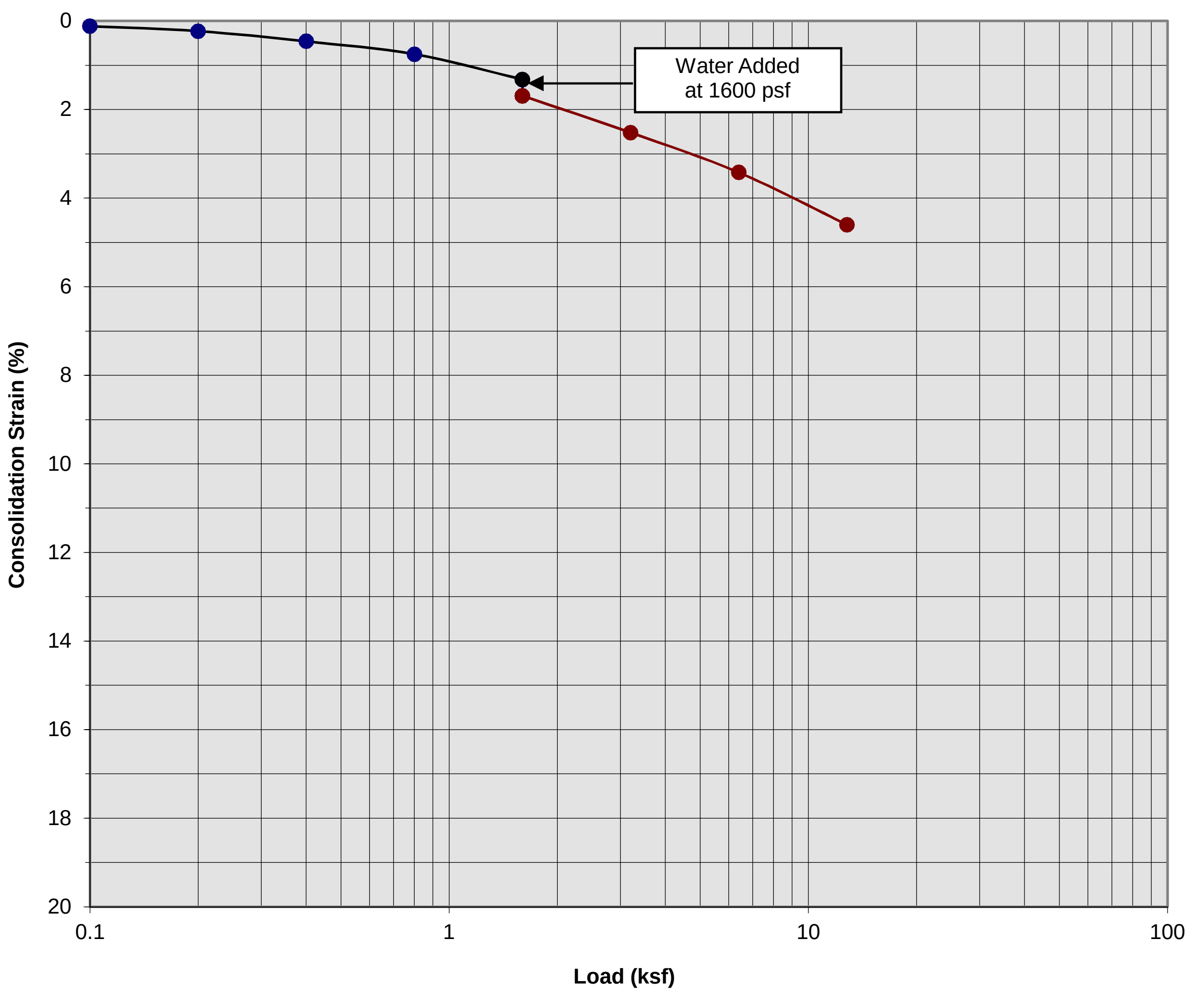
Classification: Light Gray fine Sand

Boring Number:	B-9	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	5 to 6	Initial Dry Density (pcf)	82.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	87.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.53

Orange Show Warehouse
San Bernardino, California
Project No. 13G157
PLATE C- 6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

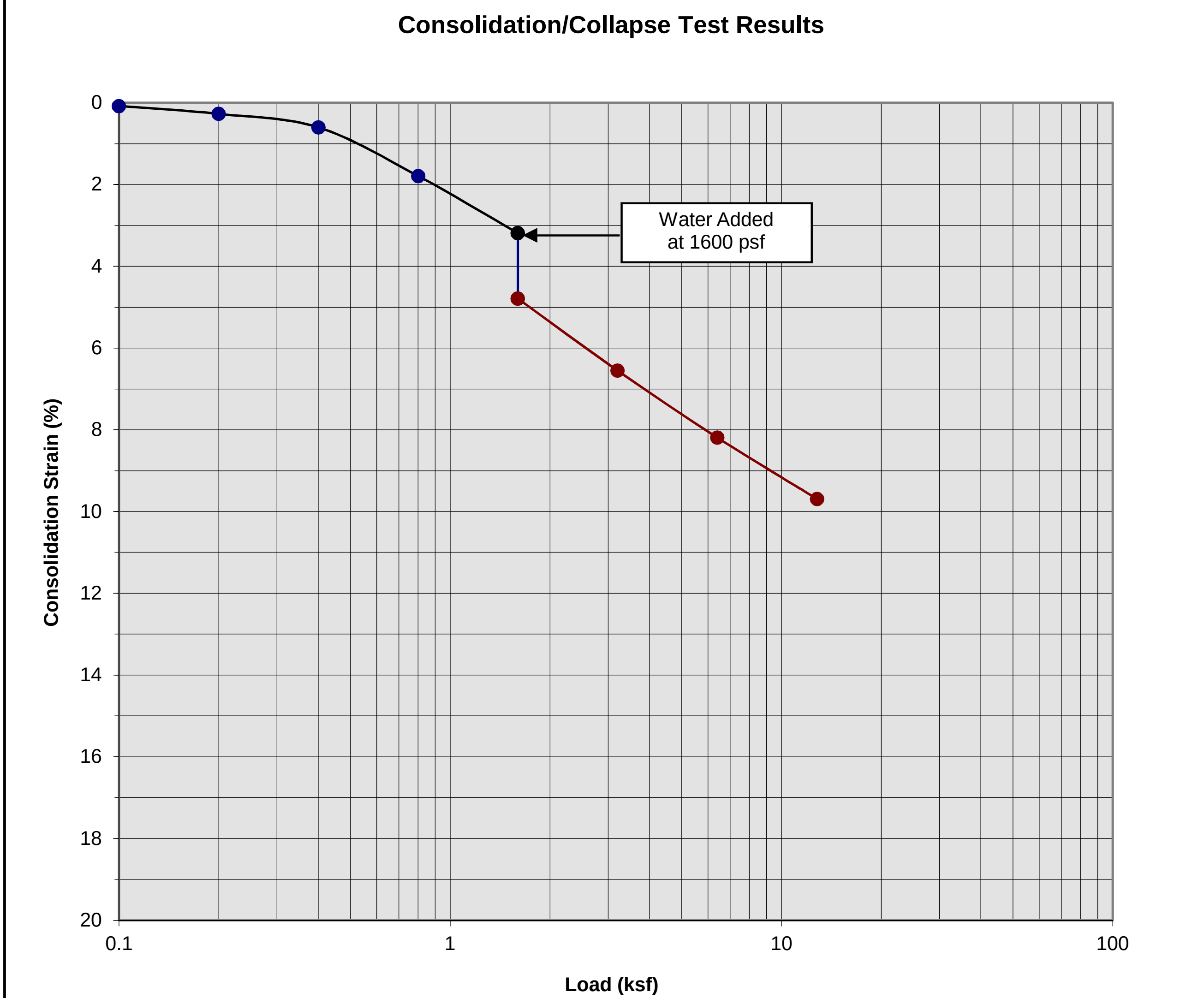


Classification: Light Gray fine Sand

Boring Number:	B-9	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	7 to 8	Initial Dry Density (pcf)	86.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	91.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.37

Orange Show Warehouse
San Bernardino, California
Project No. 13G157
PLATE C- 7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: Light Gray Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-11	Initial Moisture Content (%)	1
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	1 to 2	Initial Dry Density (pcf)	100.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.60

Orange Show Warehouse
San Bernardino, California
Project No. 13G157
PLATE C- 8

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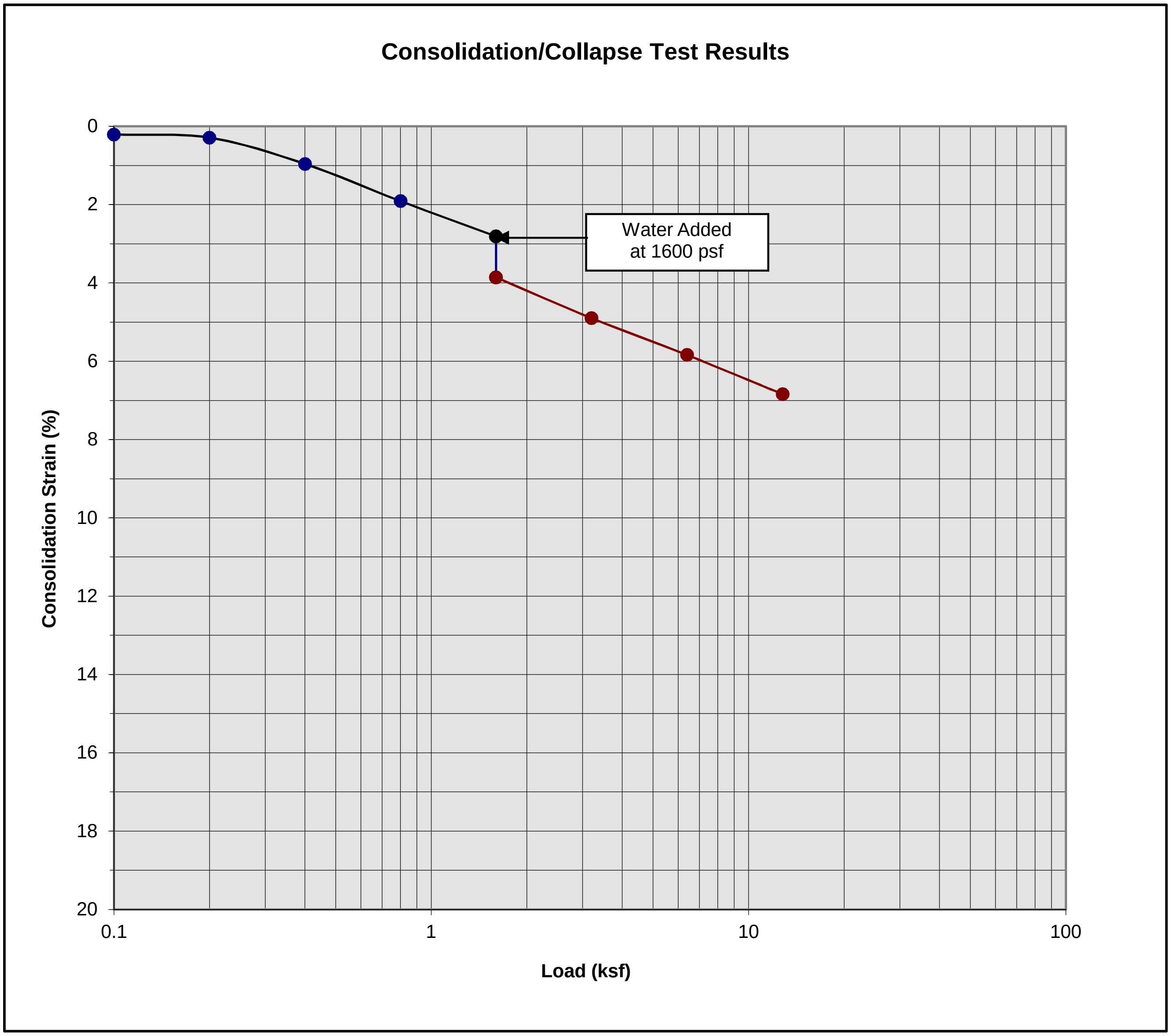
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



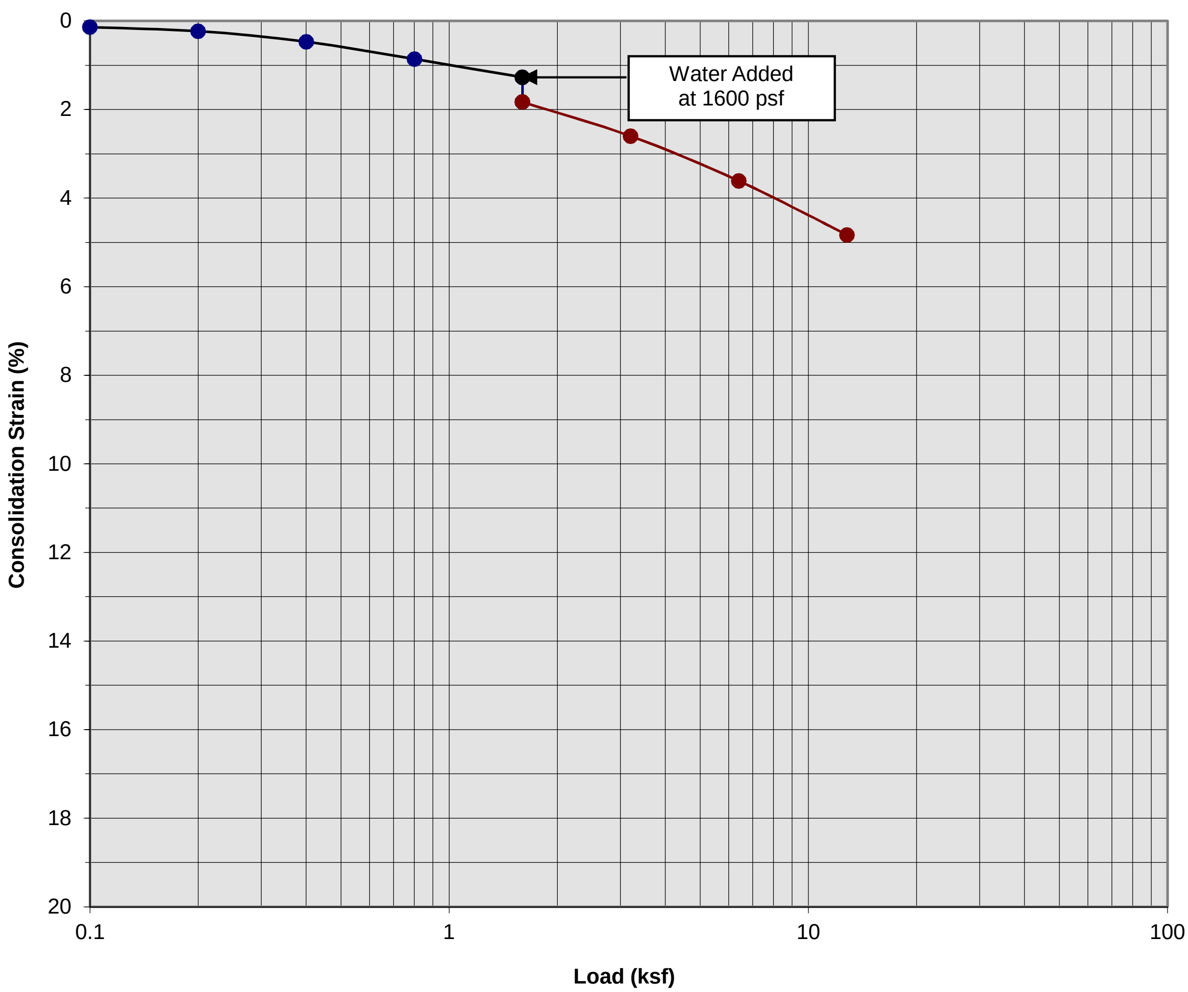
Classification: Light Gray Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-11	Initial Moisture Content (%)	1
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	3 to 4	Initial Dry Density (pcf)	89.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	97.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.05

Orange Show Warehouse
San Bernardino, California
Project No. 13G157
PLATE C- 9

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

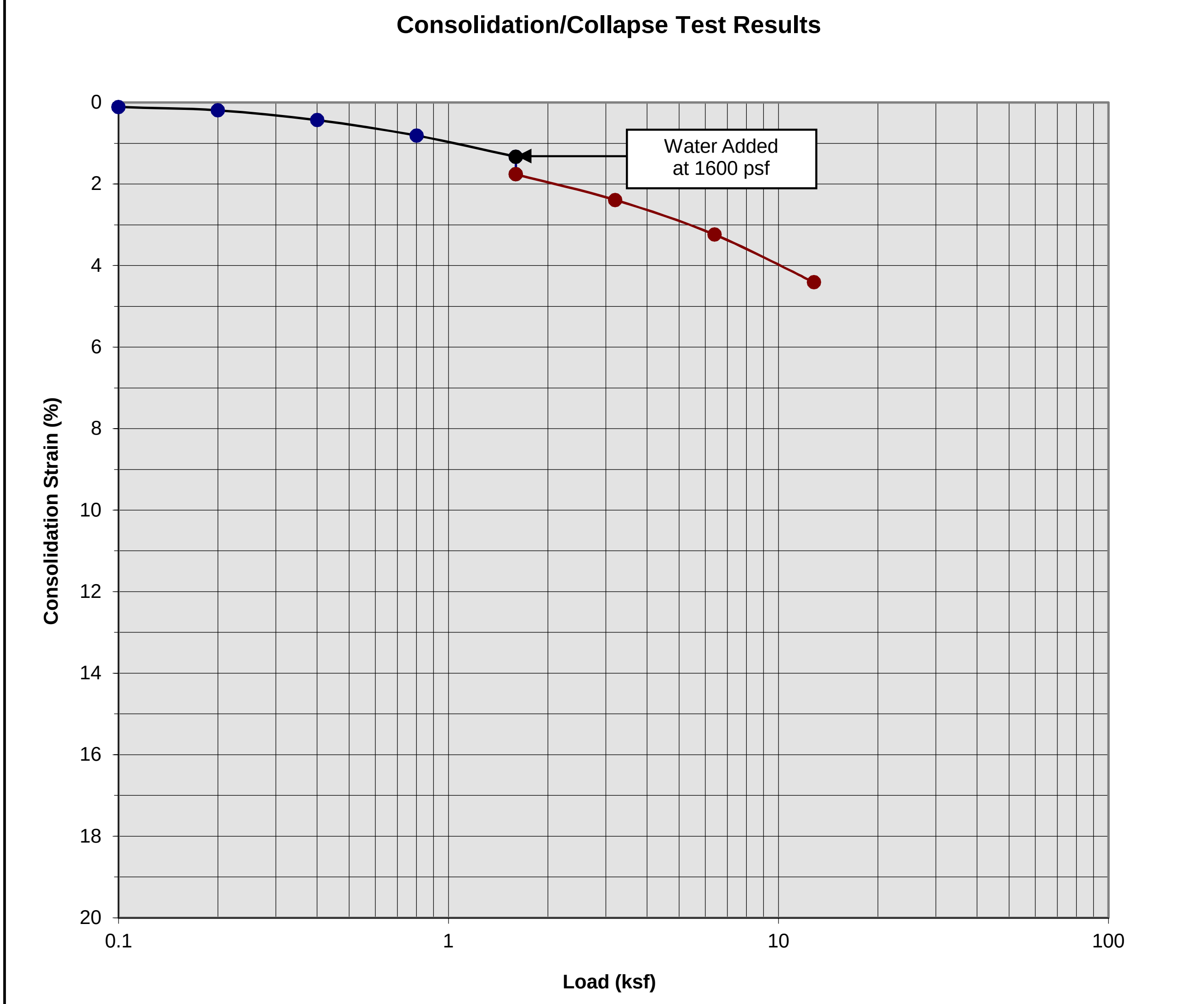


Classification: Light Gray Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-11	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	26
Depth (ft)	5 to 6	Initial Dry Density (pcf)	91.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	95.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.56

Orange Show Warehouse
San Bernardino, California
Project No. 13G157
PLATE C- 10

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL A California Corporation



Classification: Light Gray Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-11	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	29
Depth (ft)	7 to 8	Initial Dry Density (pcf)	93.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	97.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.43

Orange Show Warehouse
San Bernardino, California
Project No. 13G157

PLATE C- 11

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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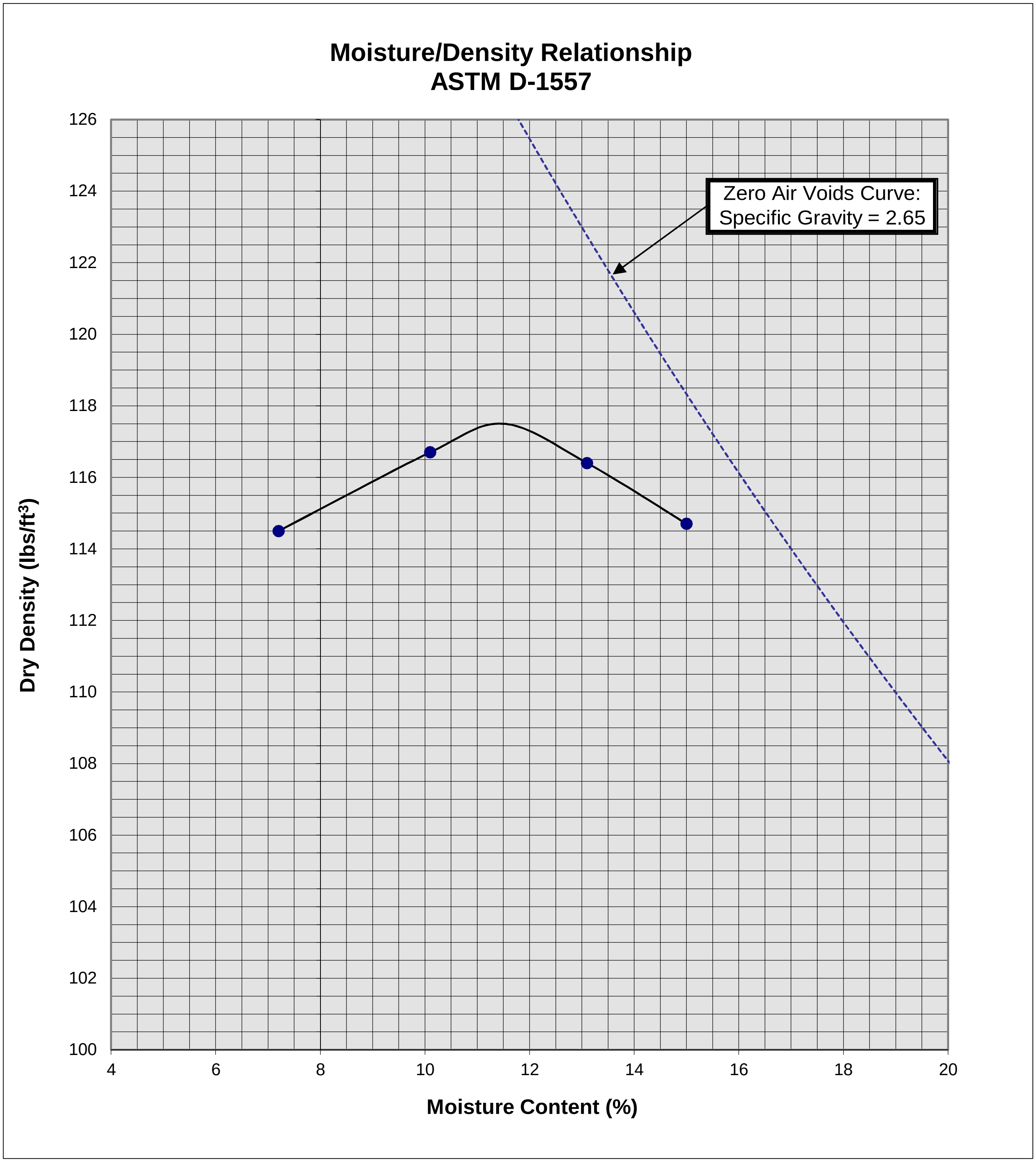


Soil ID Number		B-10 @ 0 to 5'
Optimum Moisture (%)		11.5
Maximum Dry Density (pcf)		117.5
Soil Classification	Gray Brown fine Sand, trace to little medium Sand	

Orange Show Warehouse
San Bernardino, California
Project No. 13G157
PLATE C-12

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

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

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

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

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

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

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

Site Preparation

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

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

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

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

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

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

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

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

<u>Compacted Fills</u>

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

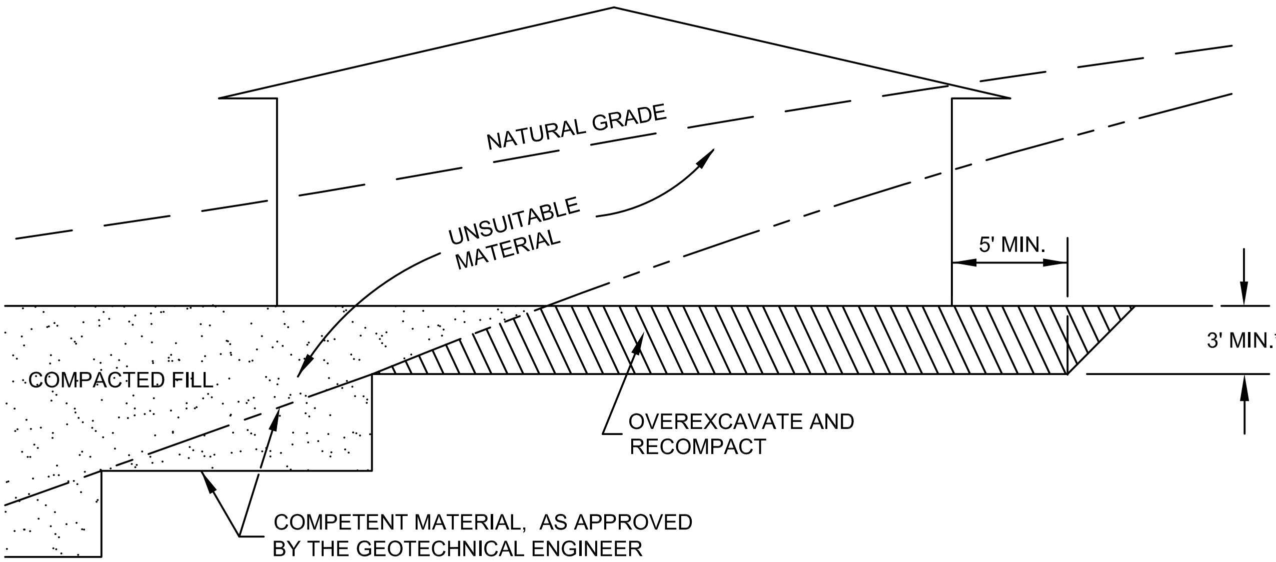
Subdrains

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

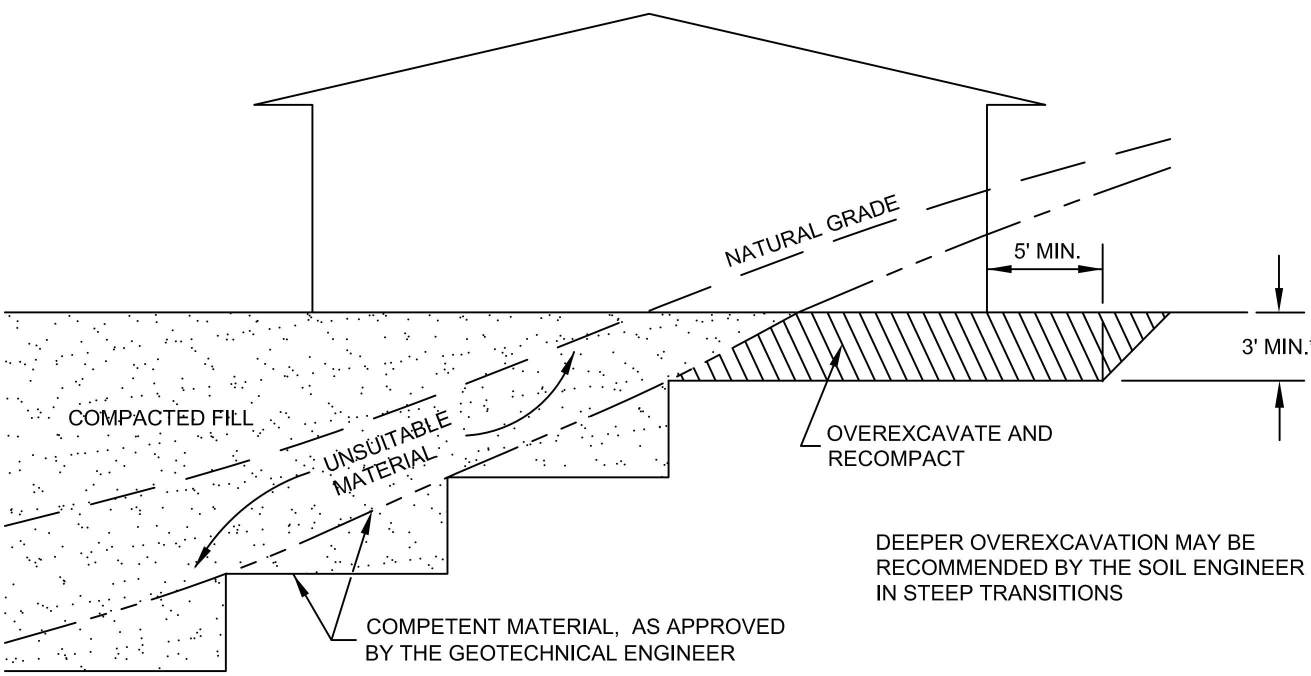
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

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

CUT LOT



CUT/FILL LOT (TRANSITION)



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

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

TRANSITION LOT DETAIL

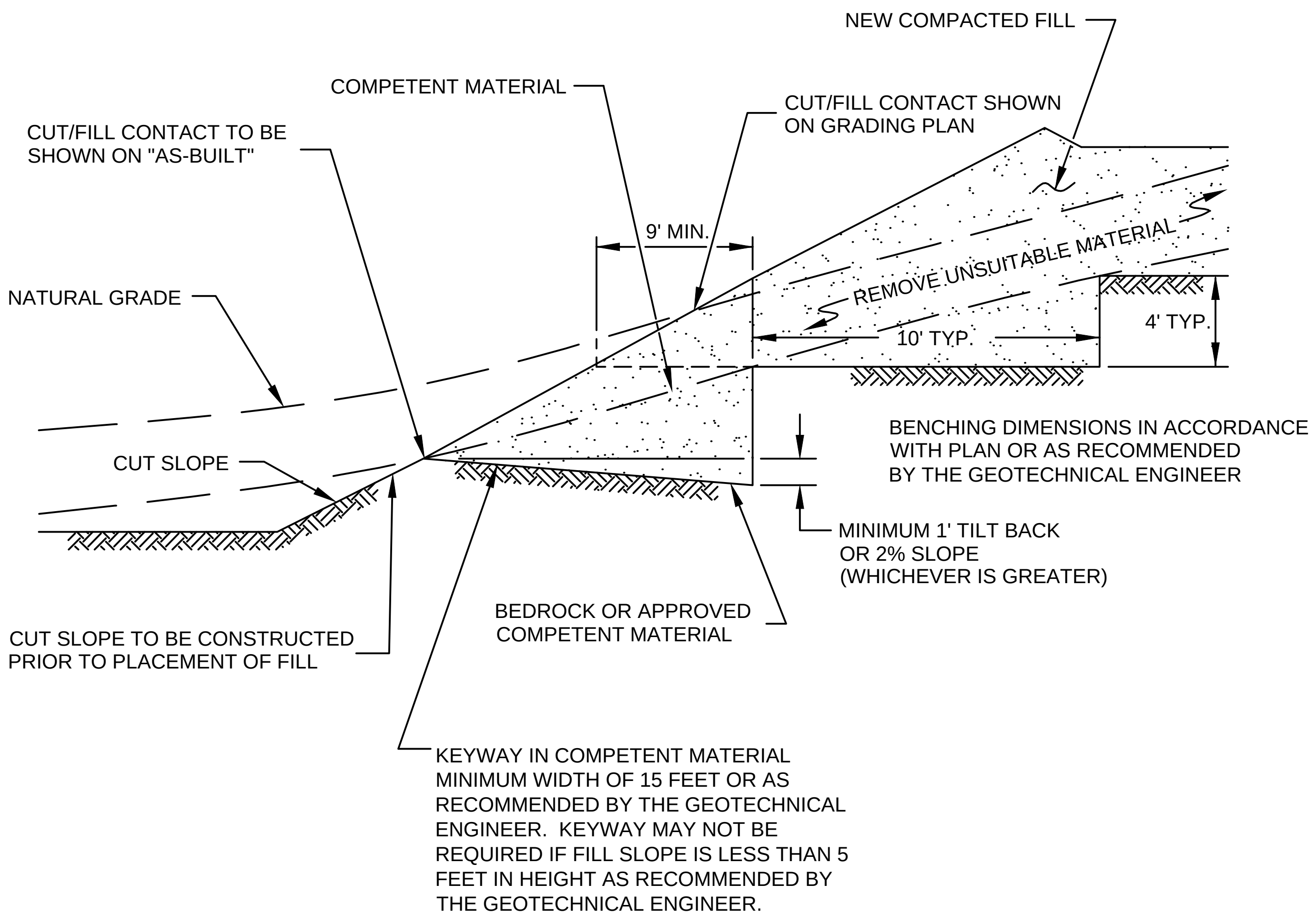
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

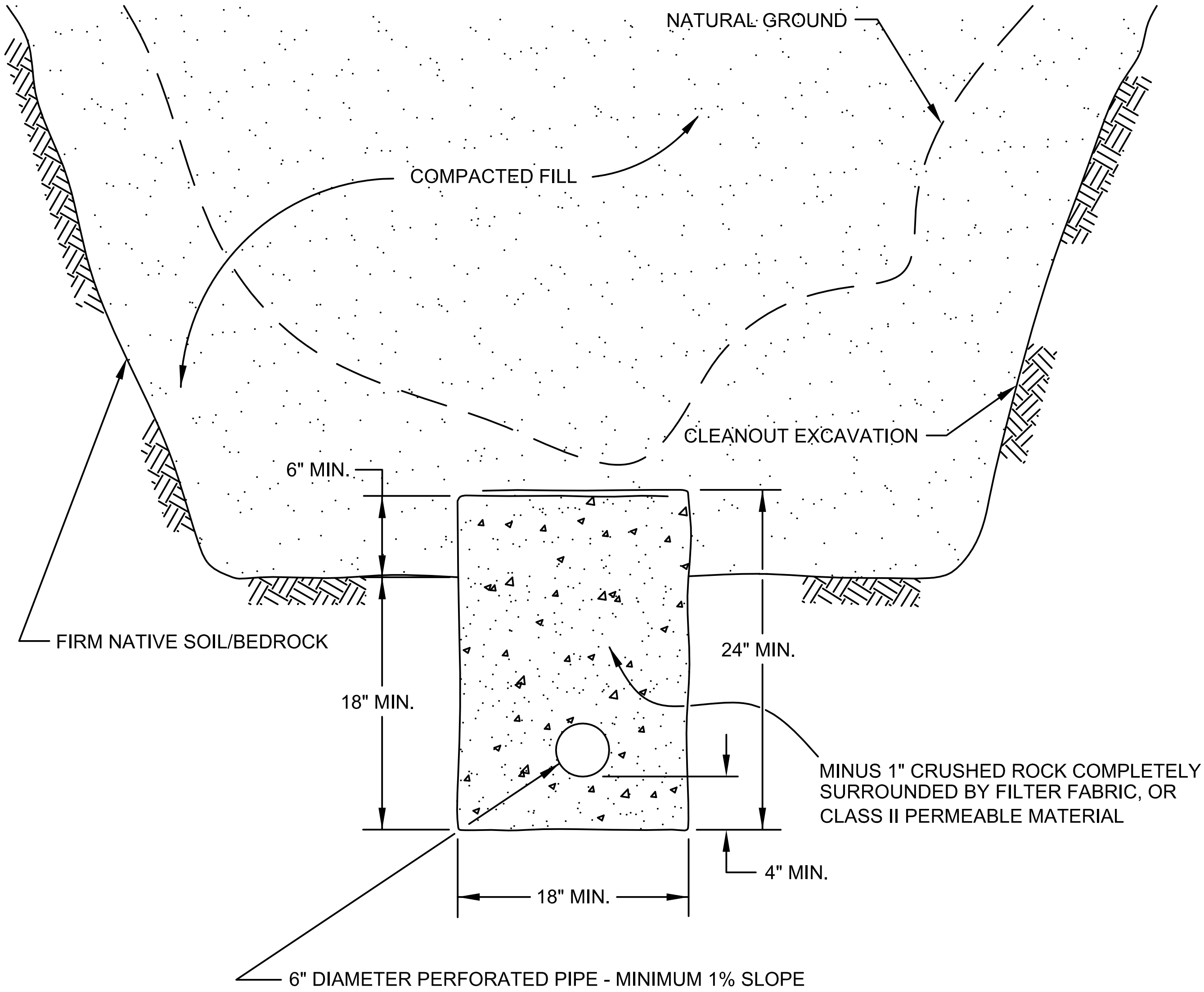
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

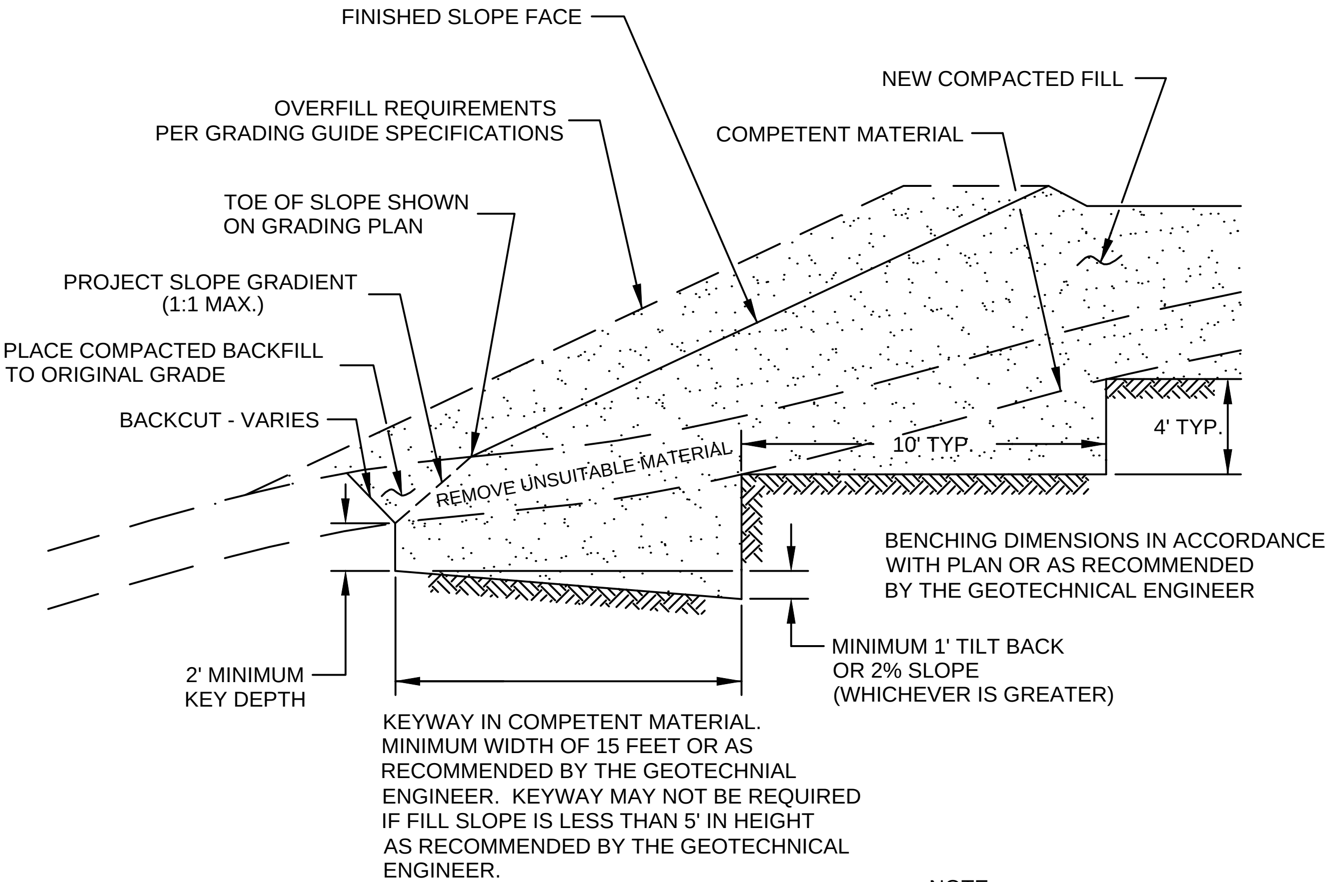
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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

FILL ABOVE NATURAL SLOPE DETAIL

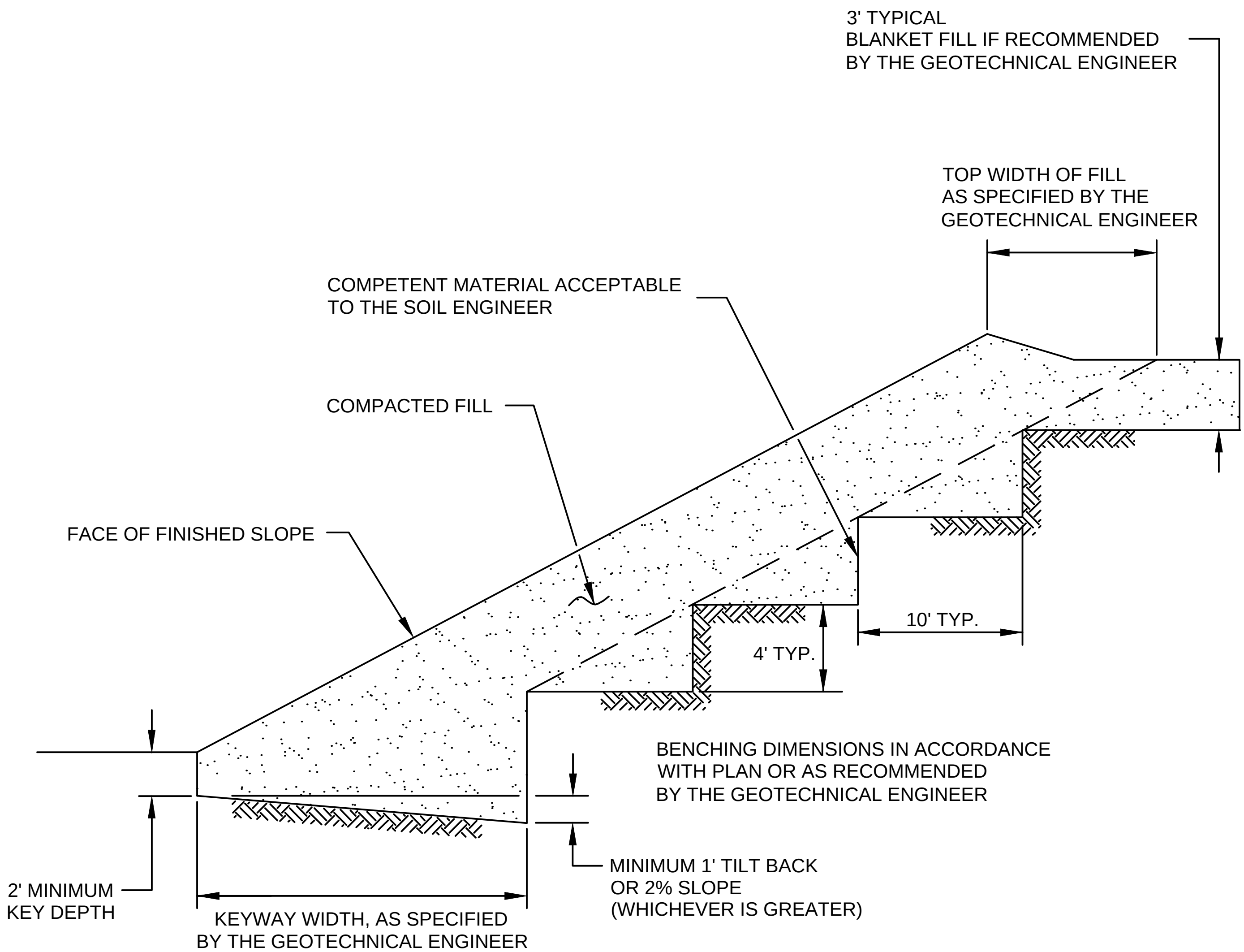
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo
SOUTHERN
CALIFORNIA
GEOTECHNICAL



STABILIZATION FILL DETAIL

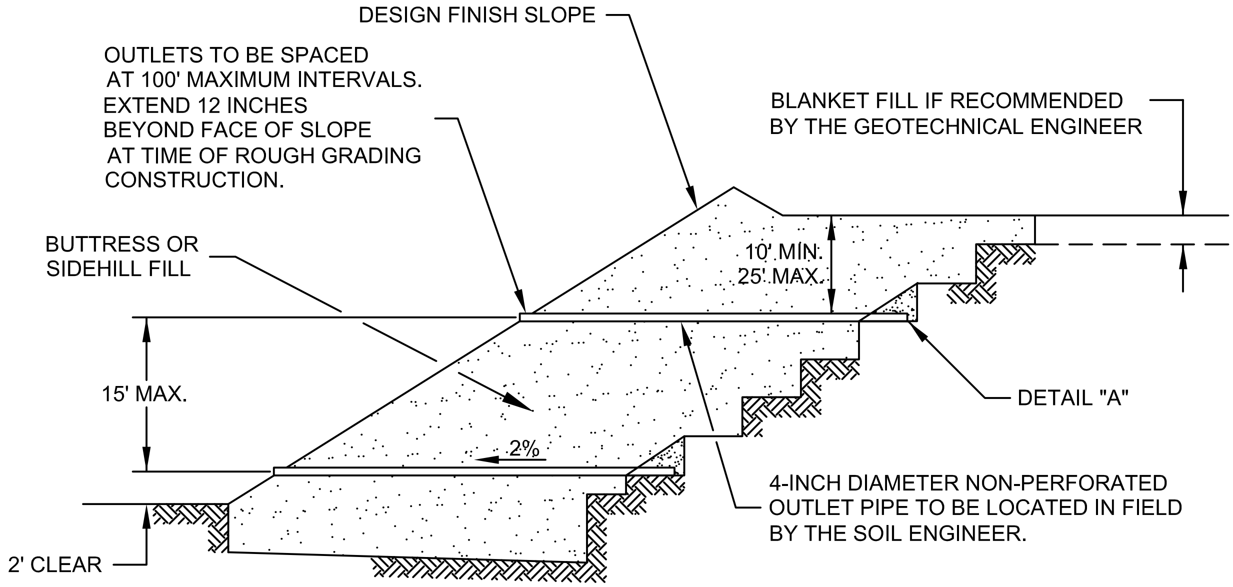
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SOUTHERN CALIFORNIA GEOTECHNICAL



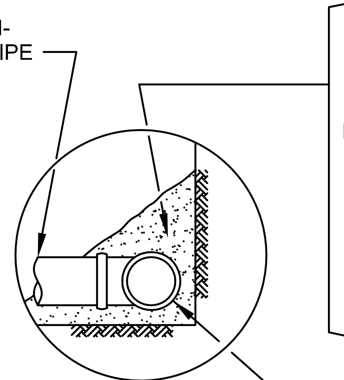
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50



DETAIL "A"

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

SLOPE FILL SUBDRAINS

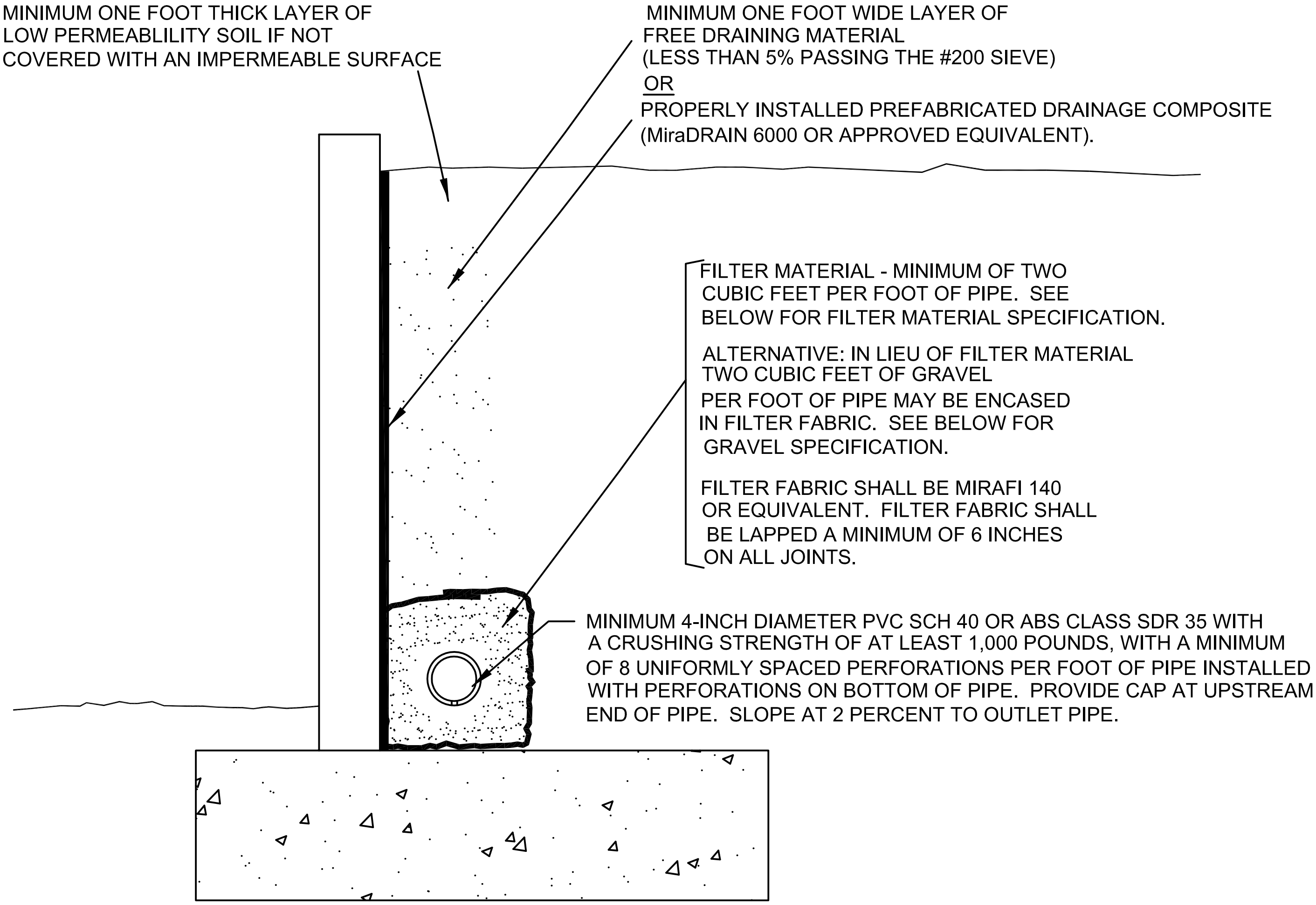
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

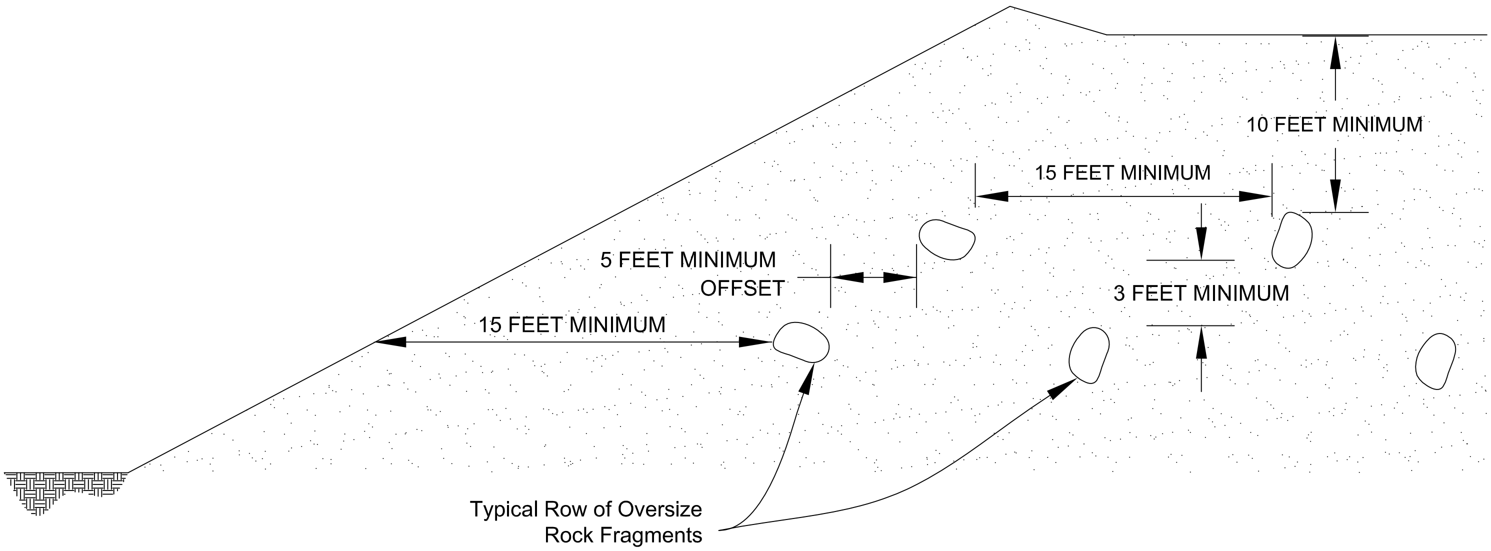
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

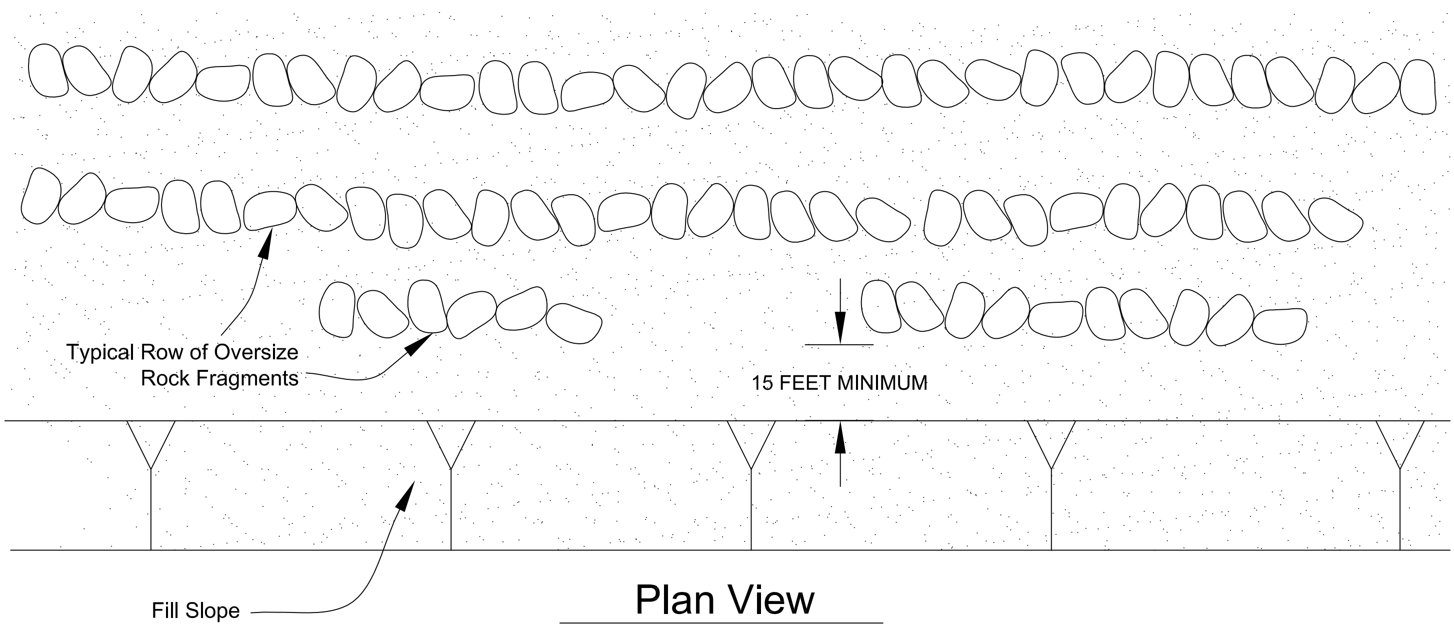
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

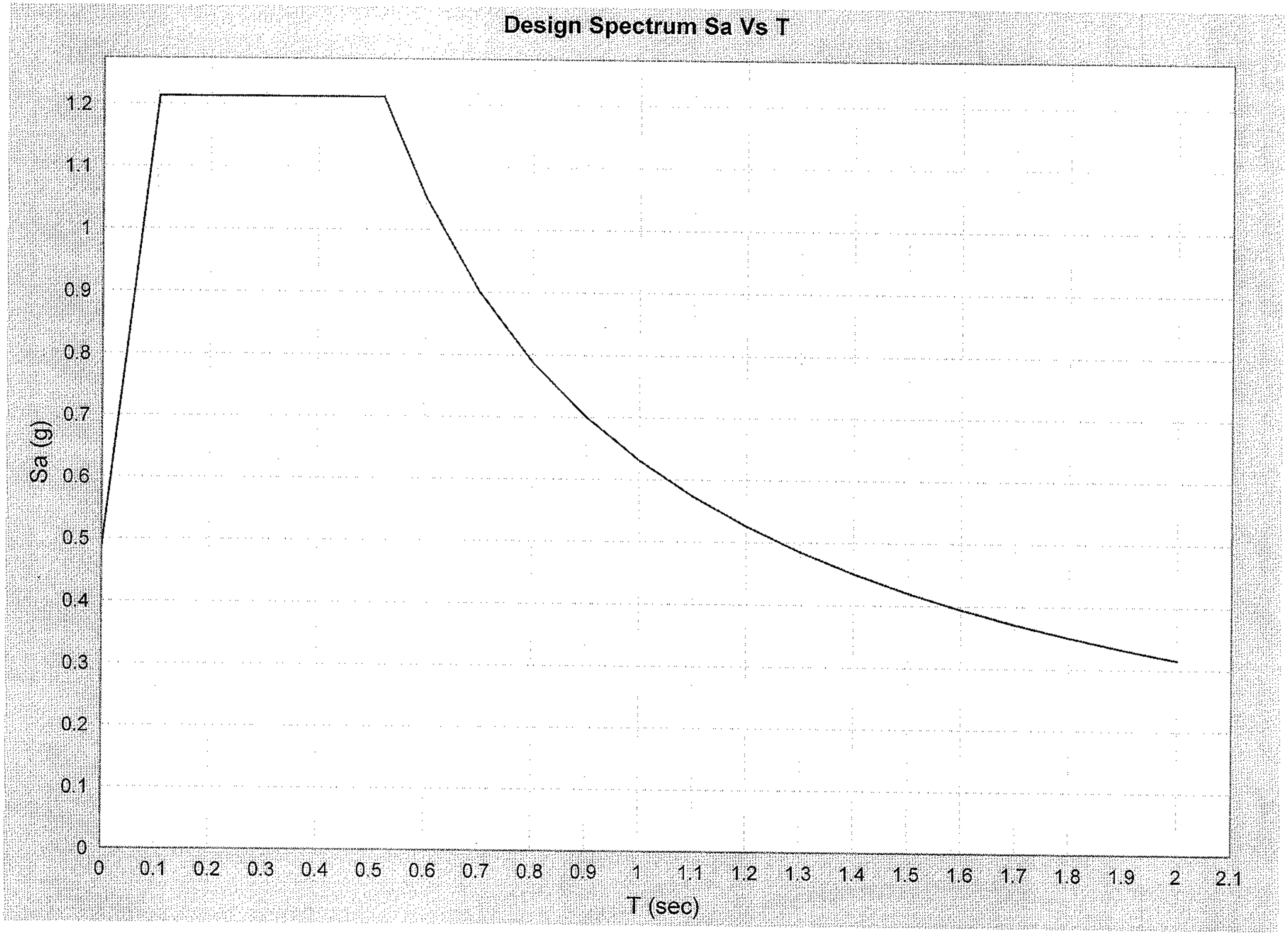
NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E


Design Spectrum Sa Vs T
Sa (g)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
T (sec)
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

```
Conterminous 48 States
2009 International Building Code
Latitude = 34.006775
Longitude = -118.153502
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    Sa
  (sec)    (g)
   0.2    1.820 (Ss, Site Class B)
   1.0    0.631 (S1, Site Class B)
```

```
Conterminous 48 States
2009 International Building Code
Latitude = 34.006775
Longitude = -118.153502
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    Sa
  (sec)    (g)
   0.2    1.820 (SMs, Site Class D)
   1.0    0.946 (SM1, Site Class D)
```

```
Conterminous 48 States
2009 International Building Code
Latitude = 34.006775
Longitude = -118.153502
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    Sa
  (sec)    (g)
   0.2    1.213 (SDs, Site Class D)
   1.0    0.631 (SD1, Site Class D)
```

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Orange Show Warehouse	Design Acceleration	0.45 (g)
Project Location	San Bernardino, CA	Design Magnitude	6.75
Project Number	13G157	Historic High Depth to Groundwater	10 (ft)
Engineer	DWN	Current Depth to Groundwater	40 (ft)

Boring No. B-3

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=6.75)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)				(6)	(7)	(8)	(9)		
4.5	0	10	5		120		1.3	1.83	0.75	0.0	0.0	600	600	600	0.99	0.05	0.06	0.29	N/A	Above Water Table
9.5	10	13	11.5	8	120		1.3	1.20	0.75	9.4	9.4	1380	1286	1380	0.97	0.10	0.13	0.31	0.44	**Liquefiable**
16	13	18	15.5	17	120	4	1.3	1.04	0.85	19.5	19.5	1860	1517	1860	0.96	0.21	0.28	0.35	0.80	**Liquefiable**
21	18	22	20	11	120	26	1.3	0.91	0.95	12.4	18.3	2400	1776	2400	0.95	0.20	0.26	0.38	0.68	**Liquefiable**
26	22	28	25	13	120	77	1.3	0.82	0.95	13.1	20.7	3000	2064	3000	0.94	0.22	0.29	0.40	N/A	Non-liquefiable: PI>12
30	28	33	30.5	25	120	37	1.3	0.74	0.95	22.8	32.4	3660	2381	3660	0.93	INDET	INDET	0.42	N/A	Non-liquefiable
36	33	36	34.5	18	120	49	1.3	0.70	1	16.3	24.5	4140	2611	4140	0.89	0.28	0.36	0.41	0.87	**Liquefiable**
36	36	38	37	18	120	7	1.3	0.67	1	15.7	16.0	4440	2755	4440	0.87	0.17	0.23	0.41	0.55	**Liquefiable**
41	38	40.5	39.25	21	120	6	1.3	0.65	1	17.8	17.9	4710	2885	4710	0.85	0.19	0.25	0.41	0.62	**Liquefiable**
41	40.5	43	41.75	21	120	72	1.3	0.64	1	17.4	25.9	5010	3029	4901	0.83	0.30	0.39	0.40	N/A	Non-liquefiable: PI>12
46	43	48	45.5	29	120	80	1.3	0.63	1	23.6	33.3	5460	3245	5117	0.80	INDET	INDET	0.40	N/A	Non-liquefiable
51	48	50	49	17	120	88	1.3	0.61	1	13.6	21.3	5880	3446	5318	0.78	0.23	0.30	0.39	N/A	Non-liquefiable: PI>12

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 $\frac{\tau_{av}}{\sigma_o'} = \frac{0.65(\sigma_o)}{\sigma_o'} * \frac{a_{max}}{g} * r_d$

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Orange Show Warehouse
Project Location	San Bernardino, CA
Project Number	13G157
Engineer	DWN

Boring No. B-3

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	N_{CORR}	$(N_1)_{60\text{-}CORR}$	Liquefaction Factor of Safety	Cyclic Stress Ratio Induced by Design Earthquake (M=6.75)	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)		
4.5	0	10	5	0.0	0	0.0	N/A	0.29	1.31	0.22			0.0	0.00	Above Water Table
9.5	10	13	11.5	9.4	0	9.4	0.44	0.31	1.31	0.23			2.7	0.97	**Liquefiable**
16	13	18	15.5	19.5	0	19.5	0.80	0.35	1.31	0.26			1.6	0.94	**Liquefiable**
21	18	22	20	12.4	2	14.4	0.68	0.38	1.31	0.29			2.0	0.96	**Liquefiable**
26	22	28	25	13.1	5	18.1	N/A	0.40	1.31	0.31			0.0	0.00	Non-liquefiable: PI>12
30	28	33	30.5	22.8	3	25.8	N/A	0.42	1.31	0.32			0.0	0.00	Non-liquefiable
36	33	36	34.5	16.3	4	20.3	0.87	0.41	1.31	0.32			1.6	0.56	**Liquefiable**
36	36	38	37	15.7	1	16.7	0.55	0.41	1.31	0.31			1.8	0.43	**Liquefiable**
41	38	40.5	39.25	17.8	1	18.8	0.62	0.41	1.31	0.31			1.6	0.49	**Liquefiable**
41	40.5	43	41.75	17.4	5	22.4	N/A	0.40	1.31	0.31			0.0	0.00	Non-liquefiable: PI>12
46	43	48	45.5	23.6	5	28.6	N/A	0.40	1.31	0.30			0.0	0.00	Non-liquefiable
51	48	50	49	13.6	5	18.6	N/A	0.39	1.31	0.30			0.0	0.00	Non-liquefiable: PI>12
										Total Deformation (in)				4.35	

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=6.75 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)

LIQUEFACTION EVALUATION

Project Name	Orange Show Warehouse	Design Acceleration	0.45	(g)
Project Location	San Bernardino, CA	Design Magnitude	6.75	
Project Number	13G157	Historic High Depth to Groundwater	10	(ft)
Engineer	DWN	Current Depth to Groundwater	38	(ft)

Boring No. B-10

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=6.75)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)				(6)	(7)	(8)	(9)		
4.5	0	10	5		120		1.3	1.83	0.75	0.0	0.0	600	600	600	0.99	0.05	0.06	0.29	N/A	Above Water Table
9.5	10	13	11.5	4	120	5	1.3	1.20	0.75	4.7	4.7	1380	1286	1380	0.97	0.06	0.08	0.31	0.27	**Liquefiable**
16	13	18	15.5	23	120	6	1.3	1.04	0.85	26.4	26.5	1860	1517	1860	0.96	0.31	0.41	0.35	1.17	**Liquefiable**
21	18	23	20.5	8	120	88	1.3	0.90	0.95	8.9	15.7	2460	1805	2460	0.95	0.17	0.22	0.38	N/A	Non-liq: PI > 12
26	23	26	24.5	15	120	94	1.3	0.82	0.95	15.3	23.3	2940	2035	2940	0.94	0.26	0.34	0.40	0.85	**Liquefiable**
26	26	30	28	15	120	28	1.3	0.77	0.95	14.3	20.8	3360	2237	3360	0.93	0.23	0.29	0.41	0.72	**Liquefiable**
31	30	33	31.5	12	120	79	1.3	0.73	0.95	10.8	17.9	3780	2438	3780	0.92	0.19	0.25	0.42	N/A	Non-liq: PI > 12
36	33	38	35.5	44	120		1.3	0.69	1	39.2	39.2	4260	2669	4260	0.89	INDET	INDET	0.41	N/A	Non-liquefiable
41	38	43	40.5	67	120		1.3	0.65	1	56.8	56.8	4860	2957	4704	0.84	INDET	INDET	0.41	N/A	Non-liquefiable
46	43	48	45.5	50	120		1.3	0.63	1	41.1	41.1	5460	3245	4992	0.80	INDET	INDET	0.40	N/A	Non-liquefiable
51	48	50	49	35	120	21	1.3	0.62	1	28.2	34.4	5880	3446	5194	0.78	INDET	INDET	0.39	N/A	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 $\frac{\tau_{av}}{\sigma_o'} = \frac{0.65(\sigma_o)}{\sigma_o'} * \frac{a_{max}}{g} * r_d$

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Orange Show Warehouse
Project Location	San Bernardino, CA
Project Number	13G157
Engineer	DWN

Boring No.	B-10

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=6.75)	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)		
4.5	0	10	5	0.0	0	0.0	N/A	0.29	1.31	0.22			0.0	0.00	Above Water Table
9.5	10	13	11.5	4.7	1	5.7	0.27	0.31	1.31	0.23			3.5	1.26	**Liquefiable**
16	13	18	15.5	26.4	1	27.4	1.17	0.35	1.31	0.26			0.1	0.06	**Liquefiable**
21	18	23	20.5	8.9	5	13.9	N/A	0.38	1.31	0.29			0.0	0.00	Non-liq: PI > 12
26	23	26	24.5	15.3	5	20.3	0.85	0.40	1.31	0.30			1.5	0.55	**Liquefiable**
26	26	30	28	14.3	2	16.3	0.72	0.41	1.31	0.31			1.9	0.90	**Liquefiable**
31	30	33	31.5	10.8	5	15.8	N/A	0.42	1.31	0.32			0.0	0.00	Non-liq: PI > 12
36	33	38	35.5	39.2	0	39.2	N/A	0.41	1.31	0.32			0.0	0.00	Non-liquefiable
41	38	43	40.5	56.8	0	56.8	N/A	0.41	1.31	0.31			0.0	0.00	Non-liquefiable
46	43	48	45.5	41.1	0	41.1	N/A	0.40	1.31	0.30			0.0	0.00	Non-liquefiable
51	48	50	49	28.2	2	30.2	N/A	0.39	1.31	0.30			0.1	0.02	Non-liquefiable
										Total Deformation (in)				2.79	

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=6.75 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)