APPENDIX E1

GEOTECHNICAL FEASIBILITY STUDY

GEOTECHNICAL FEASIBILITY STUDY
PROPOSED COMMERCIAL/INDUSTRIAL BUILDING

1494 Waterman Avenue
San Bernardino, California
for
Hillwood

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

November 14, 2016

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

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

Attention: Mr. Ned Sciortino

Project No.: **16G167-1R**

Subject: **Geotechnical Feasibility Study**
Proposed Commercial/Industrial Building
1494 Waterman Avenue
San Bernardino, California

Gentlemen:

In accordance with your request, we have conducted a geotechnical feasibility study 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.

It should be noted that this investigation was focused on determining the geotechnical feasibility of the proposed development. It was not intended to be a design level investigation. Future studies will be necessary to refine the preliminary design parameters that are presented within this current report.

Geotechnical Design Considerations

- The results of the preliminary liquefaction evaluation indicate that total dynamic settlements of 0 to 0.69± inches could occur at the site during the design seismic event concurrent with historically high groundwater levels. The design level geotechnical investigation for this site should include a comprehensive liquefaction evaluation.
- The subsurface conditions encountered at the boring locations generally consist of a surficial layer of possible fill soils underlain by moderate strength native alluvium. The upper zone of alluvium possesses loose to medium dense relative densities. In their present condition, these soils are not considered suitable for support of the proposed structure. Remedial grading will be required to remove a portion of these soils and replace them as compacted structural fill.
- The groundwater table is considered to have been present at a depth of greater than 50± feet at the time of the subsurface exploration.

Preliminary Site Preparation Recommendations

- Demolition of the existing buildings and associated improvements including asphaltic concrete parking lots and driveways, and flatwork will be required at this site. Demolition should include all subsurface remnants of the existing structures, including foundations, floor slabs, and any utilities that will not be reutilized.
- Initial site stripping should include removal of the existing vegetation including turf grass, as well as any underlying topsoil, and trees. Stripping should also include the removal of any tee root masses. These materials should be disposed of offsite or in non-structural areas of the property.
- Overexcavation will be necessary within the proposed building area to remove any existing fill soils along with a portion of the near-surface native alluvium. Overexcavation to depths on the order of 3 to 5± feet below existing grade and 3± feet below proposed pad grade is anticipated to be necessary. Overexcavation within the foundation areas is expected to extend to depths of 2 to 3± feet below proposed foundation bearing grade.
- No significant overexcavation is expected to be necessary in new pavement or flatwork areas, unless zones of unsuitable existing fill or native alluvium are encountered.

Preliminary Foundation Design Parameters

- Spread footing foundations, supported in newly placed structural fill soils.
- Maximum, net allowable soil bearing pressure: 2,500 to 3,000 lbs/ft^2. The design of the proposed structures should include sufficient rigidity to resist the differential settlements that

may occur as a result of liquefaction. The magnitude of the liquefaction induced settlements should be further refined during the design level geotechnical investigation.

- The estimated allowable bearing pressures provided above should be refined during the design level geotechnical investigation, based on actual column loads and detailed settlement analyses.

Preliminary Building Floor Slab Recommendations

- Conventional Slab-on-Grade, 6 inches thick
- The design of the floor slabs will depend in large part on the results of the future geotechnical study, including a more detailed liquefaction evaluation. The floor slab should include sufficient rigidity to resist the effects of differential settlements that may occur during liquefaction.
- The actual thickness and reinforcement of the floor slabs should be determined by the structural engineer.

Preliminary Pavement Thickness Recommendations

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

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

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 15P410, dated October 14, 2015. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory geotechnical testing, and geotechnical engineering analysis to determine the geotechnical feasibility of the proposed development. This report also contains preliminary design criteria for building foundations, building floor slab, and parking lot pavements. The evaluation of the environmental aspects of this site was beyond the scope of services for this feasibility study.

It should be noted that additional subsurface exploration, laboratory testing and engineering analysis will be necessary to provide a design level geotechnical investigation with specific foundation, floor slab, and grading recommendations.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located on the west side of Waterman Avenue at the intersection of Waterman Avenue and Park Center Circle in San Bernardino, California. The site is also referenced by the street address 1494 Waterman Avenue. The site is bounded to the north by a vacant lot and single family residences, to the west by the Twin Creek channel, to the south by the Santa Ana River, and to the east by Waterman Avenue. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in Appendix A of this report.

The site consists of an irregular-shaped parcel, approximately 60.89 acres in size. The site is currently utilized as a golf course and is identified as the San Bernardino Public Golf Course. A clubhouse building and two other associated structures are located in the northwestern area of the site. An asphaltic concrete parking lot is located northeast of the clubhouse. The asphalt pavements are in fair condition with light to moderate cracking throughout. The ground surface cover throughout the golf course consists of turf grass with multiple large trees lining the fairways. Several small lakes/ponds/water hazards and sand traps are located throughout the golf course.

Topographic information was obtained from a conceptual grading plan prepared by Thienes Engineering, Inc. This plan generally indicates the site topography to be relatively level, with the exception of some areas with moderately sloping terrain and some localized variations, including golf hazards and berms. The overall site topography slopes downward to the west at gradients ranging from 1 to 2 percent. However, several terraced areas, located within the central and northeastern region of the site possess slope inclinations of up to 3h:1v (horizontal to vertical). The terraced areas are generally 4 to 10± feet higher in elevation than the surrounding adjacent grades. The existing site grades range from an elevation of 1010± feet mean sea level (msl) in the northeastern portion of the site to an elevation of 983± feet msl in the southwestern portion of the site.

3.2 Proposed Development

Based on a site plan prepared by HPA, the site will be developed with one (1) new commercial/industrial building. The building will be located in the central area of the site and will be 1,078,480± ft^2 in size. The building will be constructed in a cross dock configuration with dock high doors along the north and south sides of the building. The building will be surrounded by asphaltic concrete pavements in the parking and drive areas, Portland cement concrete pavements in the loading dock areas, and may include concrete flatwork and landscape planters throughout the site.

Detailed structural information has not been provided. It is assumed that the building will be a single-story structure of tilt-up concrete construction, supported on conventional shallow foundations with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 80 kips and 3 to 5 kips per linear foot, respectively.

The proposed development is not expected to include any significant amounts of below grade construction, such as basements or crawl spaces. Based on the conceptual site plan, cuts of up to 4 feet and fills of 1 to 7± feet will be required in order to achieve the proposed building pad grade.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration performed for this project consisted of five (5) borings advanced to depths of 20 to 50± feet below existing site grades. Two (2) of the borings were extended to depths of 50± feet as a part of the liquefaction evaluation. A third boring, Boring No. B-1, was intended to extend to a depth of 50± as part of the liquefaction evaluation but was terminated at a shallower depth due refusal on very dense soils encountered at the site. All of the borings were logged during drilling by a member of our staff.

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

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

4.2 Geotechnical Conditions

Artificial Fill

Soils identified as possible fill were encountered at the ground surface at Boring Nos. B-3 and B-4 extending to depths of 4½ and 5½± feet below the existing site grades. The possible fill soils generally consist of loose to medium dense silty fine sands and fine to medium sands. These possible fill soils possess some indicators of fill but also resemble the underlying native soil.

Alluvium

Disturbed alluvial soils were encountered at the ground surface at one of the boring locations, Boring No. B-1. These soils generally consist of loose silty fine sands and extend to a depth of 2½± feet below existing grades. These soils are classified as disturbed alluvium because they resemble the underlying native soils, however these soils, at the ground surface, are expected to have been disturbed as part of the current site use.

Native alluvium was encountered beneath the disturbed soils, possible fill soils, or at the ground surface, at all of the boring locations. The near-surface alluvial soils generally consist of loose to medium dense fine sands and silty sands with varying fine to coarse sand content and zones of stiff to very stiff silty clays, extending to depths of 12 to 24± feet. At greater depths, the alluvium generally consists of medium dense to very dense fine to medium sands, silty fine sands and stiff to hard silty clays extending to the maximum depth explored of 50± feet.

Groundwater

Groundwater was not encountered during drilling of the borings. In addition, delayed readings taken within the open boreholes did not identify any free water. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 50± feet at the time of the subsurface exploration.

Research of historic high groundwater levels was performed as a part of the site-specific liquefaction evaluation. 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.

Dry Density and Moisture Content

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

Consolidation

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

Grain Size Analysis

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

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on selected samples of various soil strata encountered at the site. 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. Soils with a PI greater than 18 are not considered to be susceptible to liquefaction when the moisture content of the soil is less than 80 percent of the liquid limit. The results of the Atterberg Limits testing are presented on the boring logs.

Soluble Sulfates

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

Sample Identification	**Soluble Sulfates (%)**	**ACI Classification**
B-2 @ 0 to 5 feet	0.004	Negligible
B-4 @ 0 to 5 feet	0.005	Negligible

Maximum Dry Density and Optimum Moisture Content

Two (2) representative near-surface bulk samples were tested for their maximum dry densities and optimum moisture contents. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil type or soil mixes may be necessary at a later date. The results of the testing are plotted on Plates C-9 and C-10 in Appendix C of this report.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on results of our document review, field exploration, laboratory testing, and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. However, several geotechnical issues are present at the subject site which may require specialized design or construction techniques to overcome. These geotechnical issues are discussed further in subsequent sections of this report.

The preliminary recommendations and conclusions in this report should be supplemented by a detailed geotechnical investigation in order to prepare final grading plans as well as foundation and floor slab designs. The recommendations within this report should be considered preliminary in nature and may be superseded by the detailed geotechnical investigation to be conducted at a later date.

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

The 2013 California Building Code (CBC) was adopted by all municipalities within Southern California on January 1, 2014. 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 2013 CBC Seismic Design Parameters have been generated using U.S. Seismic Design Maps, a web-based software application developed by the United States Geological Survey. This software application, available at the USGS web site, calculates seismic design parameters in accordance with the 2013 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 as Plate E-1 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:

2013 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	2.482
Mapped Spectral Acceleration at 1.0 sec Period	S_1	1.137
Site Class	---	F*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	2.482
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.706
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.655
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	1.137

*The 2013 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 7-10. However, Section 20.3.1 of ASCE 7-10 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of the structure is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. Therefore, if the proposed structure has a fundamental period greater than 0.5 seconds, a site specific seismic hazards analysis would be required and additional subsurface exploration would be necessary.

Ground Motion Parameters

For the liquefaction evaluation, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2013 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-10. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-10. The web-based software application U.S. Seismic Design Maps (described in the previous section) was used to determine PGA_M, which is 0.954g. A portion of the program output is included as Plate E-2 of this report. An associated earthquake magnitude was obtained from the 2008 USGS Interactive Deaggregation application available on the USGS website. The deaggregated modal magnitude is 7.0, based on the peak ground acceleration and NEHRP soil classification D.

Liquefaction

The California Geological Survey (CGS) has not yet conducted detailed seismic hazards mapping in the area of the subject site. The general liquefaction susceptibility of the site was determined by research of the San Bernardino County 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 moderate to high liquefaction susceptibility. Therefore, the scope of this geotechnical investigation was expanded to include a site-specific liquefaction evaluation.

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

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

The historic high groundwater depth was obtained from 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 approximately 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 using data obtained at Boring Nos. B-3 and B-5, which were advanced to a depth of 50 ± feet. As previously noted, Boring No. B-1 was intended to extend to a depth of 50± feet in order to evaluate the liquefaction potential at that boring location. However, this boring was terminated at a depth of 30± feet due to auger refusal on very dense soils. Therefore, the liquefaction analysis within Boring No. B-1 was performed on soils extending to a depth of 30± feet. The liquefaction potential was analyzed for each of the boring locations utilizing a PGA_M of 0.954g related to a 7.0 magnitude seismic event.

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

Conclusions and Recommendations

The results of the liquefaction analysis identified potentially liquefiable soil strata at Boring No. B-1, between the depth of 10 to 12 feet, and at Boring No. B-5, between depths of 10 to 12 feet and 17 to 22± feet. However, the results of the liquefaction analysis did not identify any potentially liquefiable soil strata at Boring No. B-3. Soils which are located above the historic groundwater table (10 feet), or possessing factors of safety in excess of 1.3 are considered non-liquefiable. Several strata of silty clay were determined to be non-liquefiable due to their cohesive characteristics and the results of the Atterberg limits testing with respect to the criteria of Bray and Sancio (2006) and Special Publication 117A. Settlement analyses were conducted for each of the potentially liquefiable strata. The results of the settlement analyses indicate a potential total settlement of 0.25± inches at Boring No. B-1, and 0.69± inches at Boring No. B-5.

Based on the estimated total settlements, differential settlements are expected to be on the order of ½± inch. The estimated differential settlement can be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of approximately 0.001 inch per inch. These differential settlements are considered to be within the structural tolerances of a typical building supported on a shallow foundation system provided that structural mitigation measures are implemented. 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

Possible fill sols were encountered at the ground surface at two of the boring locations, extending to depths of 4½ to 5½± feet. In addition, the near-surface native alluvial soils possess low to moderate strengths, and are composed of loose to medium dense fine sands and silty sands. The results of laboratory testing indicate that the near-surface soils possess low strengths and a minor potential for consolidation settlement. The near-surface soils are not considered suitable, in their present condition, to support the foundation loads of the new structure. Remedial grading will be necessary within the proposed building area to remove and replace existing fill materials as well as the upper zone of native alluvium, as compacted structural fill.

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

Settlement

The recommended remedial grading will remove and replace any existing fill soils from the building pad area, as well as the upper portion of the low strength native alluvium, and replace these materials as compacted structural fill. Following completion of the recommended remedial grading, post-construction static settlements of the buildings are expected to be within tolerable limits.

Expansion

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

Soluble Sulfates

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

Shrinkage/Subsidence

Removal and recompaction of the near surface native soils is estimated to result in an average shrinkage of 8 to 12 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 0.10± feet. Additional borings and in-place density tests should be performed at the time of the design level investigation in order to confirm these values.

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

The grading plans provided to our office consisted of a conceptual grading plan which did not include precise grading information. In addition, foundation plans were not available at the time of this report. It is therefore recommended that we be provided with copies of the final 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

Demolition of the existing buildings and associated improvements, including asphaltic concrete parking lots, driveways, and concrete flatwork will be required at this site. Demolition should include all subsurface remnants of the existing structures, including foundations, floor slabs, any utilities that will not be reutilized with the proposed development. Any debris resultant from demolition should be disposed of offsite in accordance with local regulations. Alternatively, asphalt and concrete debris may be crushed to a maximum 2-inch particle size, well mixed with on-site soils, and incorporated into new structural fills or crushed to make miscellaneous base, if desired. Any excavations associated with demolition should be backfilled with compacted fill soils.

Initial site stripping should include removal of any surficial vegetation, as well as any underlying topsoil or other organic materials. Based on conditions encountered at the time of the subsurface exploration, stripping of existing turf grass and topsoil and large trees is expected to be necessary. Site stripping should also include the removal of any root masses from the existing trees. After removal, these materials should be disposed of off-site. The actual extent of stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pad

Remedial grading will be necessary within the proposed building pad area to remove any existing fill soils, the upper portion of the low strength native alluvium, and any soils disturbed during stripping and demolition procedures. The depth of overexcavation should be determined during the design level geotechnical investigation. On a preliminary basis, overexcavation to depths of 3 to 5± feet below existing grade and at least 3 feet below pad grade should be anticipated. Overexcavation within the foundation areas will likely be in the range of 2 to 3± feet below the foundation bearing grade.

Treatment of Existing Soils: Retaining Walls and Site Walls

Although not indicated on the site plan, it may be necessary to construct some small retaining walls or site walls at or near the existing surface grade. Overexcavation will also be necessary in these areas to remove the existing fill soils and lower strength potentially compressible alluvium. The overexcavation depth should be expected to be on the order of 2 to 3± feet below proposed foundation bearing grade, and to depths of 3 to 4± feet below existing grade.

Treatment of Existing Soils: Parking and Drive Areas

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

The grading recommendations presented above for the proposed parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within the proposed parking areas. The grading recommendations presented above do not mitigate the extent of undocumented fill soils and compressible 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, all of the existing undocumented fill soils and existing low strength, potentially compressible, native alluvium within these areas should be removed and replaced as structural fill.

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 2013 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 fine sands and silty sands. 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.

Groundwater

The static groundwater table at this site is considered to be present at a depth in excess of 50± feet. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

6.5 Preliminary Foundation Design Recommendations

Based on the preceding geotechnical design considerations and preliminary grading recommendations, it is assumed that the new buildings will be underlain by newly placed structural fill soils, extending to depths of at least 2 to 3 feet below foundation bearing grade. Based on this subsurface profile and assuming that the proposed structure can tolerate the estimated liquefaction-induced settlements, the proposed structure may be supported on conventional spread footing foundations.

The foundation design parameters presented below provide anticipated ranges for the allowable soil bearing pressures. These ranges should be refined during the subsequent design level geotechnical investigation.

Spread Footing Foundation Design Parameters

New square and rectangular footings may be designed using the following approximate values:

- Maximum, net allowable soil bearing pressure: 2,500 to 3,000 lbs/ft^2.

General Foundation Design Recommendations

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

Estimated Foundation Settlements

Typically, foundations designed in accordance with the preliminary foundation design parameters presented above will experience total and differential static settlements of less than 1.0 and 0.5 inches, respectively. A detailed settlement analysis should be conducted as part of the design level geotechnical investigation, once detailed foundation loading information is available.

The estimated settlements provided above are based only on static conditions. As discussed previously, portions of the subject site are underlain by potentially liquefiable soils. Additional liquefaction-induced settlements will occur during the design seismic event. Design of the foundations for the proposed structure should allow for the occurrence of these liquefaction-induced settlements.

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: 275 - 325 lbs/ft^3
- Friction Coefficient: 0.30 to 0.35

6.6 Preliminary Floor Slab Design and Construction

Subgrades which will support new floor slab should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floors of the new structure may be constructed as conventional slabs-on-grade supported on newly placed structural fill soils. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.

- Modulus of Subgrade Reaction: k = 100 to 150 psi/in.

- Minimum slab reinforcement: Reinforcement is not required for expansive soil conditions. However, slab reinforcement may be required to resist the effects of liquefaction-induced settlements, or for other structural design considerations. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading, after completion of the design-level geotechnical investigation.

- Slab underlayment: If moisture sensitive floor coverings will be used the minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area where such moisture sensitive floor coverings are anticipated. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. 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.

The actual design of the floor slab should be based on the results of the future detailed geotechnical investigation, and completed by the structural engineer to verify adequate thickness and reinforcement.

6.7 Preliminary Retaining Wall Design and Construction

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

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near surface soils generally consist of fine sands and silty fine sands. Based on their classifications, the sand and silty sand materials are expected to possess a friction angle of at least 30 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density.

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

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type: On-Site Sands and Silty Sands
Internal Friction Angle (ϕ)		30°
Unit Weight		120 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	40 lbs/ft^3
	Active Condition (2h:1v backfill)	65 lbs/ft^3
	At-Rest Condition (level backfill)	60 lbs/ft^3

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

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

Seismic Lateral Earth Pressures

In accordance with the 2013 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

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

Backfill Material

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

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

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D 1557). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

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

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

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

6.8 Preliminary Pavement Design Parameters

Presented below are preliminary recommendations for pavements that may be required around the perimeters of the proposed structures. Grading recommendations for these pavement areas should be developed during the design level geotechnical investigation.

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. Based on their classification, these materials are expected to possess good to excellent pavement support characteristics, with R-values in the range of 50 to 70. Since R-value testing was not included in the scope of services for this feasibility study, 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

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)**			
	Automobile Parking and Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic		
		(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3½	4	5
Aggregate Base	3	4	5	5
Compacted Subgrade	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)**		
	Automobile and Light Truck Traffic (TI = 5.0 & 6.0)	Truck Traffic	
		(TI = 7.0)	(TI = 8.0)
PCC	5	6	7
Compacted Subgrade (95% minimum compaction)	12	12	12

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

7.0 GENERAL COMMENTS

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

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

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

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

8.0 REFERENCES

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

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

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

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

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

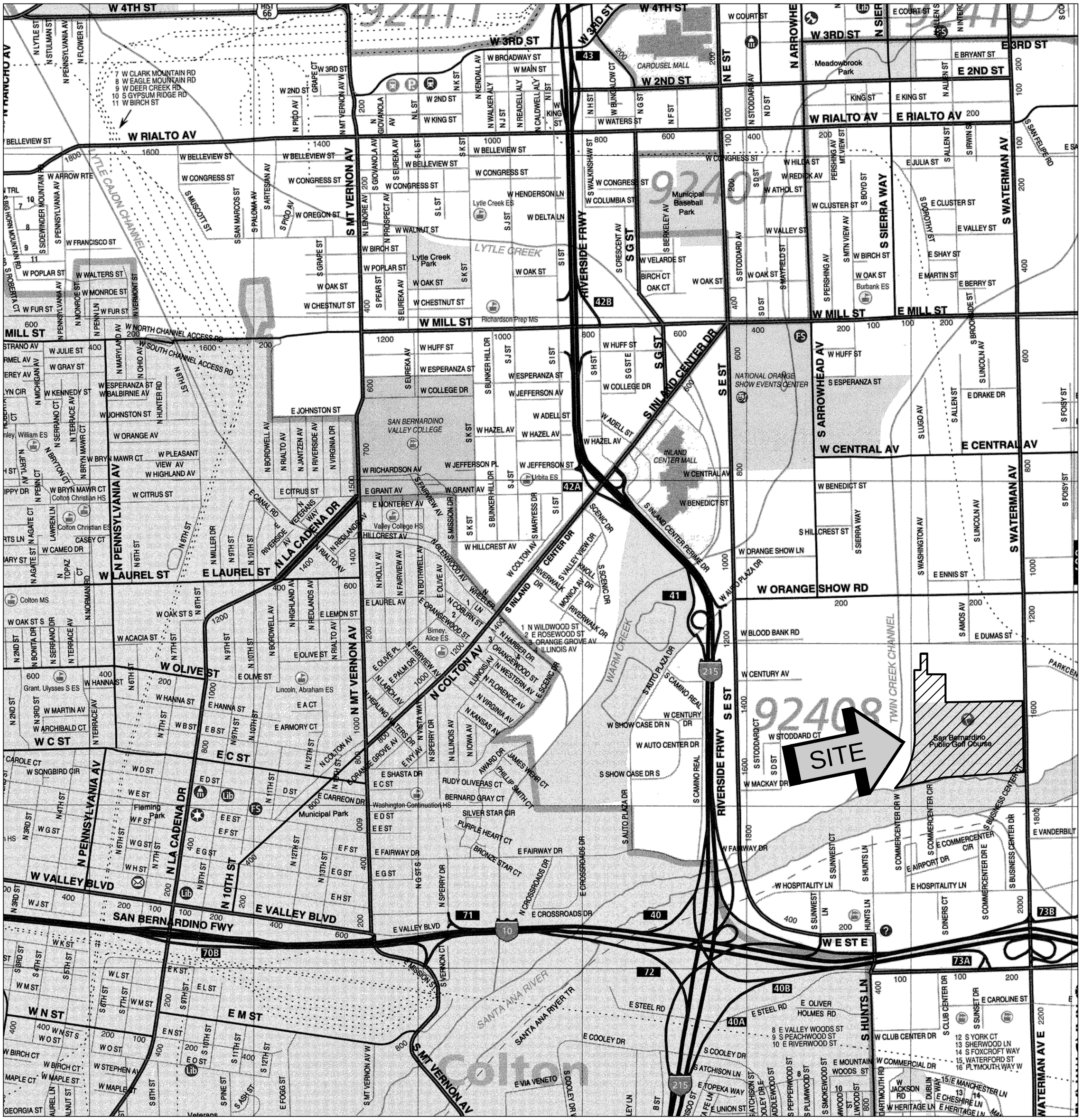
Southern California Earthquake Center (SCEC), University of Southern California, "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California," Committee formed 1997.

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

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

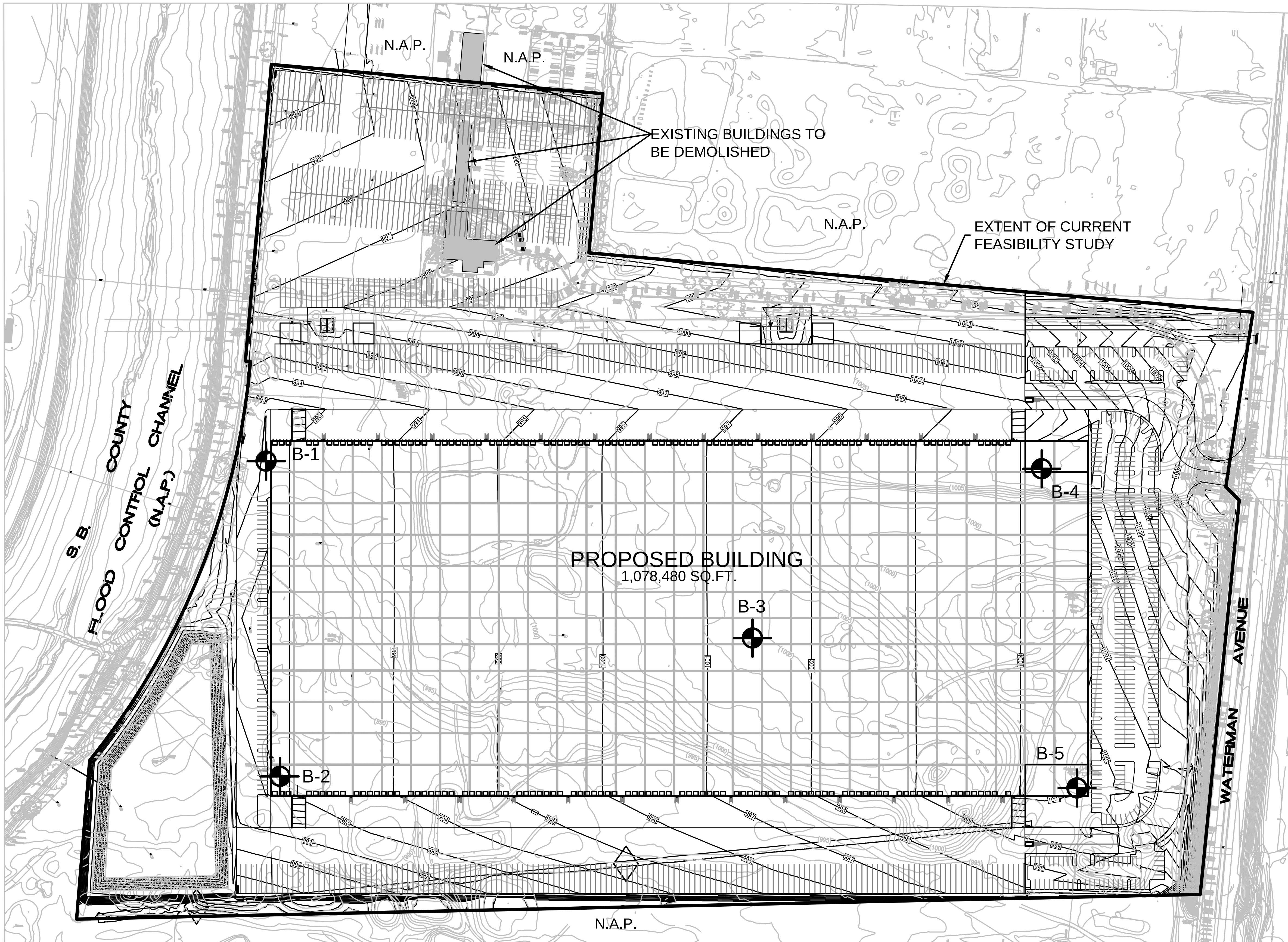
Youd, T. L. and Idriss, I. M. (Editors), "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," Salt Lake City, UT, January 5-6 1996, NCEER Technical Report NCEER-97-0022, Buffalo, NY.

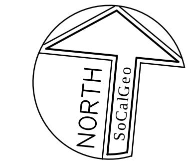
APPENDIX A



SOURCE: SAN BERNARDINO COUNTY
THOMAS GUIDE, 2013

SITE LOCATION MAP	
PROPOSED COMMERCIAL/INDUSTRIAL BUILDING	
SAN BERNARDINO, CALIFORNIA	
SCALE: 1" = 2400'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JLH CHKD: JAS	
SCG PROJECT 16G167-1	
PLATE 1	





GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

NOTE: BASE MAP PROVIDED BY THIENES ENGINEERING, INC.

BORING LOCATION PLAN

PROPOSED COMMERCIAL/INDUSTRIAL BUILDING

SAN BERNARDINO, CALIFORNIA

SCALE: 1" = 200'

DRAWN: PM
CHKD: DWN

SCG PROJECT
16G167-1R

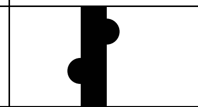
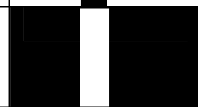
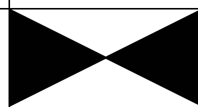
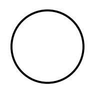
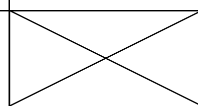
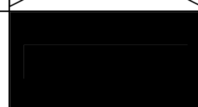
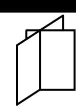
PLATE 2

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.: 16G167	DRILLING DATE: 10/14/16	WATER DEPTH: Dry
PROJECT: Proposed C/I Bldg	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 9 feet
LOCATION: San Bernardino, California	LOGGED BY: Jason Hiskey	READING TAKEN: 30 mins

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 995 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	4			DISTURBED ALLUVIUM: Light Gray Brown Silty fine Sand, trace Iron oxide staining, trace fine root fibers, loose-moist		17					
	X	11			ALLUVIUM: Light Gray fine Sand, little Silt, medium dense-dry to damp		2					
5												
	X	10			Light Gray fine Sand, trace medium Sand, loose to medium dense-dry		1					
	X	13			Light Gray fine to coarse Sand, little coarse Gravel, medium dense-dry		1			4		
10												
	X	40			Gray to Gray Brown fine to coarse Sand, trace fine to coarse Gravel, dense-dry to damp		2					
15												
	X	43					2					
20												
	X	34			Light Gray Brown fine Sand, little medium Sand, dense-dry to damp		2					
25												
	X	32	2.0		Gray to Gray Brown Silty fine Sand with occasional 1 to 2" thick Silty Clay to Clayey Silt lenses with moderate Organic content, dense-very moist		44					
30												
					Boring Terminated at 30' due to refusal on very dense soils							

TBL 16G167.GPJ SOCALGEO.GDT 11/16/16

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
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BORING NO. B-2

JOB NO.: 16G167	DRILLING DATE: 10/13/16	WATER DEPTH: Dry
PROJECT: Proposed C/I Bldg	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 15 feet
LOCATION: San Bernardino, California	LOGGED BY: Jason Hiskey	READING TAKEN:

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 990 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray Brown fine Sand, little medium Sand, trace Iron oxide staining, loose to medium dense-damp to moist							
		18				103	8					
		9				93	4					
5		4				97	6					
					Gray Brown fine Sandy Silt, trace fine root fibers, trace Iron oxide staining, loose-very moist		21					
					Light Gray fine to coarse Sand, little fine Gravel, occasional Cobbles, loose-damp							
		12				100	4					
		10				105	3					
10												
					Gray Brown Silty fine Sand with 1" Silty Clay laminations, medium dense to dense-moist							
		30					11					
15												
					Dark Gray fine to medium Sand, little Silt, medium dense-moist							
		17					8					
			1.0		Gray Brown Silt, little Clay, very stiff-very moist to wet		37					
20												

Boring Terminated at 20'

SOUTHERN CALIFORNIA GEOTECHNICAL
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BORING NO. B-3

JOB NO.: 16G167	DRILLING DATE: 10/14/16	WATER DEPTH: Dry
PROJECT: Proposed C/I Bldg	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 35 feet
LOCATION: San Bernardino, California	LOGGED BY: Jason Hiskey	READING TAKEN: 30 mins

SURFACE ELEVATION: 1004.5 feet MSL

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	8			POSSIBLE FILL: Light Gray Silty fine Sand, trace fine root fibers, loose-dry to damp		2					
	X	11			POSSIBLE FILL: Light Gray Brown Silty fine Sand, trace medium Sand, 1" Silty Clay lenses, trace fine root fibers, medium dense-dry to damp		3					
5												
	X	22	4.5		ALLUVIUM: Light Gray Brown Silty Clay, very stiff-damp to moist		10					
	X	16			Light Gray Brown Silty fine Sand to fine Sandy Silt, little calcareous nodules, trace Clay, trace fine Gravel, medium dense-damp		7			47		
10												
	X	17	3.5		Light Gray Silty Clay, abundant calcareous nodules, little fine Sand, very stiff-very moist		26	42	19	86		
15												
	X	24			Light Gray Brown Silty fine Sand, medium dense-damp		4			24		
20												
	X	50			Light Gray fine Sand, trace medium Sand, trace Silt, dense to very dense-damp		3					
25												
	X	25	2.0		Dark Gray Silty Clay interbedded with fine Sand lenses, very stiff to medium dense-very moist		26			66		
30												
	X	35	0.5		Dark Gray Silty Clay interbedded with fine Sand lenses, hard to dense-very moist		24					

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 16G167
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino, California

DRILLING DATE: 10/14/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: Dry
CAVE DEPTH: 35 feet
READING TAKEN: 30 mins

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
					Dark Gray Silty Clay interbedded with fine Sand lenses, hard to dense-very moist							
					Gray fine to medium Sand, trace coarse Sand with Clayey Silt angular clasts, trace calcareous nodules, dense-damp							
	X	41					5					
40												
					Gray Brown Silty fine Sand with 2" Clayey Silt lenses, dense-very moist							
	X	36					21					
45												
					Gray Brown Silty fine Sand, very dense-moist							
	X	62					14					
50												
					Boring Terminated at 50'							

TBL 16G167.GPJ SOCALGEO.GDT 11/16/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 16G167
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino, California

DRILLING DATE: 10/13/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: Dry
CAVE DEPTH: 11 feet
READING TAKEN:

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1007.5 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
		20			POSSIBLE FILL: Light Gray Brown fine to medium Sand, trace coarse Sand, medium dense-dry to damp	102	2					
		19			POSSIBLE FILL: Brown Silty fine to medium Sand, trace coarse Sand, medium dense-moist	110	9					
5		16			ALLUVIUM: Light Brown fine to medium Sand, trace coarse Sand, medium dense-damp	103	5					
		15			Dark Brown Silty fine Sand to fine Sandy Silt, loose to medium dense-damp	105	7					
10		12			Light Gray Brown fine Sand, trace medium Sand, loose-dry to damp	91	2					
15		14			Light Gray fine to medium Sand, little coarse Sand, little fine to coarse Gravel, medium dense-dry to damp		2					
20		30			Light Gray Brown fine to coarse Sand, trace fine Gravel, medium dense to dense-dry to damp		2					
					Boring Terminated at 20'							

TBL 16G167.GPJ SOCALGEO.GDT 11/16/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 16G167
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino, California

DRILLING DATE: 10/14/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: Dry
CAVE DEPTH: 47 feet
READING TAKEN: 30 mins

SURFACE ELEVATION: 999.5 feet MSL

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray Brown fine to medium Sand, loose to medium dense-dry							
		10					1					
					Light Gray Brown fine Sand, trace Silt, loose-dry to damp							
		6					3					
5		11				87	3					
		16			Light Brown fine to medium Sand, trace fine Gravel, medium dense-dry to damp		2					
		12			Light Gray fine Sand, trace medium Sand, medium dense-dry to damp		2			2		
10												
					Dark Gray Brown Silty Clay, little fine Sand, stiff-very moist							
		11	2.5				27	40	20	84		
15												
					Gray Brown Silty fine Sand, trace Iron oxide staining, medium dense-moist							
		17					10			39		
20												
					Dark Gray Silty Clay with fine Sandy Silt lenses, trace Organic content, stiff to medium dense-very moist							
		12	1.5				39			81		
25												
		13	1.5				43	61	26	86		
30												
					Gray fine to medium Sand with 1" lenses of Clayey Silt, dense-very moist to wet							
		33	2.0				25					

TBL 16G167.GPJ SOCALGEO.GDT 11/16/16

JOB NO.: 16G167
PROJECT: Proposed C/I Bldg
LOCATION: San Bernardino, California

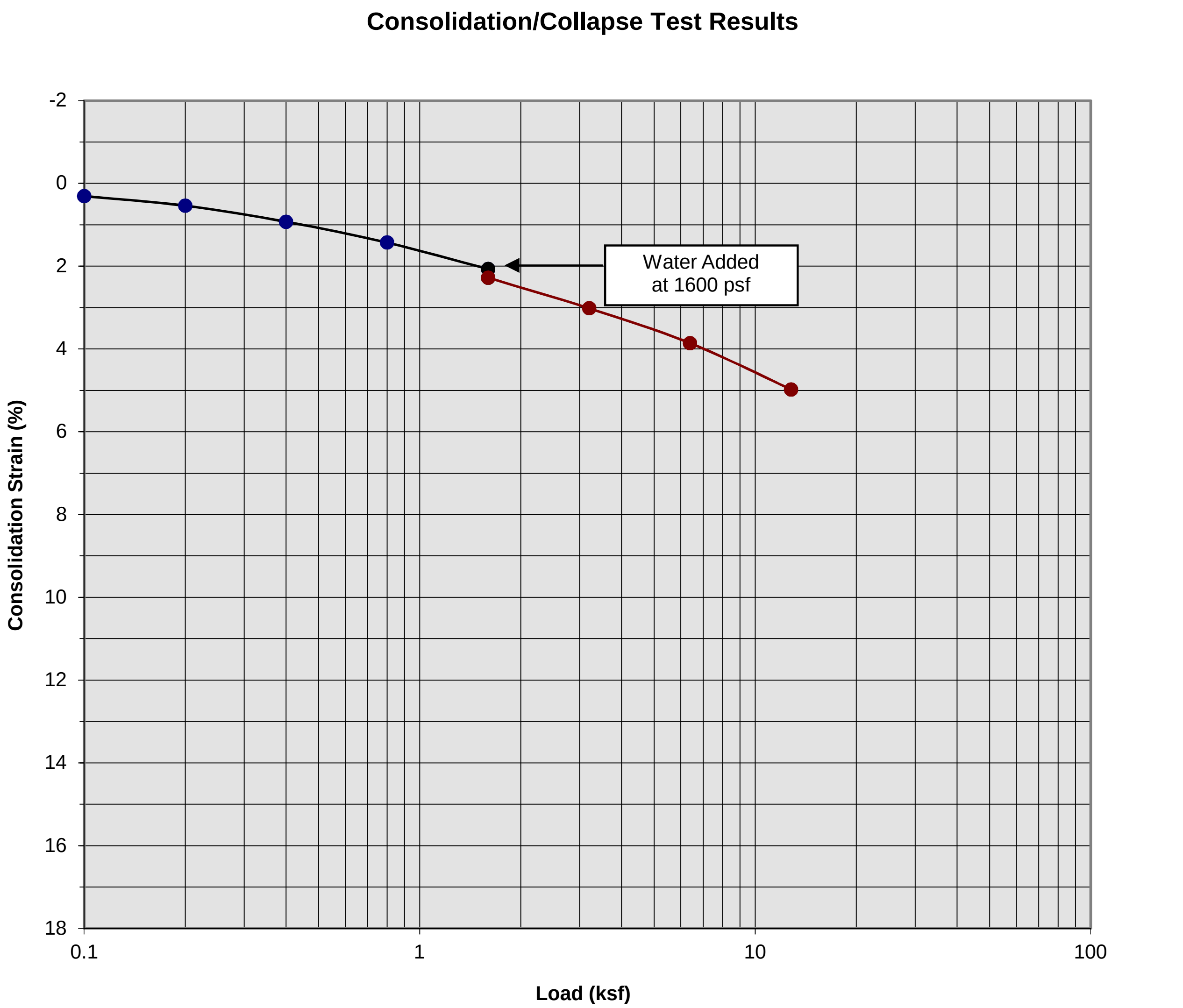
DRILLING DATE: 10/14/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: Dry
CAVE DEPTH: 47 feet
READING TAKEN: 30 mins

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
					Gray fine to medium Sand with 1" lenses of Clayey Silt, dense-very moist to wet							
					Gray Silty fine Sand, trace medium Sand, dense-moist							
	X	38					12					
40												
					Gray Brown fine Sand, trace Silt, dense-moist							
	X	40					14					
45												
					Gray Brown Silty fine Sand to fine Sandy Silt, medium dense-very moist							
	X	29					26					
50												
					Boring Terminated at 50'							

TBL 16G167.GPJ SOCALGEO.GDT 11/16/16

APPENDIX C



Classification: Light Gray Brown fine Sand, little medium Sand

Boring Number:	B-2	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	1 to 2	Initial Dry Density (pcf)	103.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.21

Proposed C/I Bldg
San Bernardino, California
Project No. 16G167
PLATE C- 1

SoCalGeo
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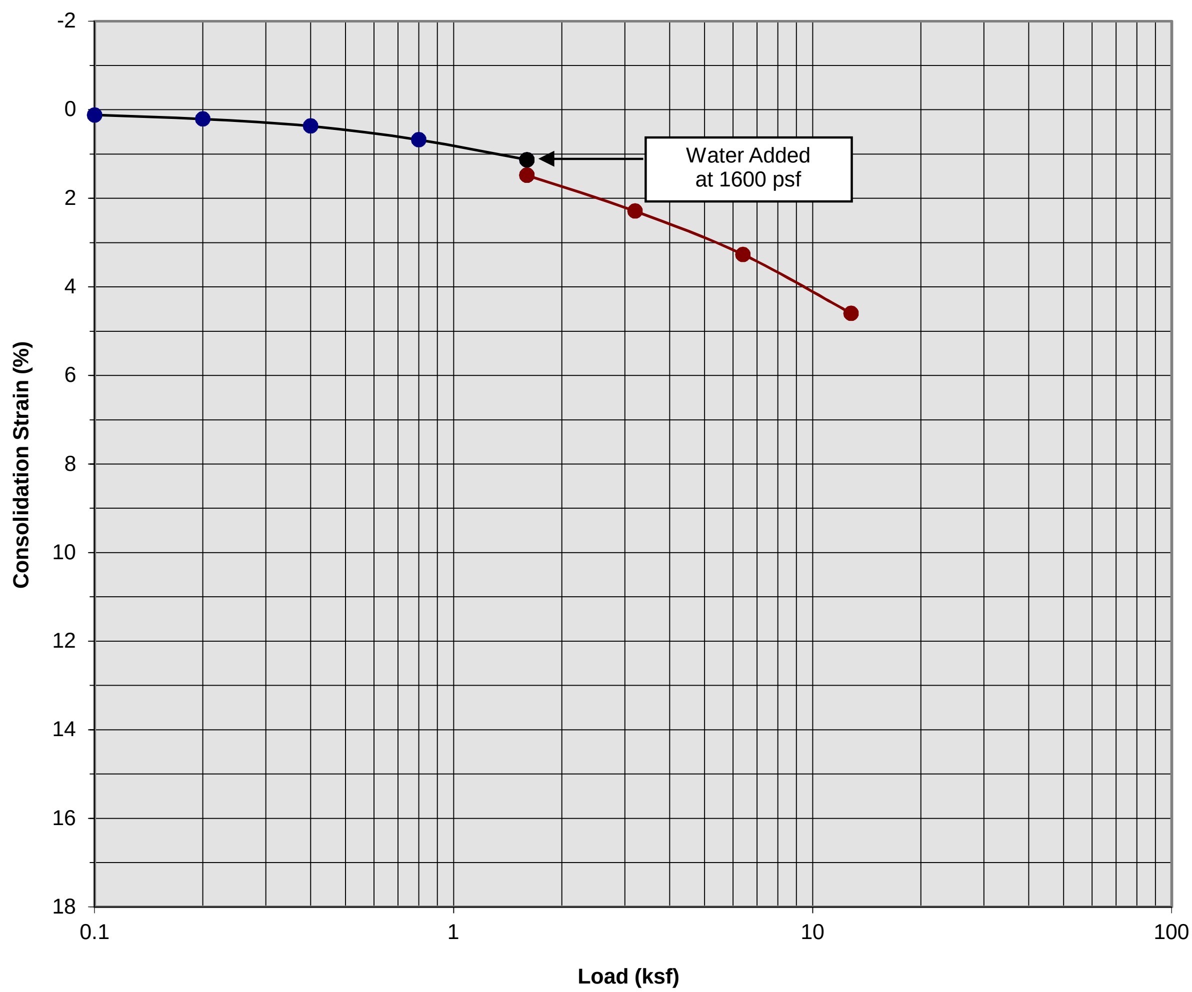
Consolidation/Collapse Test Results



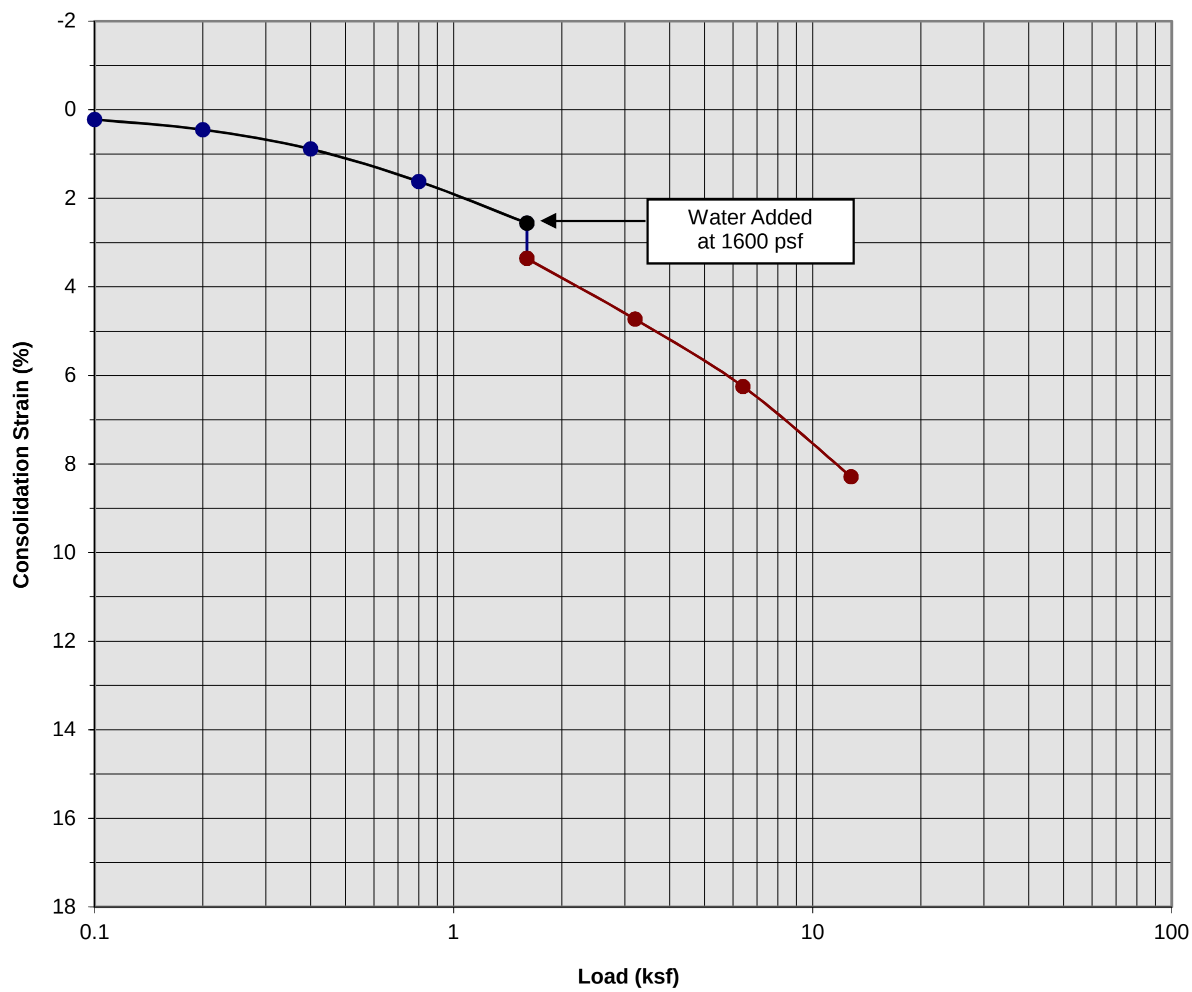
Classification: Light Gray Brown fine Sand, little medium Sand

Boring Number:	B-2	Initial Moisture Content (%)	4
Sample Number:	---	Final Moisture Content (%)	26
Depth (ft)	3 to 4	Initial Dry Density (pcf)	92.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	96.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.35

Proposed C/I Bldg
San Bernardino, California
Project No. 16G167
PLATE C- 2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



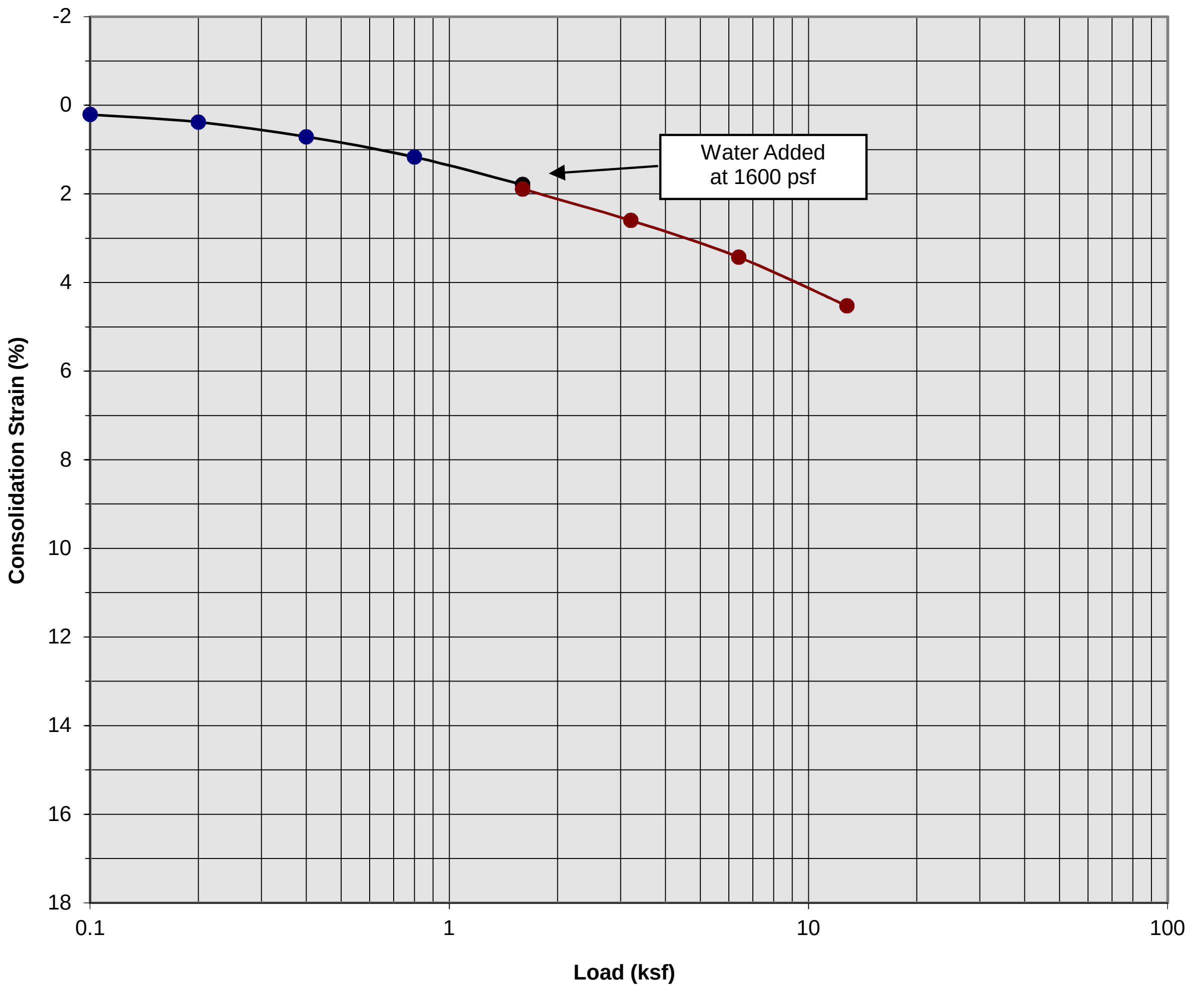
Classification: Gray Brown fine Sand, little medium Sand

Boring Number:	B-2	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	23
Depth (ft)	5 to 6	Initial Dry Density (pcf)	97.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	105.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.80

Proposed C/I Bldg
San Bernardino, California
Project No. 16G167
PLATE C- 3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: Light Gray fine to coarse Sand, little fine Gravel

Boring Number:	B-2	Initial Moisture Content (%)	4
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	7 to 8	Initial Dry Density (pcf)	99.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	101.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.10

Proposed C/I Bldg
San Bernardino, California
Project No. 16G167
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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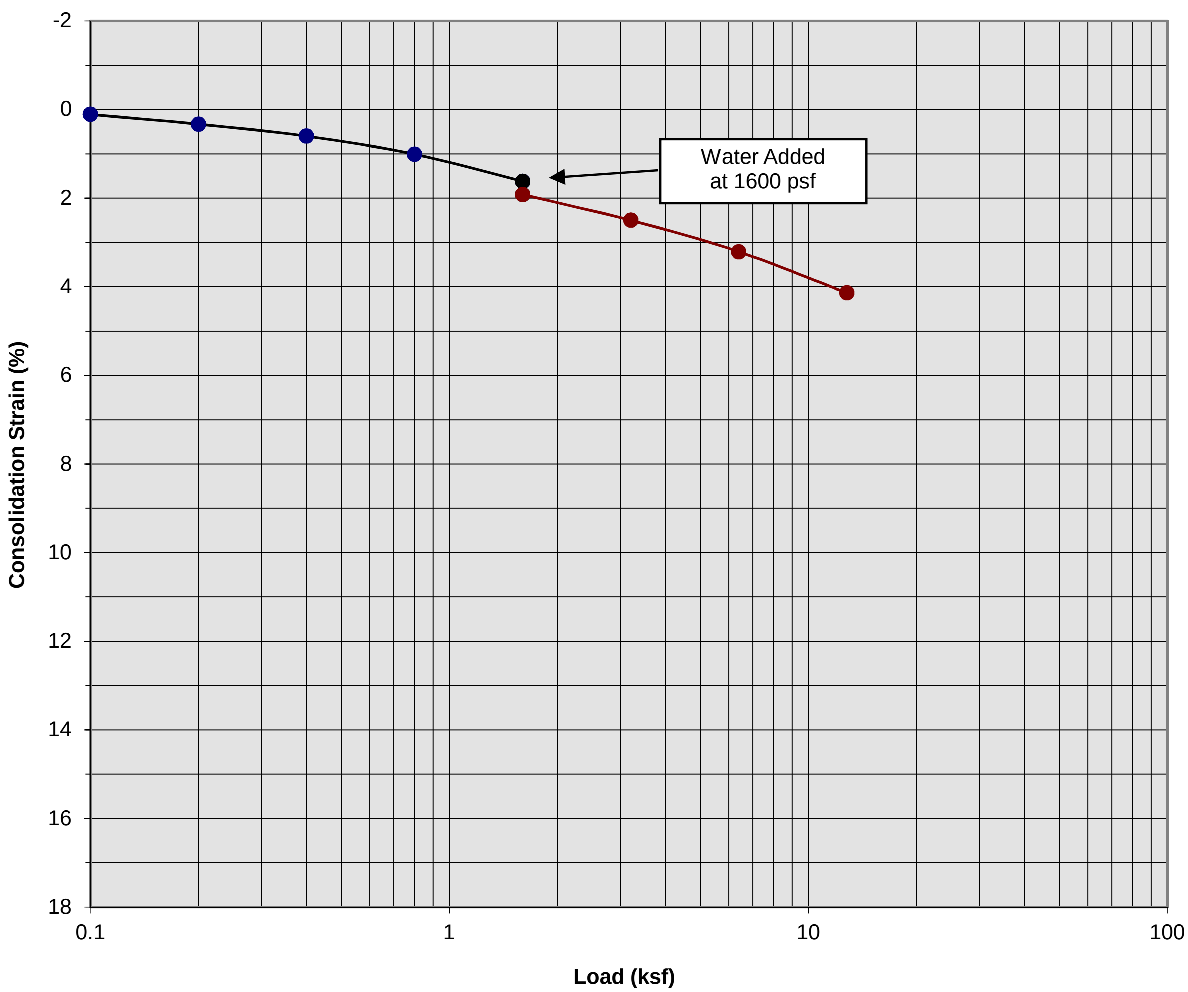
Consolidation/Collapse Test Results



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

Boring Number:	B-4	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	1 to 2	Initial Dry Density (pcf)	101.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	104.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.30

Proposed C/I Bldg
San Bernardino, California
Project No. 16G167
PLATE C- 5

SoCalGeo
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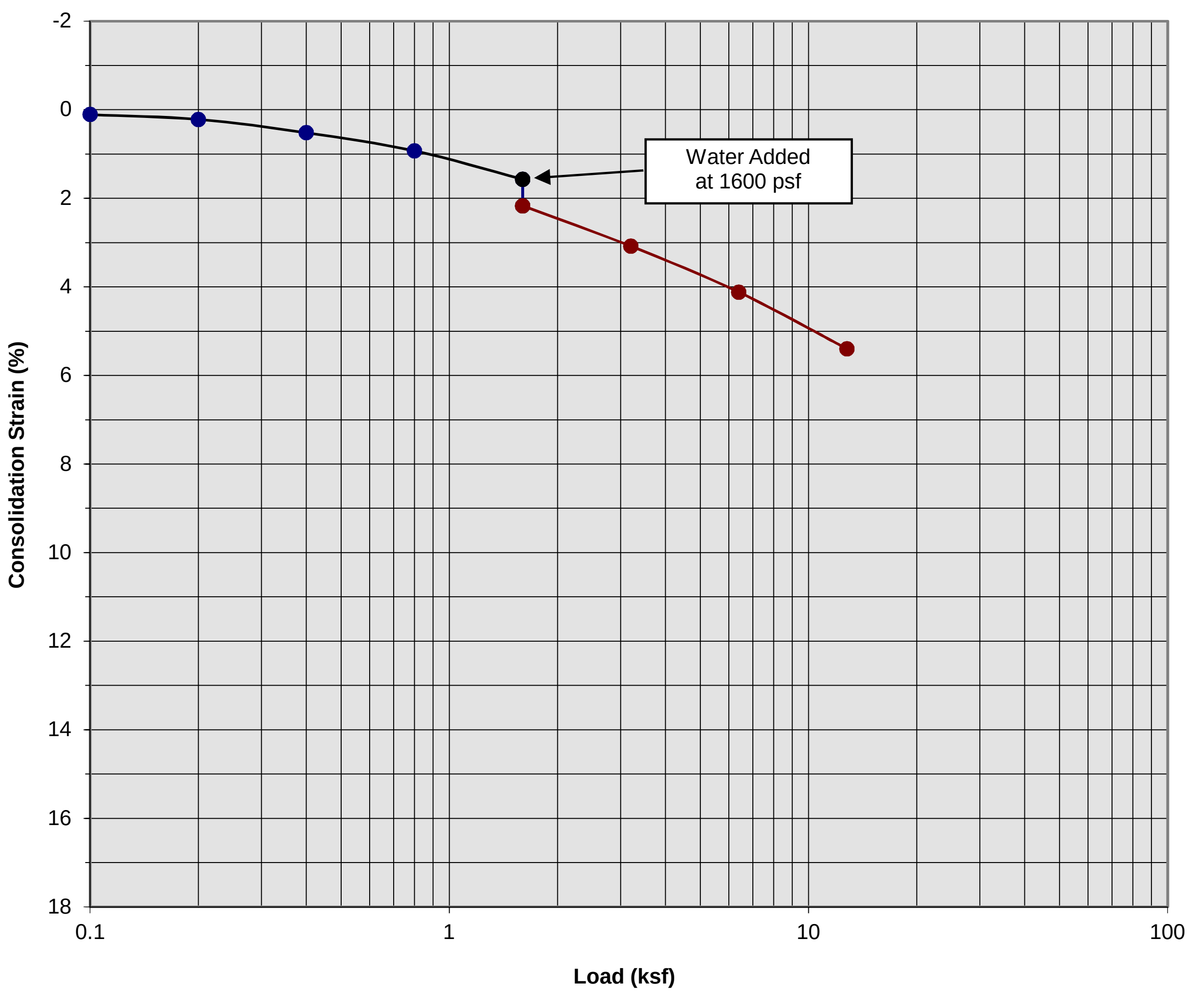
Consolidation/Collapse Test Results



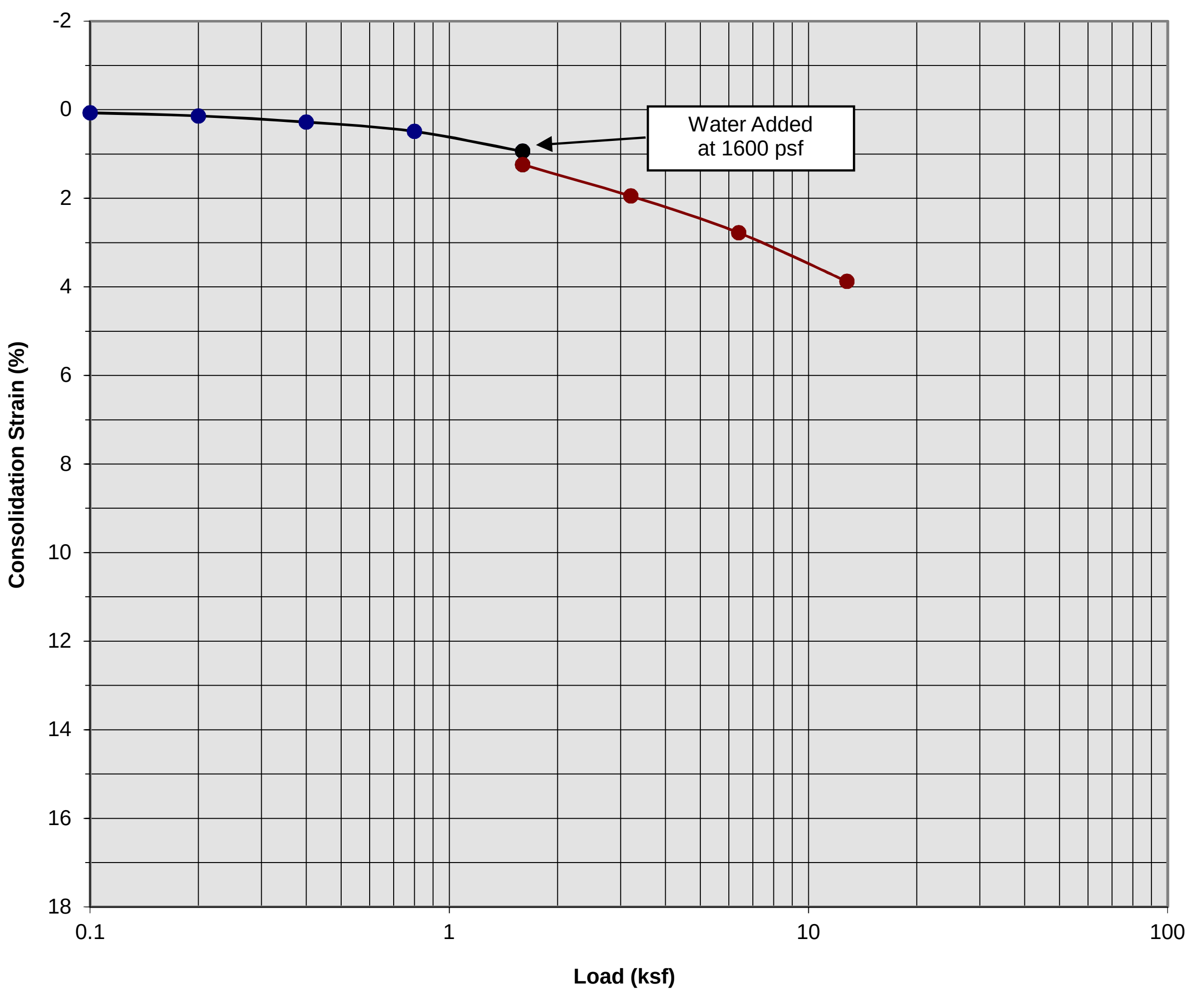
Classification: Brown Silty fine to medium Sand, trace coarse Sand

Boring Number:	B-4	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	3 to 4	Initial Dry Density (pcf)	109.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	116.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.60

Proposed C/I Bldg
San Bernardino, California
Project No. 16G167
PLATE C- 6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



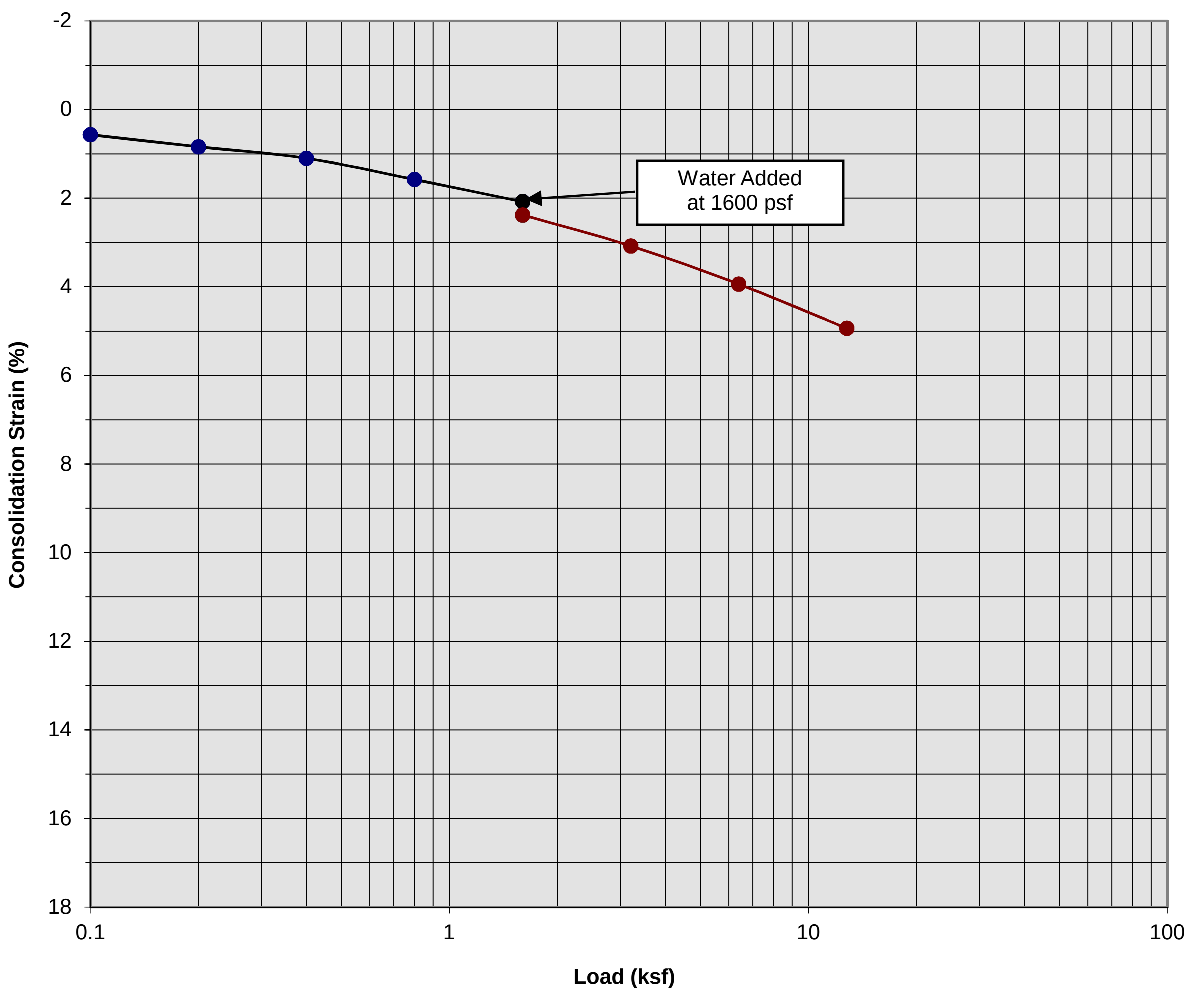
Classification: Light Brown fine to medium Sand, trace coarse Sand

Boring Number:	B-4	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	5 to 6	Initial Dry Density (pcf)	103.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	107.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.30

Proposed C/I Bldg
San Bernardino, California
Project No. 16G167
PLATE C- 7

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: Dark Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-4	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	7 to 8	Initial Dry Density (pcf)	105.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.30

Proposed C/I Bldg
San Bernardino, California
Project No. 16G167
PLATE C- 8

SoCalGeo
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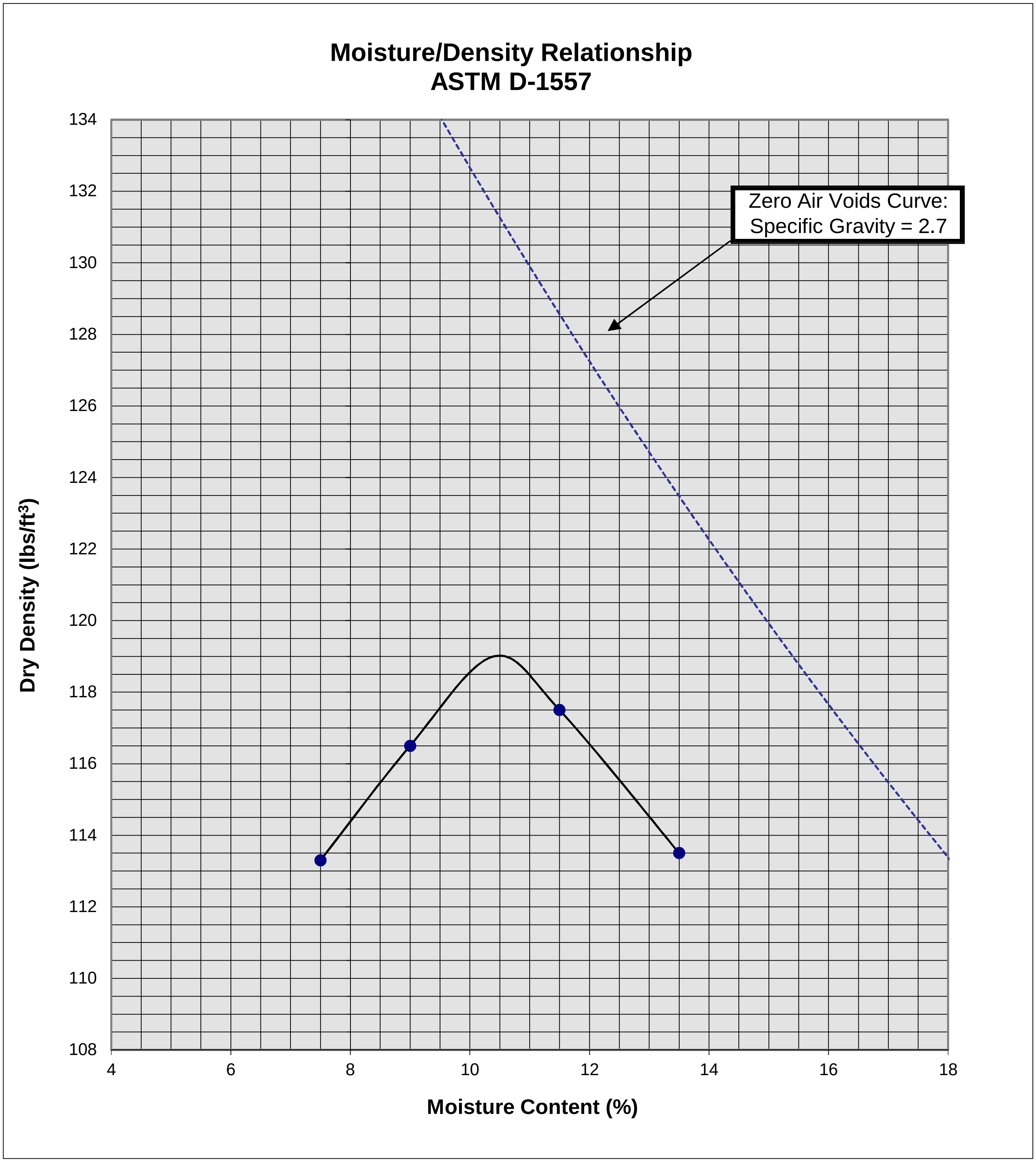

Soil ID Number	B-2 @ 0 to 5'
Optimum Moisture (%)	11
Maximum Dry Density (pcf)	119.5
Soil Classification	Gray Brown fine Sand, trace Silt, little medium to coarse Sand,

Proposed C/I Bldg
San Bernardino, California
Project No. 16G167
PLATE C-9

SOUTHERN CALIFORNIA GEOTECHNICAL
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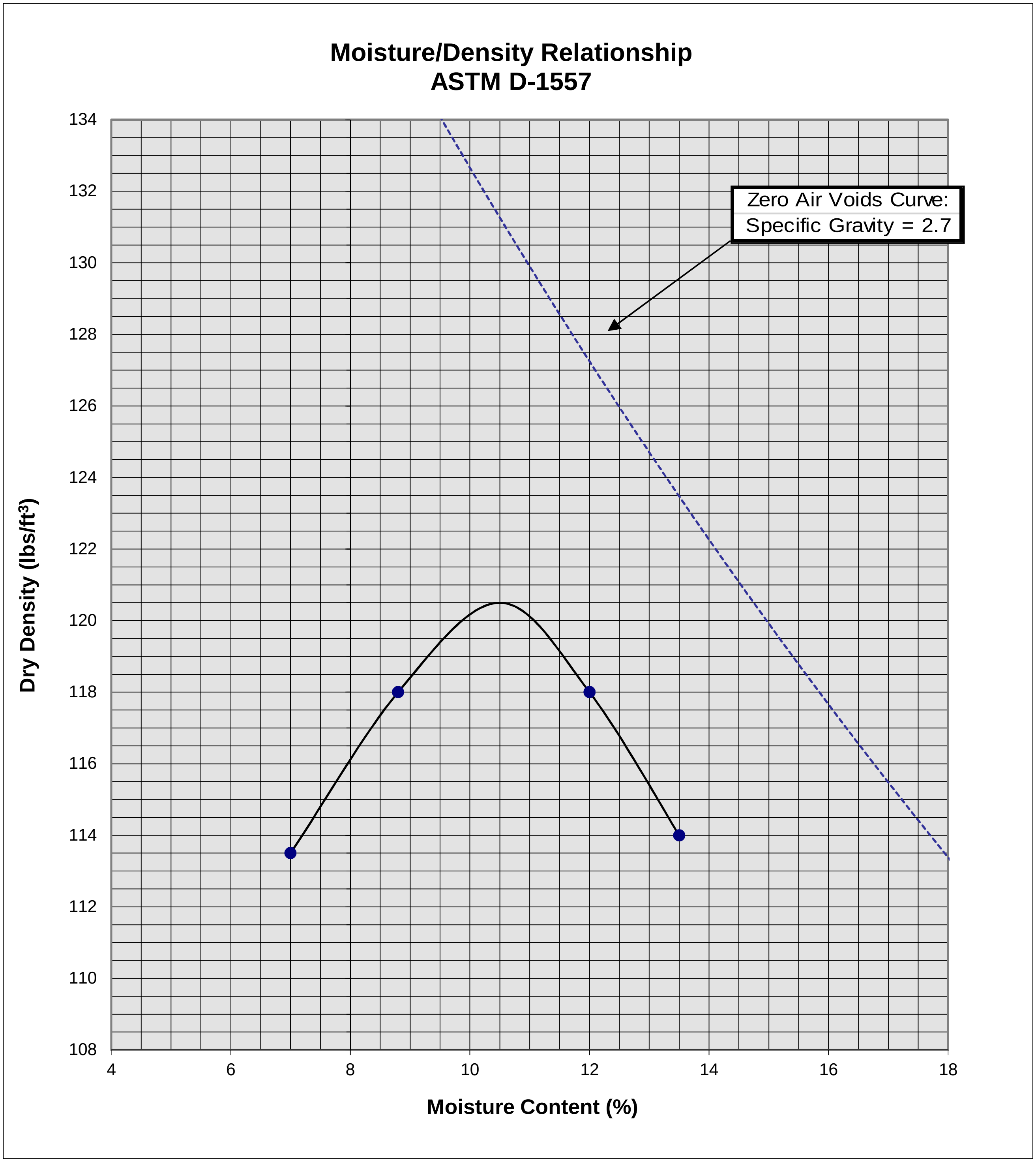


Soil ID Number			B-4 @ 0 to 5'
Optimum Moisture (%)			10.5
Maximum Dry Density (pcf)			120.5
Soil Classification	Brown fine to medium Sand, trace Silt, trace coarse Sand,		

Proposed C/I Bldg
San Bernardino, California
Project No. 16G167
PLATE C-10

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.
- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.
- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.
- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.
- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.
- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.
- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

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

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

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

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

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

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

<u>Compacted Fills</u>

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

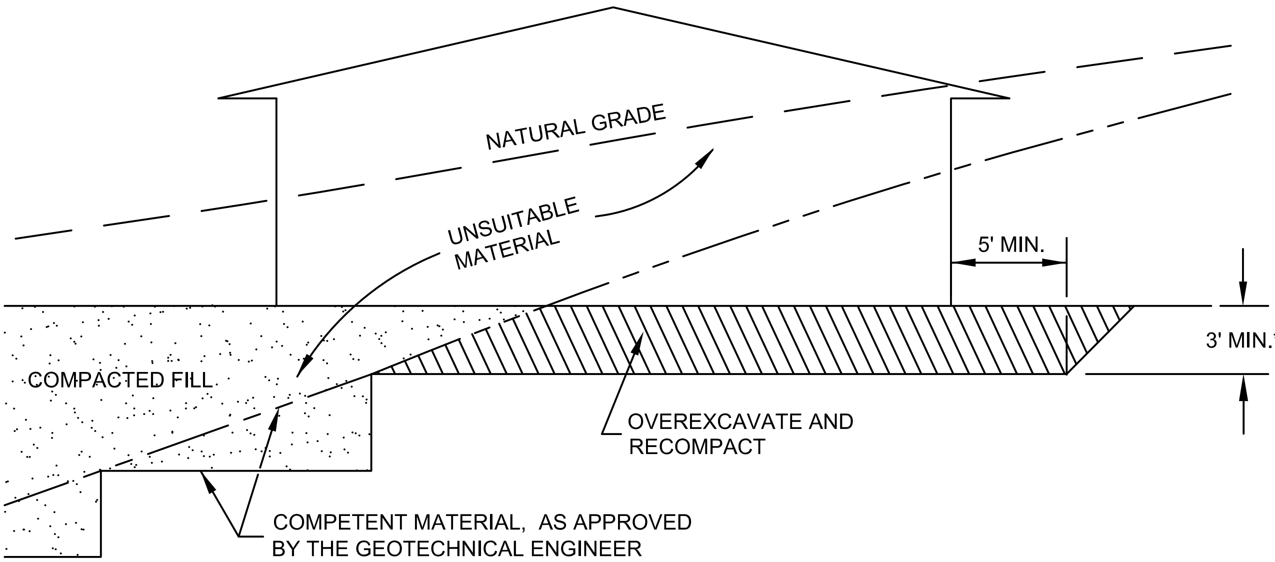
Subdrains

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

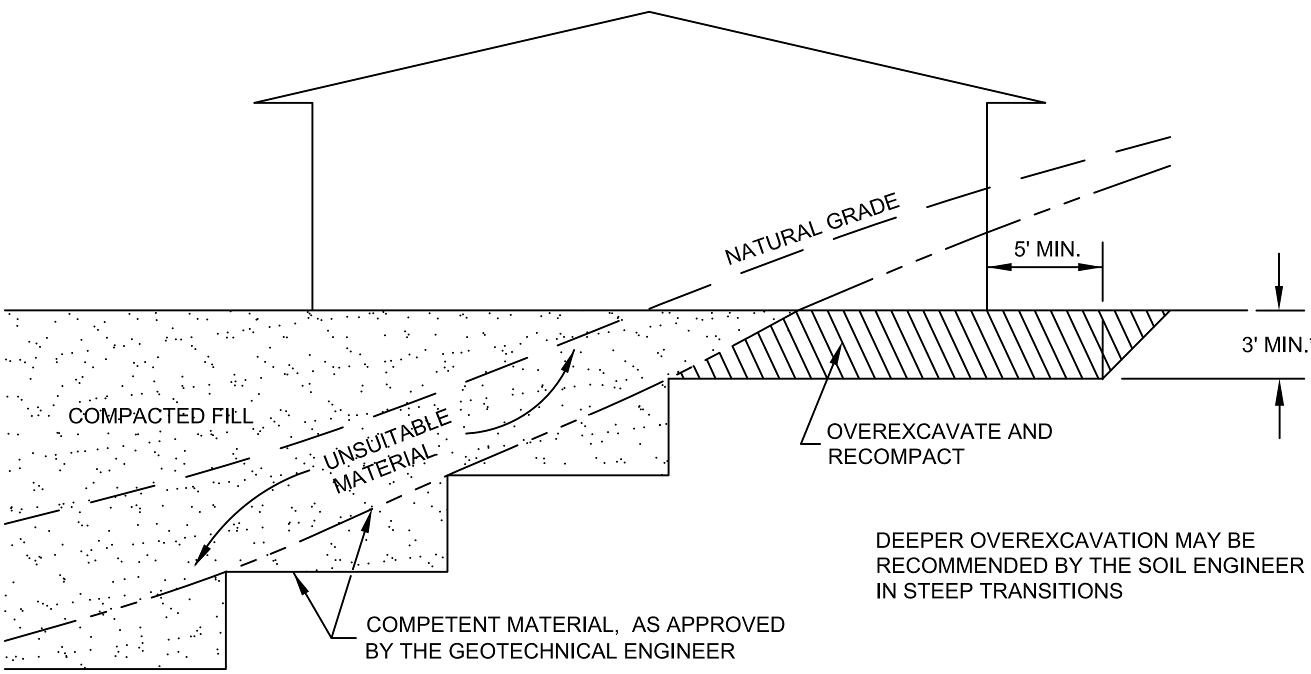
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

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

CUT LOT



CUT/FILL LOT (TRANSITION)



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

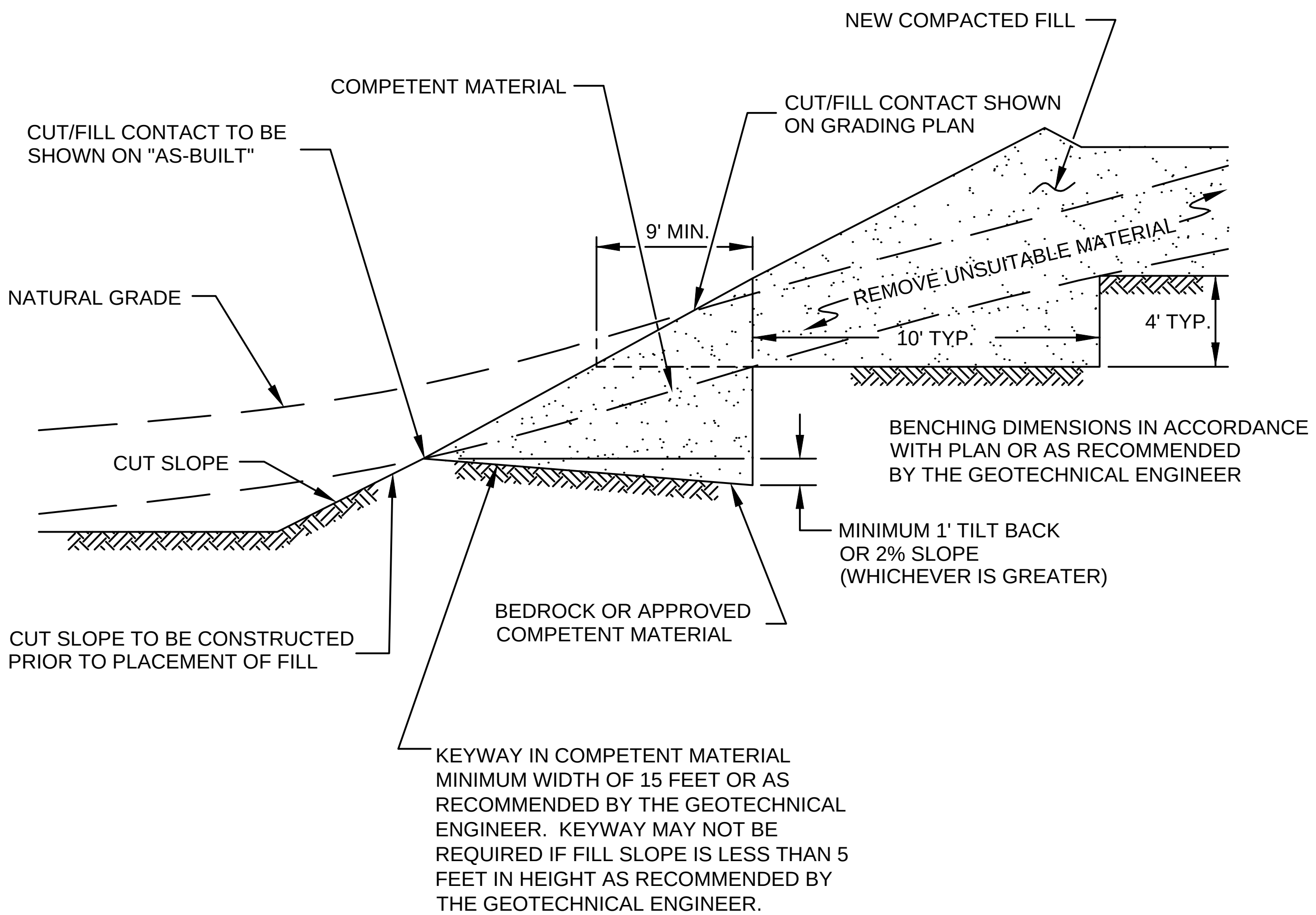
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

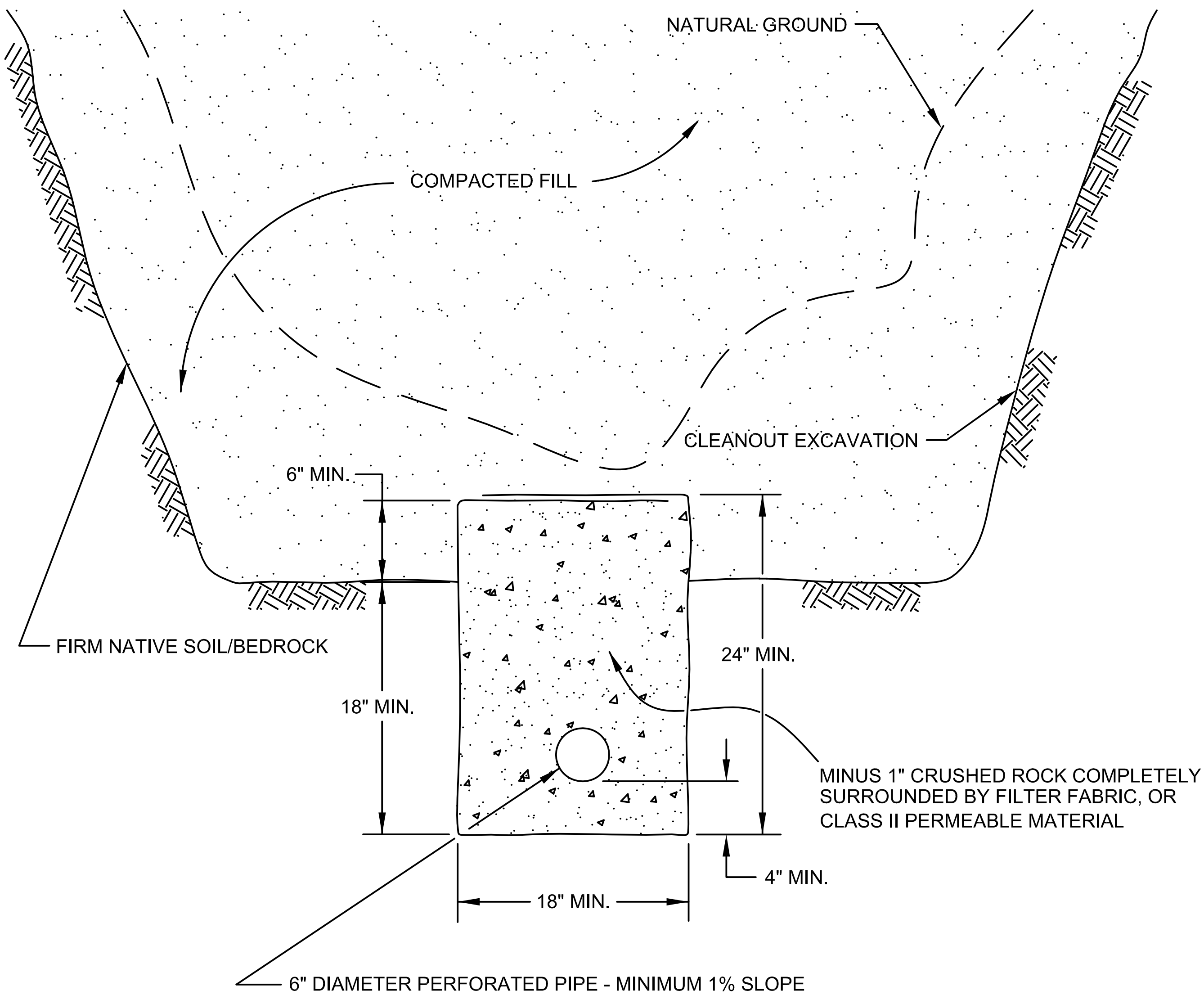
DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

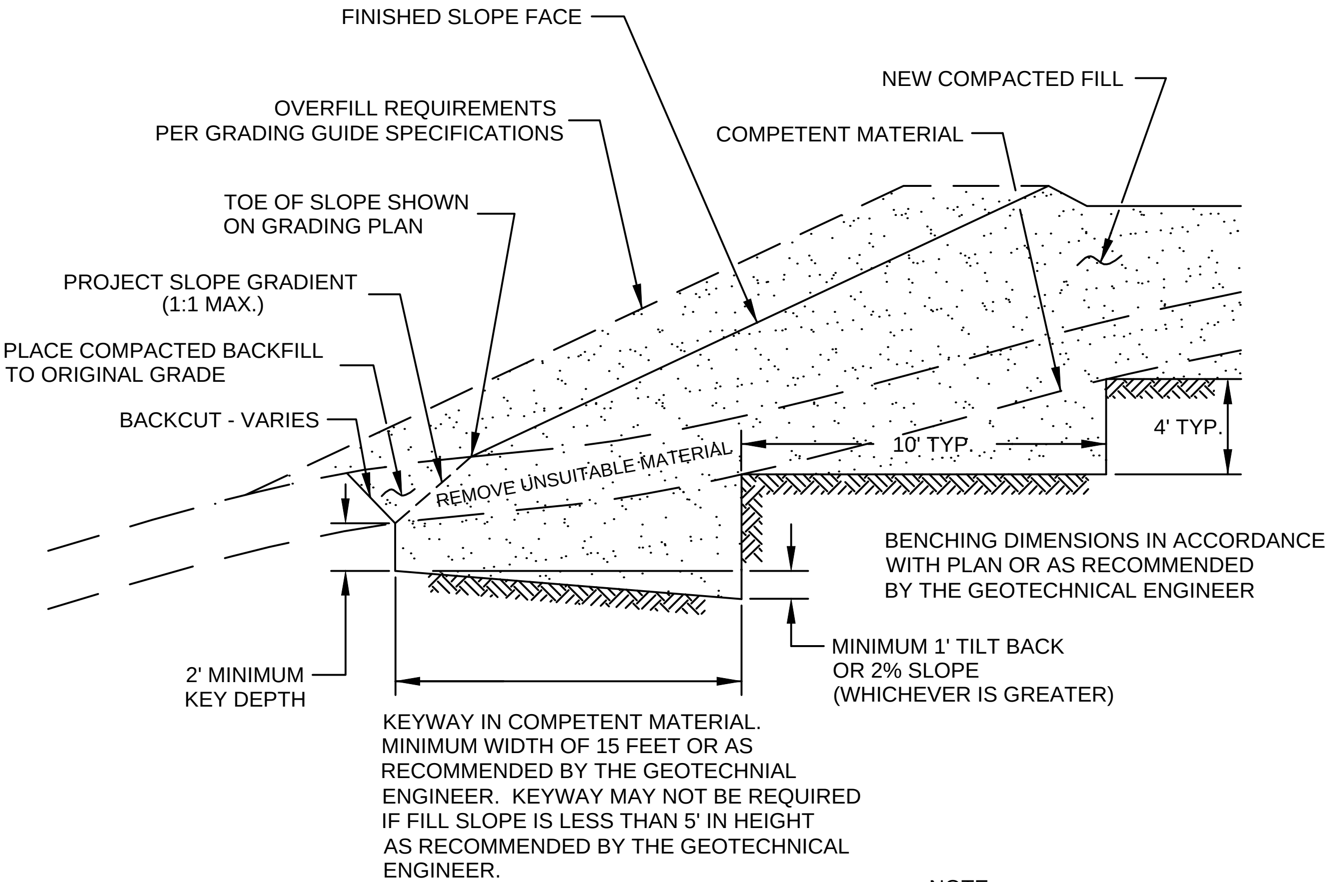
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

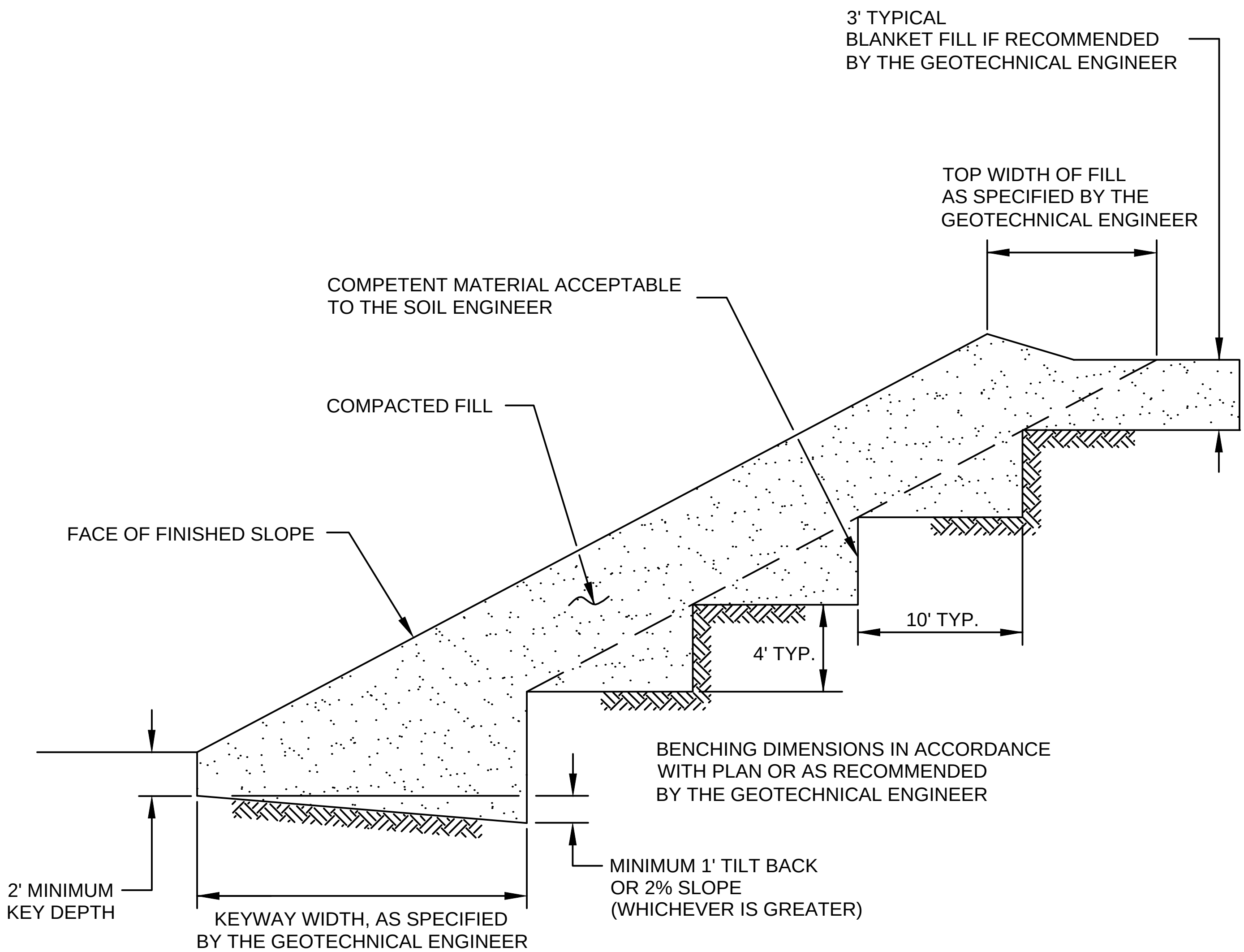
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo
SOUTHERN
CALIFORNIA
GEOTECHNICAL



STABILIZATION FILL DETAIL

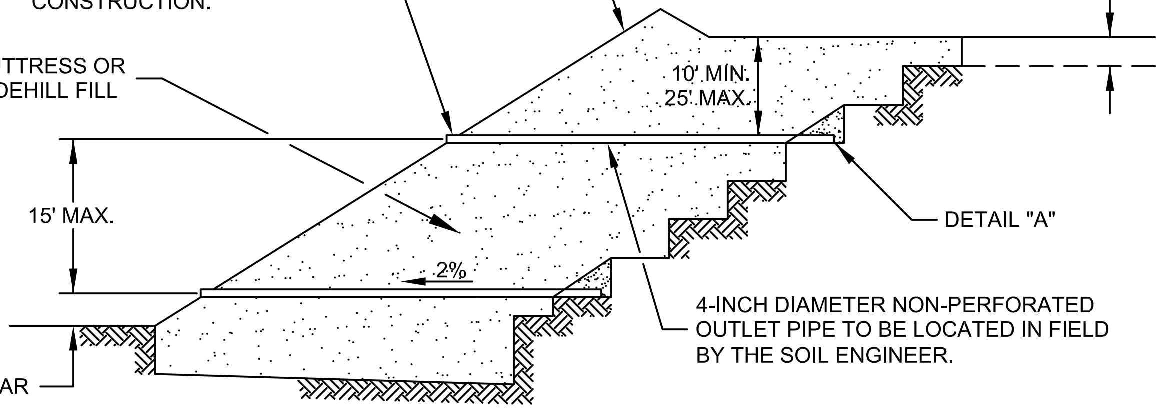
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



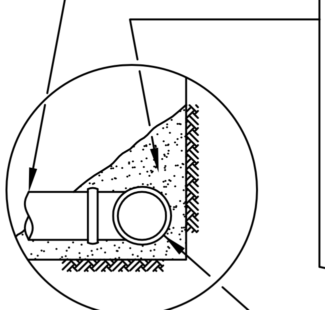
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50



DETAIL "A"

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

SLOPE FILL SUBDRAINS

GRADING GUIDE SPECIFICATIONS

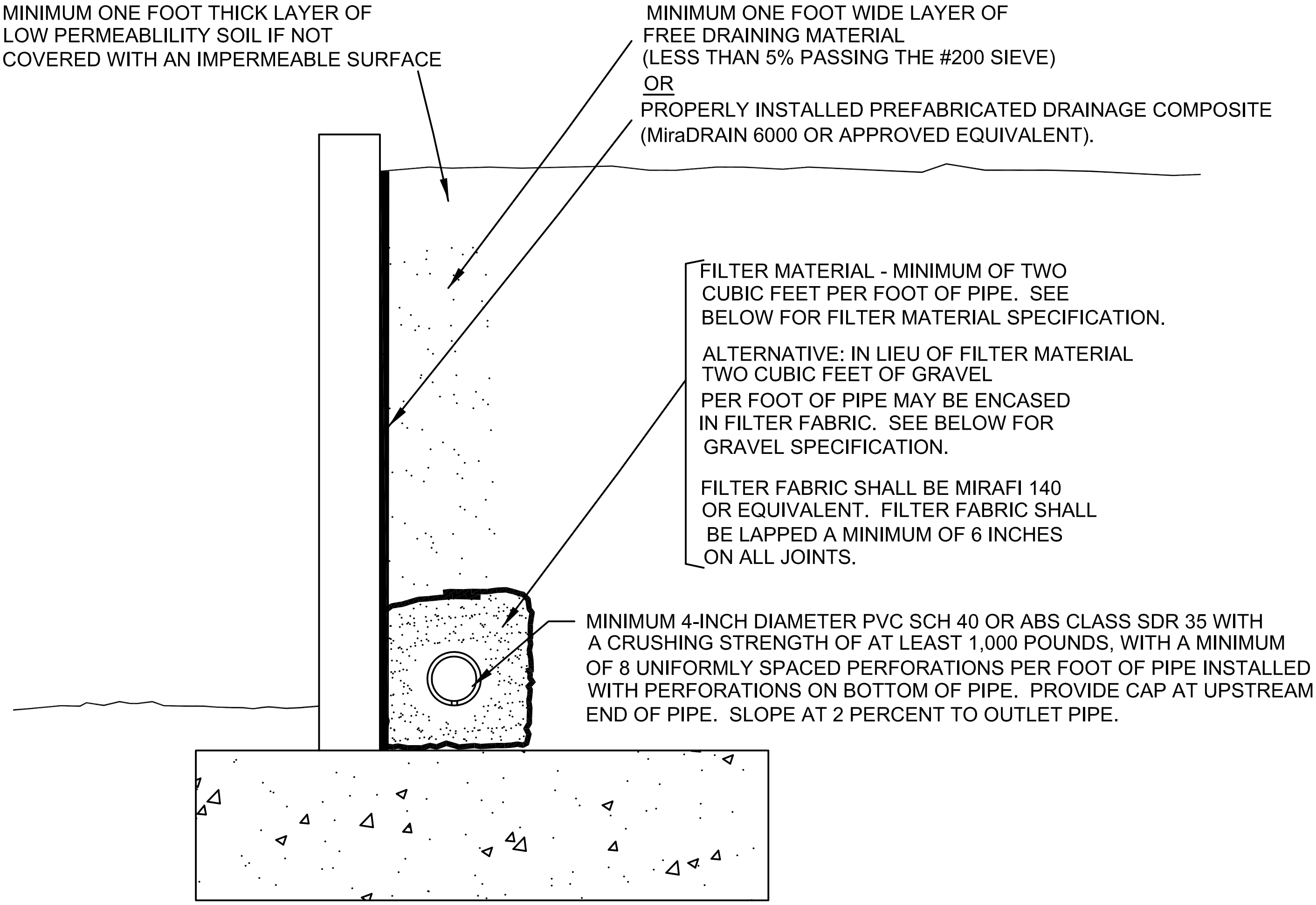
NOT TO SCALE

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

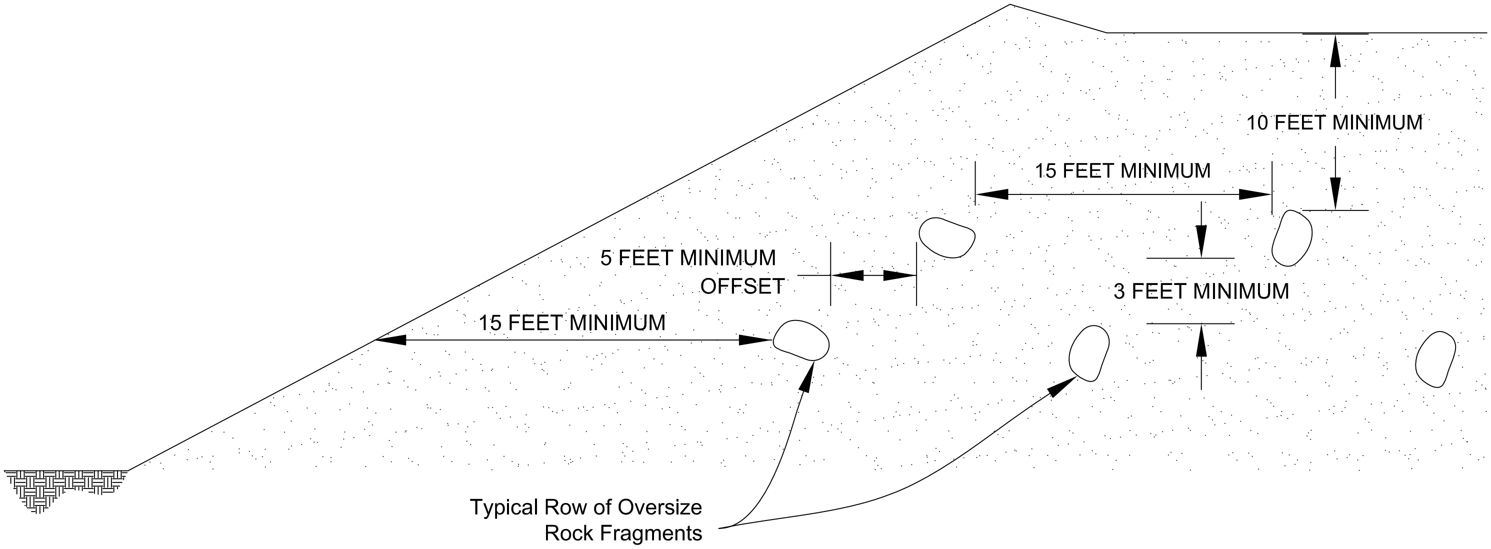
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

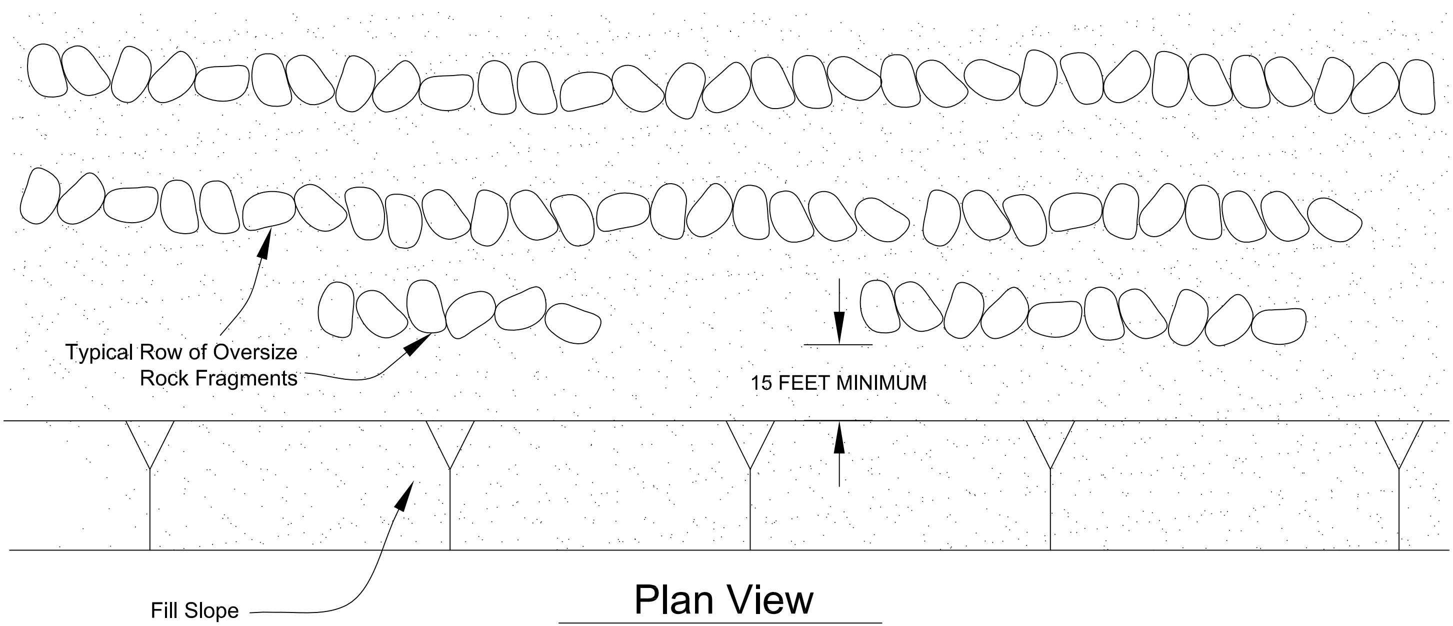
PLATE D-7

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

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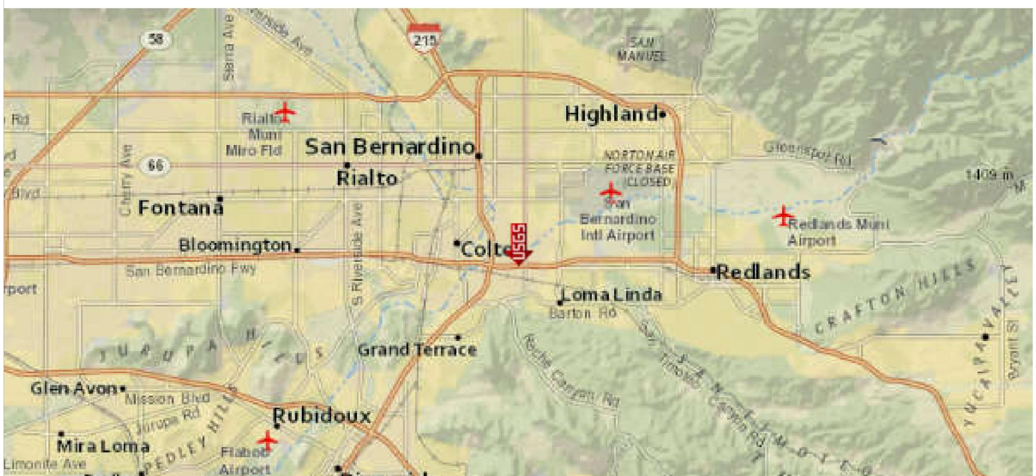
APPENDIX E

USGS Design Maps Summary Report

User–Specified Input

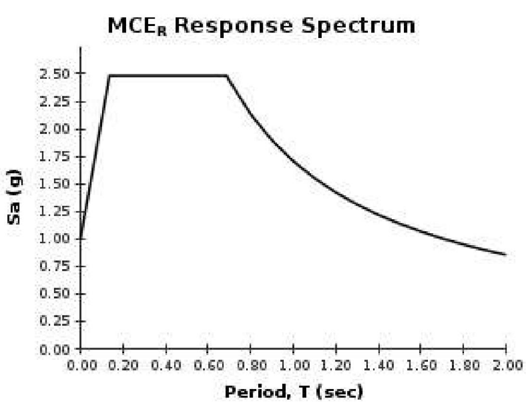
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	34.07341°N, 117.28234°W
Site Soil Classification	Site Class D – “Stiff Soil”
Risk Category	I/II/III

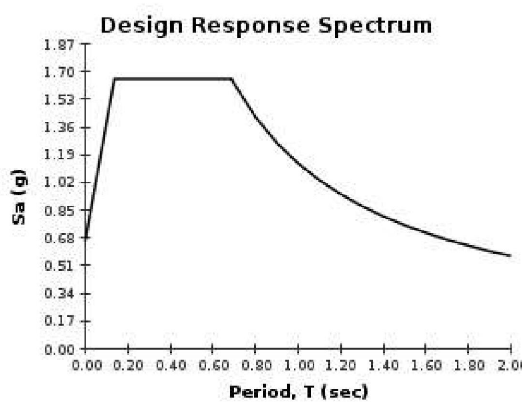


USGS–Provided Output

S_S =	2.482 g	S_{MS} =	2.482 g	S_{DS} =	1.655 g
S_1 =	1.137 g	S_{M1} =	1.706 g	S_{D1} =	1.137 g

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





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

SEISMIC DESIGN PARAMETERS	
PROPOSED COMMERCIAL/INDUSTRIAL BUILDING	
SAN BERNARDINO, CALIFORNIA	
DRAWN: AL CHKD: JAS	SOUTHERN CALIFORNIA GEOTECHNICAL
SCG PROJECT 16G167-1	
PLATE E-1	

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

From Figure 22-7 [4]	PGA = 0.954
Equation (11.8–1):	$PGA_M = F_{PGA}PGA = 1.000 \times 0.954 = 0.954$ g

Table 11.8–1: Site Coefficient F_{PGA}

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

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

For Site Class = D and PGA = 0.954 g, F_{PGA} = 1.000

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

From Figure 22-17 [5]	C_{RS} = 1.010
From Figure 22-18 [6]	C_{R1} = 0.963

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

MCE PEAK GROUND ACCELERATION
PROPOSED COMMERCIAL/INDUSTRIAL BUILDING
SAN BERNARDINO, CALIFORNIA

DRAWN: PM
CHKD: JAS
SCG PROJECT
16G167-1
PLATE E-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Proposed C/I Building	MCE$_G$ Design Acceleration	0.954	(g)
Project Location	San Bernardino, CA	Design Magnitude	7	
Project Number	16G167	Historic High Depth to Groundwater	10	(ft)
Engineer	PM	Depth to Groundwater at Time of Drilling	80	(ft)
		Borehole Diameter	6	(in)
Boring No.	B-1			

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	10	5		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	600	600	600	0.99	1.02	1.07	0.06	N/A	N/A	N/A	Above Water Table
10	10	12	11	13	120	4	1.3	1.05	1.29	1.70	0.75	29.2	29.2	120	58	120	0.97	1.17	1.1	0.44	0.57	1.25	0.45	**Liquefiable**
14.5	12	17	14.5	40	120		1.3	1.05	1.3	1.04	0.85	62.5	62.5	1740	1459	1740	0.96	1.21	1.1	2.00	2.00	0.71	2.83	Nonliquefiable
19.5	17	22	19.5	43	120		1.3	1.05	1.3	0.99	0.95	71.5	71.5	2340	1747	2340	0.93	1.21	1.05	2.00	2.00	0.77	2.58	Nonliquefiable
24.5	22	27	24.5	34	120		1.3	1.05	1.3	0.93	0.95	53.3	53.3	2940	2035	2940	0.91	1.21	1.01	2.00	2.00	0.81	2.46	Nonliquefiable
29.5	27	30	28.5	32	120		1.3	1.05	1.3	0.89	0.95	47.8	47.8	3420	2266	3420	0.89	1.21	0.98	2.00	2.00	0.83	2.41	Nonliquefiable

Notes:

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

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed C/I Building
Project Location	San Bernardino, CA
Project Number	16G167
Engineer	PM

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	10	5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	10.00		0.000		0.00	Above Water Table
10	10	12	11	29.2	0.0	29.2	0.45	0.05	-0.04	0.05	2.00		0.011		0.25	**Liquefiable**
14.5	12	17	14.5	62.5	0.0	62.5	2.83	0.00	-2.63	0.00	5.00		0.000		0.00	Nonliquefiable
19.5	17	22	19.5	71.5	0.0	71.5	2.58	0.00	-3.43	0.00	5.00		0.000		0.00	Nonliquefiable
24.5	22	27	24.5	53.3	0.0	53.3	2.46	0.00	-1.86	0.00	5.00		0.000		0.00	Nonliquefiable
29.5	27	30	28.5	47.8	0.0	47.8	2.41	0.00	-1.41	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				0.25	

Notes:

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

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

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

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

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

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

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

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

LIQUEFACTION EVALUATION

Project Name	Proposed C/I Building	MCE_G Design Acceleration	0.954 (g)
Project Location	San Bernardino, CA	Design Magnitude	7
Project Number	16G167	Historic High Depth to Groundwater	10 (ft)
Engineer	PM	Depth to Groundwater at Time of Drilling	80 (ft)
		Borehole Diameter	6 (in)

Boring No. B-3

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	10	5		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	600	600	600	0.99	1.02	1.07	0.06	N/A	N/A	N/A	Above Water Table
10	10	12	11	16	120	47	1.3	1.05	1.3	1.70	0.75	36.2	41.8	120	58	120	0.97	1.21	1.1	2.00	2.00	1.25	1.60	Nonliquefiable
14.5	12	17	14.5	17	120	86	1.3	1.05	1.27	1.07	0.85	26.8	32.3	1740	1459	1740	0.96	1.20	1.08	0.67	N/A	N/A	N/A	Non-Liq: PI>18
19.5	17	22	19.5	24	120	24	1.3	1.05	1.3	0.97	0.95	39.4	44.3	2340	1747	2340	0.93	1.21	1.05	2.00	2.00	0.77	2.58	Nonliquefiable
24.5	22	27	24.5	50	120		1.3	1.05	1.3	0.97	0.95	81.9	81.9	2940	2035	2940	0.91	1.21	1.01	2.00	2.00	0.81	2.46	Nonliquefiable
29.5	27	32	29.5	25	120	66	1.3	1.05	1.3	0.86	0.95	36.4	42.0	3540	2323	3540	0.88	1.21	0.97	2.00	2.00	0.83	2.40	Nonliquefiable
34.5	32	37	34.5	35	120		1.3	1.05	1.3	0.86	1	53.5	53.5	4140	2611	4140	0.85	1.21	0.94	2.00	2.00	0.84	2.38	Nonliquefiable
39.5	37	42	39.5	41	120		1.3	1.05	1.3	0.87	1	63.3	63.3	4740	2899	4740	0.83	1.21	0.9	2.00	2.00	0.84	2.39	Nonliquefiable
44.5	42	47	44.5	36	120		1.3	1.05	1.3	0.81	1	51.5	51.5	5340	3187	5340	0.80	1.21	0.88	2.00	2.00	0.83	2.41	Nonliquefiable
49.5	47	50	48.5	62	120		1.3	1.05	1.3	1.04	1	114.2	114.2	5820	3418	5820	0.78	1.21	0.86	2.00	2.00	0.82	2.44	Nonliquefiable

Notes:

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

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed C/I Building
Project Location	San Bernardino, CA
Project Number	16G167
Engineer	PM

Boring No. B-3

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	10	5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	10.00		0.000		0.00	Above Water Table
10	10	12	11	36.2	5.6	41.8	1.60	0.01	-0.94	0.01	2.00		0.000		0.00	Nonliquefiable
14.5	12	17	14.5	26.8	5.5	32.3	N/A	0.03	-0.24	0.00	5.00		0.000		0.00	Non-Liq: PI>18
19.5	17	22	19.5	39.4	5.0	44.3	2.58	0.00	-1.14	0.00	5.00		0.000		0.00	Nonliquefiable
24.5	22	27	24.5	81.9	0.0	81.9	2.46	0.00	-4.37	0.00	5.00		0.000		0.00	Nonliquefiable
29.5	27	32	29.5	36.4	5.6	42.0	2.40	0.01	-0.95	0.00	5.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	53.5	0.0	53.5	2.38	0.00	-1.88	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	63.3	0.0	63.3	2.39	0.00	-2.71	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	51.5	0.0	51.5	2.41	0.00	-1.71	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	114.2	0.0	114.2	2.44	0.00	-7.44	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				0.00	

Notes:

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

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

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

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

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

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

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

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

LIQUEFACTION EVALUATION

Project Name	Proposed C/I Building		MCE_G Design Acceleration	0.954 (g)
Project Location	San Bernardino, CA		Design Magnitude	7
Project Number	16G167		Historic High Depth to Groundwater	10 (ft)
Engineer	PM		Depth to Groundwater at Time of Drilling	80 (ft)
			Borehole Diameter	6 (in)
Boring No.	B-5			

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	10	5		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	600	600	600	0.99	1.02	1.07	0.06	N/A	N/A	N/A	Above Water Table
10	10	12	11	12	120	2	1.3	1.05	1.26	1.70	0.75	26.4	26.4	120	58	120	0.97	1.14	1.1	0.33	0.41	1.25	0.33	**Liquefiable**
14.5	12	17	14.5	11	120	84	1.3	1.05	1.16	1.09	0.85	16.1	21.6	1740	1459	1740	0.96	1.10	1.05	0.23	N/A	N/A	N/A	Non-Liq: PI >18
19.5	17	22	19.5	17	120	39	1.3	1.05	1.27	0.97	0.95	27.1	32.6	2340	1747	2340	0.93	1.21	1.04	0.71	0.89	0.77	1.15	**Liquefiable**
24.5	22	27	24.5	12	120	81	1.3	1.05	1.16	0.87	0.95	15.6	21.2	2940	2035	2940	0.91	1.10	1	0.22	N/A	N/A	N/A	Non-Liq: PI >18
29.5	27	32	29.5	13	120	86	1.3	1.05	1.16	0.80	0.95	15.6	21.1	3540	2323	3540	0.88	1.10	0.99	0.22	N/A	N/A	N/A	Non-Liq: PI >18
34.5	32	37	34.5	33	120		1.3	1.05	1.3	0.85	1	49.8	49.8	4140	2611	4140	0.85	1.21	0.94	2.00	2.00	0.84	2.38	Nonliquefiable
39.5	37	42	39.5	38	120		1.3	1.05	1.3	0.85	1	57.3	57.3	4740	2899	4740	0.83	1.21	0.9	2.00	2.00	0.84	2.39	Nonliquefiable
44.5	42	47	44.5	40	120		1.3	1.05	1.3	0.84	1	59.4	59.4	5340	3187	5340	0.80	1.21	0.88	2.00	2.00	0.83	2.41	Nonliquefiable
49.5	47	50	48.5	29	120		1.3	1.05	1.3	0.73	1	37.5	37.5	5820	3418	5820	0.78	1.21	0.86	1.96	2.00	0.82	2.44	Nonliquefiable

Notes:

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

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed C/I Building
Project Location	San Bernardino, CA
Project Number	16G167
Engineer	PM

Boring No. B-5

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	10	5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	10.00		0.000		0.00	Above Water Table
10	10	12	11	26.4	0.0	26.4	0.33	0.07	0.15	0.07	2.00		0.017		0.40	**Liquefiable**
14.5	12	17	14.5	16.1	5.5	21.6	N/A	0.13	0.43	0.00	5.00		0.000		0.00	Non-Liq: PI >18
19.5	17	22	19.5	27.1	5.6	32.6	1.15	0.03	-0.27	0.03	5.00		0.005		0.29	**Liquefiable**
24.5	22	27	24.5	15.6	5.5	21.2	N/A	0.14	0.46	0.00	5.00		0.000		0.00	Non-Liq: PI >18
29.5	27	32	29.5	15.6	5.5	21.1	N/A	0.14	0.46	0.00	5.00		0.000		0.00	Non-Liq: PI >18
34.5	32	37	34.5	49.8	0.0	49.8	2.38	0.00	-1.57	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	57.3	0.0	57.3	2.39	0.00	-2.19	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	59.4	0.0	59.4	2.41	0.00	-2.37	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	37.5	0.0	37.5	2.44	0.01	-0.61	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				0.69	

Notes:

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

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

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

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

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

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

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

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