GEOTECHNICAL INVESTIGATION PROPOSED COMMERCIAL/INDUSTRIAL DEVELOPMENT

SEC San Bernardino Avenue and Tippecanoe Avenue
San Bernardino, California
for
Rockefeller Group Development Corporation

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SOCALGEO
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

February 10, 2016

Rockefeller Group Development Corporation
4 Park Plaza, Suite 840
Irvine, California 92614

Attention: Mr. Michael M. Sajjadi
Vice President Design and Construction Western Division

Project No.: **16G106-1**

Subject: **Geotechnical Investigation**
Proposed Commercial/Industrial Development
SEC San Bernardino Avenue and Tippecanoe Avenue
San Bernardino, California

Gentlemen:

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

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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1.0 EXECUTIVE SUMMARY

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

Site Preparation

- Demolition of the existing structures and pavements will be necessary in order to facilitate the construction of the proposed development. Demolition should include all foundations, floor slabs, utilities and any other subsurface improvements that will not remain in place with the new development. Within the interior of the existing warehouse building (at Boring Nos. B-5 and B-6), the existing floor slab consists of a 6-inch thick reinforced concrete slab. The reinforcement consists of heavy-gauge welded wire mesh. At the exterior of the building, the existing asphaltic concrete sections observed at the boring locations consist of 2 to 3± inches of asphaltic concrete underlain by 4 to 5 inches of aggregate base. Concrete and asphalt debris may be crushed to a maximum 2-inch particle size, mixed with the on-site soils, and reused as compacted structural fill. It may also be feasible to crush these materials for use as crushed miscellaneous base (CMB).
- Initial site preparation should also include stripping of the surficial vegetation within the landscaped planter areas. Vegetation including grass, shrubs, trees, and any organic soils should be properly disposed of off-site. Root balls associated with the trees should be removed in their entirety, and the resultant excavations should be backfilled with compacted structural fill soils.
- The near-surface soils at this site consist of artificial fill and alluvium. The fill soils extend to depths of 1½ to 4½± feet and possess variable strengths and densities. The near-surface alluvial soils also possess variable strengths and densities and marginal consolidation characteristics. The fill soils and the near surface alluvial soils are not considered suitable in their present condition to support the proposed structure.
- Remedial grading is recommended to be performed within the proposed building area in order to remove all of the artificial fill soils and the upper portion of the alluvial soils. The existing soils within the proposed building areas should be overexcavated to a depth of 5 feet below existing grade and to a depth of at least 5 feet below proposed building pad subgrade elevation. The depth of overexcavation should also be sufficient to remove any existing fill soils and any soils disturbed during demolition.
- The proposed foundation influence zones should be overexcavated to a depth of 3 feet below proposed foundation bearing grade.
- Following evaluation of the subgrade by the geotechnical engineer, the exposed subgrade soils should be scarified, moisture conditioned to 2 to 4 percent above optimum, and recompacted. The resulting soils may be replaced as compacted structural fill.

Liquefaction

- Our site-specific liquefaction evaluation indicates that some of the on-site soils are subject to liquefaction during the design seismic event.
- The liquefaction analysis indicates total dynamic settlements on the order of 2.9 to 2.1± inches at Boring Nos. B-1 and B-4, respectively. The liquefaction-induced differential

settlements within the building area are expected to be on the order of 1½± inches. Assuming that this settlement occurs across a distance of 100± feet, a maximum angular distortion of less than 0.002± inches per inch would result.

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

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,500 lbs/ft^2 maximum allowable soil bearing pressure.
- Reinforcement consisting of at least six (6) No. 5 rebars (3 top and 3 bottom) in strip footings due to the presence of potentially liquefiable soils. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade, 6 inches thick.
- Minimum reinforcement of the floor slab should consist of No. 3 bars at 18-inches on center in both directions, due to the presence of potentially liquefiable soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed slab loading.
- Modules of Subgrade Reaction: 100 psi/in

Pavements

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)**		
	Autos Parking 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. 16P102, dated January 6, 2016. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. Based on the location of the subject site, this investigation also included a site specific liquefaction evaluation. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the southeast corner of Tippecanoe Avenue and San Bernardino Avenue in San Bernardino, California. The site is bounded to the north by San Bernardino Avenue, to the east by a railroad easement and a warehouse building, to the south by Victoria Avenue, and to the west by Tippecanoe Avenue. The general location of the site is illustrated on the Site Location Map included as Plate 1 in Appendix A of this report.

The subject site consists of a nearly rectangular-shaped parcel, 19.2± acres in size. The site is currently developed with a warehouse building located in the south-central region of the site. Based on the ALTA survey that was provided to our office, the building is 213,375± ft^2 in size and is of metal-frame and concrete construction. The warehouse building is currently occupied by Loma Linda University and is being utilized for storage. The exterior of the site is being utilized as trailer-truck storage. The north half and southwest portion of the building is dock-high, with a concrete floor-slab 4± feet higher in elevation than the exterior pavements. The southern portion of the building is constructed at level grade with the exterior pavements. Loading docks are located along the north, west and southeast sides of the building. The pavement areas surrounding the building consist of asphaltic concrete pavements in the automobile parking and drive areas and Portland cement concrete (PCC) pavements in the loading dock areas. The pavements are in poor condition with moderate to severe cracking throughout. Landscaped planters with several large trees are located along the southern property line.

Topographical information for the subject site was obtained from the conceptual site plan prepared by Thienes Engineering, Inc., the project civil engineer. This plan indicates that the site grades range from elevations of 1064± feet mean sea level (msl) in the southeast portion of the site to an elevation of 1052± feet msl in the western portion of the site. The overall site topography generally slopes downward to the north at a gradient of approximately 1± percent.

3.2 Proposed Development

Our office was provided with an architectural site plan, identified as Scheme 5, which was prepared by HPA. It should be noted that the boring locations were based on a previous site plan. Therefore several of the borings are not located within the proposed building footprint. However, this does not affect our design recommendations for the subject site as a comprehensive understanding of the subsurface profile was achieved and sufficient data was obtained during our site investigation.

Based on our review of this plan, the site will be developed with two (2) commercial/industrial buildings. The buildings, identified as Buildings 1 and 2, will be 81,730± ft^2 and 333,170± ft^2 in size, respectively. Building 1 will be constructed in the south-central region of the site. Loading docks will be located on the north and south sides of Building 1. Building 2 will be located in the north-central region of the site, with loading docks located along the north side. The buildings will be surrounded by asphaltic concrete pavements in the parking and drive lanes and PCC pavements in the loading dock areas.

Detailed structural information has not been provided. It is assumed that the new buildings will be single-story structures of concrete tilt-up construction, supported on conventional shallow foundations with concrete slab-on-grade floors. 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.

No significant amounts of below grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. Based on the conceptual site plan, cuts of less than 1 foot and fills on the order of 1 to 6± feet will be necessary to achieve the proposed building pad grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of eight (8) borings advanced to depths of 4½ to 50± feet below currently existing site grades. Two (2) of these borings were drilled to depths of 50± feet to evaluate the liquefaction potential of the on-site soils. Four (4) additional borings were drilled in the building areas to depths of 15 to 25± feet. These borings were performed using a conventional truck-mounted drilling rig, equipped with standard hollow stem augers. The remaining two (2) borings were drilled within the interior of the existing warehouse building to depths of 4½ and 10± feet below existing finish floor elevation. Boring No. B-5 was terminated at a depth shallower than originally planned due to a possible utility conduit which was encountered at a depth of 4½± feet. Due to access limitations, these borings were drilled using manually operated hand equipment. The existing concrete floor-slab was cored using a portable concrete coring rig equipped with a 5-inch diameter diamond tipped core barrel. All of the borings were logged during drilling by a member of our staff.

Except for the borings drilled within the existing warehouse buildings, all of 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 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. 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

Concrete Floor Slab

Boring Nos. B-5 and B-6 were drilled within the interior of the existing warehouse building through the existing PCC building slab. The existing slab section at these borings consists of 6 inches of PCC. No significant layer of underlying aggregate base or sand was observed at either of the boring locations. Wire mesh was observed within the concrete cores. The spacing of the mesh could not be determined from our cores, but appeared to be heavy gauge.

Pavements

Asphaltic concrete pavements were encountered at the ground surface at all of the remaining boring locations. The pavements consist of 2 to 3± inches of asphaltic concrete underlain by 4 to 5± inches of aggregate base.

Artificial Fill

Artificial fill soils were encountered beneath the existing floor-slab and pavements at most of the boring locations, with the exception of Boring Nos. B-2 and B-7. These fill soils extend to depths of 1½ to 4½± feet below the existing site grades and consist of loose to medium dense silty fine sands and fine sandy silts, with varying medium to coarse sand and clay content. The fill soils possess a disturbed appearance, resulting in their classification as artificial fill. Boring No. B-5 was terminated within the fill soils due to a possible utility conduit which was encountered at a depth of 4½± feet below the existing floor-slab.

Alluvium

Native alluvium was encountered beneath the fill soils or beneath the pavements at all boring locations, with the exception of Boring No. B-5 which was terminated within the fill. The alluvium generally consists of loose to dense silty fine sands, fine sands, and fine sandy silts and medium stiff to very stiff clayey silts and silty clays, with varying amounts medium to coarse sand, extending to at least the maximum depth explored of 50± feet.

Groundwater

Groundwater was not encountered at any of the borings. 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 below existing site grades, at the time of the subsurface investigation.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the historic groundwater depths in this area is Plate 2 of the U.S.G.S. Bulletin 1898: Liquefaction Susceptibility in the San Bernardino Valley and Vicinity, Southern California, which indicates that the historic high groundwater level for the site was 15 feet below the ground surface. More recent water level data was obtained from the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. The nearest monitoring well is located approximately 3000 feet southeast from the site. Water level readings within this monitoring well indicates high groundwater levels of 216± feet (December 2015) below the ground surface. Therefore, the high groundwater depth of 15 feet reported in Bulletin 1898 is considered to be conservative with respect to the recent site conditions.

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

Consolidation

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

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	0.002	Negligible
B-4 @ 0 to 5 feet	0.002	Negligible

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample was tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil type or soil mixes may be necessary at a later date. The results of the testing are plotted on Plate C-9 in Appendix C of this report.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations. The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record. The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

6.1 Seismic Design Considerations

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

Faulting and Seismicity

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

Seismic Design Parameters

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	2.176
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.987
Site Class	---	F*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	2.176
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.481
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.451
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.987

*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. <u>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.</u> 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.

<u>Ground Motion Parameters</u>

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>U.S. Seismic Design Maps</u> (described in the previous section) was used to determine PGA_M, which is 0.840g. A portion of the program output is included as Plate 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 6.95, based on the peak ground acceleration and NEHRP soil classification D.

<u>Liquefaction</u>

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 <u>San Bernardino County Official Land Use Plan, General Plan, Geologic Hazard Overlay</u>. Map FH30C for the San Bernardino South Quadrangle indicates that the subject site is located within a zone of high liquefaction susceptibility. Therefore, the scope of this geotechnical investigation was expanded to include a site-specific liquefaction evaluation.

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.

As part of the liquefaction evaluation, Boring Nos. B-1 and B-4 were extended to depths of 50± feet. The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report, using the data obtained from these borings. The liquefaction potential of the site was analyzed utilizing a PGA_M of 0.840g for a magnitude 6.95 seismic event.

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 15 feet. Therefore, the historic high groundwater table was considered to be 15 feet for the liquefaction evaluation.

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

Conclusions and Recommendations

The results of the liquefaction analysis have identified potentially liquefiable soils at the site. Several potentially liquefiable strata are located at various depths between 15 and 42± feet at both of the 50± foot deep borings. Soils which are located above the historic groundwater table, or possess factors of safety in excess of 1.3 are considered non-liquefiable. The silty clay and clayey silt strata encountered at Borings Nos. B-1 and B-4 are considered 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). Settlement analyses were conducted for each of the potentially liquefiable strata.

Based on the settlement analysis (also tabulated on the spreadsheets in Appendix F) total dynamic (liquefaction induced) settlements of 2.91 and 2.07± inches are expected at Boring Nos. B-1 and B-4, respectively, during the design level earthquake. Based on these total settlements, differential settlements of up to 1½± inches should be expected to occur during a liquefaction inducing seismic event. The estimated differential settlement could be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of less than 0.002 inches per inch. These settlements are considered to be within the structural tolerances of a typical building supported on a shallow foundation system. However, it should be noted that minor to moderate repairs, including repair of damaged drywall and stucco, etc., could be required after the occurrence of liquefaction-induced settlements.

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

6.2 Geotechnical Design Considerations

General

The near surface soils consist of artificial fill materials and native alluvial soils. The artificial fill materials extend to depths of 1½ to 4½± feet at several of the boring locations. No reports documenting the placement and compaction of these materials were provided to our office. Based on the apparent age of the existing development and the lack of any documentation, the existing fill materials at the site are considered to be undocumented fill soils. The near-surface alluvium possesses variable strengths and densities, and marginal consolidation characteristics. Based on these conditions, remedial grading is considered warranted within the proposed building areas in order to remove the artificial fill in its entirety and replace the upper portion of variable strength alluvium as compacted structural fill.

The existing building will be demolished in order to allow for construction of the new development. The demolition of the existing structure, including foundations, utilities, and any other subsurface improvements is expected to cause significant disturbance to the near surface soils. Therefore, the recommended remedial grading should also remove any soils disturbed during demolition, prior to the placement of any compacted fill materials.

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 the undocumented fill soils and a portion of the near-surface, compressible, variable density alluvial soils from the proposed building pad areas. These materials will be replaced as compacted structural fill. The native soils that will remain in place below the recommended depth of overexcavation generally possess more favorable consolidation characteristics and will not be subject to significant load increases from the foundations of the new structures. Provided that the recommended remedial grading is completed, the post-construction static settlement of the proposed structures is expected to be within tolerable limits.

Expansion

The near-surface soils present within the upper 5± feet below the existing site grades generally consist of silty sands and fine sandy silts, with trace clay content. Based on their composition, these soils are considered to be very low to non-expansive. Therefore, no design considerations related to expansive soils are considered warranted for this site. All imported fill soils should also possess very low expansive characteristics.

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 artificial fill and near-surface native soils is estimated to result in an average shrinkage of 10 to 14 percent. Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1± feet. This estimate may be used for grading in areas that are underlain by native alluvial soils.

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

Grading and Foundation Plan Review

Grading and foundation plans were not available at the time of this report. It is therefore recommended that we be provided with copies of the preliminary grading and foundation plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

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

Site Stripping and Demolition

Demolition of the existing structure and pavements will be necessary in order to facilitate the construction of the proposed development. Demolition should include all foundations, floor slabs, utilities and any other subsurface improvements that will not remain in place with the new development. Concrete and asphalt debris may be crushed to a maximum 2-inch particle size, mixed with the on-site soils, and reused as compacted structural fill. It may also be feasible to crush these materials for use as crushed miscellaneous base (CMB).

Initial site stripping should include removal of any surficial vegetation from the planter areas of the site. This should include any weeds, grasses, shrubs, and trees. Root balls associated with the trees should be removed in their entirety, and the resultant excavation should be backfilled with compacted structural fill soils. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pads

Remedial grading should be performed within the proposed building areas in order to remove the artificial fill materials, the upper portion of the alluvial soils, and any soils disturbed during the demolition of the existing site improvements. Based on conditions encountered at the boring locations, the existing soils within the proposed building areas are recommended to be overexcavated to a depth of at least 5 feet below the proposed building pad subgrade elevation and to a depth of at least 5 feet below existing grade, whichever is greater. The depth of the overexcavation should also extend to a depth sufficient to remove all artificial fill soils and any soils disturbed during demolition. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building and foundation perimeters, and to an extent equal to the depth of fill below the new foundations. If the proposed structures incorporate any exterior columns (such as for a canopy or overhang) the overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the building areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structures. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or loose, porous, or low density native soils are encountered at the base of the overexcavation.

Based on conditions encountered at the exploratory boring locations, some zones of moist to very moist clayey silts and silty clays may be encountered at or near the base of the recommended overexcavation. Where these soils are exposed at the overexcavation subgrade level, some subgrade stabilization may be required. Scarification and significant air drying of these materials may be sufficient to obtain a stable subgrade. If highly unstable soils are identified, and if the construction schedule does not allow for delays associated with drying, mechanical stabilization will be necessary. In this event, the geotechnical engineer should be contacted for supplementary recommendations. Typically, an unstable subgrade can be stabilized using a suitable geotextile fabric, such as Mirafi RS380i and/or an 18-inch thick layer of coarse (2 to 4 inch particle size) crushed stone.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, moisture treated to 2 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining and site walls should be overexcavated to a depth of 2 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pad. Any undocumented fill soils within any of these foundation areas should be removed in their entirety. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils, as discussed for the building areas. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Parking and Drive Areas

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

The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. Any such materials should be removed to a level of firm and unyielding soil. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to at least 2 to 4 percent above the optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

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

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

Moisture Sensitive Subgrade Soils

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

If the construction schedule dictates that site grading will occur during a period of wet weather, allowances should be made for costs and delays associated with drying the on-site soils or import of a drier, less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pad areas as well as the need for and/or the thickness of the crushed stone stabilization layer, discussed in Section 6.3 of this report.

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 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pads will be underlain by structural fill soils used to replace existing fill and near-surface alluvial soils. These new structural fill soils are expected to extend to depths of at least 3 feet below proposed foundation bearing grades, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, the proposed structures may be supported on shallow foundations.

<u>Foundation Design Parameters</u>

New square and rectangular footings may be designed as follows:

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

- Minimum wall/column footing width: 14 inches/24 inches.

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

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

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

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

<u>Foundation Construction</u>

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

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

Estimated Foundation Settlements

Post-construction total and differential settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch. These settlements are in addition to the liquefaction-induced settlements previously discussed in Section 6.1 of this report. However, the likelihood of these two settlements combining is considered remote. The static settlements are expected to occur in a relatively short period of time after the building loads being applied to the foundations, during and immediately subsequent to construction. It should be noted that the projected potential dynamic settlement is related to a major seismic event and a conservative historic high groundwater level.

Lateral Load Resistance

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

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

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

6.6 Floor Slab Design and Construction

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

- Minimum slab thickness: 6 inches.

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

- Minimum slab reinforcement: Minimum slab reinforcement: No. 3 bars at 18 inches on-center, in both directions, due to the liquefaction potential of the encountered soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed loading.

- Slab underlayment: If moisture sensitive floor coverings will be used the minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area where such moisture sensitive floor coverings are anticipated. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview.

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

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

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement. The steel reinforcement recommendations presented above are based on standard geotechnical practice, given the magnitude of predicted liquefaction-induced settlements, and the structure type proposed for the site. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements discussed in Section 6.1.

6.7 Retaining Wall Design and Construction

New retaining walls are expected to be necessary in the truck court areas. Additionally, although not indicated on the site plan, the proposed development may require some small retaining walls (less than 5± feet in height) to facilitate the new site grades and in loading dock areas.

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 silty fine sands and fine sandy silts. Based on their classifications, these 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 Silty Sands and Sandy Silts
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

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

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

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

Seismic Lateral Earth Pressures

In 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, extending to a depth of at least 2 feet below the proposed bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

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

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

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557). 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 Pavement Design Parameters

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

Pavement Subgrades

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

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)**		
	Autos Parking 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.

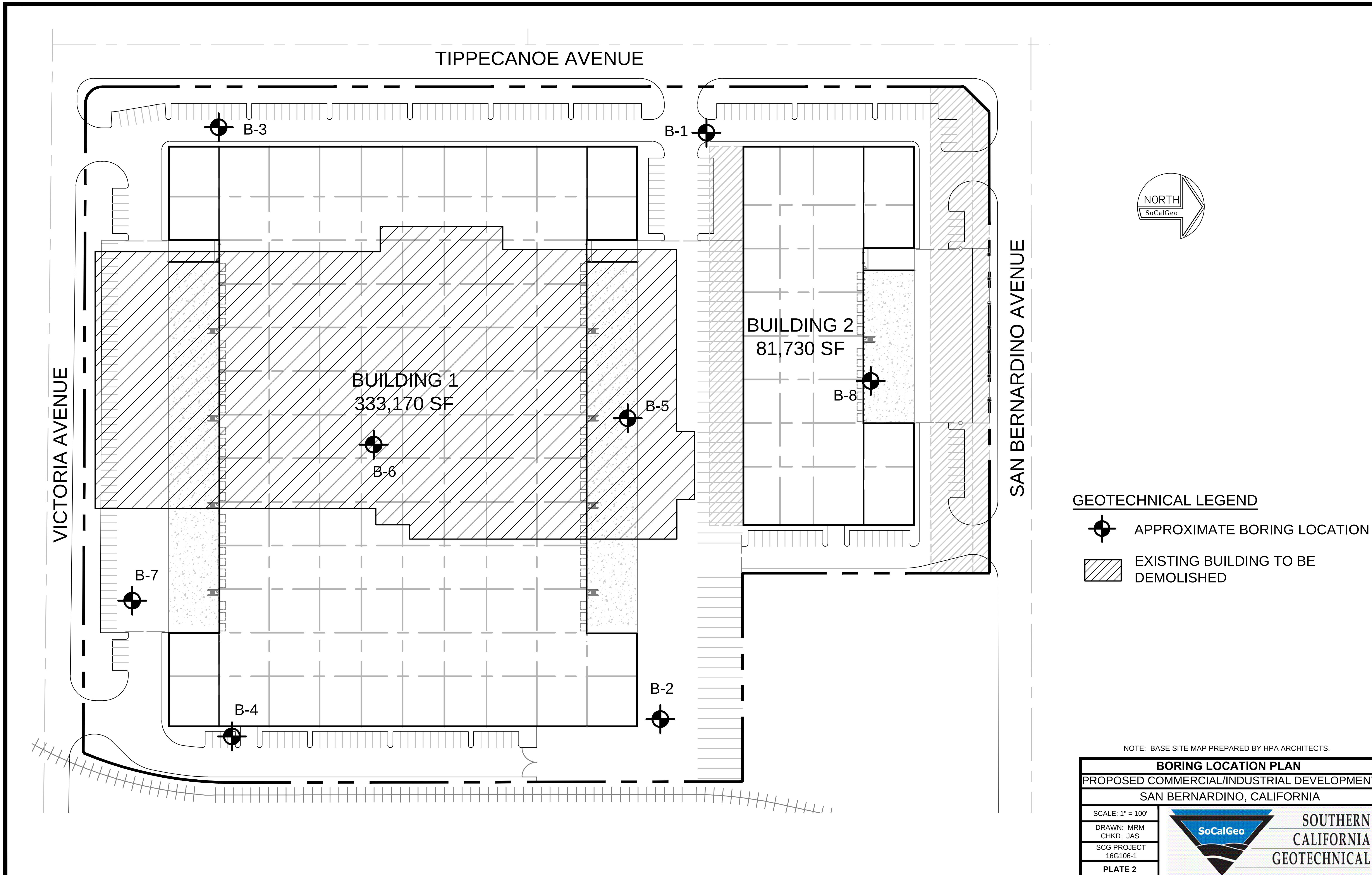
Tokimatsu, K. and Yoshimi, Y., "*Empirical Correlations of Soil Liquefaction Based on SPT N-value and Fines Content,"* Seismological Research Letters, Eastern Section Seismological Society Of America, Volume 63, Number 1, p. 73.

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

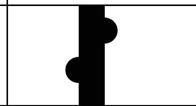
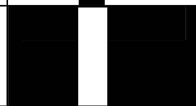
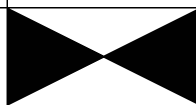
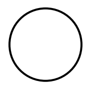
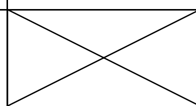
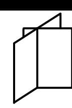
SOURCE: SAN BERNARDINO COUNTY
THOMAS GUIDE, 2013

SITE LOCATION MAP	
PROPOSED COMMERCIAL/INDUSTRIAL DEVELOPMENT	
SAN BERNARDINO, CALIFORNIA	
SCALE: 1" = 2400'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JL CHKD: JAS	
SCG PROJECT 16G106-1	
PLATE 1	


TIPPECANOE AVENUE
VICTORIA AVENUE
SAN BERNARDINO AVENUE
NORTH
SoCalGeo
BUILDING 1
333,170 SF
BUILDING 2
81,730 SF
B-1
B-2
B-3
B-4
B-5
B-6
B-7
B-8
GEOTECHNICAL LEGEND
APPROXIMATE BORING LOCATION
EXISTING BUILDING TO BE DEMOLISHED
NOTE: BASE SITE MAP PREPARED BY HPA ARCHITECTS.
BORING LOCATION PLAN
PROPOSED COMMERCIAL/INDUSTRIAL DEVELOPMENT
SAN BERNARDINO, CALIFORNIA
SCALE: 1" = 100'
DRAWN: MRM
CHKD: JAS
SCG PROJECT
16G106-1
PLATE 2
SoCalGeo
SOUTHERN
CALIFORNIA
GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

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

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

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

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

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

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

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

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

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

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BORING NO. B-1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 16G106
PROJECT: Proposed C/I Development
LOCATION: San Bernardino, California

DRILLING DATE: 1/19/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					3± inches Asphaltic concrete, 5± inches Aggregate base							
	X	14			FILL: Dark Gray fine Sandy Silt, trace to little Clay, trace medium Sand, mottled, medium dense-very moist		21					
	X	12			ALLUVIUM: Light Gray Brown Silty fine Sand, medium dense-moist		11					
5												
	X	11			Light Gray Brown fine Sand, trace Silt, medium dense-damp		5					
					Light Gray Brown Silty fine Sand, trace medium Sand, medium dense-very moist		16					
	X	14			Light Gray fine Sand, little Silt, medium dense-moist		8					
10												
	X	7	2.5		Gray Clayey Silt, trace fine Sand, trace caclareous nodules, trace Iron oxide staining, medium stiff-very moist		34					
					Light Gray Silty fine Sand, trace medium Sand, loose-moist		14			46		
15												
	X	11			Gray Brown Clayey Silt, trace Iron oxide staining, stiff-very moist		34	48	32	90		
					Gray Brown Silty fine Sand to fine Sandy Silt, trace Clay, trace Iron oxide staining, medium dense-very moist		18			51		
20												
	X	16			Light Gray Brown Silty fine Sand, medium dense-moist		12			49		
					Gray Brown Silt, litlte fine Sand, little Clay, trace Iron oxide staining, medium dense to very stiff-very moist		30			88		
25												
	X	9			Gray Silty fine Sand, loose-moist to very moist		16			38		
			1.5		Gray Clayey Silt, trace fine Sand, trace Iron oxide staining, stiff-very moist		36			93		
30												
	X	12	3.0				43	48	29	94		

TBL 16G106.GPJ SOCALGEO.GDT 2/11/16

JOB NO.: 16G106	DRILLING DATE: 1/19/16	WATER DEPTH: Dry
PROJECT: Proposed C/I Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 38 feet
LOCATION: San Bernardino, California	LOGGED BY: Matt Manni	READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	(Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Gray Clayey Silt, trace fine Sand, trace Iron oxide staining, stiff-very moist							
					Light Gray Brown Silty fine Sand, trace Iron oxide staining, medium dense-moist							
	X	29					14					
40												
					Gray Silty fine Sand, dense-moist							
	X	32					11					
45												
	X	32					13					
50												
					Boring Terminated at 50'							

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 16G106	DRILLING DATE: 1/19/16	WATER DEPTH: Dry
PROJECT: Proposed C/I Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 13 feet
LOCATION: San Bernardino, California	LOGGED BY: Matt Manni	READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
					2± inches Asphaltic concrete, 4± inches Aggregate base							
	X	15			ALLUVIUM: Gray Brown Silty fine Sand, trace Clay, trace medium Sand, slightly porous, trace calcareous veining, medium dense-very moist	104	15					
	X	15			Gray Brown Silty fine Sand, trace medium Sand, slightly porous, trace calcareous nodules, loose to medium dense-moist	98	11					
5	X	14				111	10					
	X	11	3.5		Light Gray Brown Clayey Silt interbedded with fine Sandy Silt, trace Iron oxide staining, slightly porous, medium stiff to loose-moist	98	19					
	X	15			Light Gray Brown fine Sand, little Silt, trace Iron oxide staining, medium dense-damp	93	7					
10												
					Gray Brown fine Sandy Silt, trace Iron oxide staining, loose-moist to very moist							
	X	14				95	17					
15												
					Gray Brown Clayey Silt, abundant Iron oxide staining, slightly porous, stiff-very moist							
	X	14	3.0			82	37					
20												

Boring Terminated at 20'

TBL 16G106.GPJ SOCALGEO.GDT 2/11/16

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 16G106
PROJECT: Proposed C/I Development
LOCATION: San Bernardino, California

DRILLING DATE: 1/19/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					2± inches Asphaltic concrete, 4± inches Aggregate base							
		9			FILL: Gray Brown fine Sandy Silt, little Clay, trace medium to coarse Sand, mottled, loose-very moist	93	24					
					ALLUVIUM: Light Gray Brown Silty fine Sand, trace calcareous nodules, slightly porous, loose-damp to moist							
		25			Light Gray Brown Silty fine to medium Sand, little coarse Sand, medium dense-damp to moist	110	8					
5		19				104	7					
		13			Gray Brown fine Sandy Silt, trace Clay, trace Iron oxide staining, loose-moist to very moist	101	18					
			4.5		Gray Brown Clayey Silt, abundant Iron oxide staining, stiff-very moist							
		27			Gray Brown Silty fine Sand, trace Iron oxide staining, medium dense-moist	103	12					
10					Light Gray Brown fine Sand, trace Silt, trace medium Sand, medium dense-moist							
					Gray Brown Clayey Silt, trace Iron oxide staining, slightly porous, medium stiff-very moist							
15		11				89	36					
20		4			@ 19 to 20 feet, soft	73	47					
					Gray Silty fine Sand, trace Iron oxide staining, medium dense-damp							
25		21				99	7					
					Boring Terminated at 25'							

TBL 16G106.GPJ SOCALGEO.GDT 2/11/16

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 16G106 | DRILLING DATE: 1/19/16 | WATER DEPTH: Dry
PROJECT: Proposed C/I Development | DRILLING METHOD: Hollow Stem Auger | CAVE DEPTH: 31 feet
LOCATION: San Bernardino, California | LOGGED BY: Matt Manni | READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					3± inches Asphaltic concrete, 4± inches Aggregate base							
	X	4			FILL: Gray Brown Silty fine Sand, loose-very moist		18					
	X	8			ALLUVIUM: Light Gray Brown fine Sand, trace to little Silt, trace medium Sand, loose-damp to moist		8					
5												
	X	13			@ 6 to 10 feet, trace coarse Sand, medium dense		5					
	X	16					3					
10												
					Gray Brown Silty Clay, trace fine Sand, trace Iron oxide staining, slightly porous, medium stiff to stiff-very moist							
	X	8	1.0				34					
15					Gray Brown Silty fine Sand to fine Sandy Silt, trace Clay, trace Iron oxide staining, trace medium Sand, loose-moist to very moist		15			44		
					Gray Brown Clayey Silt, trace fine Sand, trace Iron oxide staining, trace calcareous nodules, medium stiff-very moist							
	X	7	3.0				32	47	30	81		
20												
					Light Gray Brown Silty fine Sand, trace Iron oxide staining, medium dense-damp to moist							
	X	16					10			33		
25												
	X	22					6			18		
30												
					Gray Silty fine Sand interbedded with Gray Clayey Silt, medium dense-moist							
	X	21					16			79		

TBL 16G106.GPJ SOCALGEO.GDT 2/11/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 16G106 | DRILLING DATE: 1/19/16 | WATER DEPTH: Dry
PROJECT: Proposed C/I Development | DRILLING METHOD: Hollow Stem Auger | CAVE DEPTH: 31 feet
LOCATION: San Bernardino, California | LOGGED BY: Matt Manni | READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Gray Silty fine Sand interbedded with Gray Clayey Silt, medium dense-moist							
	X	19			Gray Brown fine Sandy Silt, trace Iron oxide staining, medium dense-very moist		27			80		
40					Gray Brown Silty fine Sand, trace Iron oxide staining, medium dense-damp		7			27		
45	X	35			Gray Brown Silty fine to medium Sand, trace coarse Sand, dense-damp		8					
50	X	16	2.0		Gray Clayey Silt, little fine Sand, trace Iron oxide staining, very stiff-very moist		28	40	26	84		

Boring Terminated at 50'

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 16G106	DRILLING DATE: 1/19/16	WATER DEPTH: Dry
PROJECT: Proposed C/I Development	DRILLING METHOD: Hand Auger	CAVE DEPTH: 3 feet
LOCATION: San Bernardino, California	LOGGED BY: Joseph Lozano Leon	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					6± inches Portland cement concrete, no discernible Aggregate base							
	X				FILL: Gray Brown Silty fine Sand, trace Clay, mottled, loose to medium dense-moist	105	10					
	X				FILL: Gray Brown fine Sandy Silt, mottled, loose-moist	100	15					

Boring Terminated at 4½' due to refusal on possible Utility Conduit

TBL 16G106.GPJ SOCALGEO.GDT 2/11/16

TEST BORING LOG **PLATE B-5**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 16G106
PROJECT: Proposed C/I Development
LOCATION: San Bernardino, California

DRILLING DATE: 1/19/16
DRILLING METHOD: Hand Auger
LOGGED BY: Joseph Lozano Leon

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					6± inches Portland cement concrete, no discernible Aggregate base							
	X				FILL: Dark Gray Brown Silty fine Sand to fine Sandy Silt, mottled, medium dense-moist	110	10					
	X					101	12					
5	X				ALLUVIUM: Gray Silty fine Sand, trace medium to coarse Sand, medium dense-damp	92	7					
					Gray fine Sand, little Silt, medium dense-dry to damp							
	X				Gray fine Sandy Silt to Silty fine Sand, medium dense-dry to damp	111	3					
	X				Gray Brown Silty fine Sand, medium dense-moist	105	10					
10												

Boring Terminated at 10'

TBL 16G106.GPJ SOCALGEO.GDT 2/11/16

TEST BORING LOG

BORING NO. B-7

JOB NO.: 16G106
PROJECT: Proposed C/I Development
LOCATION: San Bernardino, California

DRILLING DATE: 1/19/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					3± inches Asphaltic concrete, 4± inches Aggregate base							
	X	10			ALLUVIUM: Gray Brown Silty fine Sand, trace medium to coarse Sand, trace calcareous nodules, slightly porous, loose to medium dense-moist		13					
	X	9			Light Gray Brown Silty fine to medium Sand, trace coarse Sand, loose-damp to moist		8					
5												
	X	8			Light Gray Brown Silty fine Sand, trace medium Sand, trace Clay, trace Iron oxide staining, loose to medium dense-moist to very moist		19					
	X	14					11					
10												
					Light Gray Brown Clayey Silt, lilttle fine Sand, trace Iron oxide staining, stiff-very moist							
	X	14					34					
15												

Boring Terminated at 15'

BORING NO. B-8

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 16G106
PROJECT: Proposed C/I Development
LOCATION: San Bernardino, California

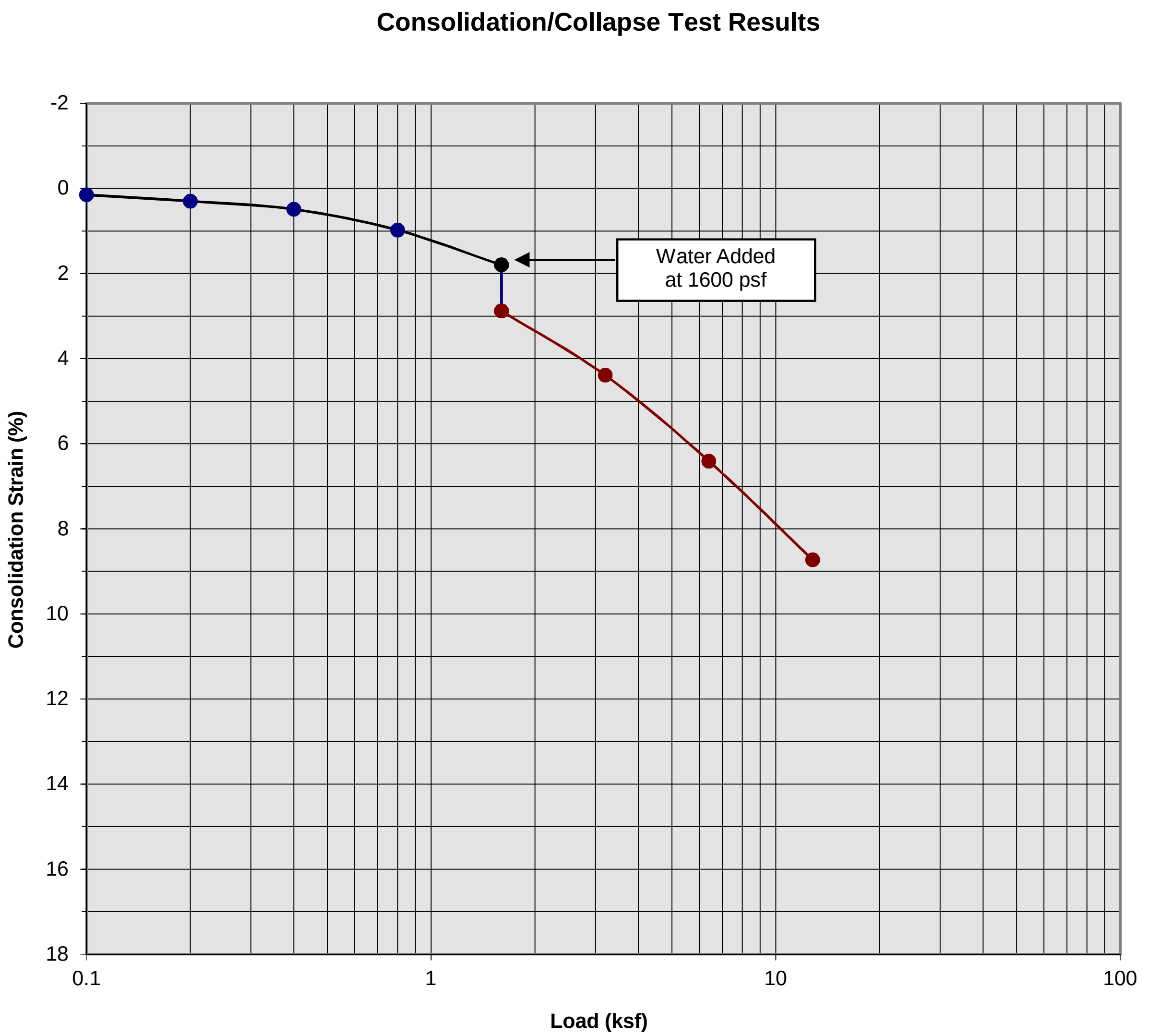
DRILLING DATE: 1/19/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

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

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)		SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
					3± inches Asphaltic concrete, 5± inches Aggregate base							
	X	10			FILL: Gray Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, mottled, loose to medium dense-moist		12					
	X	12			ALLUVIUM: Light Gray Brown Silty fine Sand, trace medium Sand, trace Iron oxide staining, medium dense-damp to moist		14					
5												
	X	12					6					
	X	7			Gray to Gray Brown Clayey Silt, trace fine Sand, trace Iron oxide staining, trace calcareous nodules, medium stiff-very moist		30					
10												
					Gray Brown Silty fine Sand, trace medium Sand, medium dense-moist							
	X	15					9					
15												
					Boring Terminated at 15'							

TBL 16G106.GPJ SOCALGEO.GDT 2/11/16

APPENDIX C



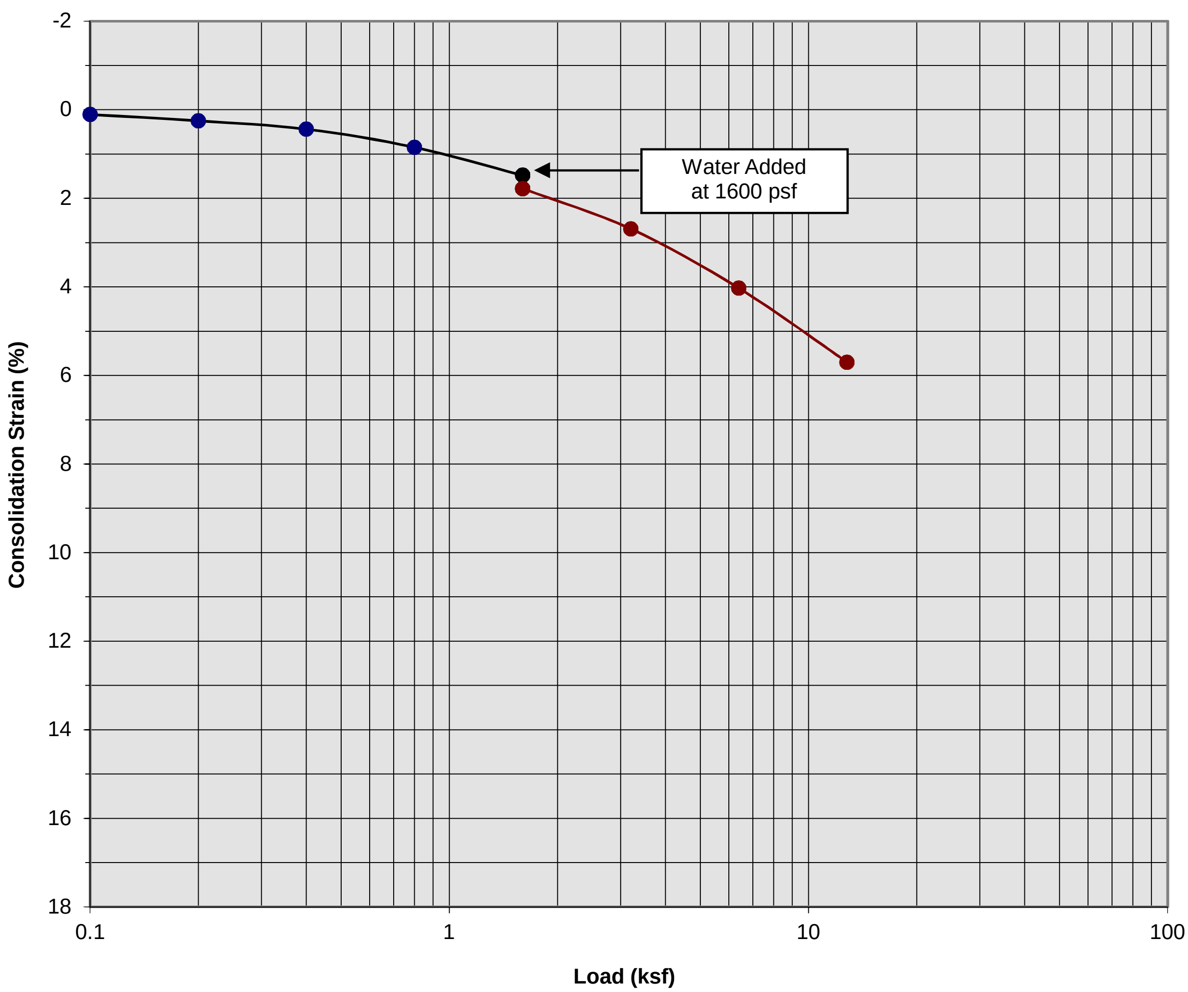
Classification: Gray Brown Silty fine Sand, trace Clay, trace medium Sand

Boring Number:	B-2	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	1 to 2	Initial Dry Density (pcf)	103.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	113.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.08

Proposed C/I Development
San Bernardino, California
Project No. 16G106
PLATE C- 1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



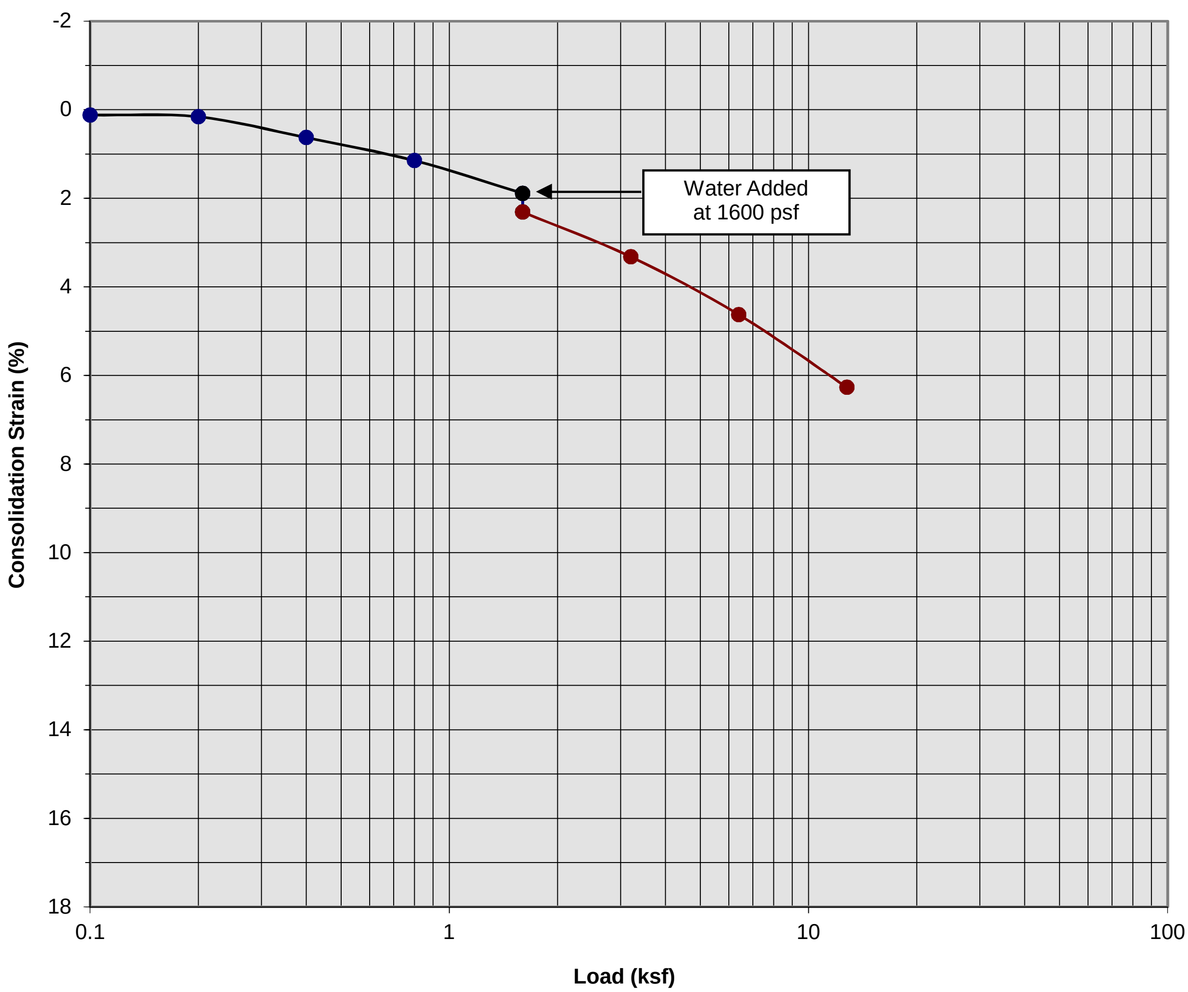
Classification: Gray Brown Silty fine Sand, trace medium Sand

Boring Number:	B-2	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	3 to 4	Initial Dry Density (pcf)	97.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	103.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.30

Proposed C/I Development
San Bernardino, California
Project No. 16G106
PLATE C- 2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

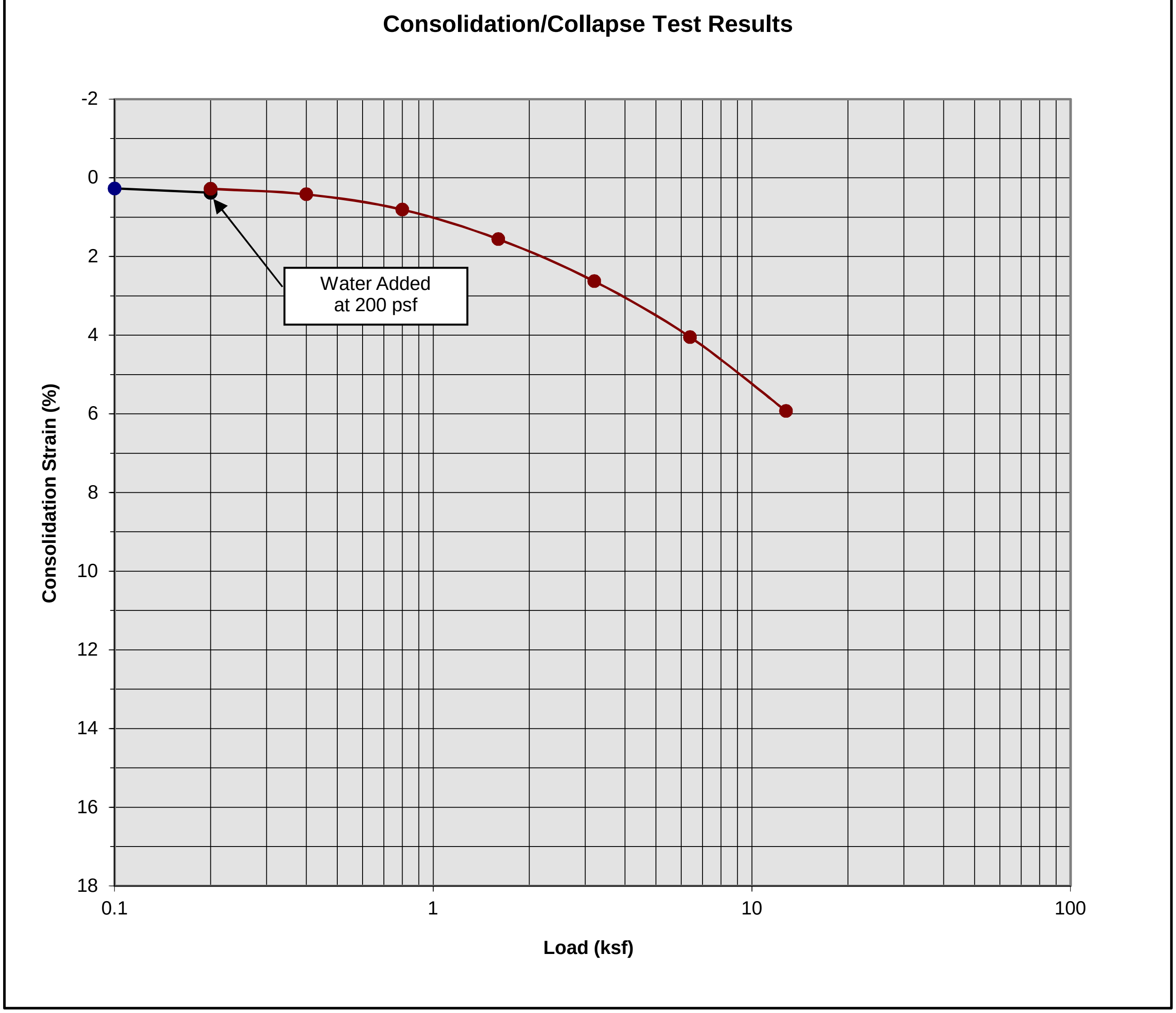


Classification: Gray Brown Silty fine Sand, trace medium Sand

Boring Number:	B-2	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	5 to 6	Initial Dry Density (pcf)	110.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	117.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.42

Proposed C/I Development
San Bernardino, California
Project No. 16G106
PLATE C- 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



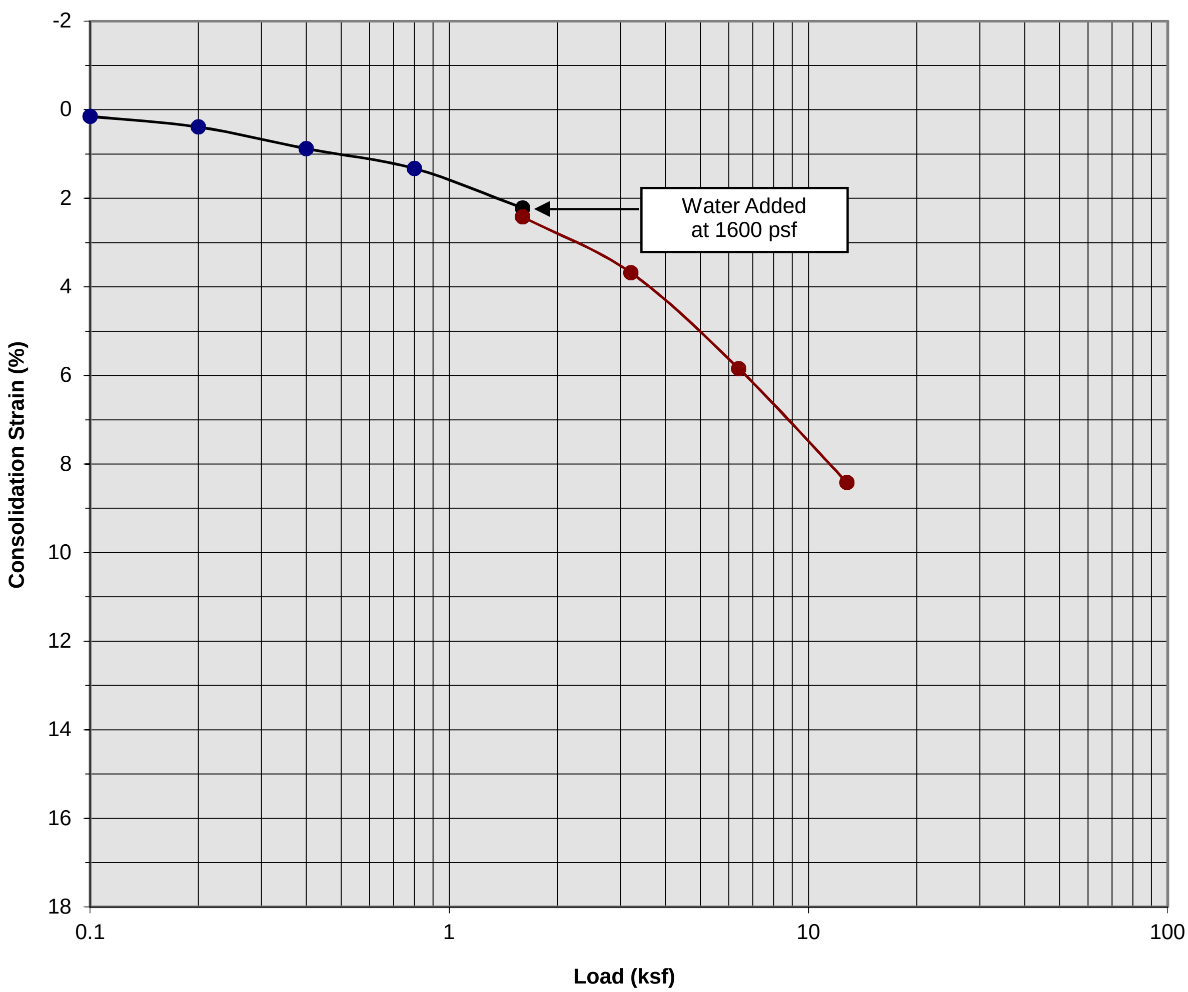
Classification: Light Gray Brown Clayey Silt to fine Sandy Silt

Boring Number:	B-2	Initial Moisture Content (%)	19
Sample Number:	---	Final Moisture Content (%)	25
Depth (ft)	7 to 8	Initial Dry Density (pcf)	97.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	103.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.10

Proposed C/I Development
San Bernardino, California
Project No. 16G106
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



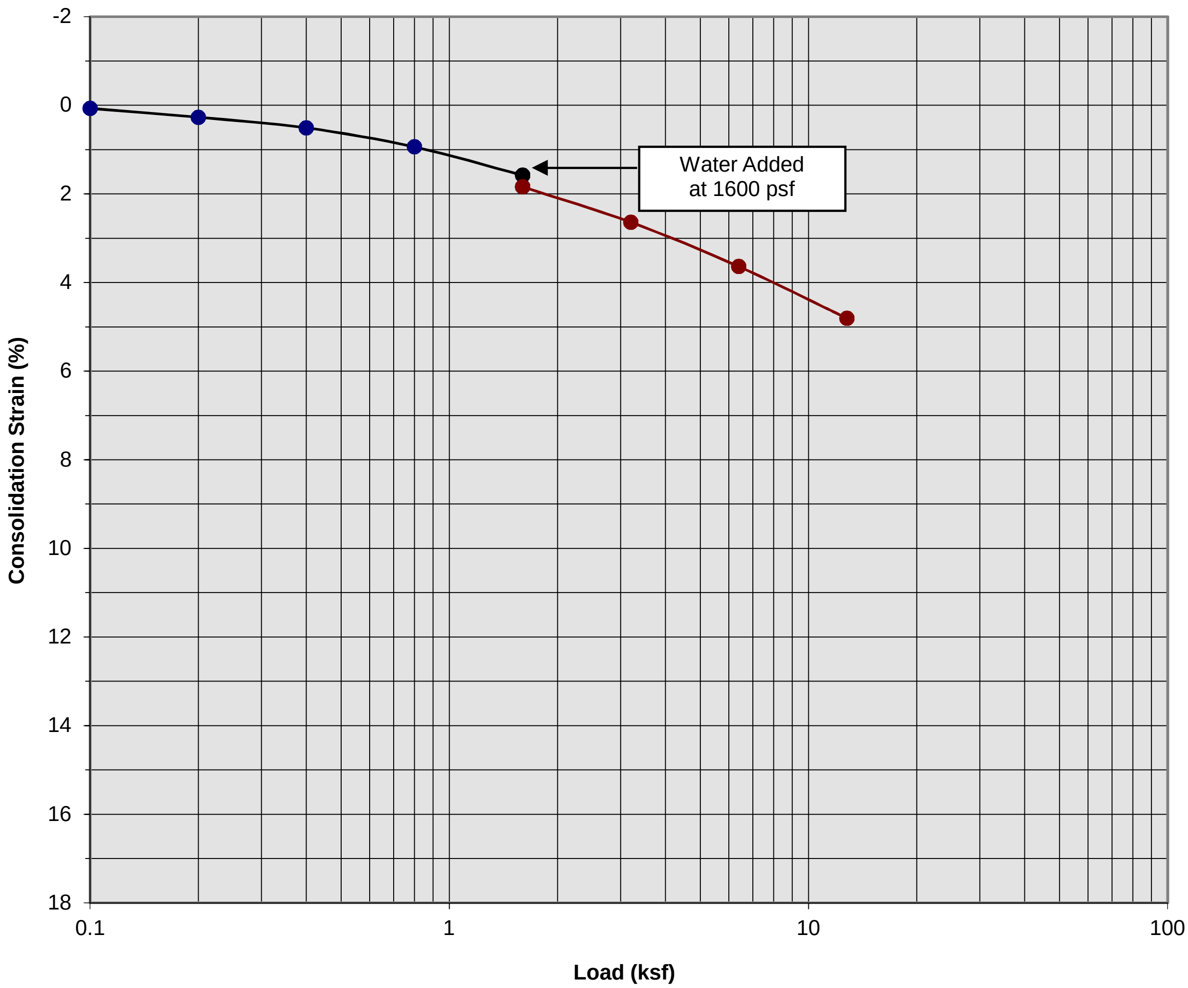
Classification: Light Gray Brown Silty fine Sand

Boring Number:	B-3	Initial Moisture Content (%)	24
Sample Number:	---	Final Moisture Content (%)	27
Depth (ft)	1 to 2	Initial Dry Density (pcf)	93.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	102.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.20

Proposed C/I Development
San Bernardino, California
Project No. 16G106
PLATE C- 5

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



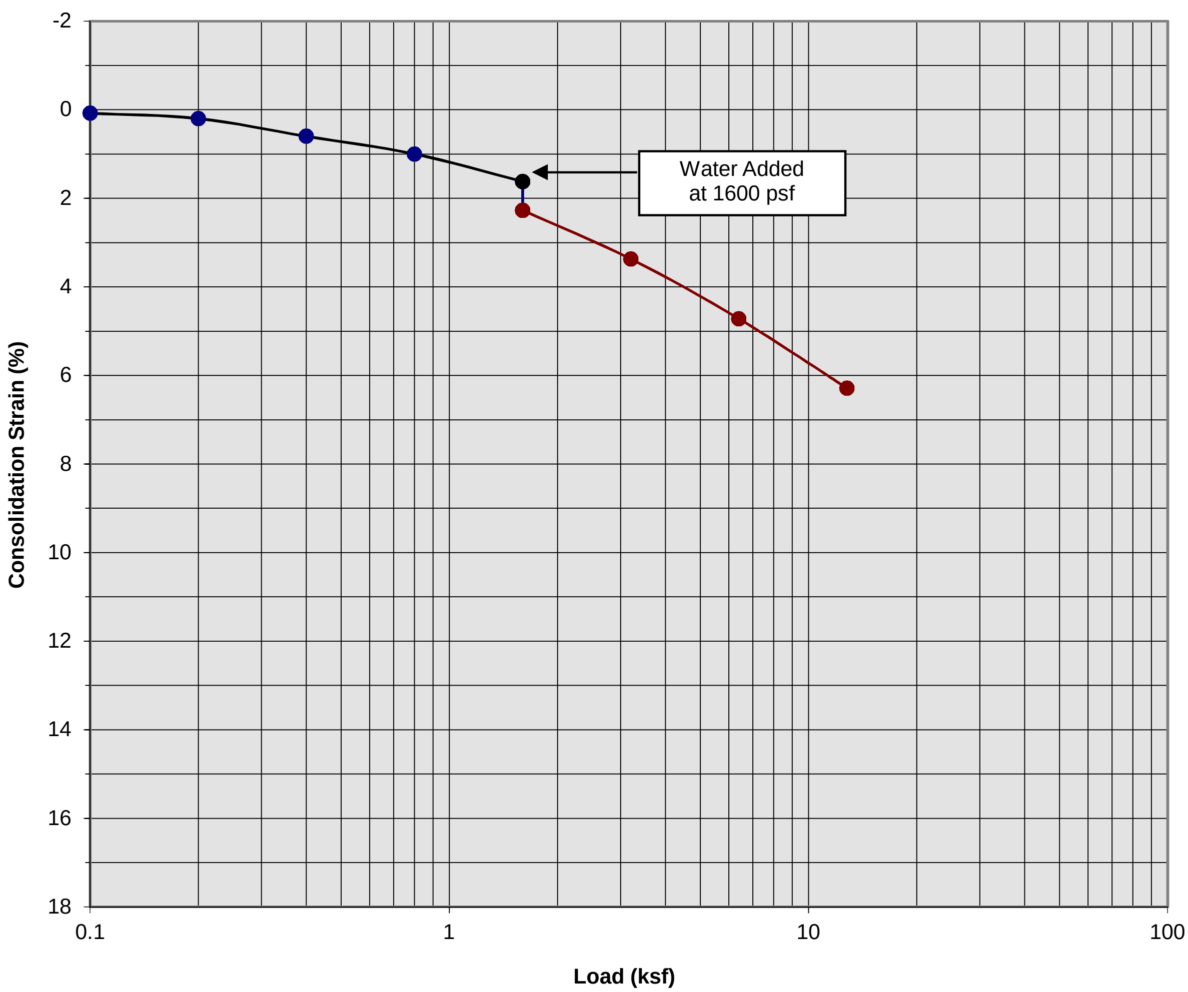
Classification: Light Gray Brown Silty fine to medium Sand, little coarse Sand

Boring Number:	B-3	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	3 to 4	Initial Dry Density (pcf)	110.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.26

Proposed C/I Development
San Bernardino, California
Project No. 16G106
PLATE C- 6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL A California Corporation

Consolidation/Collapse Test Results



Classification: Light Gray Brown Silty fine to medium Sand, little coarse Sand

Boring Number:	B-3	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	5 to 6	Initial Dry Density (pcf)	104.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.65

Proposed C/I Development
San Bernardino, California
Project No. 16G106
PLATE C- 7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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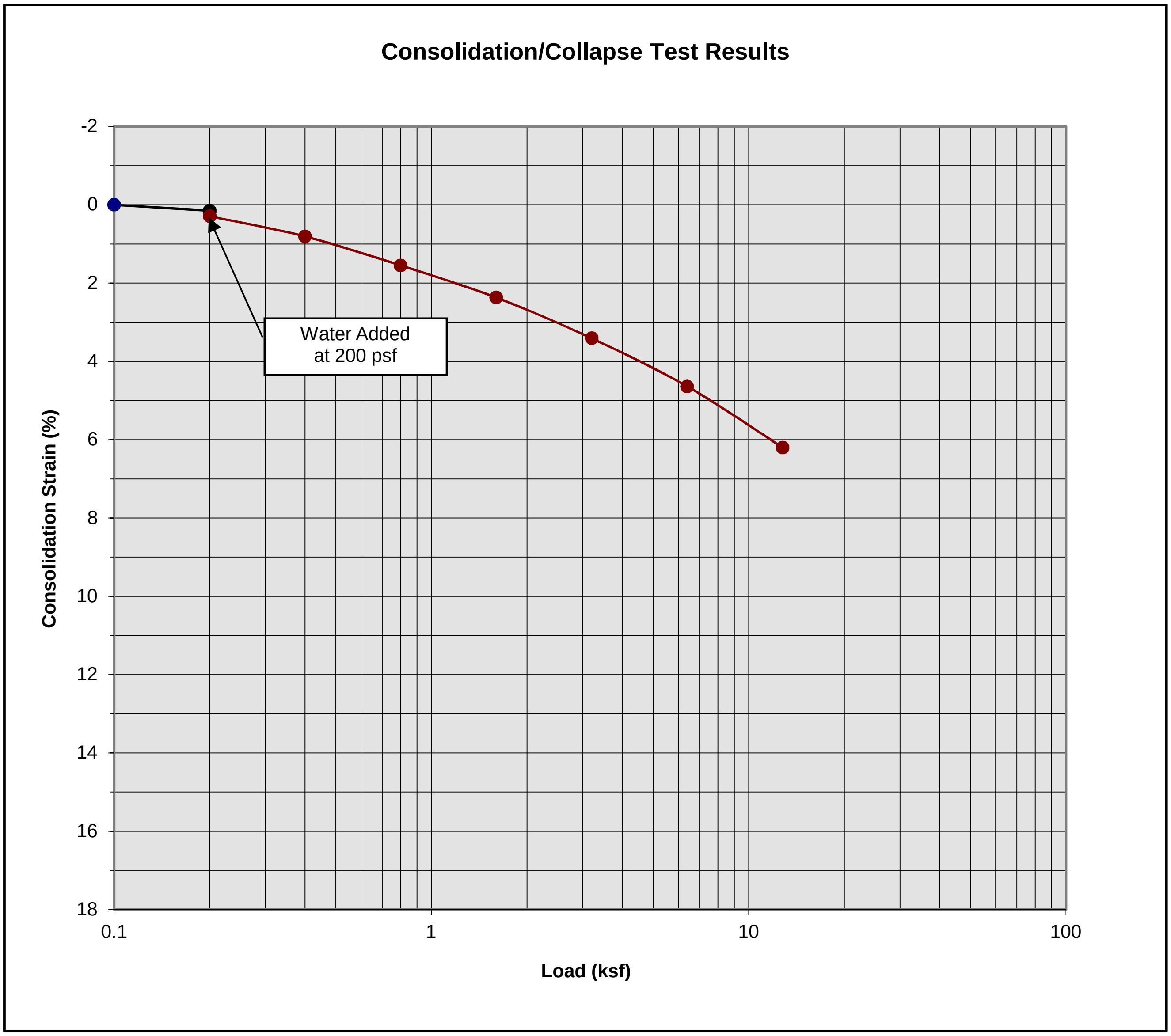

Classification: Gray Brown Clayey Silt

Boring Number:	B-3	Initial Moisture Content (%)	18
Sample Number:	---	Final Moisture Content (%)	23
Depth (ft)	7 to 8	Initial Dry Density (pcf)	101.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	107.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.14

Proposed C/I Development
San Bernardino, California
Project No. 16G106
PLATE C-8

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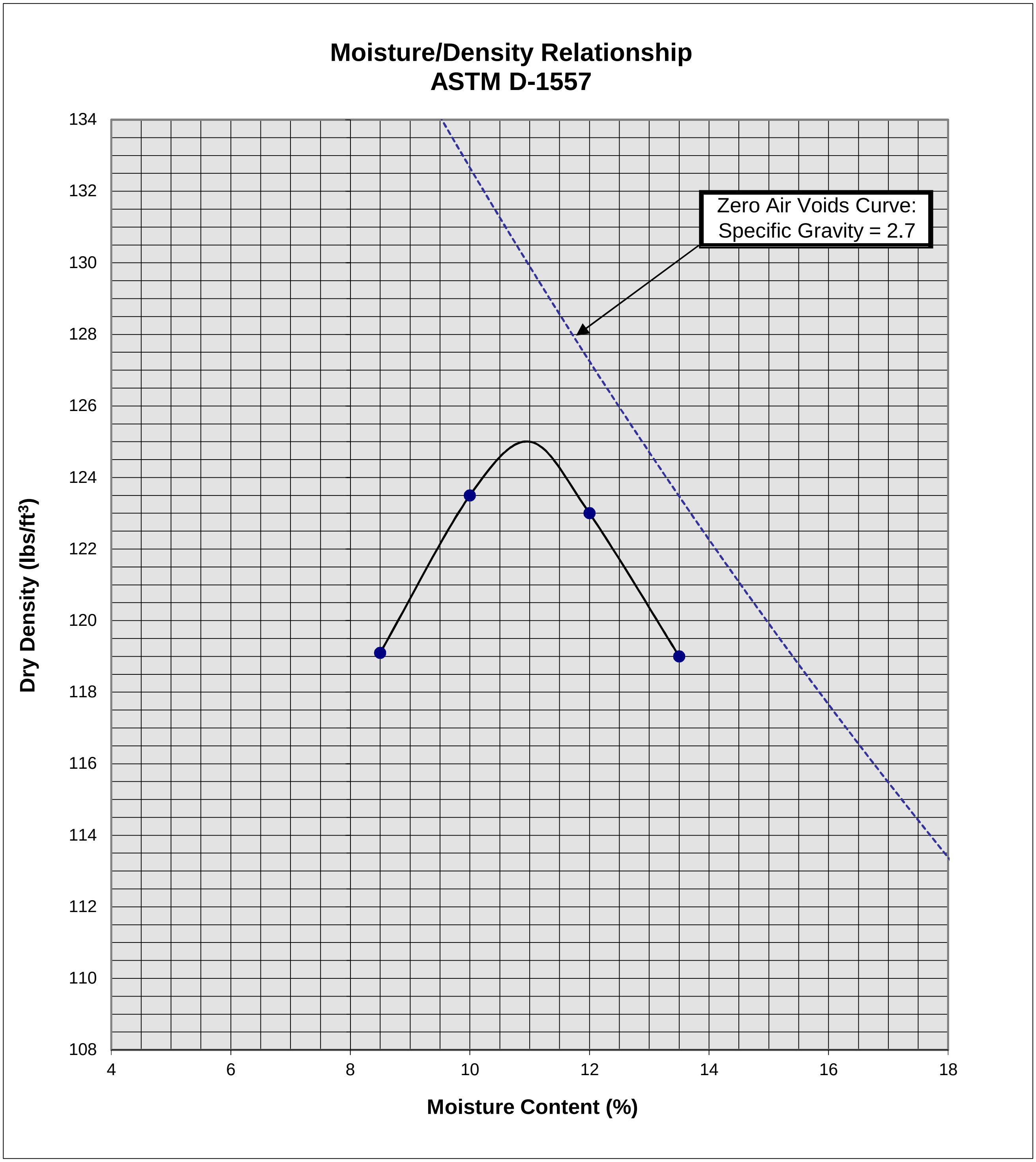

Soil ID Number	B-2 @ 0 to 5'
Optimum Moisture (%)	11
Maximum Dry Density (pcf)	125
Soil Classification	Gray Brown Silty fine Sand, trace Clay, trace Gravel

Proposed C/I Development
San Bernardino, California
Project No. 16G106
PLATE C-9

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APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

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

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

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

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

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

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

Site Preparation

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

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

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

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

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

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

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

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

<u>Compacted Fills</u>

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

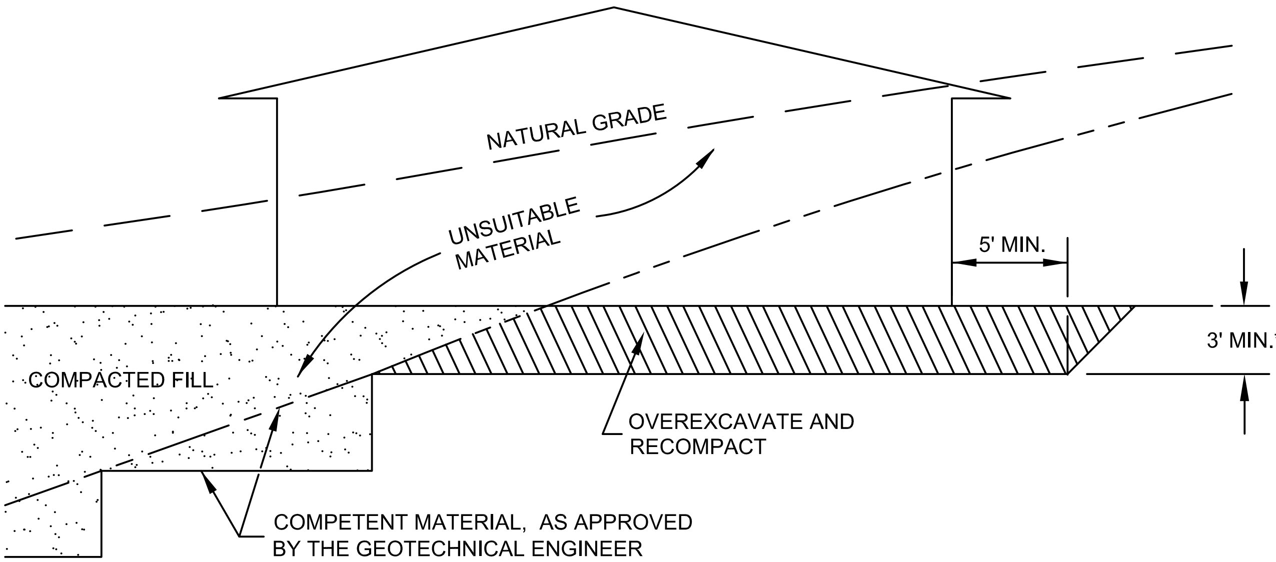
Subdrains

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

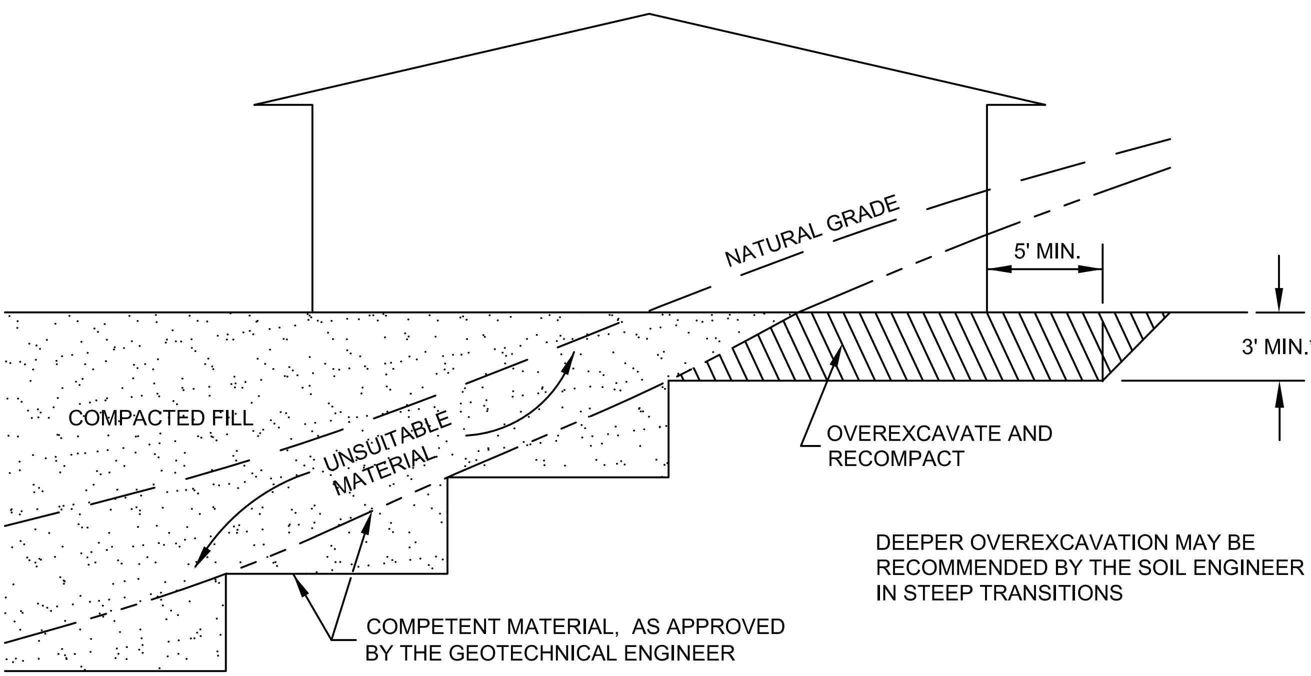
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

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

CUT LOT



CUT/FILL LOT (TRANSITION)



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

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

TRANSITION LOT DETAIL

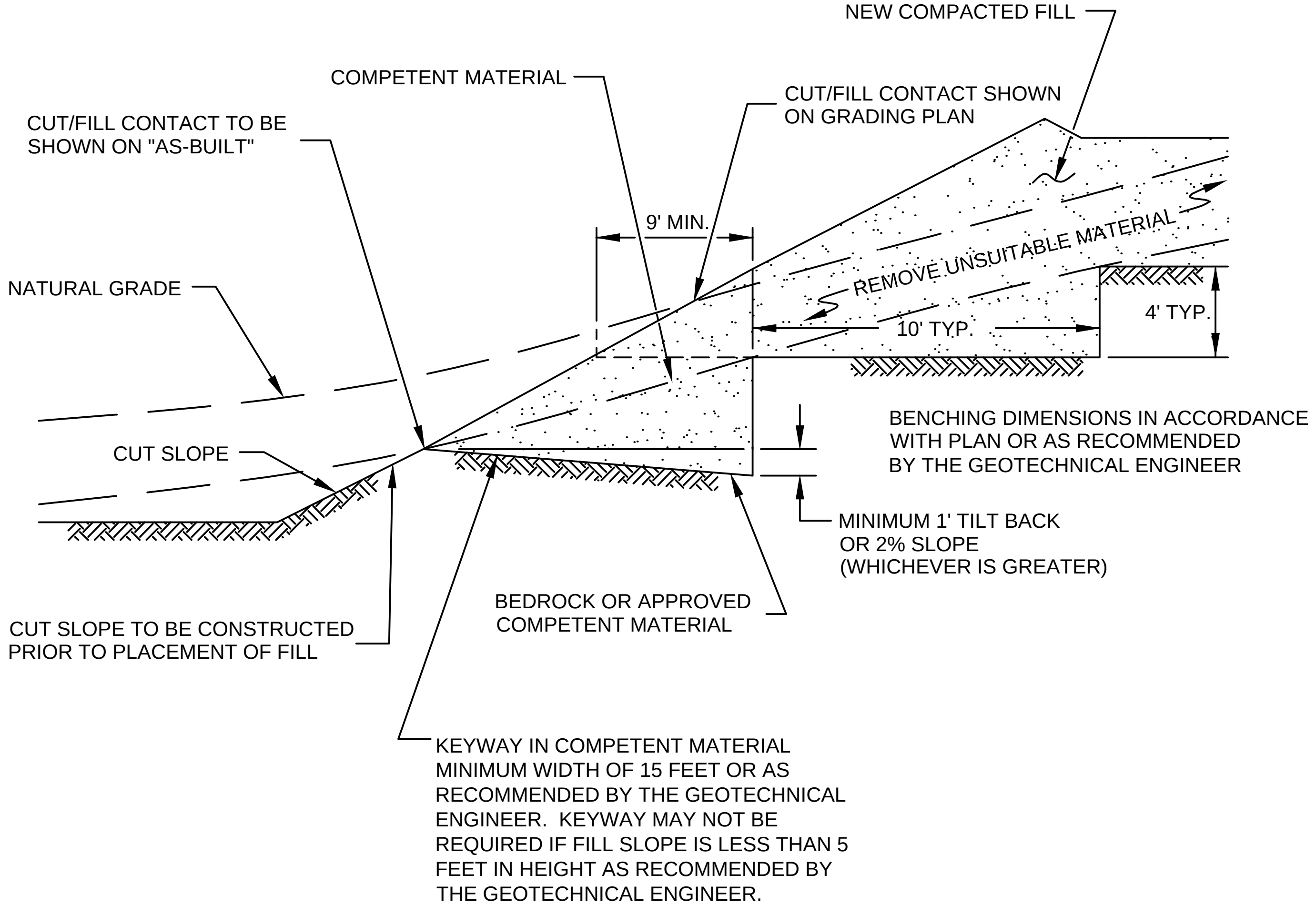
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

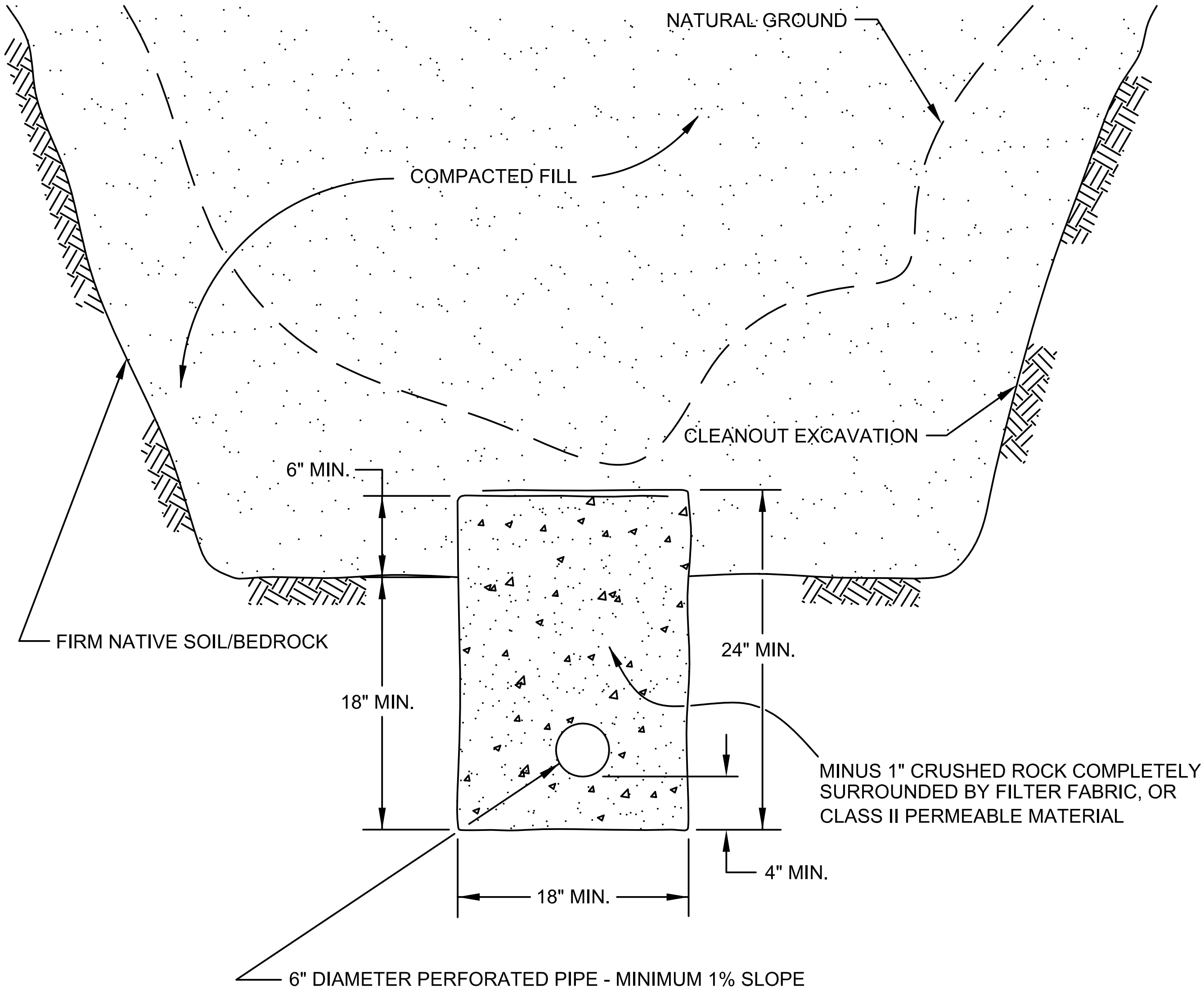
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

GRADING GUIDE SPECIFICATIONS

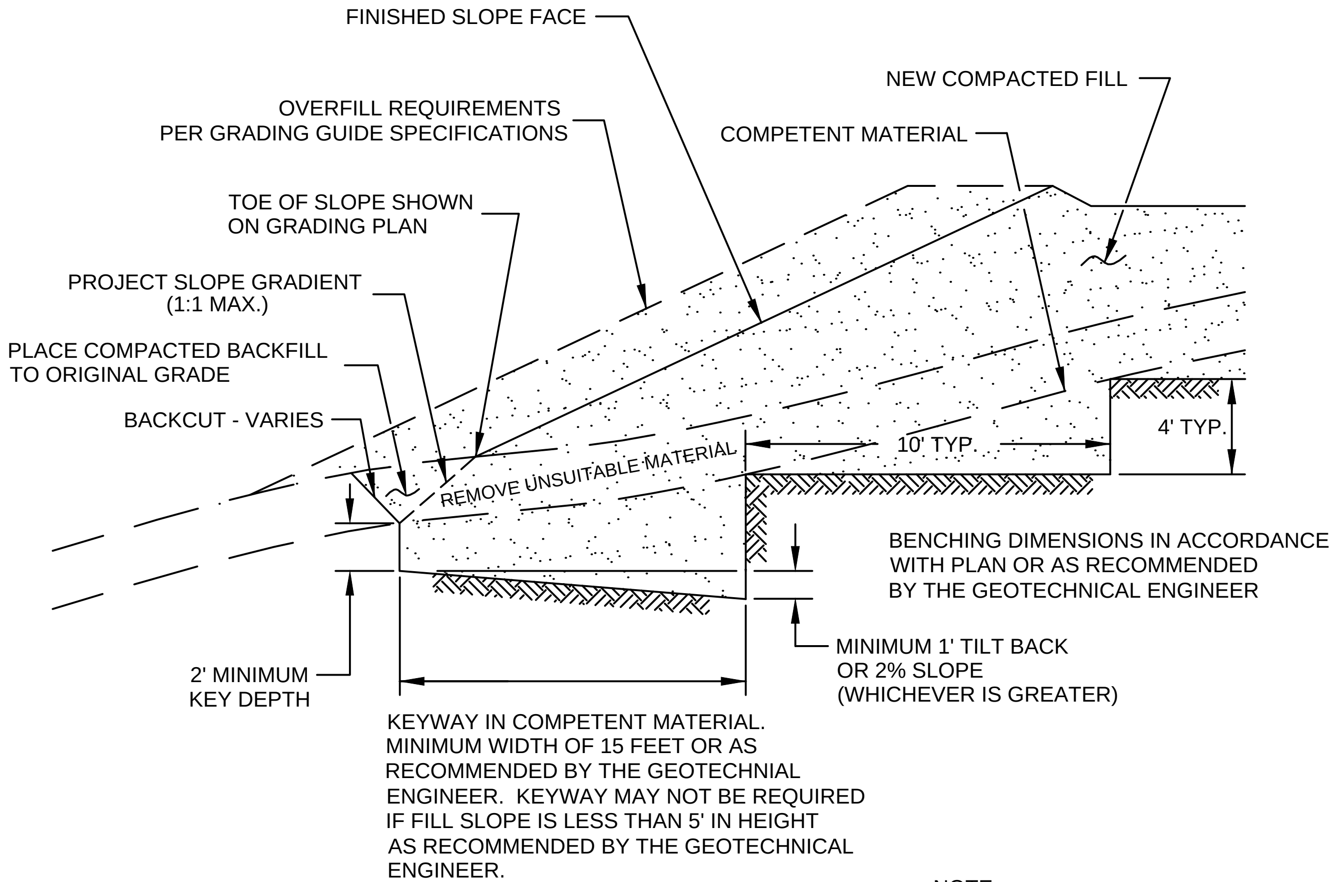
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



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

FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

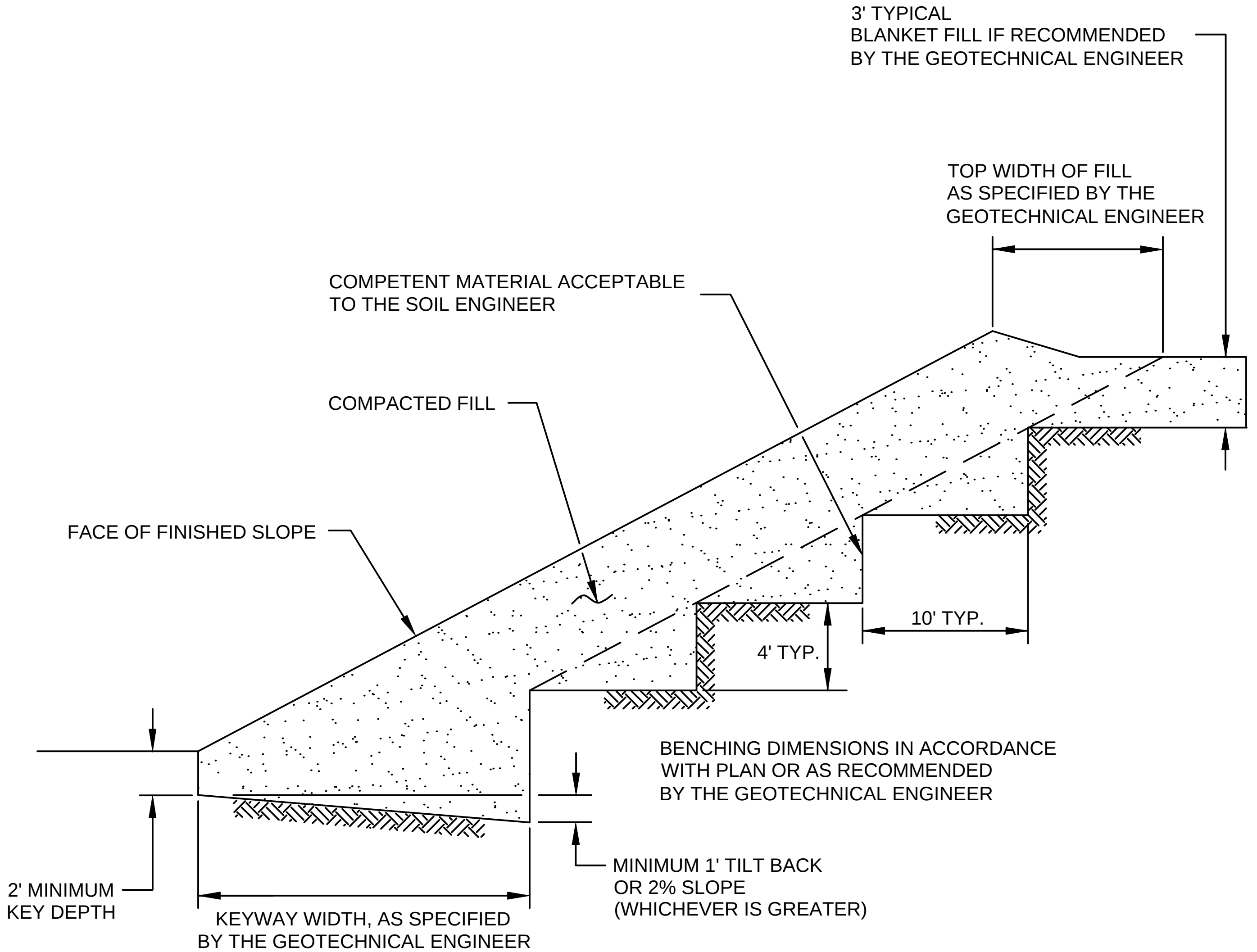
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

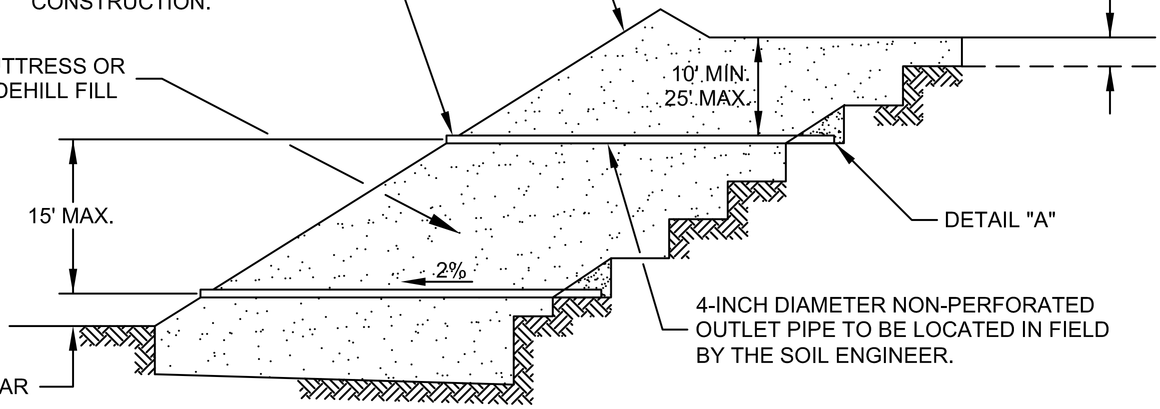
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL



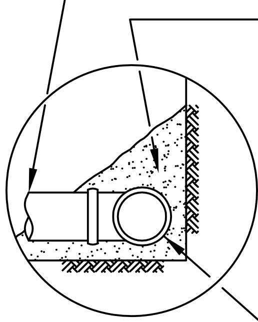
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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



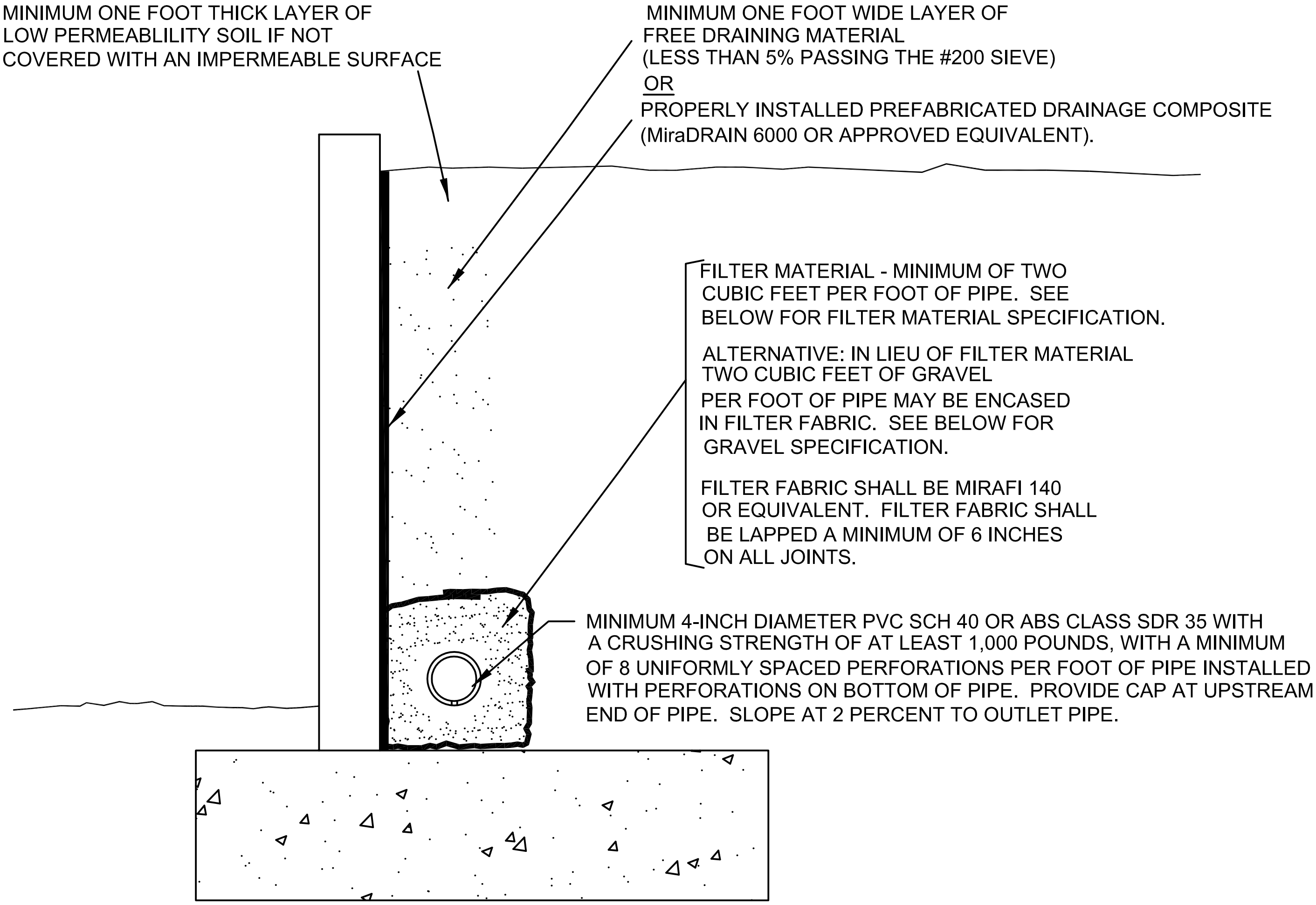
DETAIL "A"

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

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

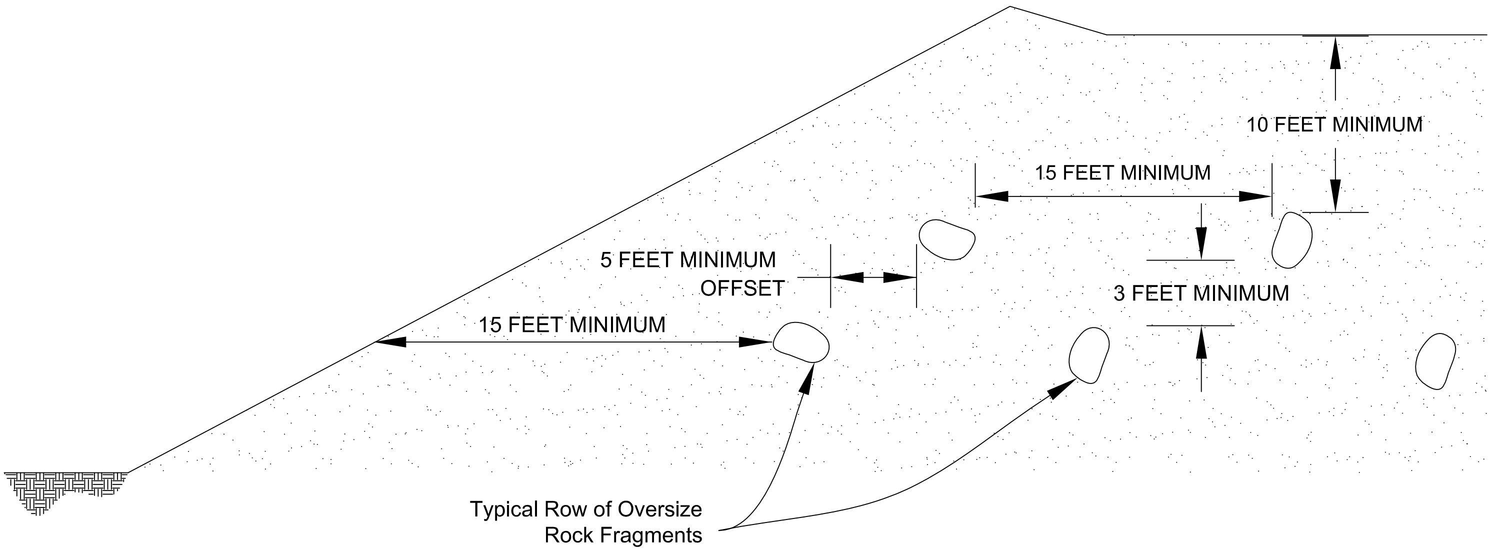
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

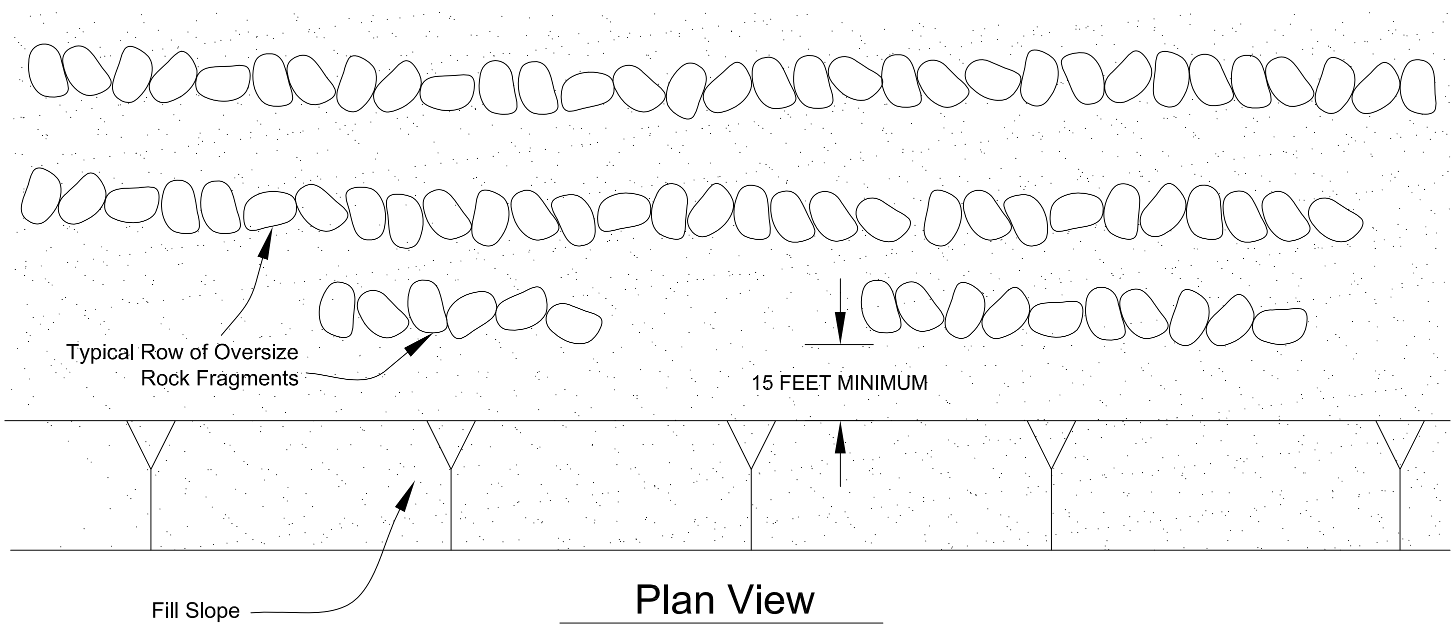
DRAWN: JAS
CHKD: GKM

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

SoCalGeo

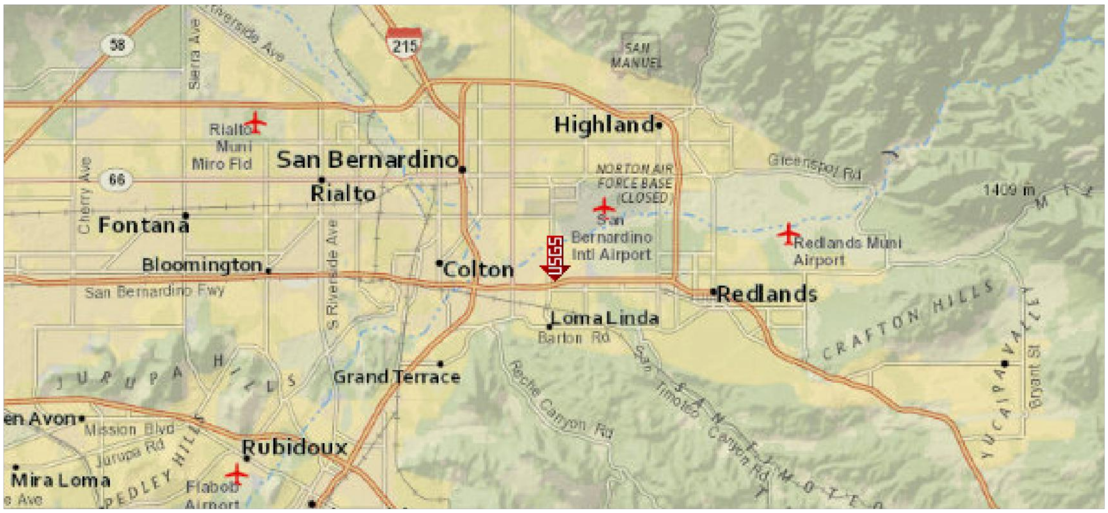
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

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	34.07607°N, 117.25874°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

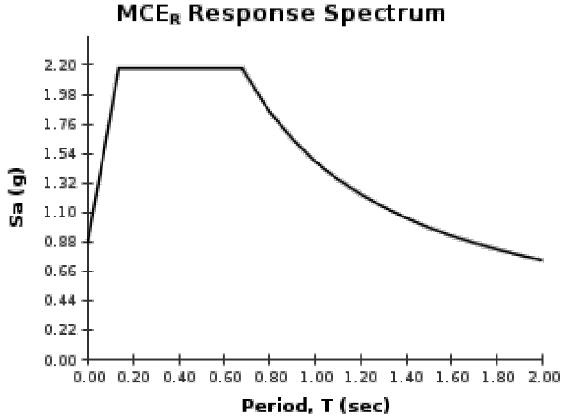


USGS–Provided Output

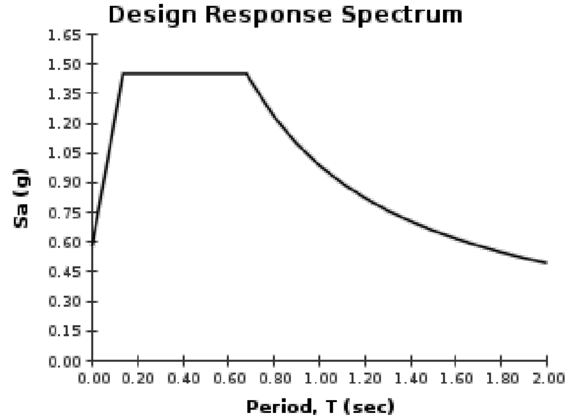
S_S =	2.176 g	S_{MS} =	2.176 g	S_{DS} =	1.451 g
S_1 =	0.987 g	S_{M1} =	1.481 g	S_{D1} =	0.987 g

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

MCE_R Response Spectrum



Design Response Spectrum



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

SEISMIC DESIGN PARAMETERS	
PROPOSED COMMERCIAL/INDUSTRIAL DEVELOPMENT	
SAN BERNARDINO, CALIFORNIA	
DRAWN: JL CHKD: JAS	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
SCG PROJECT 16G106-1	
PLATE E-1	

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

From Figure 22-7 [4]	$PGA = 0.840$
Equation (11.8–1):	$PGA_M = F_{PGA}PGA = 1.000 \times 0.840 = 0.84$ g

Table 11.8–1: Site Coefficient F_{PGA}

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

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

For Site Class = D and PGA = 0.840 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]	$C_{RS} = 1.016$
From Figure 22-18 [6]	$C_{R1} = 0.969$

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

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

DRAWN: PM
CHKD: JAS
SCG PROJECT 16G106-1
PLATE E-2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Proposed C/I Development	MCE_G Design Acceleration	0.840 (g)
Project Location	San Bernardino, CA	Design Magnitude	6.95
Project Number	16G106	Historic High Depth to Groundwater	15 (ft)
Engineer	PM	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-1		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.95)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
9	0	15	7.5	7	120	46	1.3	1.05	1.12	1.49	0.75	11.9	17.5	900	900	900	0.98	1.08	1.1	0.18	0.21	N/A	N/A	Above Water Table
15	15	17	16	7	120	46	1.3	1.05	1.1	1.05	0.85	9.4	15.0	1920	1858	1920	0.95	1.06	1.01	0.16	0.17	0.53	0.31	**Liquefiable**
19.5	17	19	18	11	120	90	1.3	1.05	1.16	0.99	0.95	16.5	22.0	2160	1973	2160	0.94	1.11	1.01	0.23	N/A	N/A	N/A	Non-Liq:12<PI<18, w<.8*LL
19.5	19	22	20.5	11	120	51	1.3	1.05	1.15	0.94	0.95	15.4	21.0	2460	2117	2460	0.93	1.10	1	0.22	0.24	0.59	0.41	**Liquefiable**
23.5	22	24	23	16	120	49	1.3	1.05	1.23	0.91	0.95	23.1	28.8	2760	2261	2760	0.91	1.18	0.99	0.42	0.49	0.61	0.80	**Liquefiable**
23.5	24	27	25.5	16	120	88	1.3	1.05	1.22	0.87	0.95	22.0	27.5	3060	2405	3060	0.90	1.17	0.98	0.36	0.41	0.63	0.66	**Liquefiable**
29.5	27	29	28	9	120	38	1.3	1.05	1.1	0.80	0.95	10.3	15.9	3360	2549	3360	0.89	1.07	0.98	0.16	0.17	0.64	0.27	**Liquefiable**
29.5	29	32	30.5	9	120	93	1.3	1.05	1.1	0.77	0.95	9.9	15.4	3660	2693	3660	0.87	1.06	0.97	0.16	N/A	N/A	N/A	Non-Liq: PI>18
34.5	32	37	34.5	12	120	94	1.3	1.05	1.14	0.74	1	13.8	19.3	4140	2923	4140	0.85	1.09	0.96	0.20	N/A	N/A	N/A	Non-Liq: PI>18
39.5	37	42	39.5	29	120		1.3	1.05	1.3	0.79	1	40.6	40.6	4740	3211	4740	0.82	1.23	0.87	2.00	2.00	0.66	3.02	Non-Liquefiable
44.5	42	47	44.5	32	120		1.3	1.05	1.3	0.78	1	44.1	44.1	5340	3499	5340	0.79	1.23	0.85	2.00	2.00	0.66	3.02	Non-Liquefiable
49.5	47	50	48.5	32	120		1.3	1.05	1.3	0.75	1	42.7	42.7	5820	3730	5820	0.77	1.23	0.83	2.00	2.00	0.66	3.04	Non-Liquefiable

Notes:

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

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed C/I Development
Project Location	San Bernardino, CA
Project Number	16G106
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	$(N_1)_{60-CS}$	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)			
9	0	15	7.5	11.9	5.6	17.5	N/A	0.21	0.64	0.00	15.00		0.000		0.00	Above Water Table
15	15	17	16	9.4	5.6	15.0	0.31	0.28	0.76	0.28	2.00		0.029		0.69	**Liquefiable**
19.5	17	19	18	16.5	5.5	22.0	N/A	0.13	0.41	0.00	2.00		0.000		0.00	Non-Liq:12<PI<18, w<.8*L
19.5	19	22	20.5	15.4	5.6	21.0	0.41	0.14	0.46	0.14	3.00		0.022		0.79	**Liquefiable**
23.5	22	24	23	23.1	5.6	28.8	0.80	0.06	-0.01	0.05	2.00		0.011		0.26	**Liquefiable**
23.5	24	27	25.5	22.0	5.5	27.5	0.66	0.06	0.07	0.06	3.00		0.014		0.51	**Liquefiable**
29.5	27	29	28	10.3	5.6	15.9	0.27	0.25	0.72	0.25	2.00		0.028		0.66	**Liquefiable**
29.5	29	32	30.5	9.9	5.5	15.4	N/A	0.26	0.74	0.00	3.00		0.000		0.00	Non-Liq: PI>18
34.5	32	37	34.5	13.8	5.5	19.3	N/A	0.17	0.55	0.00	5.00		0.000		0.00	Non-Liq: PI>18
39.5	37	42	39.5	40.6	0.0	40.6	3.02	0.01	-0.85	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	44.1	0.0	44.1	3.02	0.00	-1.12	0.00	5.00		0.000		0.00	Non-Liquefiable
49.5	47	50	48.5	42.7	0.0	42.7	3.04	0.00	-1.01	0.00	3.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				2.91	

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 Development	MCE_G Design Acceleration	0.840 (g)
Project Location	San Bernardino, CA	Design Magnitude	6.95
Project Number	16G106	Historic High Depth to Groundwater	15 (ft)
Engineer	PM	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-4		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.95)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
14.5	0	15	7.5	8	120	44	1.3	1.05	1.15	1.45	0.85	15.5	21.1	900	900	900	0.98	1.11	1.1	0.22	N/A	N/A	N/A	Above Water Table
14.5	15	17	16	8	120	44	1.3	1.05	1.11	1.05	0.85	10.8	16.4	1920	1858	1920	0.95	1.07	1.01	0.17	0.18	0.53	0.34	**Liquefiable**
19.5	17	22	19.5	7	120	81	1.3	1.05	1.1	0.95	0.95	9.5	15.0	2340	2059	2340	0.93	1.06	1	0.16	N/A	N/A	N/A	Non-Liq:12<PI<18, w<.8*LL
24.5	22	27	24.5	16	120	33	1.3	1.05	1.22	0.88	0.95	22.4	27.9	2940	2347	2940	0.90	1.17	0.98	0.38	0.43	0.62	0.70	**Liquefiable**
29.5	27	32	29.5	22	120	18	1.3	1.05	1.3	0.85	0.95	31.3	35.4	3540	2635	3540	0.88	1.23	0.94	1.22	1.41	0.64	2.19	Non-Liquefiable
34.5	32	37	34.5	21	120	79	1.3	1.05	1.3	0.80	1	29.9	35.5	4140	2923	4140	0.85	1.23	0.91	1.22	1.37	0.66	2.09	Non-Liquefiable
39.5	37	39.5	38.3	19	120	80	1.3	1.05	1.24	0.75	1	24.3	29.9	4590	3139	4590	0.83	1.19	0.92	0.48	0.52	0.66	0.79	**Liquefiable**
39.5	39.5	42	40.8	19	120	27	1.3	1.05	1.23	0.73	1	23.4	28.6	4890	3283	4890	0.81	1.18	0.91	0.41	0.44	0.66	0.67	**Liquefiable**
44.5	42	47	44.5	35	120	84	1.3	1.05	1.3	0.83	1	51.4	56.9	5340	3499	5340	0.79	1.23	0.85	2.00	2.00	0.66	3.02	Non-Liquefiable
49.5	47	50	48.5	16	120	61	1.3	1.05	1.17	0.65	1	16.6	22.2	5820	3730	5820	0.77	1.11	0.92	0.24	N/A	N/A	N/A	Non-Liq:12<PI<18, w<.8*LL

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 Development
Project Location	San Bernardino, CA
Project Number	16G106
Engineer	PM

Boring No. B-4

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)			
14.5	0	15	7.5	15.5	5.6	21.1	N/A	0.14	0.46	0.00	15.00		0.000		0.00	Above Water Table
14.5	15	17	16	10.8	5.6	16.4	0.34	0.24	0.70	0.24	2.00		0.027		0.65	**Liquefiable**
19.5	17	22	19.5	9.5	5.5	15.0	N/A	0.27	0.75	0.00	5.00		0.000		0.00	Non-Liq:12<PI<18, w<.8*L
24.5	22	27	24.5	22.4	5.5	27.9	0.70	0.06	0.05	0.06	5.00		0.013		0.79	**Liquefiable**
29.5	27	32	29.5	31.3	4.1	35.4	2.19	0.02	-0.47	0.00	5.00		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	29.9	5.5	35.5	2.09	0.02	-0.47	0.00	5.00		0.000		0.00	Non-Liquefiable
39.5	37	39.5	38.3	24.3	5.5	29.9	0.79	0.05	-0.08	0.05	2.50		0.009		0.28	**Liquefiable**
39.5	39.5	42	40.8	23.4	5.2	28.6	0.67	0.06	0.00	0.06	2.50		0.012		0.35	**Liquefiable**
44.5	42	47	44.5	51.4	5.5	56.9	3.02	0.00	-2.16	0.00	5.00		0.000		0.00	Non-Liquefiable
49.5	47	50	48.5	16.6	5.6	22.2	N/A	0.12	0.40	0.00	3.00		0.000		0.00	Non-Liq:12<PI<18, w<.8*LL
											Total Deformation (in)				2.07	

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)