Appendix D Geotechnical Report

Appendices

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GEOTECHNICAL INVESTIGATION PROPOSED MULTI-UNIT RESIDENTIAL DEVELOPMENT

SEC Berry Street and Mercury Lane
Brea, California
for
Peregrine Construction, Inc.

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

April 20, 2018

Peregrine Construction, Inc.
7545 Irvine Center Drive, Suite 200
Irvine, California 92618

Attention: Mr. John Atherton
President

Project No.: **18G118-1**

Subject: **Geotechnical Investigation**
Proposed Multi-Unit Residential Development
SEC Berry Street and Mercury Lane
Brea, California

Gentlemen:

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

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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1.0 EXECUTIVE SUMMARY

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

Geotechnical Design Considerations

- Our site-specific liquefaction evaluation indicates that some of the soils encountered at the borings are subject to liquefaction during the design seismic event.
- Our analysis indicates total settlements of 1.2 to 3.4± inches. These settlements are assumed to occur over a horizontal distance of 50± feet, indicating an angular distortion of approximately 0.003 inches per inch.
- Existing fill soils were encountered at Boring Nos. B-1, B-3 and B-4 extending to depths of 2½ to 4½± feet.
- The existing fill soils generally possess variable strengths. No documentation regarding their placement or compaction was available, therefore these soils are not considered suitable to support the proposed structure. Remedial grading will be necessary within the building area. It is anticipated that planned cuts to reach proposed subgrades will remove all of the encountered fill soils.
- Groundwater was encountered at depths of 25 to 27± feet at all of our boring locations.
- As of the writing of this report, grading plans and foundation plans were not available for the proposed development. Based on the existing site topography and site plans provided to us by the client, we expect that cuts of up to 10 to 12± feet will be necessary in order to construct a below-ground parking level with a footprint nearly covering the entire subject site. Preliminary grading and foundation design recommendations have been included in subsequent sections of this report.
- Based on the subsurface conditions encountered at the subject site, the proposed building may be supported on conventional shallow foundations. However, this assumption is subject to review of the grading plans and foundation plans, when this information becomes available. Due to the relatively large anticipated foundation loads and other considerations, it may be desirable or necessary to support the proposed building on an alternate foundation system such as a mat foundation or a deep foundation system.

Site Preparation

- Remedial grading is recommended to be performed within the new building pad areas. The overexcavation should extend to a depth of at least 5 feet below the existing grade, 3 feet below the proposed pad grade and to a depth sufficient to remove all of the existing undocumented fill soils.
- It is expected that some of the soils encountered at the base of the recommended overexcavations within the building pad area will possess elevated moisture contents. Some drying and/or stabilization of the overexcavation subgrade may be necessary.
- Additional overexcavation should be performed within the influence zones of the new foundations, to provide for a new layer of compacted structural fill extending to a depth of at least 7 feet below foundation bearing grades.

- After overexcavation has been completed, the resulting subgrade should then be scarified to a depth of 10 to 12 inches, and moisture conditioned or air dried to a moisture content of 0 to 4 percent above the optimum. The previously excavated soils may then be replaced as compacted structural fill.
- Due to the liquefaction potential of the on-site soils, the new layer of structural fill within the proposed building area is recommended to incorporate geotextile reinforcement, to provide additional rigidity within this structural fill layer. Two (2) perpendicular layers of Mirafi RS580i, or equivalent, are considered suitable for this purpose
- The new parking area and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, thoroughly moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 3,500 lbs/ft^2 maximum allowable soil bearing pressure. 2,500 lbs/ft^2 maximum allowable soil bearing pressure for footings where the described lateral extents are not met.
- Reinforcement consisting of at least six (6) No. 5 rebars (3 top and 3 bottom) in strip footings, due to the liquefaction potential of the on-site soils. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade, 6 inches thick.
- Reinforcement consisting of at least No. 4 bars at 16 inches on center, in both directions due to the presence of expansive soils and the liquefaction potential of the on-site soils. The actual floor slab reinforcement to be determined by the structural engineer. Additional reinforcement may be necessary for structural considerations.

Temporary Shoring

- Temporary shoring will be required during construction of the proposed subterranean level. Detailed shoring recommendations are presented in Section 6.9 of this report and a diagram illustrating the earth pressures that will be exerted on the shoring is presented as Plate 3 in Appendix A of this report.

Pavements

ASPHALT PAVEMENTS (R = 20)				
Materials	**Thickness (inches)**			
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic	
			(TI = 6.0)	(TI = 7.0)
Asphalt Concrete	3	3	3½	4
Aggregate Base	5	8	10	12
Compacted Subgrade	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS			
Materials	**Thickness (inches)**		
	Auto Parking & Drives (TI = 5.0)	Truck Traffic	
		(TI =6.0)	(TI =7.0)
PCC	5	6	7
Compacted Subgrade	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 18P164, dated February 15, 2018. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. 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 site is located on the southeast corner of Berry Street and Mercury Lane in Brea, California. The site is bordered to the north by Mercury Lane, to the east and south by existing commercial development, and to the west by Berry Street. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in Appendix A of this report.

The subject site consists of a rectangular-shaped property that is 1.01± acres in size. The site is currently undeveloped. Ground surface cover consists of exposed soils with some medium trees and shrubs located on the southwest corner of the site and along the northern perimeter of the site. The eastern portion of the site is currently being utilized and a temporary tractor trailer storage lot. A drainage swale and a 2:1 slope sloping downward to the east are located along the eastern perimeter of the site on the adjoining property is. The subject site is surrounded with chain link fencing as well as a chain link fence that is running north to south through the center of the site.

Detailed topographic information was not available at the time of this report. However, based on visual observations, the site topography slopes downward to the south at an estimated gradient of approximately 1 to 2 percent. There was estimated to be 3 to 4± feet of elevation differential across the site.

3.2 Proposed Development

Our office was provided with architectural plans that was prepared by Humphreys & Partners Architects, L.P. (H&P), dated April 4, 2018. Based on our review of these documents, the site will be developed with a 120-unit multi-story residential development.

The site massing study identifies the ground surface adjacent to the proposed building as "Level 1." The study indicates that the vertical distance from the ground level to the proposed finish floor of subterranean level "Level B1" is 10½± feet. The study also indicates that the vertical distances between floors above the ground level range from 9 feet to 12 feet. The study identifies a total of 7 levels consisting of the subterranean Level B1, the ground level (Level 1), levels 2 through 5, and the roof deck. The total height of the apartment complex is 65 feet 10 inches above the ground level, as indicated on the H&P study.

Based on our review of the building sections included in the H&P study, our understanding of the project layout is described below:

- The lowest parking garage floor is located at a depth of approximately 10½ feet below the ground level and the lowest parking garage walls retain up to approximately 10½ feet of soil.

- Levels B-1, 1, and 2 are planned for residential parking and limited residential units.
- A podium level is planned for Level 3, which contains a courtyard, residential units, amenity rooms and a laundry room.
- Levels 4 and 5 contain residential units and a laundry room.
- The walls of the residential units do not retain any soil.
- The proposed apartment complex is located approximately 5 to 10 feet horizontally from the surrounding property lines.

Detailed structural information was not available at the time of this proposal. Based on conversations with the project architect, the lowest two levels will likely be of reinforced concrete construction and the remaining levels will consist of wood-frame construction. Prior to the preparation of this report, the project architect was not able to provide anticipated maximum column and wall loads for the apartment complex. Also, it is our understanding that no structural engineer is currently on the design team. Therefore, we were not provided with any load information from the project structural engineer prior to writing this report. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 300 to 600 kips and 3 to 10 kips per linear foot, respectively.

Based on the existing topography and in order to facilitate the construction of the subterranean level at the subject site, it is estimated that cuts of up to 10 to 12± feet and fills of up to 3 to 6± feet may be necessary to achieve the new site grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of five (5) borings, advanced to a depth of 75± feet below currently existing site grades. All of the borings were logged during drilling by a member of our staff.

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

The approximate locations of the borings are indicated 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

Pavements

Pavements consisting of 5 to 6± inches of aggregate base were encountered at the ground surface at Boring Nos. B-1, B-2 and B-5.

Artificial Fill

Fill soils were encountered at the ground surface or beneath the existing pavements at Boring Nos. B-1, B-3 and B-4, extending to depths of 2½ to 4½± feet below the existing site grades. The fill soils generally consist of loose to medium dense silty fine to coarse sands with varying amounts of clay and fine to coarse gravel. The fill soils possess a disturbed appearance and varying amounts of debris, including brick and asphaltic concrete fragments, resulting in their classification as fill.

Alluvium

Native alluvium was encountered below the pavements and artificial fill soils at all of the boring locations, extending to at least the maximum depth explored of 75± feet below existing site grades.

The native alluvial soils extending to depths of 8 to 12± feet generally consist of stiff to very stiff fine sandy clays with occasional layers of loose to medium dense silty fine sands. Beneath these soils, the native alluvium generally consists of medium dense fine to coarse sands, silty fine sands and clayey sands extending to depths of 27 to 33½± feet at Boring Nos. B-1, B-2, B-4, and B-5, and extending to a depth of 47± feet at Boring No. B-3. Beneath these soils, the native alluvium generally consists of stiff to hard sandy clays and silty clays and dense to very dense clayey sands and fine to coarse sands, with varying gravel and cobble content and occasional layers of medium dense fine to coarse sands and clayey sands extending to the maximum depth explored of 75± feet.

Groundwater

Free water was encountered during drilling at depth of 25 to 27± feet. Based on the water level measurements and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at depths of 25 to 27± feet below existing site grades at the time of the subsurface investigation.

As part of our research of historic groundwater levels we reviewed CA DMG Open-File Report 97-09 for the La Habra Quadrangle. Plate 1.2 of OFR 97-09 is a map which displays the historically highest ground water levels using contour lines. The water levels mapped in the vicinity of the subject site indicate the historic high groundwater table to be at a depth of 10± feet.

5.0 LABORATORY TESTING

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

Classification

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

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-9 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

Three (3) representative bulk samples have been tested for their maximum dry densities and optimum moisture contents. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing and are presented on Plates C-10 through C-12. Additional testing of other soil types or soil mixes may be necessary at a later date.

Direct Shear

Direct shear tests were performed on two (2) selected soil samples to determine their shear strength parameters. These tests were performed in accordance with ASTM D-3080. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Three samples of the same soil are prepared by remolding them to 90± percent compaction and near optimum moisture. Each of the three samples are then loaded with different normal loads and the resulting shear strength is determined for that particular normal load. The shearing of the samples is performed at a rate slow enough to permit the dissipation of excess pore water pressure. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The results of the direct shear tests are presented on Plates C-13 and C-14.

Expansion Index

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

Sample Identification	**Expansion Index**	**Expansive Potential**
B-1 @ 8 to 12 feet	47	Low
B-1 @ 20 to 25 feet	2	Very Low

Soluble Sulfates

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

Sample Identification	**Soluble Sulfates (%)**	**Sulfate Classification**
B-1 @ 20 to 25 feet	0.006	Negligible
B-3 @ 5 to 10 feet	0.004	Negligible

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

6.1 Seismic Design Considerations

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

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. In addition, no evidence of faulting was observed during our field exploration. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

Beginning January 1, 2017, the 2016 CBC was adopted by all municipalities within Southern California. The CBC provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

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

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	2.047
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.751
Site Class	---	F*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	2.047
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.127
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.364
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.751

*The 2013 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site *coefficients* are to be determined in accordance with Section 11.4.7 of ASCE 7-10. However, Section 20.3.1 of ASCE 7-10 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental periods of the structures are less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. **If the proposed structure has a fundamental period greater than 0.5 seconds, a site specific seismic hazards analysis will be required and additional subsurface exploration will be necessary.**

Ground Motion Parameters

For the liquefaction evaluation, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2016 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-10. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-10. The web-based software application U.S. Seismic Design Maps (described in the previous section) was used to determine PGA_M, which is 0.795g. A portion of the program output is included as Plate E-2 of this report. An associated earthquake magnitude was obtained from the USGS Unified Hazard Tool, Interactive Deaggregation application available on the USGS website. The deaggregated modal magnitude is 6.71, based on the peak ground acceleration and soil classification D.

Liquefaction

Research of the Seismic Hazards Zones Map for the La Habra Quadrangle, published by the California Geological Survey (CGS) indicates that the site subject site is located within a liquefaction hazard zone. Based on this mapping, and the subsurface conditions encountered at the borings, the scope of this investigation included a detailed liquefaction evaluation in order to determine the site-specific liquefaction potential.

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

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

The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report. The liquefaction analysis was performed for Boring Nos. B-2 and B-4, which were advanced to depths of more than 50± feet. The liquefaction potential was analyzed at the boring locations utilizing a PGA_M of 0.795g related to a 6.71 magnitude seismic event. The liquefaction evaluation was performed using the reported historic high groundwater depth of 10 feet.

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

Conclusions and Recommendations

The results of the liquefaction analysis have identified potentially liquefiable soil strata at Boring No. B-2, between depths of 27 and 32± feet and at Boring No. B-4, between depths of 17 and 32± feet. Soils which are located above the historic groundwater table (10 feet), or possessing factors of safety in excess of 1.3 are considered non-liquefiable. Settlement analysis was conducted for the potentially liquefiable strata. The result of the settlement analysis indicates potential total settlements of 1.16± inches and 3.37± inches at Boring Nos. B-2 and B-4, respectively.

Based on the estimated total settlements, liquefaction-induced differential settlements are expected to be on the order of 2.2± inches. The estimated differential settlement can be assumed to occur across a distance of 50 feet, indicating a maximum angular distortion on the order of 0.003 inches per inch. These differential settlements are considered to be within the structural tolerances of a typical building supported on a shallow foundation system provided that structural mitigation measures are implemented. However, it should be noted that minor to moderate repairs, including repair of damaged drywall and stucco, etc., could be required after the occurrence of liquefaction-induced settlements.

Based on our understanding of the proposed development and the client's risk tolerances, it is considered feasible to support the proposed building on a shallow foundation system. Such foundation systems can be designed to resist the effects of the anticipated differential settlements, to the extent that the structure would not catastrophically fail. Designing the proposed building to remain completely undamaged during a major seismic event is not considered to be economically feasible. Based on this understanding, the use of a shallow foundation system is considered to be the most economical means of supporting the proposed building.

In order to support the proposed building on shallow foundations (such as spread footings) the structural engineer should verify that the structure would not catastrophically fail due to the predicted dynamic differential settlements. Any utility connections to the structure should be designed to withstand the estimated differential settlements. It should also be noted that minor to moderate repairs, including releveling, restoration of utility connections, repair of damaged drywall and stucco, etc., would likely be required after occurrence of the 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 to the buildings 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

Existing variable strength fill soils were encountered at Boring Nos. B-1, B-3 and B-4 extending to depths of 2½ to 4½± feet. No documentation regarding their placement or compaction was available, therefore these soils are not considered suitable to support the proposed structure. The proposed building will incorporate one subterranean parking garage encompassing a majority of the subject site. We anticipate that this underground parking garage will be constructed at a depth of 10 to 12± feet below existing site grades. As such, the excavation for the proposed underground parking garage will remove all of the existing fill soils encountered during our subsurface exploration program.

The soils encountered at depths of 10 to 12± feet below existing site grades consist of medium stiff to stiff fine sandy clays and medium dense silty fine sands and clayey fine sands. These soils possess variable consolidation characteristics. Some remedial grading of these materials is recommended in order to provide uniform support characteristics for the new structure and to help reduce settlement potentials.

As discussed previously, detailed structural information was not available as of the writing of this report. Therefore, we are providing preliminary geotechnical design parameters. However, it should be understood that these recommendations are based on preliminary assumptions and will require review and may be revised upon review of the grading and foundation plans. Factors which may affect the grading and foundation design recommendations include the depth of the proposed below-grade parking level, foundation loads, and required settlement tolerances. It may be necessary to perform additional subsurface exploration, lab testing and/or engineering analysis in order to update the grading and foundation design recommendations after the foundation loads and other associated building information becomes available.

As discussed in a previous section of this report, potentially liquefiable soils were identified at the site. The presence of the recommended layer of newly placed compacted structural fill above these potentially liquefiable soils will help to reduce surface manifestations that could occur as a result of liquefaction. The foundation and floor slab design recommendations presented in 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.

A significant design concern for this project is the potential difficulty associated with making excavations near adjacent structures, adjacent property lines, adjacent streets and other adjacent improvements. Provisions should be made to incorporate temporary or permanent shoring in the design of the proposed structure. More detailed recommendations for shoring are presented in subsequent sections of this report.

Settlement

As previously discussed, the proposed below-ground parking garage will require an excavation extending to a depth of at least 10 to 12± feet throughout most of the subject site.

The recommended remedial grading will remove a portion of the existing fill soils and replace these materials as compacted structural fill. As a result, the building foundations and floor slab will be underlain by a newly placed layer of certified fill. The existing native alluvial soils that will remain in place below the recommended depth of overexcavation possess relatively high strengths and will not be subject to significant stress increases due to the applied foundation loads. Therefore, provided that the recommended remedial grading is completed, the post-construction static settlement of the proposed structure is expected to be within tolerable limits.

Expansion

Expansion index tests were performed on representative soil samples. These tests indicates that the on-site soils possess a very low to low expansion potential (EIs = 2 and 47). The recommendations contained in this report are made with respect to this condition. **Based on the presence of expansive soils at this site, special care should be taken to properly moisture condition all existing subgrade soils and newly placed fill soils, and to maintain these soils at moisture contents above the optimum moisture content.** Due to the significant amount of grading expected to be performed at this site, it is recommended that additional expansion index testing be performed subsequent to grading to confirm the actual conditions at the building pad subgrade elevations. Based on the varied expansion potentials, and with respect to the relatively large volume of grading which is proposed, it is expected that the finished subgrade soils will possess a low to medium expansion potential.

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

Based on the results of the laboratory testing, removal and recompaction of the loose to medium dense undocumented fill and near-surface native alluvial soils, extending to depths of 14± feet, is estimated to result in an average shrinkage of 10 to 16 percent. Please note that based on the variation of soil type, in-place densities and in-place moisture contents throughout the subject site, the local variation of shrinkage ranges from 0 to 26 percent. It should be noted that the potential shrinkage estimate is based on dry density testing performed on small-diameter samples taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1± feet. These estimates are based on previous experience and the subsurface conditions encountered at the boring locations. The actual amount of subsidence is expected to be variable and will be dependent on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely.

Grading and Foundation Plan Review

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

6.3 Site Grading Recommendations

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

Site Stripping

Initial site preparation should include removal of any surficial vegetation. Based on conditions encountered at the time of the subsurface exploration, moderate stripping of native grass and weed growth will generally be required in the northwestern region of the site and in the landscaped planters located in the new building area. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pad

It is anticipated that cuts to grade within the proposed building area will remove all of the existing fill soils. Based on conditions encountered at the boring locations and in order to help reduce the potential for excessive differential settlement due to the differing support conditions provided by the native soils at depths of 10 to 12± feet, the existing soils within the proposed building area are recommended to be overexcavated to a depth of at least 5 feet below existing grade and to a depth of at least 3 feet below proposed building pad subgrade elevation, whichever is greater.

Additional overexcavation should be performed within the influence zones of the new foundations, to provide for a new layer of compacted structural fill extending at least 7 feet below the foundation bearing grade in order to limit potential settlements to within tolerable limits.

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 placed below the foundation bearing grade, whichever is greater. If the proposed structure incorporates any exterior columns (such as for a canopy or overhang) the area of overexcavation should also encompass these areas.

Based on the configuration of the proposed structures with respect to the apparent property lines, it appears that the horizontal limits of overexcavation will not be achievable along any of the four property lines. Temporary shoring will be necessary. The geotechnical engineer should provide supplemental recommendations in this regard, after grading plans have been prepared. Additional considerations related to excavations adjacent to the existing improvements are presented in Section 6.4 of this report.

Following completion of the overexcavation, the subgrade soils within the building area should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed.

Based on conditions encountered at the exploratory boring locations, moist to very moist soils may be encountered at or near the base of the recommended overexcavation. Scarification and air drying of these materials may be sufficient to obtain a stable subgrade. However, if highly unstable soils are identified, and if the construction schedule does not allow for delays associated with drying, mechanical stabilization, usually consisting of coarse crushed stone and/or geotextile, may be necessary. If unstable subgrade conditions are encountered, the geotechnical engineer should be contacted for supplementary recommendations.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches and moisture conditioned or air dried to achieve a moisture content of 0 to 4 percent above the optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The building pad area may then be raised to grade, using the previous excavated on-site soils.

Based on the predicted liquefaction induced settlements that could occur at this site, it is recommended that the new structural fill incorporate geotextile reinforcement, to provide additional rigidity and thereby reduce the potential for excessive differential seismic settlements. Two layers of Mirafi RS580i, or equivalent, are recommended to be used for this reinforcement. The first layer should be placed on the approved overexcavation subgrade, which has been processed as described above. The edges of adjacent geotextile pieces should be overlapped at least 1 foot. The geotextile layer should extend across the entire overexcavation area and at least 5 feet outside the building footprint. The second layer of geotextile reinforcement should be placed 1 foot above the first layer, separated by a layer of new structural fill. The two layers of reinforcement should be placed in perpendicular directions. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Non-Building Retaining Walls and Site Walls

The existing soils within the areas of any proposed non-building retaining walls should be overexcavated to a depth of 3 feet below foundation bearing grade and replaced as compacted structural fill, as discussed above for the proposed building pad. Subgrade soils in areas of non-retaining site walls should be overexcavated to a depth of 2 feet below proposed bearing grade. In both cases, the overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Flatwork Areas

Subgrade preparation in the new flatwork areas should initially consist of the removal of all soils disturbed during stripping operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength fill 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.

Treatment of Existing Soils: At-Grade 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 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 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed parking area assume that the owner and/or developer can tolerate minor amounts of settlement within the proposed parking areas. The grading recommendations presented above do not completely mitigate the extent of the existing fill soils or lower strength alluvial soils in the parking areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking and drive areas should be overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 0 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the CBC and the grading code of the city of Brea.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

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

Utility Trench Backfill

In general, all utility trench backfill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Brea. 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 consist of sands, silty sands, sandy clays and clayey sands. These materials may be subject to minor caving within shallow excavations. Where caving does occur, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 2h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Remedial grading for the proposed structure will require excavation immediately adjacent to all four property lines. The contractor should take all necessary provisions to protect any improvements on the adjacent properties. Based on the planned development and the encountered soil conditions, it is anticipated that shoring will be required. Geotechnical shoring recommendations are included in later sections of this report.

Geotextile Reinforcement

As discussed in Section 6.3 of this report, it is recommended that geotextile reinforcement be incorporated into the new layer of structural fill. All contractors performing work on this project should be notified of the presence of this geotextile and should take appropriate precautions to avoid damaging it during later foundation or utility construction activities. The installation of the geotextile should be performed by a contractor familiar with this procedure.

Elevator Equipment Shafts

It is expected that the new apartment building will incorporate elevators. Typically, these elevators require installation of relatively large-diameter steel pipes as part of the elevator counterweights. It is expected that the pipes will be installed within slightly oversized borings. Where these pipes are installed, the annulus between the borehole wall and the elevator pipe should be backfilled with a lean concrete slurry or grout. Placement of loose backfill soils around these pipes could result in localized settlement of the structural soils and/or foundation elements.

Groundwater

The static groundwater table at this site is considered to exist at a depth of 25± feet. In addition, based on our research, the historic high groundwater table at the site is 10± feet below ground surface. Also, our corrective grading recommendations include overexcavation extending at least 7 feet beneath the proposed bearing grades. Therefore, depending on several factors including, but not limited to, the size of the foundations and the weather conditions, groundwater may impact grading or foundation construction activities. A dewatering system may be required in order to help facilitate grading and/or construction. This dewatering system would be designed and implemented by other members of the design and construction team.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by structural fill soils used to replace artificial fill soils and the upper portion of the near surface native alluvium. In the areas of the proposed building, the structural fill soils will extend at least 7 feet below foundation bearing grades, assuming that the structure will be supported on shallow foundations.

Based on the subsurface profile, it is expected that the proposed building can be supported on shallow foundations. However, this recommendation is subject to review of the grading plans and foundation loads when this information becomes available. The proposed building is partially underlain by potentially liquefiable soils. Based on these considerations, it may be desirable to support the proposed building on an alternative foundation system, such as a mat foundation or a deep foundation system. Recommendations for alternative foundation systems can be provided following review of the grading plans and foundation loads for this building. Additional subsurface exploration may be necessary in order to provide an alternative foundation design. Until such information becomes available, it is assumed that the proposed building can be supported on conventional shallow foundations.

Building Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 3,500 lbs/ft^2.
- Maximum, net allowable soil bearing pressure for wall footings constructed along the edges of the overexcavation where previously described lateral extents are not met: 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), due to the liquefaction potential of the soils at the site.
- It is recommended that any isolated column footings be structurally connected to adjacent columns and/or the perimeter foundations in both perpendicular directions using grade beams. The grade beam system should be designed by the structural engineer.
- Minimum foundation embedment: 12 inches into newly placed structural fill soils, and at least 24 inches below adjacent exterior grade.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind loads. However, **the bearing pressure may not be increased when considering seismic loads due to the liquefaction potential of the soils at the subject site**. The minimum steel reinforcement recommended above is based on standard geotechnical practice, given the magnitude of predicted liquefaction-induced settlements, and the structure type proposed for this site. 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.

Foundation Construction

It is 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 engineered fill soils, compacted at least 90 percent relative compaction. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

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

Estimated Foundation Settlements

Post-construction total and differential settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively, under static conditions. 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.

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: 280 lbs/ft^3
- Friction Coefficient: 0.27

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

- Minimum slab thickness: 6 inches.

- Minimum slab reinforcement: Minimum slab reinforcement: No. 4 bars at 16 inches on-center, in both directions, due to the expansive potential and 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 then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of the proposed slab. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

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

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

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

6.7 Concrete Flatwork Design and Construction

Presented below are recommendations for flatwork which will be subject only to pedestrian traffic. Based on the results of the laboratory testing presented in Section 5.0 of this report, the on-site soils possess a very low to low expansion potential. The concrete flatwork should incorporate the following characteristics:

- Concrete Thickness: 4 inches

- Reinforcement: No. 4 Bars at 18 inches on center in both directions.

- Subgrade Preparation: Compact all flatwork subgrade soils to 90 percent of the ASTM D-1557 maximum dry density.

- Where the flatwork is adjacent to a landscape planter or another area with exposed soil, it should incorporate a turned down edge. This turned down edge should be at least 12 inches in depth and 6 inches in width. The turned down edge should incorporate longitudinal steel reinforcement consisting of at least one No. 4 bar.

- Flatwork which is constructed immediately adjacent to the new structure should be dowelled into the perimeter foundations in a manner determined by the structural engineer.

These recommendations are contingent upon additional expansion index testing being conducted at the completion of rough grading, to verify the actual expansion potential of the flatwork subgrade soils.

6.8 Retaining Wall Design and Construction

It is expected that retaining walls will be required around the perimeter of the proposed subterranean parking garage and maintenance room levels. Based on information from the architect, these subterranean walls will be up to 10 to 12± feet in height. The parameters recommended for use in the design of these walls are presented below:

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near surface soils generally consist of silty sands, clayey sands and sandy clays. Based on the results of direct shear testing, included in Appendix C of this report, these soils possess a friction angle of at least 28 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 Soils
Internal Friction Angle (ϕ)	28°
Unit Weight	120 lbs/ft^3

Equivalent Fluid Pressure:	Active Condition (level backfill)	43 lbs/ft^3
	Active Condition (2h:1v backfill)	76 lbs/ft^3
	At-Rest Condition (level backfill)	64 lbs/ft^3

Regardless of the backfill type, the walls should be designed using a soil-footing coefficient of friction of 0.27 and an equivalent passive pressure of 280 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 pressures 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, such as the perimeter walls of the below-grade parking garage and maintenance room levels.**

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

Seismic Lateral Earth Pressures

In accordance with the 2016 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations. The seismic earth pressures for this project have been developed utilizing the Mononabe-Okabe Method in accordance with guidance published by the Structural Engineers Association of California (SEAOC) in their 2010 Convention Proceedings.

The recommended seismic pressure distribution on the basement walls is triangular in shape, with a maximum magnitude of 24H lbs/ft^2, where H is the overall height of the wall. The maximum pressure should be assumed to occur at the base of the wall, decreasing to 0 at the top of the wall. The peak ground acceleration for the subject site is 0.795g, in accordance with ASCE 7-10 Section 11.8.3. The seismic pressure distribution is based on the Mononobe-Okabe equation, utilizing a design acceleration of 0.463g, which is equal to 0.58 times the peak ground acceleration, in accordance with the referenced SEAOC document. In calculating the total pressure exerted on the below grade walls (static plus seismic) during a seismic event, the seismic lateral earth pressure should be added to the active earth pressure, not the at-rest static earth pressure.

Surcharge Loads

The subterranean retaining walls should be designed to withstand lateral pressures due to traffic surcharges and surcharge loads from the residential building foundations. A traffic surcharge load of 250 lbs/ft^2 should be applied to the subterranean retaining wall adjacent to Berry Street and

Mercury Lane and a traffic surcharge of 100 lbs/ft^2 should be applied anywhere else traffic is anticipated. Additional surcharge loads, as determined by the architect or structural engineer may also be appropriate to account for heavily loaded areas, including the residential building foundations, adjacent to the subterranean walls.

Retaining Wall Foundation Design

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

Backfill Material

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 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 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.

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

In addition, groundwater seeping into the parking garage through the walls or floors will most likely be viewed as a major nuisance. A subsurface drainage system for the retaining walls should be designed and constructed to eliminate the development of pore water pressure behind the basement walls as well as to eliminate seepage of water into the parking garage and mechanical room level retaining walls. It is anticipated that this subsurface dewatering system will include one or more sump/pumps which properly and adequately drain to one or more appropriate above-ground surface outlet/s or storm drain outlet/s. The design and selection of the subsurface drainage system for the below-grade walls is outside the scope of our report.

6.9 Temporary Shoring Recommendations

Shoring will be required during grading and/or foundation construction activities. The following recommendations assume that the retained soil heights will not exceed 15± feet. If surcharge loads are located within 15± feet of the shoring, the effect of these loads upon the shoring system must be considered by the shoring engineer. The shoring should be designed to withstand the effects of nearby surcharge loads, including traffic from the two adjacent streets, and any construction loads or traffic.

Lateral Earth Pressures

It is assumed that the soil behind the shoring system will be relatively level. It is assumed that the shoring will consist of either sheet piles or soldier piles and lagging. The shoring may be a braced design or a cantilever design. Plate 3, enclosed in Appendix A of this report, illustrates the lateral earth pressure distributions for both cantilevered shoring and restrained (braced) shoring. The earth pressures shown on Plate 3 are based on static conditions. As discussed previously, if surcharge loads are imposed upon the shoring, they must be considered by the shoring engineer. This should include surcharges related to automobile traffic for the shoring systems adjacent to Berry Street and Mercury Lane, and for any construction equipment. A traffic surcharge load of 250 lbs/ft^2 should be applied to shoring adjacent to Berry Street and Mercury Lane and a traffic surcharge of 100 lbs/ft^2 should be applied to shoring adjacent to anywhere else traffic is anticipated. A construction surcharge of 75 lbs/ft^2 should be used elsewhere, as required. These loads assume normal construction traffic, consisting of lightly loaded vehicles and storage of small amounts of materials. If large stockpiles of soil, concentrated pallet loads, or crane loads are expected, SCG should be contacted for additional surcharge load recommendations. The passive resistance value of the soil below the level of excavation may be assumed to be 330 lbs/ft^2, per foot of depth.

Shoring Construction

If soldier piles are utilized, they should be spaced no closer than 3 times the nominal soldier pile diameter. The contractor should take all necessary provisions to assure firm contact between the retained soils and the shoring system. A 2-sack cement slurry may be used to fill voids where inadequate contact between the shoring system and the retained soils are observed.

Since the shoring system will be designed as a cantilever wall, some deflection will occur. In order to develop the full active pressure, a deflection of at least ½ to 1½± inches is expected to occur at the top of the shoring system. The design of the shoring system as well as the protection of adjacent improvements should take this deflection into consideration

6.10 Pavement Design Parameters

Site preparation in these pavement areas 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 majority of the planned non-basement parking and drive areas at the subject site will be supported by the concrete deck above the subterranean parking garage. It is anticipated that any new at-grade pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing fill or near-surface alluvial soils. The near-surface on-site soils generally consist of silty sands, clayey sands and sandy clays. Based on their classification, these materials are expected to possess fair to good pavement support characteristics, with R-values ranging from 20 to 30. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon a conservatively assumed R-value of 20. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

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

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

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

ASPHALT PAVEMENTS (R = 20)				
Materials	**Thickness (inches)**			
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic	
			(TI = 6.0)	(TI = 7.0)
Asphalt Concrete	3	3	3½	4
Aggregate Base	5	8	10	12
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)**		
	Auto Parking & Drives (TI = 5.0)	Truck Traffic	
		(TI =6.0)	(TI =7.0)
PCC	5	6	7
Compacted Subgrade	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. 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.

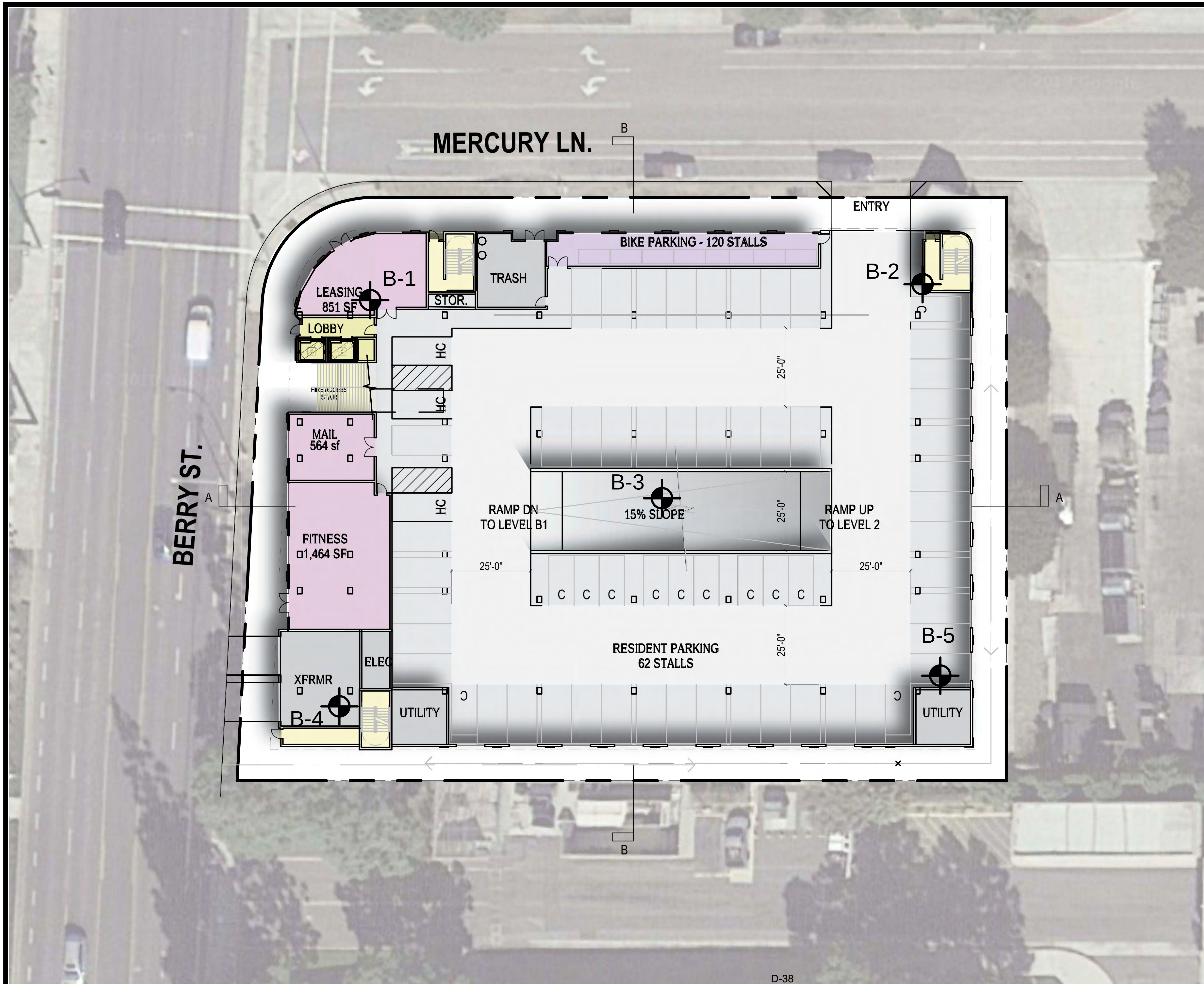
Tokimatsu, K. and Yoshimi, Y., "*Empirical Correlations of Soil Liquefaction Based on SPT N-value and Fines Content,"* Seismological Research Letters, Eastern Section Seismological Society Of America, Volume 63, Number 1, p. 73.

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: THOMAS GUIDE, 2013

SITE LOCATION MAP	
PROPOSED MULTI-UNIT RESIDENTIAL DEVELOPMENT	
BREA, CALIFORNIA	
SCALE: 1" = 2400'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JLH CHKD: RGT	
SCG PROJECT 18G118-1	
PLATE 1	



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

NOTE: SITE PLAN PREPARED BY HUMPHREYS & PARTNERS ARCHITECTS, L.P.

BORING LOCATION PLAN - FIRST LEVEL

PROPOSED MULTI-UNIT RESIDENTIAL DEVELOPMENT

BREA, CALIFORNIA

SCALE: 1" = 30'

DRAWN: JLL
CHKD: RGT

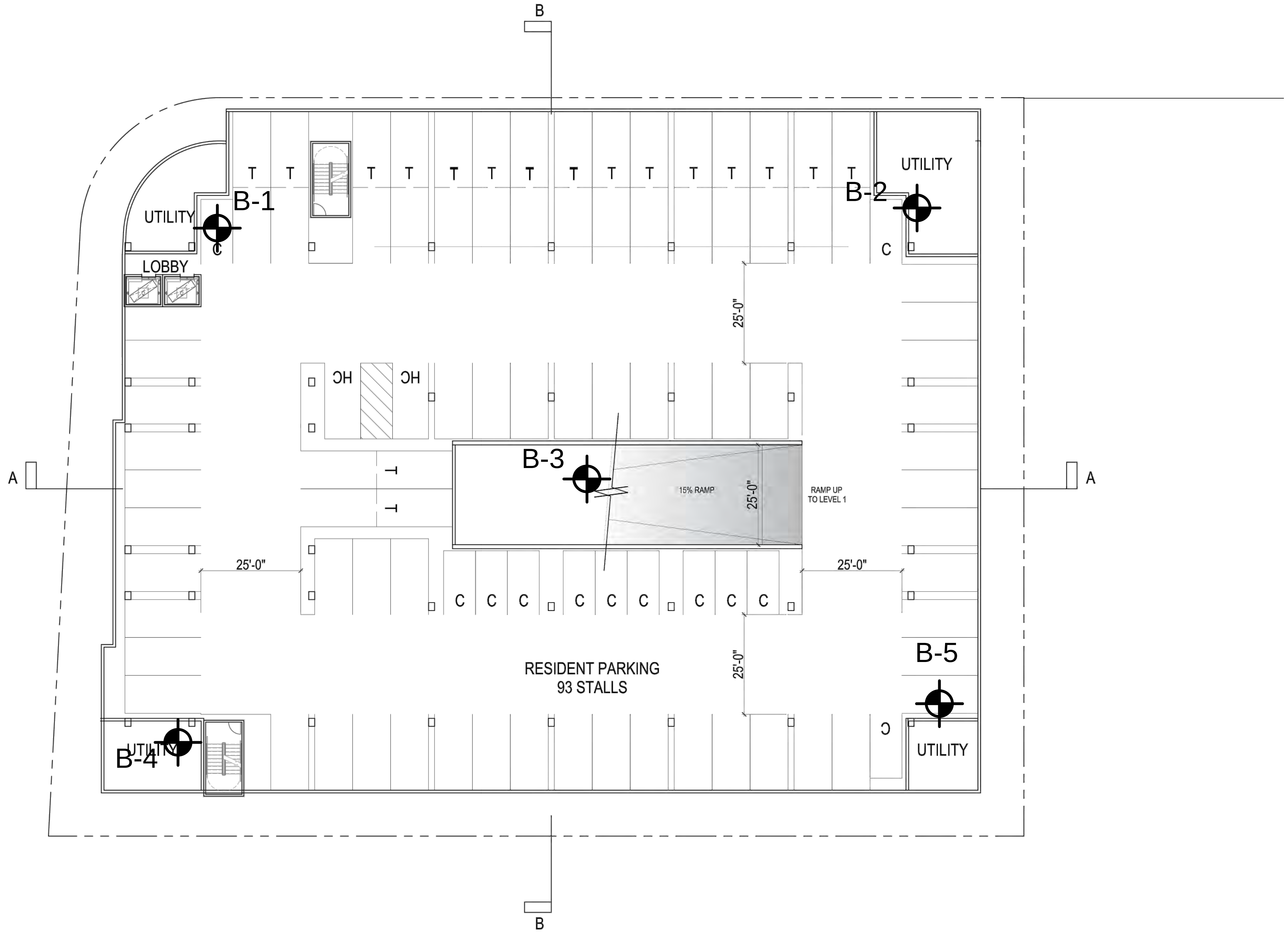
SCG PROJECT
18G118-1

PLATE 2A

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION



NOTE: SITE PLAN PREPARED BY HUMPHREYS & PARTNERS ARCHITECTS, L.P.

BORING LOCATION PLAN - BASEMENT LEVEL

PROPOSED MULTI-UNIT RESIDENTIAL DEVELOPMENT

BREA, CALIFORNIA

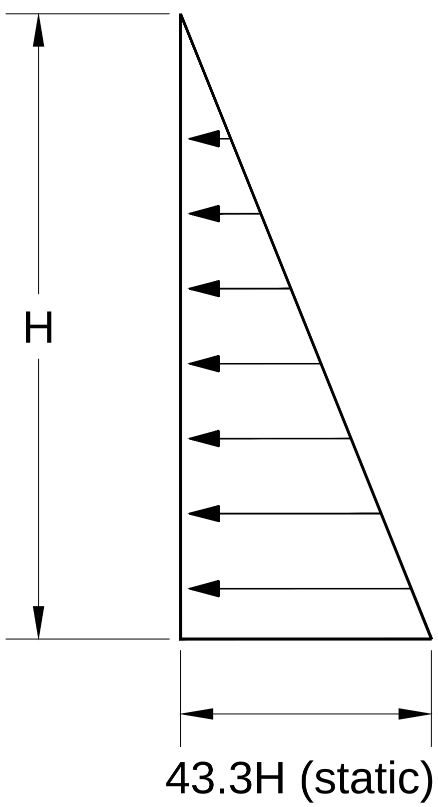
SCALE: 1" = 30'

DRAWN: JLL
CHKD: RGT

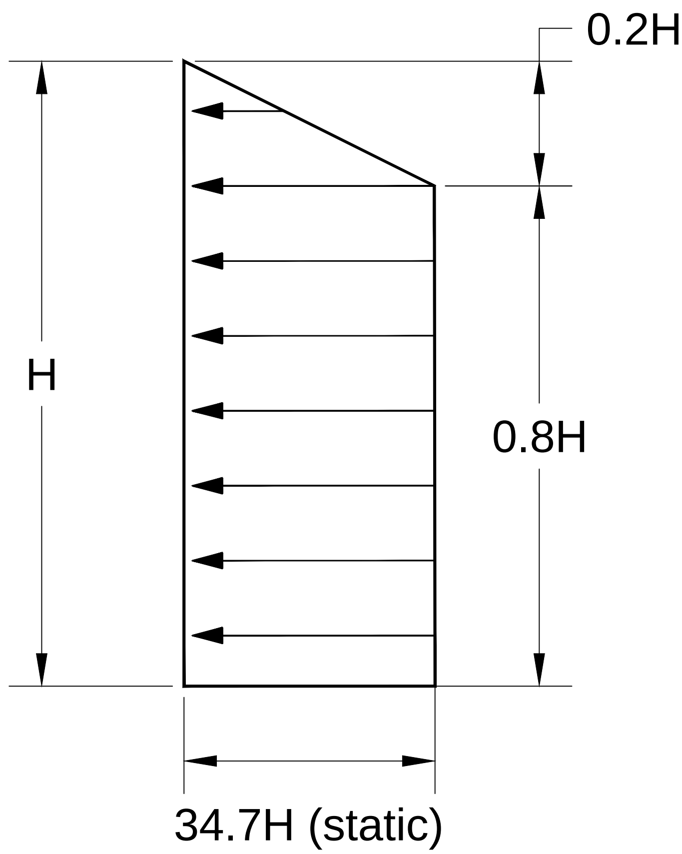
SCG PROJECT
18G118-1

PLATE 2B

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



CANTILEVERED SHORING



BRACED SHORING

NOTES: THE LATERAL EARTH PRESSURES DEPICTED IN THIS DIAGRAM DO NOT INCLUDE ANY SURCHARGE LOADS.

IN ADDITION TO THE STATIC LOAD, A SEISMIC LOAD SHOULD BE APPLIED FOR PERMANENT SHORING.

PRESSURE DIAGRAM FOR SHORING

NOT TO SCALE

DRAWN: AL
CHKD: RGT

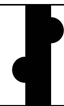
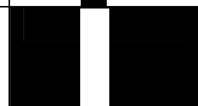
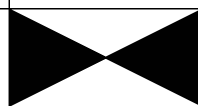
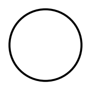
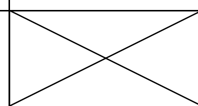
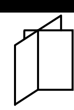
SCG PROJECT
18G118-1

PLATE 3

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.: 18G118
PROJECT: Proposed MFR Development
LOCATION: Brea, California

DRILLING DATE: 3/20/18
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 27 feet
CAVE DEPTH: ---
READING TAKEN: 1.5 hrs

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
				6± inches Aggregate base							
	X	16		FILL: Dark Brown Silty fine Sand, trace medium to coarse Sand, trace Clay, Brick fragments, fine to coarse Gravel, loose to medium dense-moist	106	11					
	X	9			89	10					
5	X	12		ALLUVIUM: Brown Silty fine Sand, trace calcareous veining, loose-moist	93	9					
	X	16	4.5+	Brown fine Sandy Clay, trace medium to coarse Sand, fine Gravel, some calcareous veining, very stiff-moist to very moist	101	15					
10	X	18	4.5+		90	13					
				Brown Silty fine Sand, trace Clay, slightly porous, calcareous veining, medium dense-moist							
15	X	22			88	6					
				Brown Gravelly fine to coarse Sand, trace Clay clasts, medium dense-damp							
20	X	31			102	3					EI = 2 @ 20 to 25 feet
	X	43			105	4					
	X	18		Brown Silty fine Sand, trace fine Gravel, trace calcareous veining, medium dense-moist	102	11					
25	X	44		Brown Gravelly fine to coarse Sand, some Cobbles, Iron oxide staining, medium dense-moist to wet	112	8					
	X	39		@ 27 feet, Water encountered during drilling	88	8					
30	X	32			107	10					
				Dark Brown Clayey fine Sand to fine Sandy Clay, medium dense to stiff-wet							
	X	19	1.5		94	29					

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

TEST BORING LOG

PLATE B-1a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 18G118
PROJECT: Proposed MFR Development
LOCATION: Brea, California

DRILLING DATE: 3/20/18
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 27 feet
CAVE DEPTH: ---
READING TAKEN: 1.5 hrs

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Dark Brown Clayey fine Sand to fine Sandy Clay, medium dense to stiff-wet							
					Brown fine to coarse Sand, trace to little Gravel, some Cobbles, trace Clay clasts, medium dense-wet							
40	X	28				98	26					
45	X	40				106	18					
					Brown Clayey fine Sand to fine Sandy Clay, trace medium Sand, trace calcareous veining, trace medium to coarse Sand, some fine Gravel, dense to hard-very moist							
50	X	49	4.5+			117	13					
55	X	24	2.5			116	16					
					Brown fine to coarse Sand, little fine Gravel, some Cobbles, trace Clay clasts, dense to very dense-wet							
60	X	65				108	16					
65	X	77				114	14					
					Brown Silty Clay, trace to little fine Sand, very stiff-very moist							
	X	24	4.0			99	25					

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 18G118
PROJECT: Proposed MFR Development
LOCATION: Brea, California

DRILLING DATE: 3/20/18
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 27 feet
CAVE DEPTH: ---
READING TAKEN: 1.5 hrs

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
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)	
					Brown Silty Clay, trace to little fine Sand, very stiff-very moist							
75		26	3.0			106	22					
					Boring Terminated at 75'							

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

TEST BORING LOG

PLATE B-1c

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 18G118
PROJECT: Proposed MFR Development
LOCATION: Brea, California

DRILLING DATE: 3/20/18
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 27 feet
CAVE DEPTH: 57 feet
READING TAKEN: 30 mins

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					5± inches Aggregate base							
	X	14	4.5+		ALLUVIUM: Brown fine Sandy Clay, slightly porous, trace calcareous veining and nodules, very stiff-moist to very moist		11					
	X	15	4.5+				11					
5												
	X	13	4.5+				15					
	X	13			ALLUVIUM: Brown Clayey fine Sand, slighlty porous, some calcareous veining, medium dense-moist to very moist		12					
10												
					Light Brown fine Sand, trace medium to coarse Sand, medium dense-dry							
	X	22					2					
15												
					Brown fine to coarse Sand, trace Silt, Clay, fine Gravel, occasional Cobbles, medium dense-damp to moist							
	X	26					5			10		
20												
	X	24					10			15		
25												
					Brown Clayey fine Sand, trace fine Gravel, medium dense-wet @ 27 feet, Water encountered during drilling							
	X	15					23			31		
30												
					Brown Clayey fine to coarse Sand, trace fine Gravel, trace Silt, dense-wet							
	X	32					23					

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

TEST BORING LOG

PLATE B-2a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 18G118
PROJECT: Proposed MFR Development
LOCATION: Brea, California

DRILLING DATE: 3/20/18
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 27 feet
CAVE DEPTH: 57 feet
READING TAKEN: 30 mins

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Brown Clayey fine to coarse Sand, trace fine Gravel, trace Silt, dense-wet							
					Brown fine to medium Sand, dense-wet							
40	X	42					23					
					Brown fine to medium Sand, trace coarse Sand, some fine Gravel, trace Clay, dense-wet							
45	X	44					17					
					Brown fine Sand, trace medium to coarse Sand, trace fine Gravel, trace Clay, medium dense-wet							
50	X	22					18			14		
					Brown fine Sandy Clay with occasional fine to medium Sand lenses, some calcareous veining, very stiff-very moist to wet							
55	X	29					22					
					Brown fine to medium Sand, little Clay, trace coarse Sand, fine Gravel, dense-wet							
60	X	34					16					
					Brown Clayey fine Sand to fine Sandy Clay, trace medium Sand, medium dense to very stiff-very moist to wet							
65	X	20	2.5				20					
					Thinly interbedded lenses of Brown Clayey fine Sand and Silty Clay, trace fine Gravel, trace calcaroeus veining, medium dense to stiff-very moist to wet							
	X	18	1.5				29					

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

TEST BORING LOG

PLATE B-2b

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 18G118	DRILLING DATE: 3/20/18	WATER DEPTH: 27 feet
PROJECT: Proposed MFR Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 57 feet
LOCATION: Brea, California	LOGGED BY: Jason Hiskey	READING TAKEN: 30 mins

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Thinly interbedded lenses of Brown Clayey fine Sand and Silty Clay, trace fine Gravel, trace calcaroeus veining, medium dense to stiff-very moist to wet							
	X	34	4.0				17					
75												
					Boring Terminated at 75'							

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

TEST BORING LOG

PLATE B-2c

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 18G118
PROJECT: Proposed MFR Development
LOCATION: Brea, California

DRILLING DATE: 3/20/18
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 27 feet
CAVE DEPTH: 70 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
			FILL: Brown Silty fine Sand, some Asphaltic concrete fragments, fine to coarse Gravel, medium dense-damp							
	32			108	4					
			ALLUVIUM: Brown fine Sandy Clay, some calcareous veining, slightly porous, stiff to very stiff-damp to moist							
	9	1.0		84	8					
5	13	3.5		82	11					
	31	4.5+		104	13					
			Brown Clayey fine Sand, slightly porous, slightly cemented, little calcareous veining, medium dense-damp to moist							
10	20			91	9					
15	27			97	6					
			Brown fine to coarse Sand, occasional Cobbles, some fine Gravel, some Clay clasts, medium dense-damp							
20	37			105	5					
25	28			110	5					
			Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, trace Clay, medium dense-wet @ 27 feet, Water encountered during drilling							
30	17				23					
			Brown fine Sand, medium dense-wet							
	23			98	23					

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

TEST BORING LOG

PLATE B-3a

BORING NO. B-3

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 18G118
PROJECT: Proposed MFR Development
LOCATION: Brea, California

DRILLING DATE: 3/20/18
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 27 feet
CAVE DEPTH: 70 feet
READING TAKEN: At Completion

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
					Brown fine Sand, medium dense-wet							
					Brown fine to coarse Sand, occasional Cobbles, trace Clay clasts, trace fine Gravel, medium dense to dense-wet							
		20					23					
40												
		51				116	14					
45												
					Gray to Brown fine Sandy Clay with occasional thinly interbedded fine Sand lenses, some Iron oxide staining, very stiff-very moist							
		22					20					
50												
					Brown Clayey fine Sand, trace medium to coarse Sand, fine Gravel, medium dense-very moist							
		38				116	14					
55												
					Brown fine to coarse Sand, trace Clay clasts, trace fine Gravel, occasional Cobbles, dense-wet							
		42					19					
60												
		62										No Sample Recovered
65												
					Brown Silty Clay, trace calcareous veining, very stiff-very moist to wet							
		20					25					

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

TEST BORING LOG

PLATE B-3b

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 18G118
PROJECT: Proposed MFR Development
LOCATION: Brea, California

DRILLING DATE: 3/20/18
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 27 feet
CAVE DEPTH: 70 feet
READING TAKEN: At Completion

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
					Brown Silty Clay, trace calcareous veining, very stiff-very moist to wet							
					Interbedded lenses of Dark Brown fine to coarse Sand and Brown fine Sandy Clay, occasional Cobbles, medium dense to stiff-wet							
75	X	18				111	18					
					Boring Terminated at 75'							

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 18G118
PROJECT: Proposed MFR Development
LOCATION: Brea, California

DRILLING DATE: 3/20/18
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 27 feet
CAVE DEPTH: 60 feet
READING TAKEN: 30 mins

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					FILL: Brown Silty fine Sand, little Clay, trace medium to coarse Sand, fine Gravel, medium dense-moist							
	X	10					11					
					ALLUVIUM: Brown Silty fine Sand to fine Sandy Clay, trace medium Sand, some calcareous veining, moderately cemented, slightly porous, medium dense to very stiff-moist							
	X	14	4.5+				10					
5												
	X	12	4.5+				12					
	X	12	4.5+				12					
10												
					Brown Silty fine Sand, trace calcareous veining, medium dense-moist							
	X	20					8					
15												
	X	14					8			36		
20												
	X	14			@ 23½ to 25 feet, very moist		21			32		
25												
					Brown fine to medium Sand, trace coarse Sand, little Silt, fine Gravel, trace Clay clasts, medium dense-wet @ 27 feet, Water encountered during drilling							
	X	18					22			9		
30												
					Brown Clayey fine Sand to fine Sandy Clay with a 2-inch thick lense of Gravelly fine to coarse Sand, medium dense to very stiff-wet							
	X	26					29					

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 18G118	DRILLING DATE: 3/20/18	WATER DEPTH: 27 feet
PROJECT: Proposed MFR Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 60 feet
LOCATION: Brea, California	LOGGED BY: Jason Hiskey	READING TAKEN: 30 mins

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Brown Clayey fine Sand to fine Sandy Clay with a 2-inch thick lense of Gravelly fine to coarse Sand, medium dense to very stiff-wet							
					Gray Brown fine to coarse Sand, occasional Cobbles, trace fine to coarse Gravel, dense-wet							
	X	33					18					
40												
	X	38					18					
45												
					Brown Clayey fine Sand, trace medium to coarse Sand, fine Gravel, medium dense to dense-very moist to wet							
	X	39					18					
50												
	X	20					19					
55												
					Gray Brown fine to coarse Sand, some Cobbles, some fine Gravel, dense-wet							
	X	37					17					
60												
					Brown fine Sandy Clay, very stiff-very moist to wet							
	X	17	3.0				22					
65												
					Interbedded lenses of Brown fine to coarse Sand and fine to medium Sandy Clay, little fine Gravel, occasional Cobbles, very dense to hard-wet							
	X	50					23					

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

TEST BORING LOG

PLATE B-4b

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 18G118
PROJECT: Proposed MFR Development
LOCATION: Brea, California

DRILLING DATE: 3/20/18
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 27 feet
CAVE DEPTH: 60 feet
READING TAKEN: 30 mins

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Interbedded lenses of Brown fine to coarse Sand and fine to medium Sandy Clay, little fine Gravel, occasional Cobbles, very dense to hard-wet							
	X	89/10"					22					
75												
					Boring Terminated at 75'							

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

TEST BORING LOG

PLATE B-4c

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 18G118
PROJECT: Proposed MFR Development
LOCATION: Brea, California

DRILLING DATE: 3/20/18
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 25 feet
CAVE DEPTH: 54 feet
READING TAKEN: 30 mins

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 Aggregate base							
		27	4.5+		ALLUVIUM: Brown Clayey fine Sand to fine Sandy Clay, trace to some calcareous veining, slightly to moderately porous, trace medium Sand, medium dense to very stiff-damp	102	5					
		18	4.5+			89	12					
5		20	3.0			85	10					
		15	4.5+			86	11					
		15			Brown Clayey fine Sand, slightly porous, medium dense-damp	97	5					
10												
		15				96	6					
15												
					Brown Silty fine Sand, trace medium Sand, medium dense-moist to very moist							
		16				87	15					
20												
					Gray Brown to Brown fine to medium Sand, trace Clay, trace coarse Sand, trace fine meduim dense-wet							
		18				104	12					
25					@ 25 feet, Water encountered during drilling							
					Brown fine to coarse Sand, some fine to coarse Gravel, trace Clay lenses and clasts, medium dense-wet							
		37				106	16					
30												
		27				113	15					
		23				114	13					
			4.5+		Gray Brown Clayey fine Sand to fine Sandy Clay, trace fine Gravel, trace Iron oxide staining, medium dense to very stiff-very							

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

TEST BORING LOG

PLATE B-5a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 18G118
PROJECT: Proposed MFR Development
LOCATION: Brea, California

DRILLING DATE: 3/20/18
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 25 feet
CAVE DEPTH: 54 feet
READING TAKEN: 30 mins

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
		22	1.5		moist	97	26					
		34			Brown fine to coarse Sand, some fine Gravel, trace Clay clasts, medium dense to very dense-wet	109	22					
40		38				114	15					
45		89/10"				113	15					
					Interbedded lenses of Brown fine Sandy Clay and Silty fine to medium Sand, dense to hard-wet							
50		60				108	19					
					Brown fine to coarse Sand, trace fine Gravel, trace Clay clasts, occasional Cobbles, dense-wet							
55		66				120	11					
60		51				106	17					
					Brown Clayey fine Sand, trace coarse Sand, dense-very moist							
65		48				111	18					
					Brown fine Sand, trace Clay clasts, dense to very dense-wet							
		61				112	15					

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

TEST BORING LOG

PLATE B-5b

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 18G118	DRILLING DATE: 3/20/18	WATER DEPTH: 25 feet
PROJECT: Proposed MFR Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 54 feet
LOCATION: Brea, California	LOGGED BY: Jason Hiskey	READING TAKEN: 30 mins

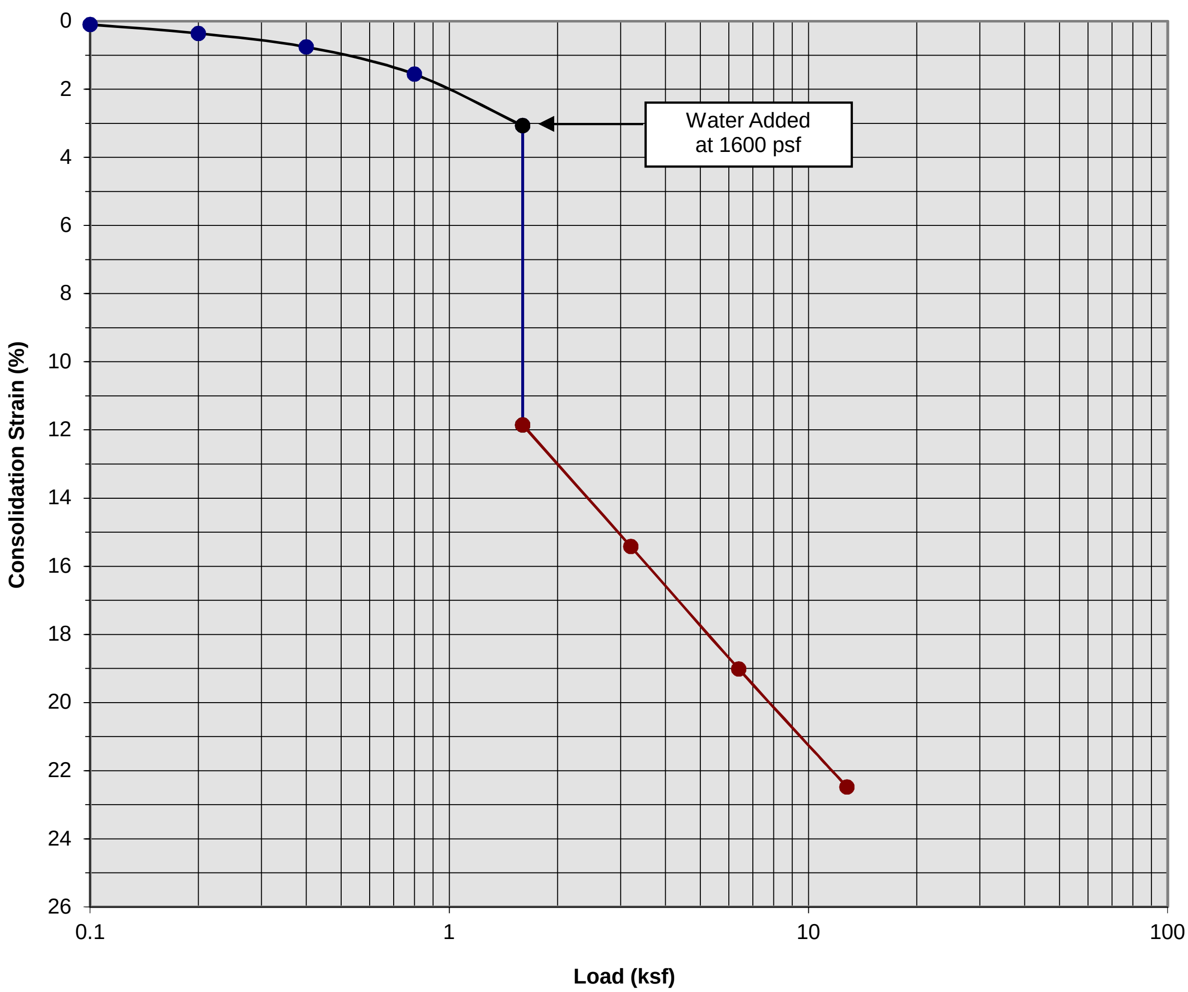
FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Brown fine Sand, trace Clay clasts, dense to very dense-wet							
75	X	69				115	17					
					Boring Terminated at 75'							

TBL 18G118.GPJ SOCALGEO.GDT 4/20/18

TEST BORING LOG **PLATE B-5c**

APPENDIX C

Consolidation/Collapse Test Results

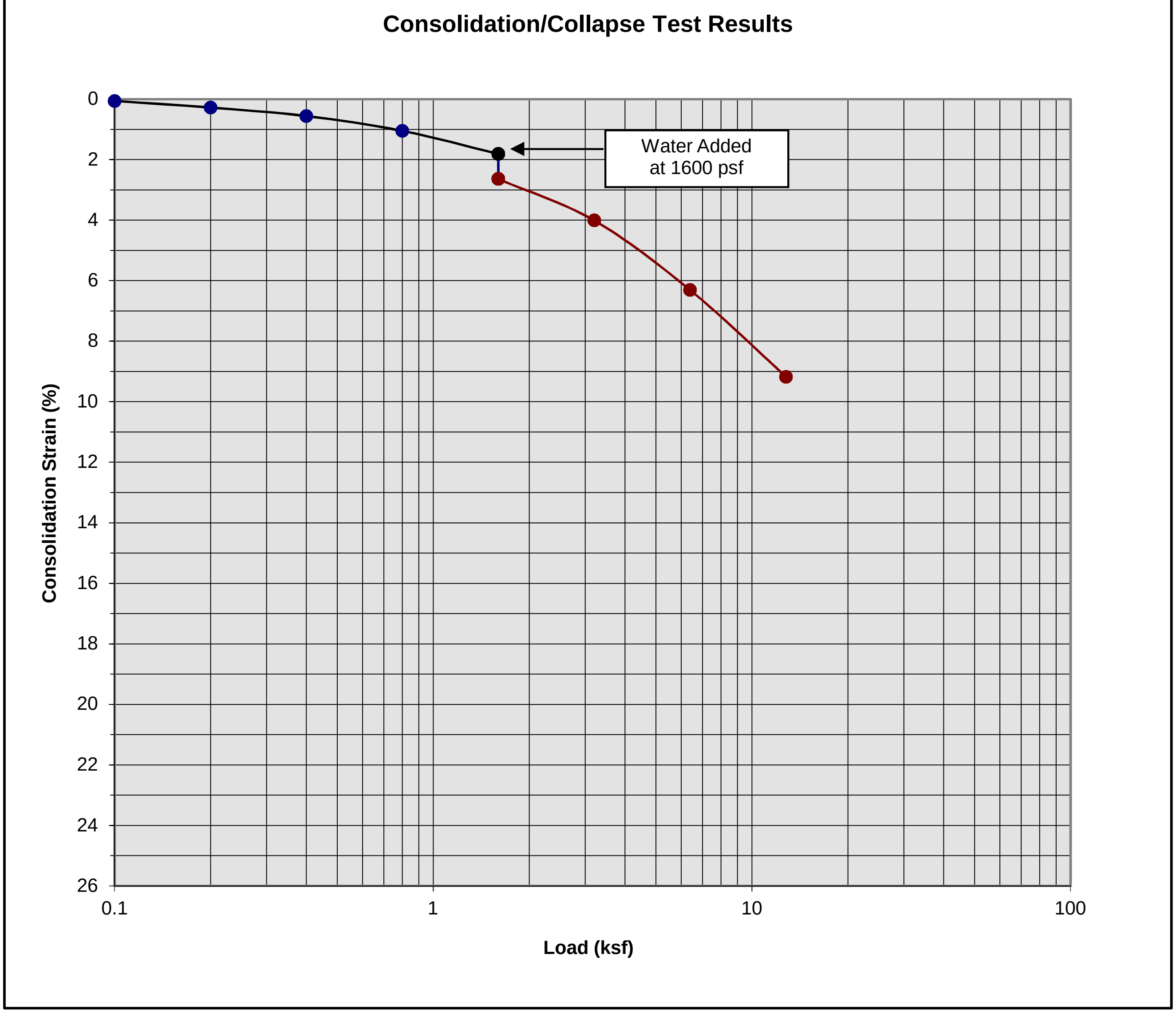


Classification: Brown Silty fine Sand

Boring Number:	B-1	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	5 to 6	Initial Dry Density (pcf)	92.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	118.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	8.78

Proposed Multi-Unit Residential Development
Brea, California
Project No. 18G118
PLATE C- 1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

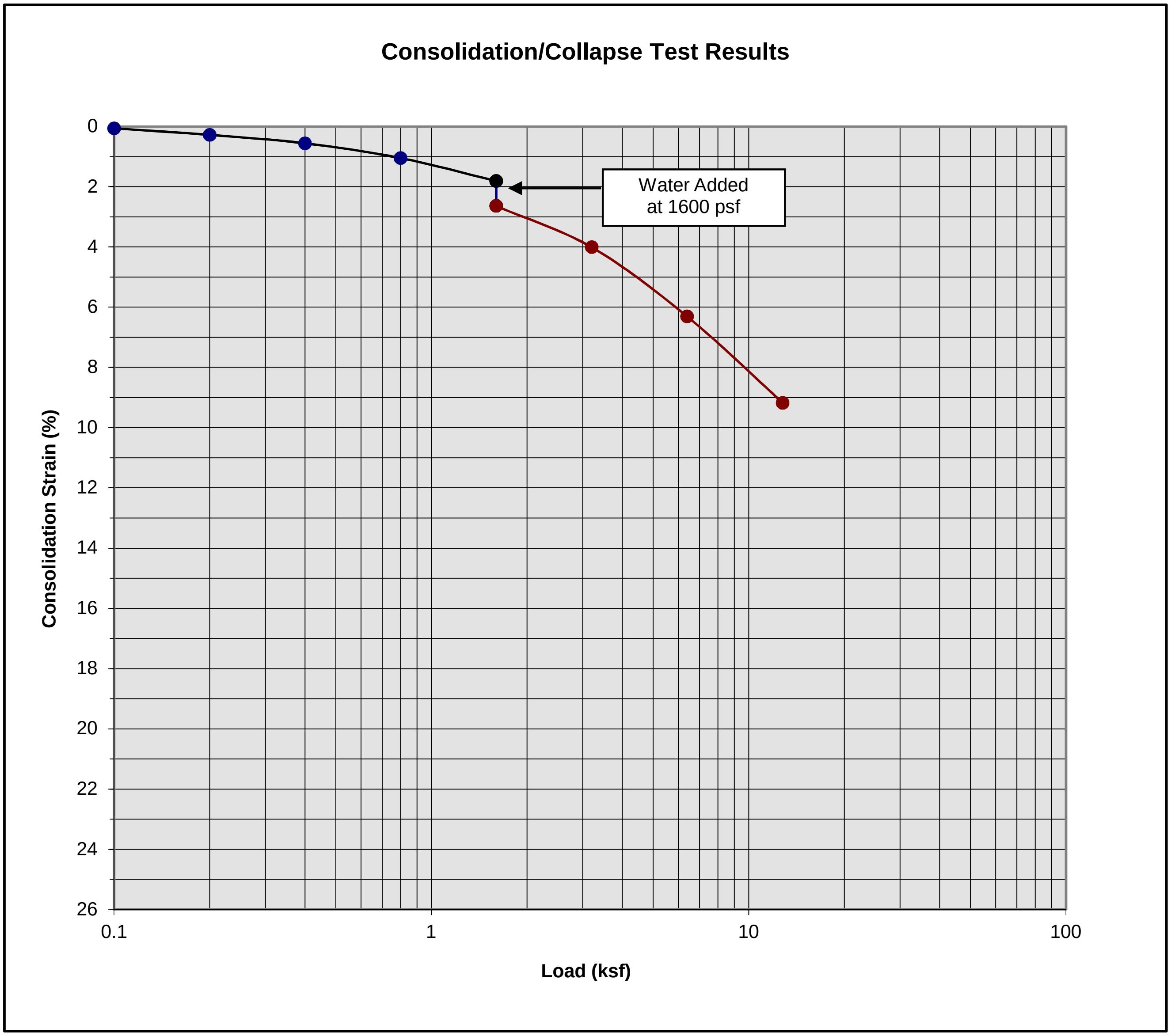


Classification: Brown fine Sandy Clay, trace medium to coarse Sand

Boring Number:	B-1	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	7 to 8	Initial Dry Density (pcf)	100.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	110.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.83

Proposed Multi-Unit Residential Development
Brea, California
Project No. 18G118
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

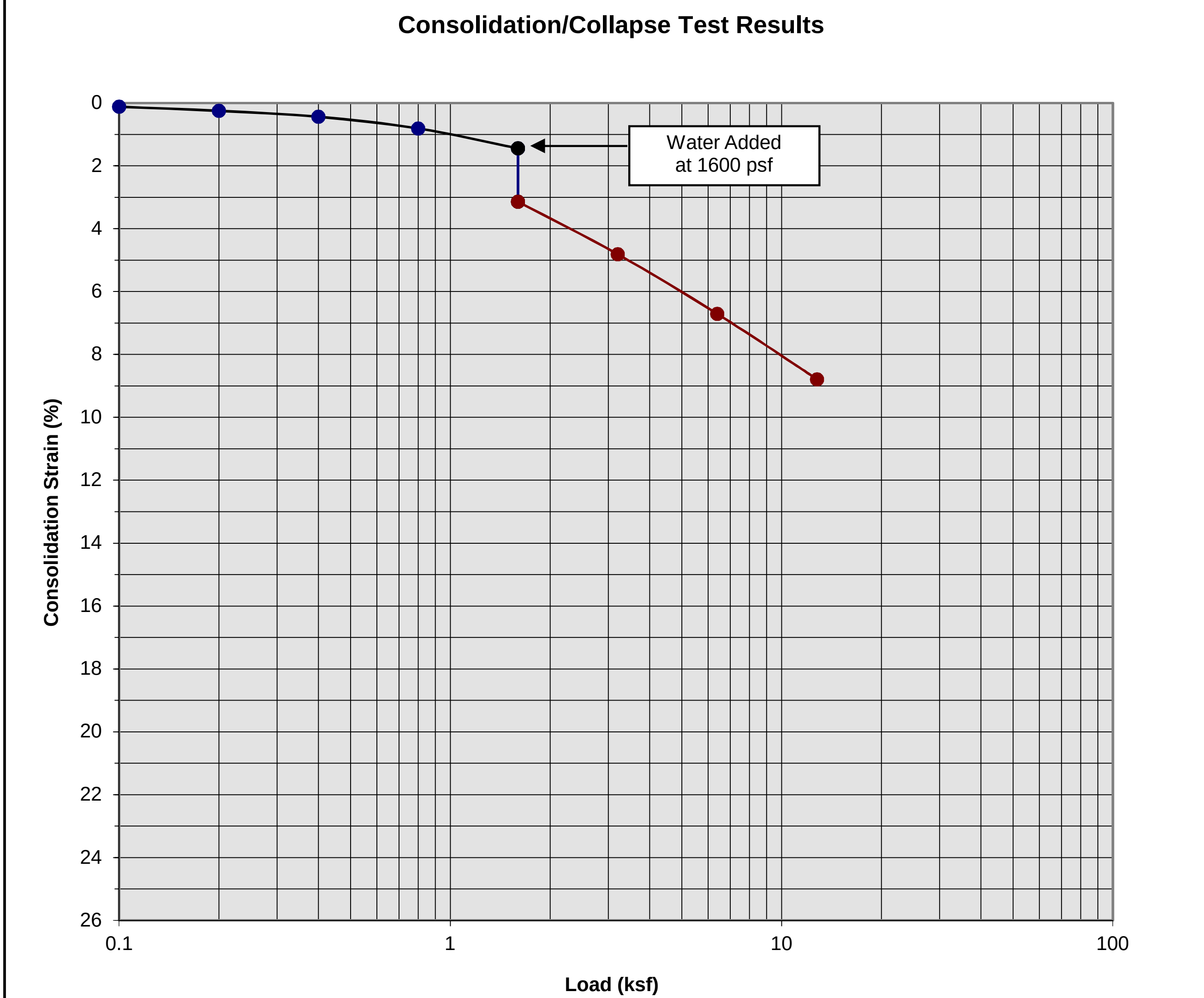


Classification: Brown fine Sandy Clay, trace medium to coarse Sand

Boring Number:	B-1	Initial Moisture Content (%)	13
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	9 to 10	Initial Dry Density (pcf)	88.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	96.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.83

Proposed Multi-Unit Residential Development
Brea, California
Project No. 18G118
PLATE C- 3

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



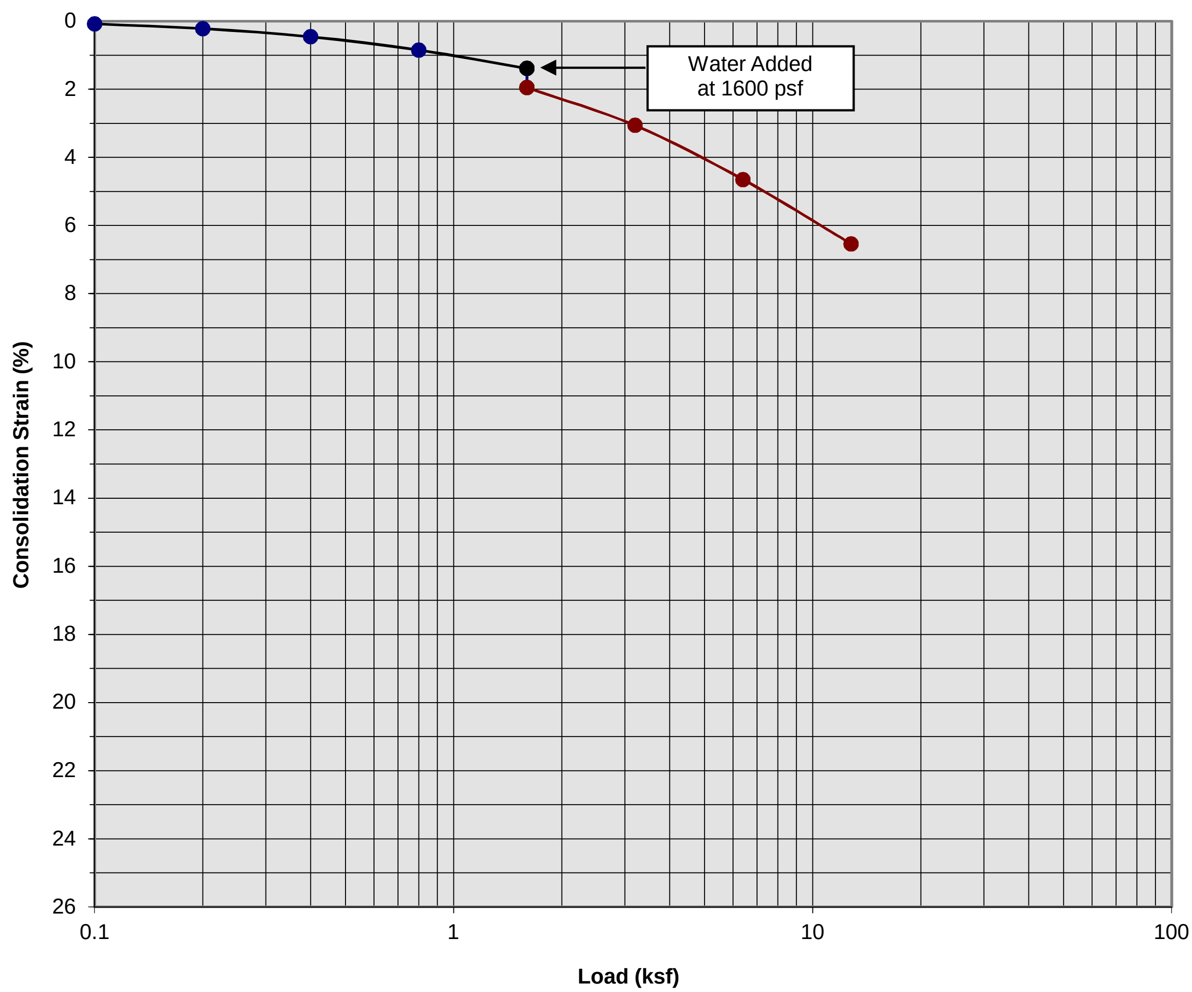
Classification: Brown Gravelly fine to coarse Sand, trace Clay clasts

Boring Number:	B-1	Initial Moisture Content (%)	4
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	21 to 22	Initial Dry Density (pcf)	105.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.70

Proposed Multi-Unit Residential Development
Brea, California
Project No. 18G118
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



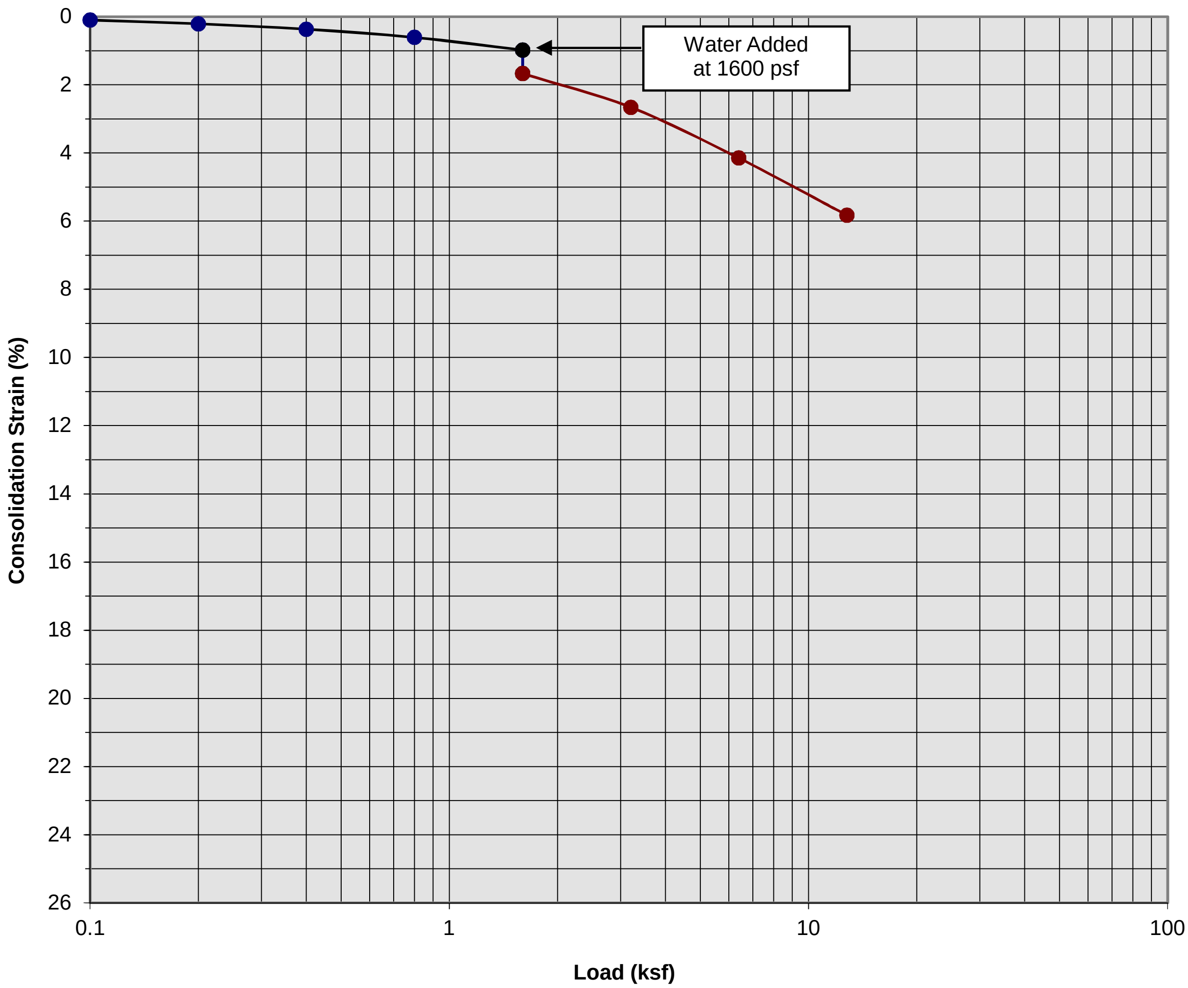
Classification: Brown Silty fine Sand, trace fine Gravel

Boring Number:	B-1	Initial Moisture Content (%)	12
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	23 to 24	Initial Dry Density (pcf)	101.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.56

Proposed Multi-Unit Residential Development
Brea, California
Project No. 18G118
PLATE C- 5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

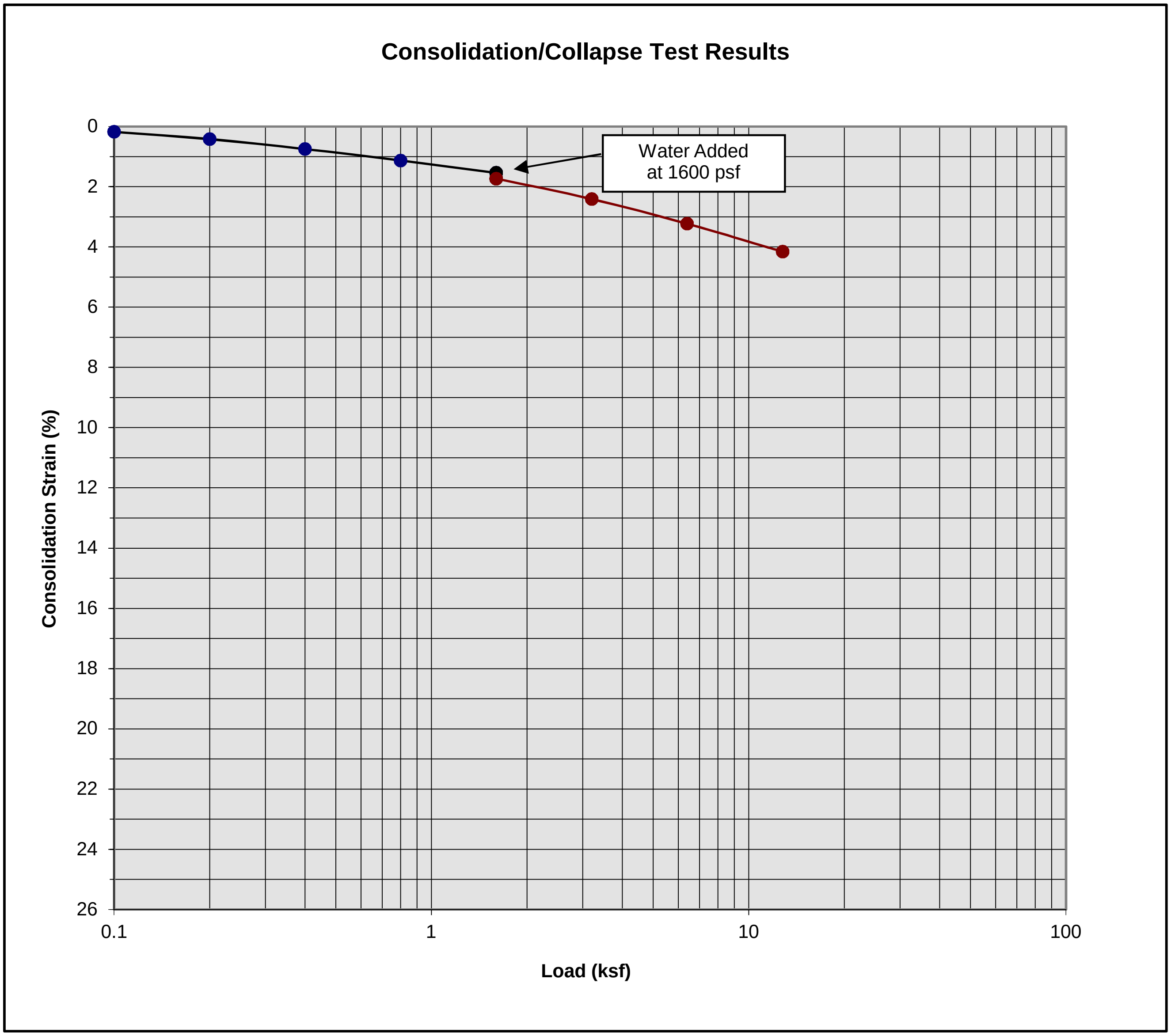


Classification: Brown Gravelly fine to coarse Sand, some Cobbles

Boring Number:	B-1	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	25 to 26	Initial Dry Density (pcf)	110.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	119.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.69

Proposed Multi-Unit Residential Development
Brea, California
Project No. 18G118
PLATE C- 6

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

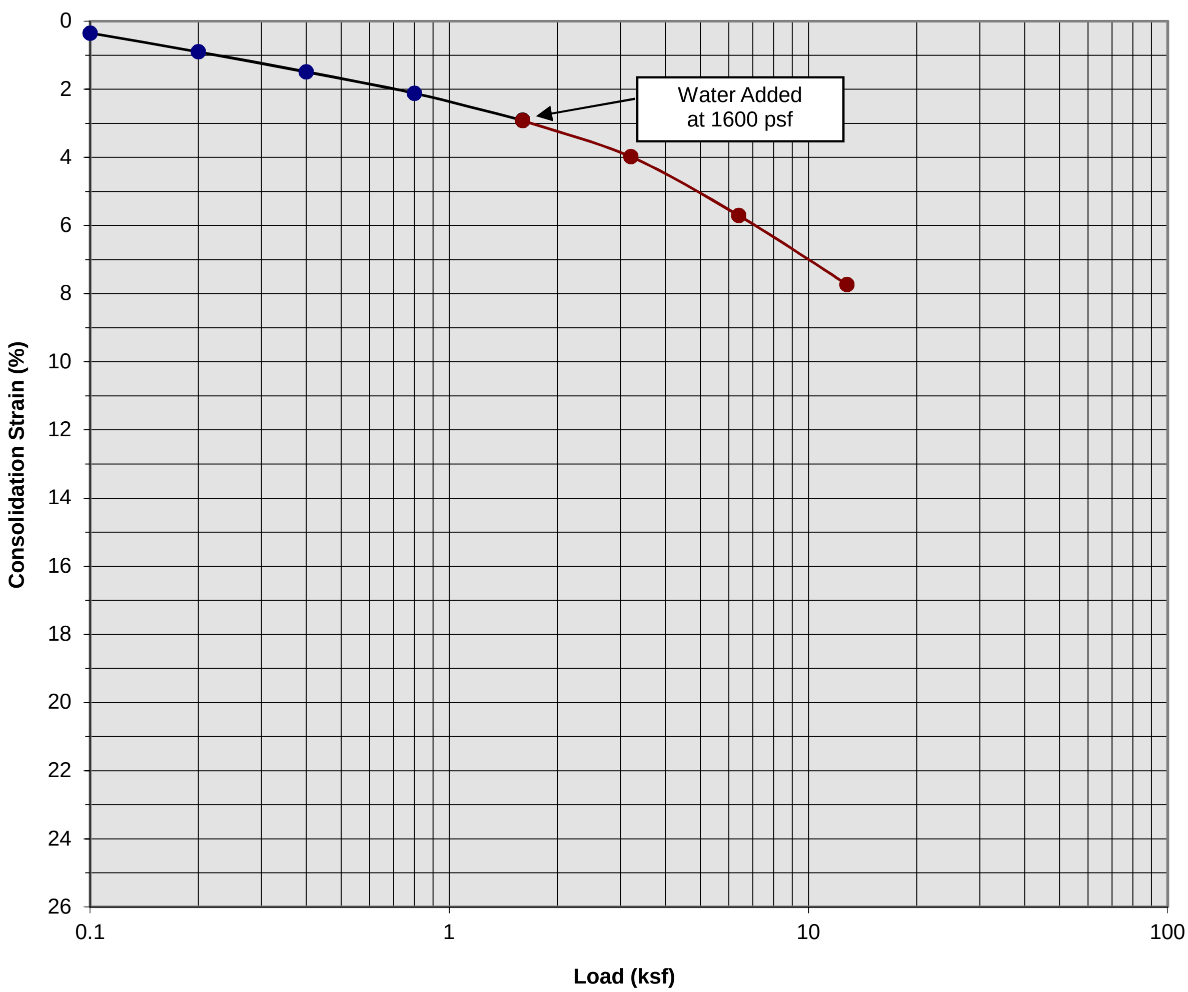


Classification: Brown Gravelly fine to coarse Sand, some Cobbles

Boring Number:	B-1	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	24
Depth (ft)	27 to 28	Initial Dry Density (pcf)	89.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	93.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.19

Proposed Multi-Unit Residential Development
Brea, California
Project No. 18G118
PLATE C- 7

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



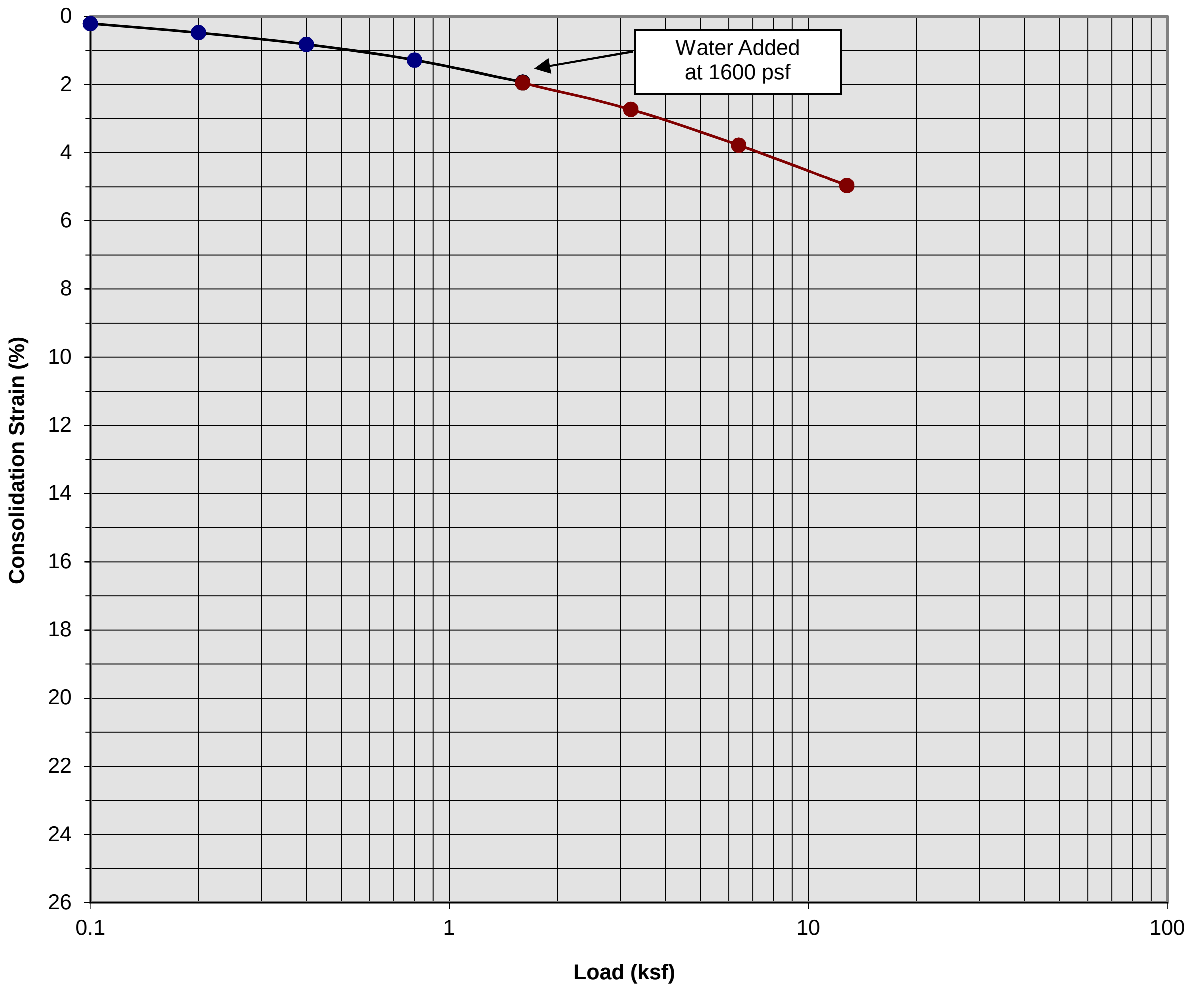
Classification: Gray Brown Clayey fine Sand to fine Sandy Clay, trace fine Gravel

Boring Number:	B-5	Initial Moisture Content (%)	13
Sample Number:	---	Final Moisture Content (%)	23
Depth (ft)	33 to 34	Initial Dry Density (pcf)	114.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	124.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.01

Proposed Multi-Unit Residential Development
Brea, California
Project No. 18G118
PLATE C- 8

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

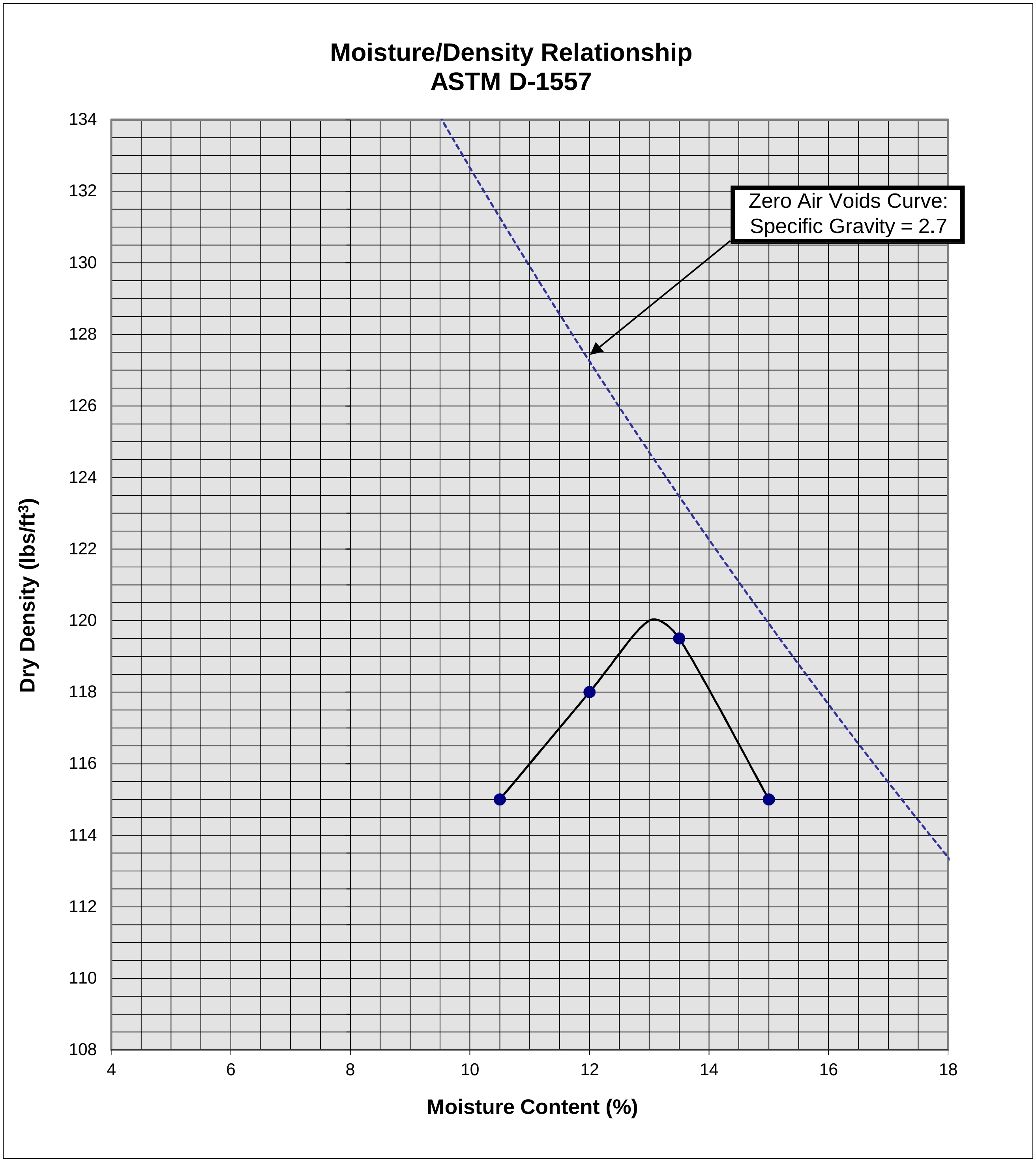


Classification: Brown fine to coarse Sand, some fine Gravel, trace Clay clasts

Boring Number:	B-5	Initial Moisture Content (%)	26
Sample Number:	---	Final Moisture Content (%)	23
Depth (ft)	35 to 36	Initial Dry Density (pcf)	96.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	102.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.02

Proposed Multi-Unit Residential Development
Brea, California
Project No. 18G118
PLATE C- 9

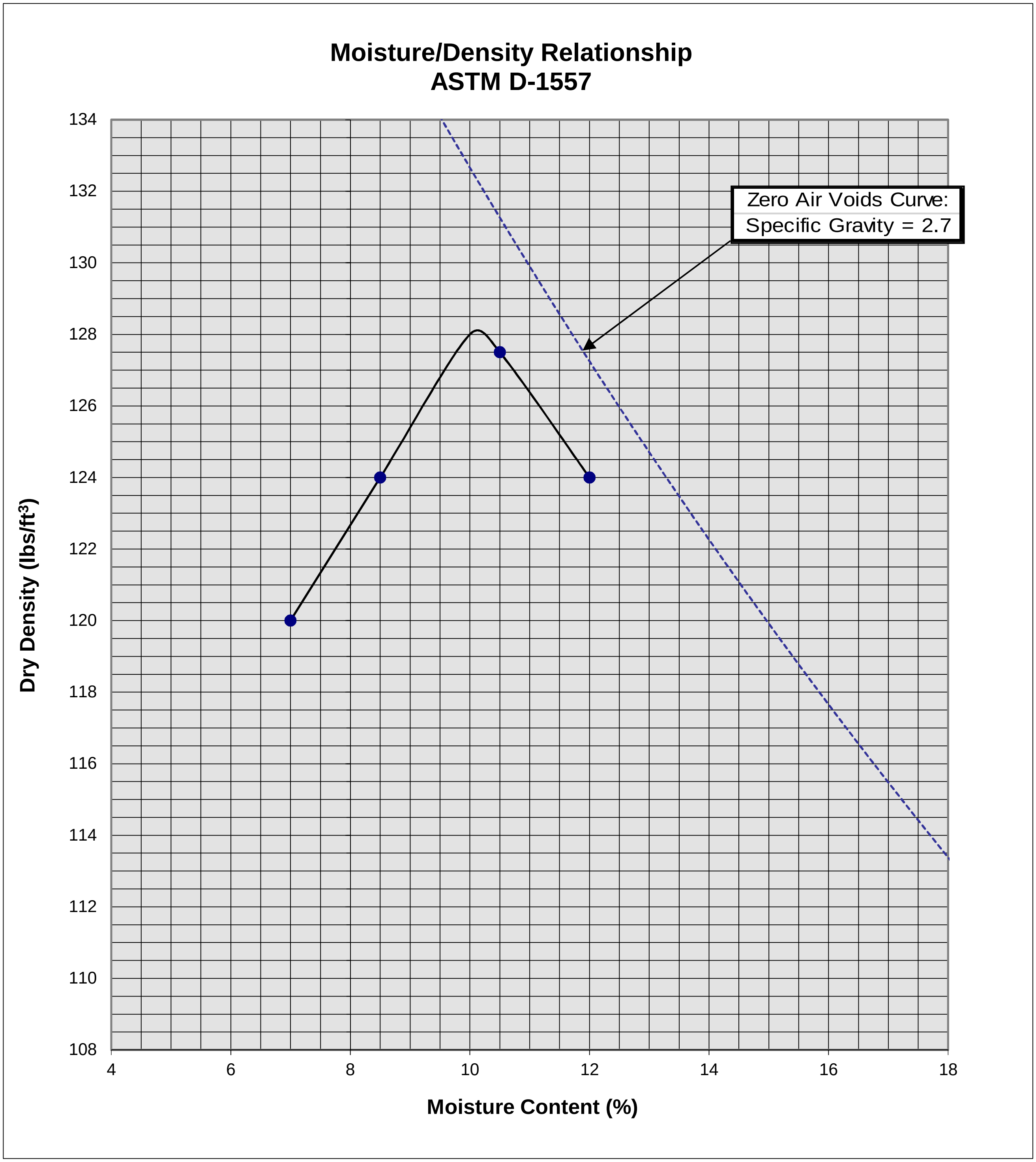
SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-3 @ 2 to 8'
Optimum Moisture (%)	13
Maximum Dry Density (pcf)	120
Soil Classification	Brown fine Sandy Clay

Proposed Residential Development
Brea, California
Project No. 18G118
PLATE C-10

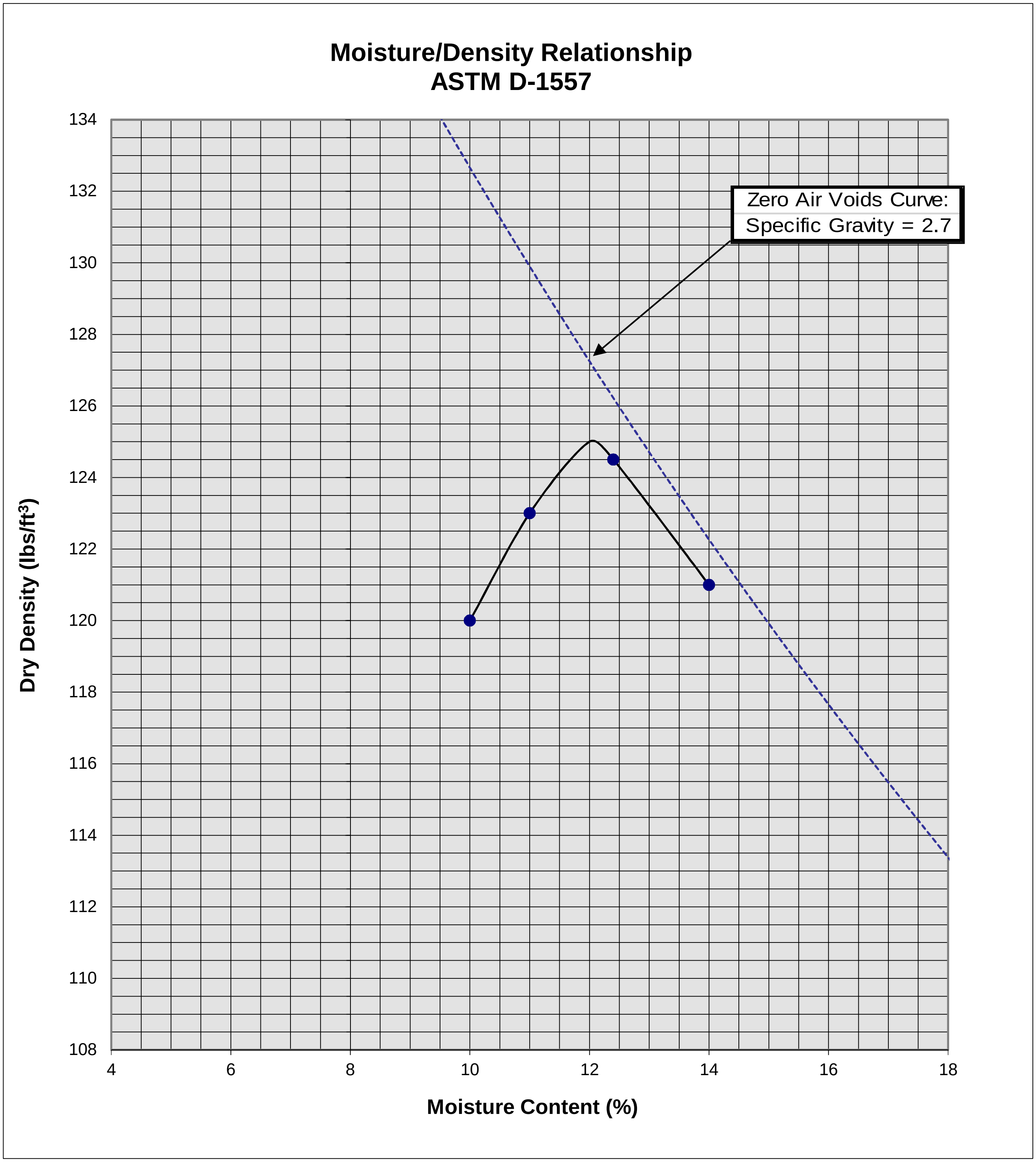
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-3 @ 0 to 5'
Optimum Moisture (%)	10
Maximum Dry Density (pcf)	128
Soil Classification	Brown Clayey fine Sand

Proposed Residential Development
Brea, California
Project No. 18G118
PLATE C-11

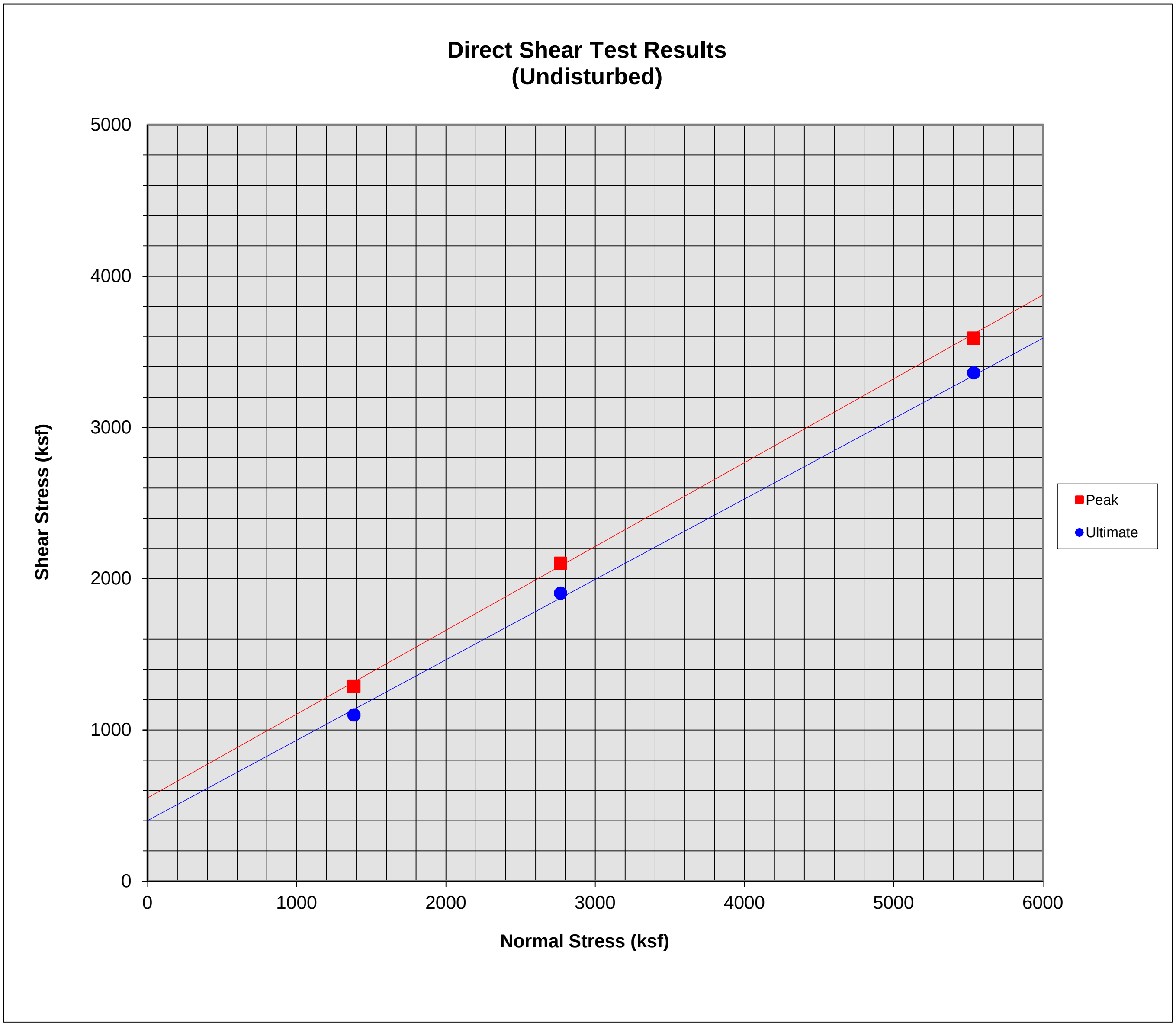
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-5 @ 0 to 5'
Optimum Moisture (%)	12
Maximum Dry Density (pcf)	125
Soil Classification	Brown Clayey fine Sand to fine Sandy Clay, trace medium Sand

Proposed Residential Development
Brea, California
Project No. 18G118
PLATE C-12

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL A California Corporation



Sample Description: B-1 @ 7 to 8 feet

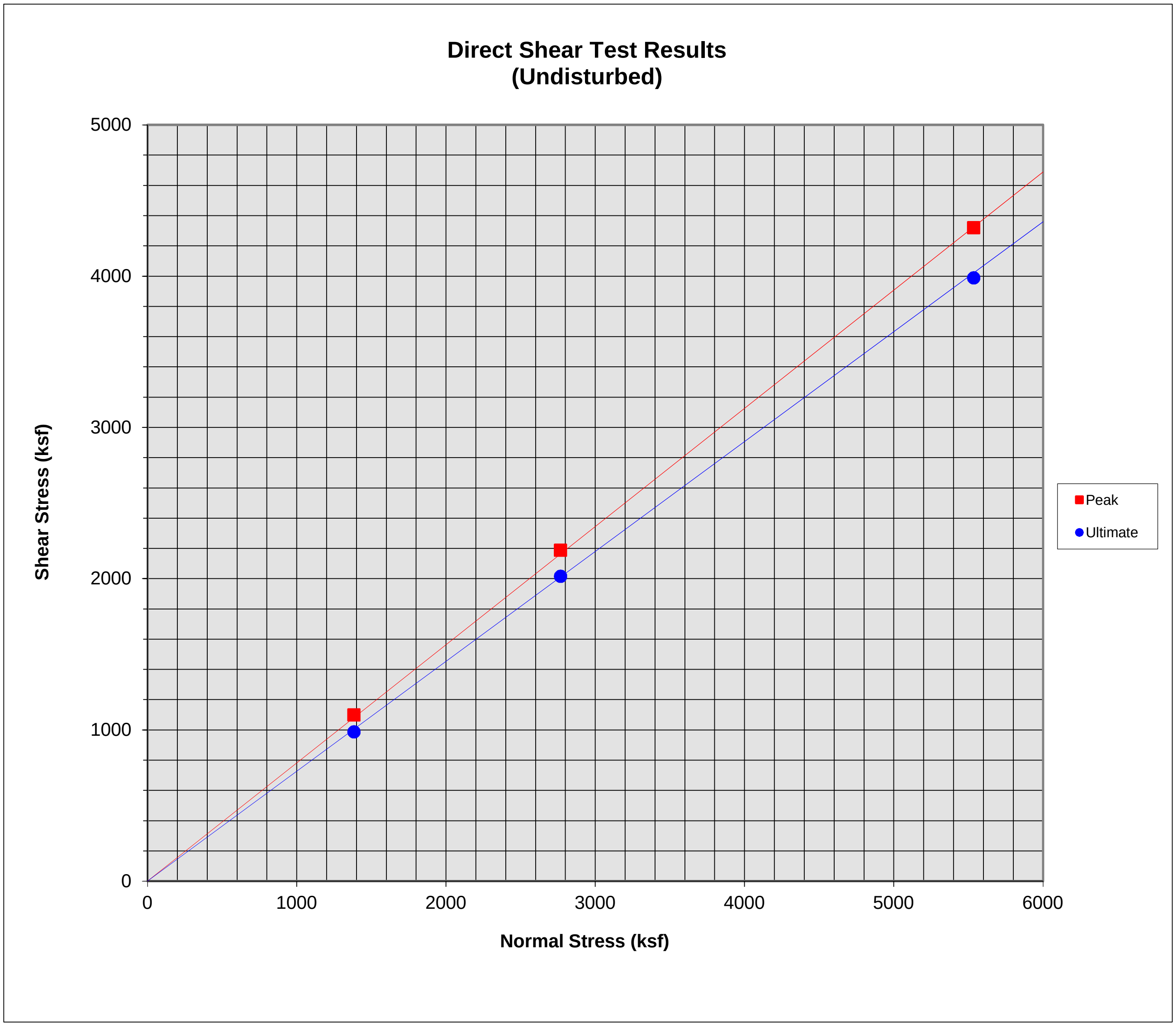
Classification: Brown fine Sandy Clay

Sample Data			Test Results	
Initial Moisture Content	13.0			
Final Moisture Content	20.0		Peak	Ultimate
Initial Dry Density	105.0	ϕ (°)	29.0	28.0
Final Dry Density	---	C (psf)	550	400
Specimen Diameter (in)	2.4			
Specimen Thickness (in)	1.0			

Proposed Residential Development
Brea, California
Project No. 18G118

PLATE C-13

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Sample Description: B-5 @ 31 to 32 feet

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

Sample Data	
Initial Moisture Content	26.0
Final Moisture Content	20.0
Initial Dry Density	95.0
Final Dry Density	---
Specimen Diameter (in)	2.4
Specimen Thickness (in)	1.0

Test Results	Peak	Ultimate
ϕ (°)	38.0	36.0
C (psf)	0	0

Proposed Residential Development
Brea, California
Project No. 18G118
PLATE C-14

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

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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.

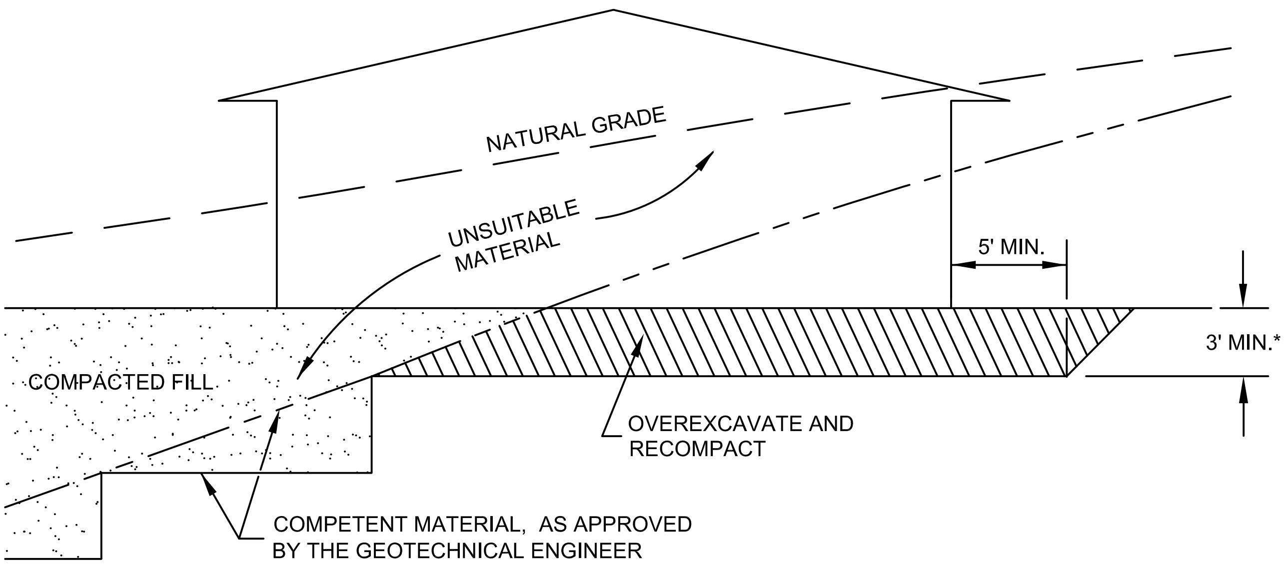
Subdrains

- 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.

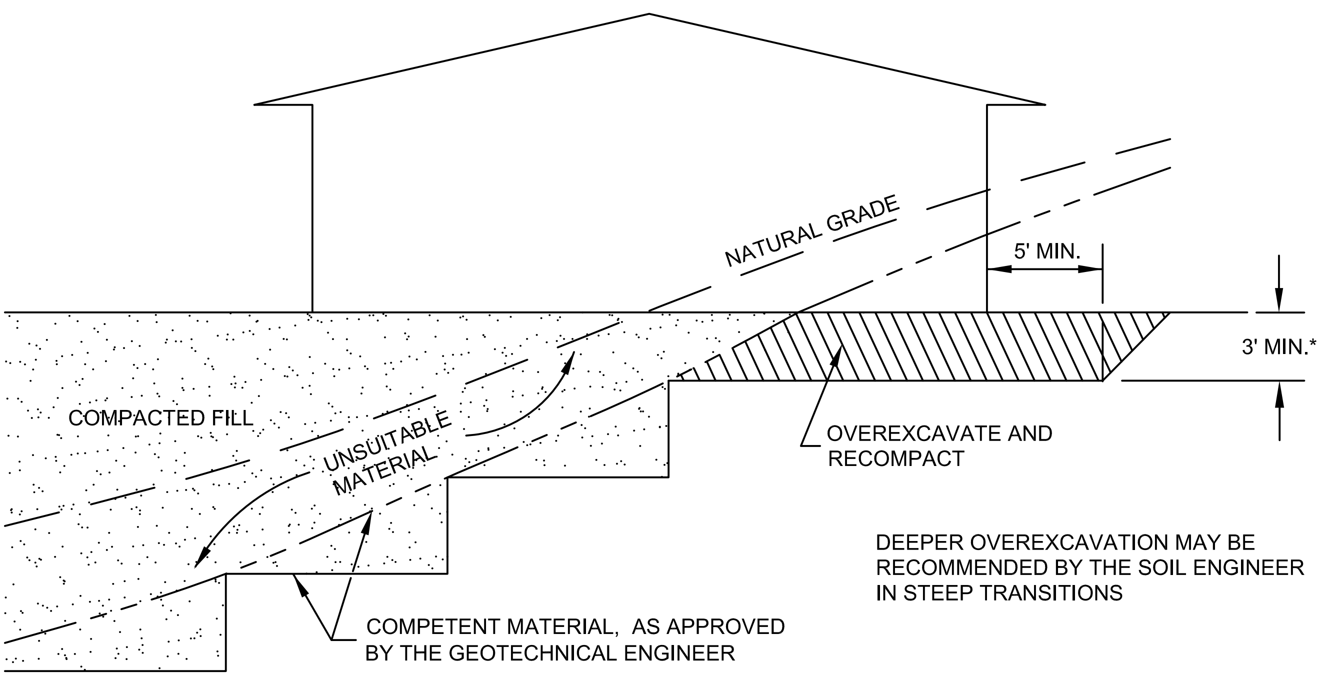
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- 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

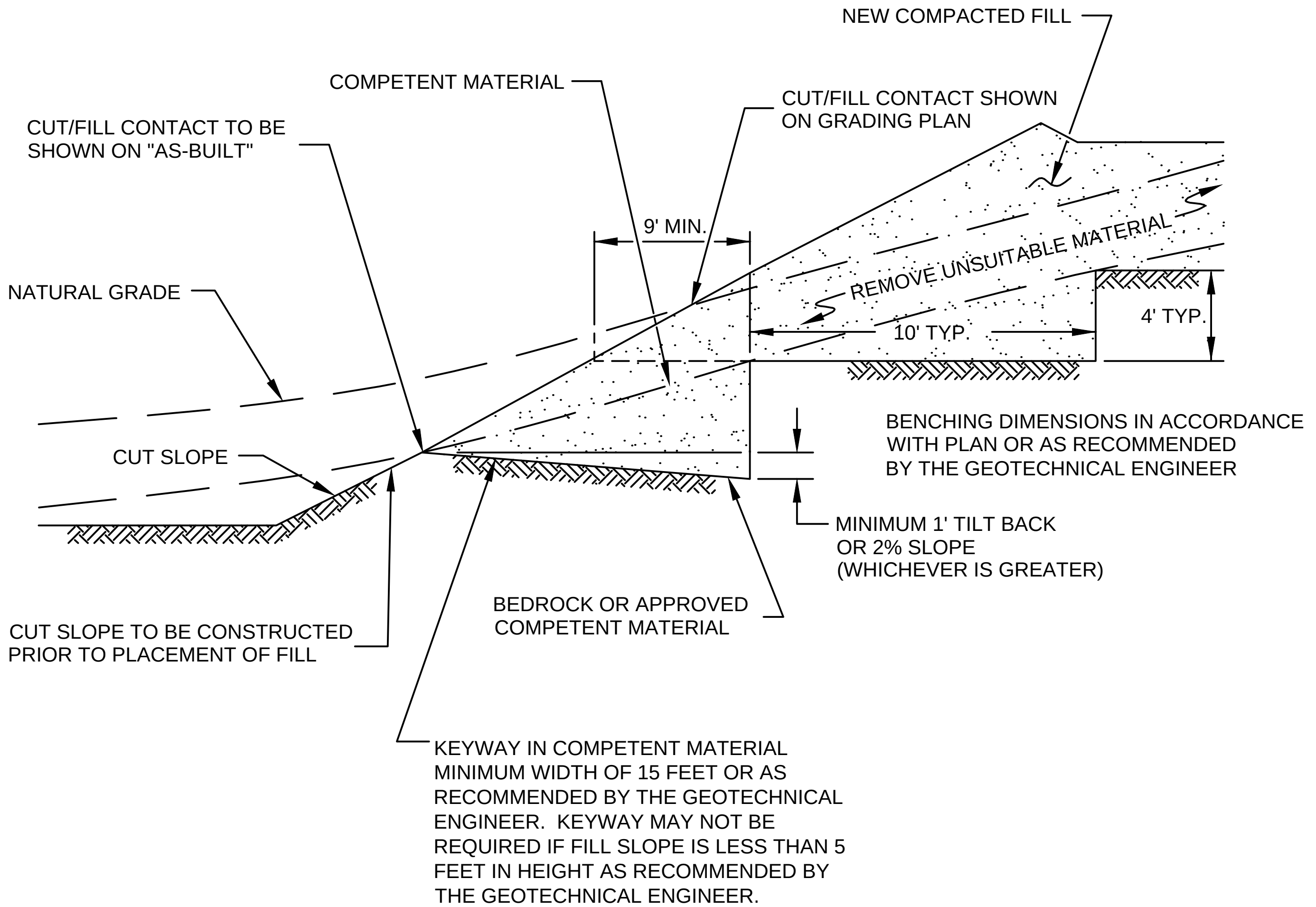
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

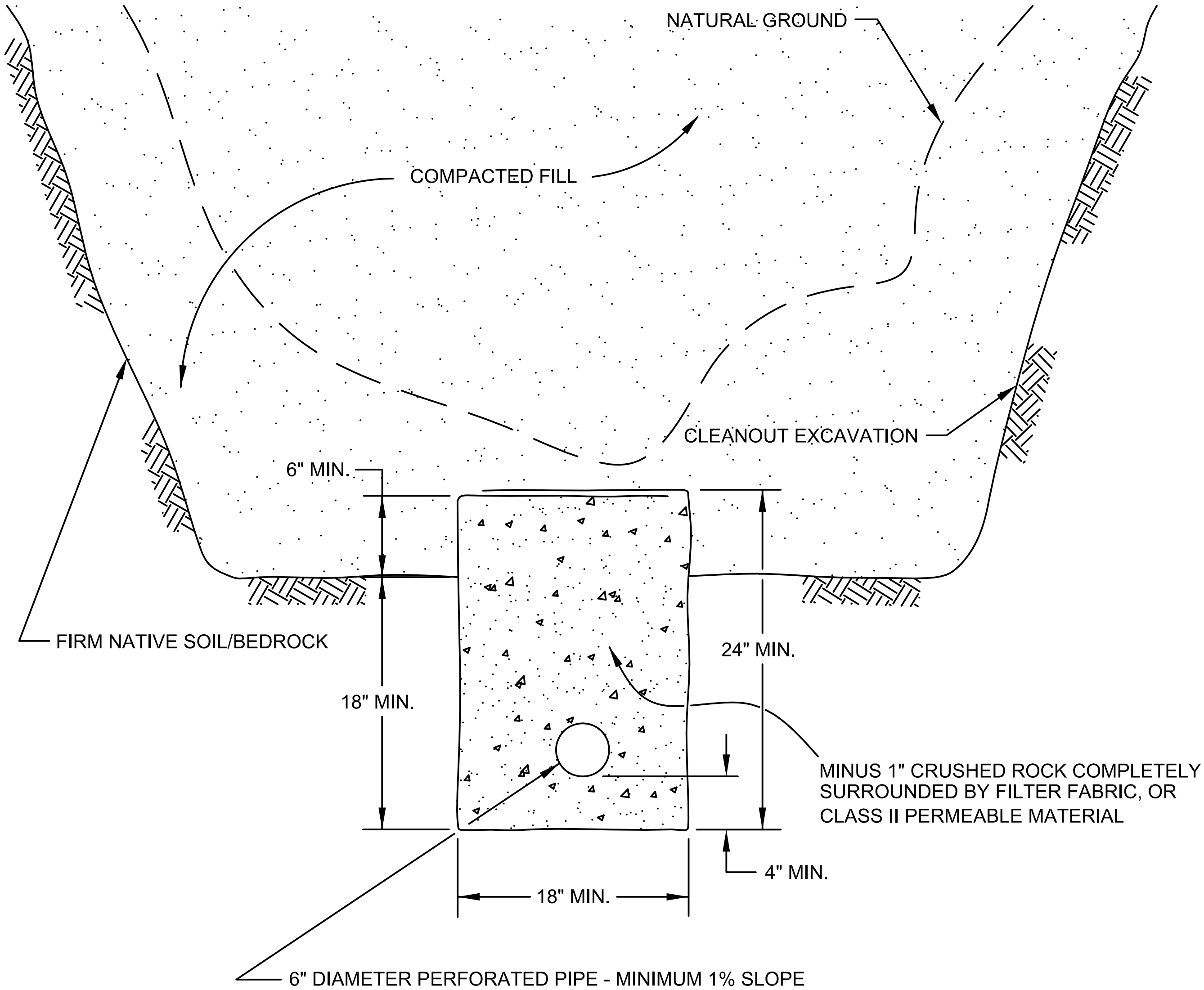
GRADING GUIDE SPECIFICATIONS

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

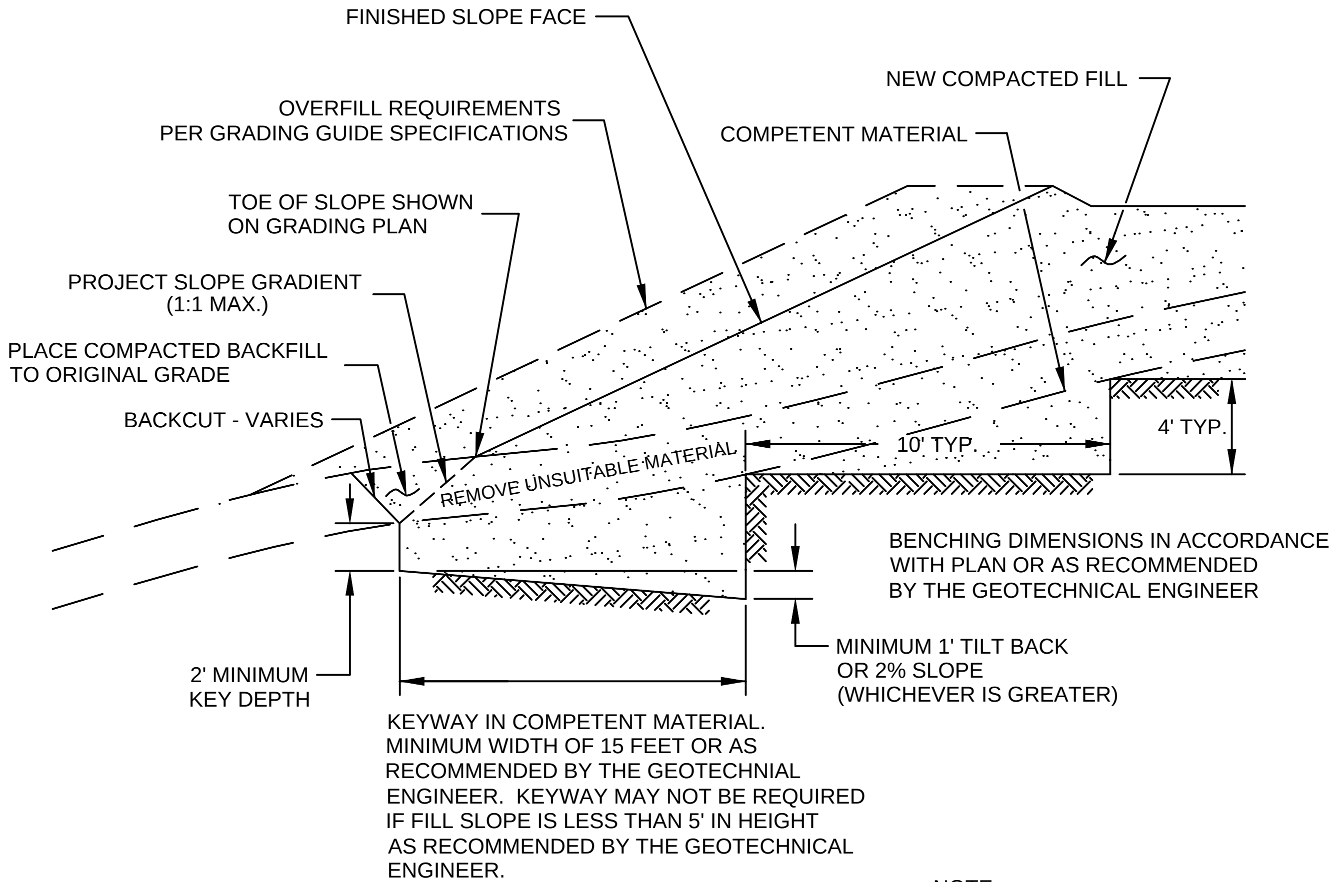
GRADING GUIDE SPECIFICATIONS

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

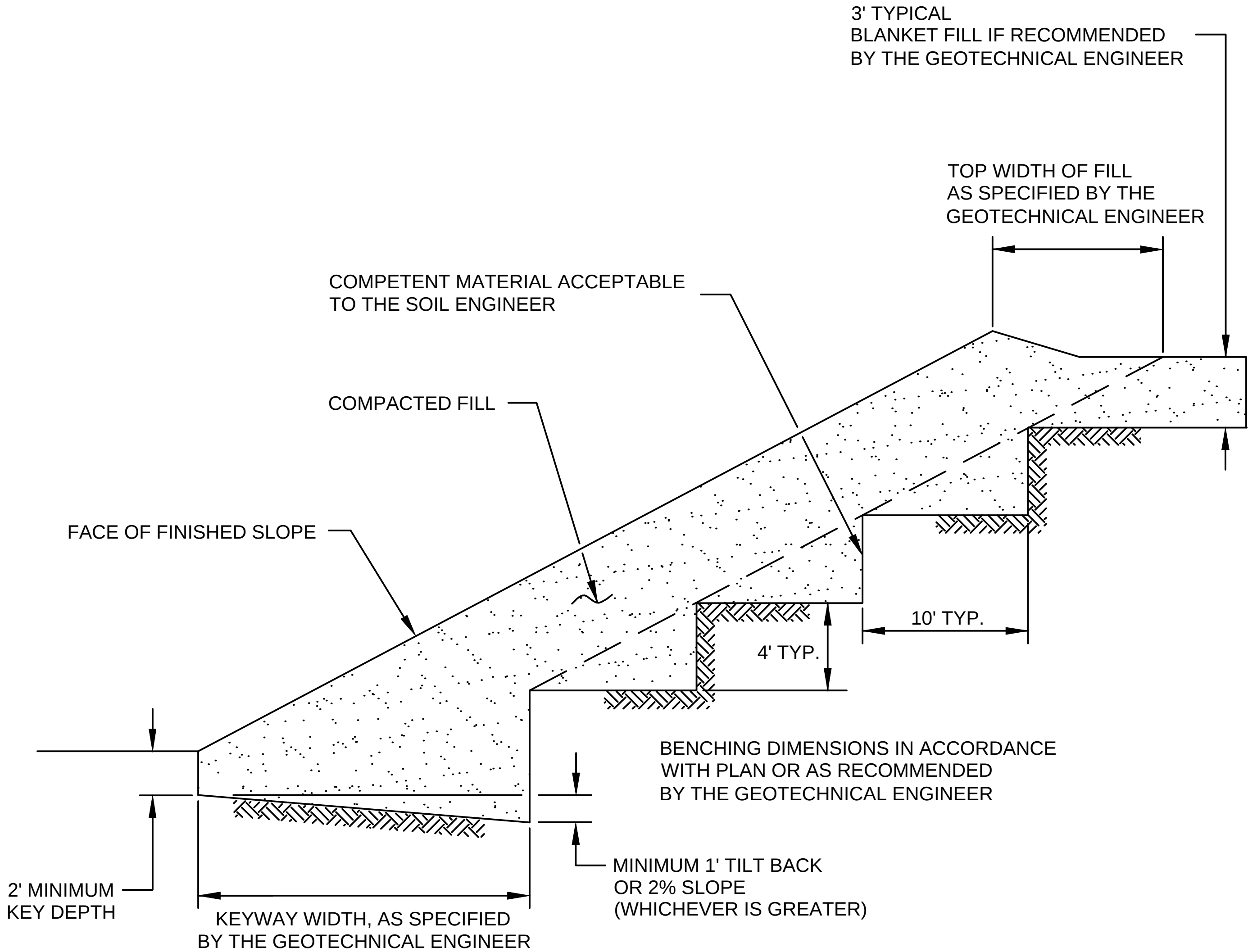
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

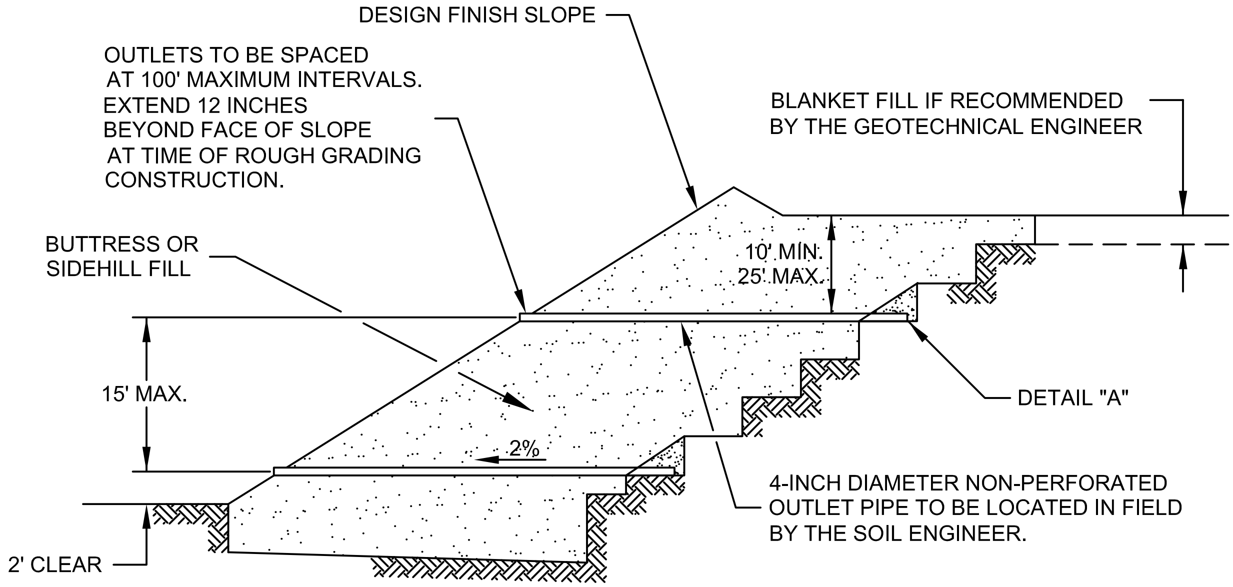
DRAWN: JAS
CHKD: GKM

PLATE D-4

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-5	



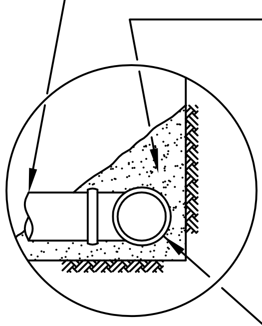
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50



DETAIL "A"

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

SLOPE FILL SUBDRAINS

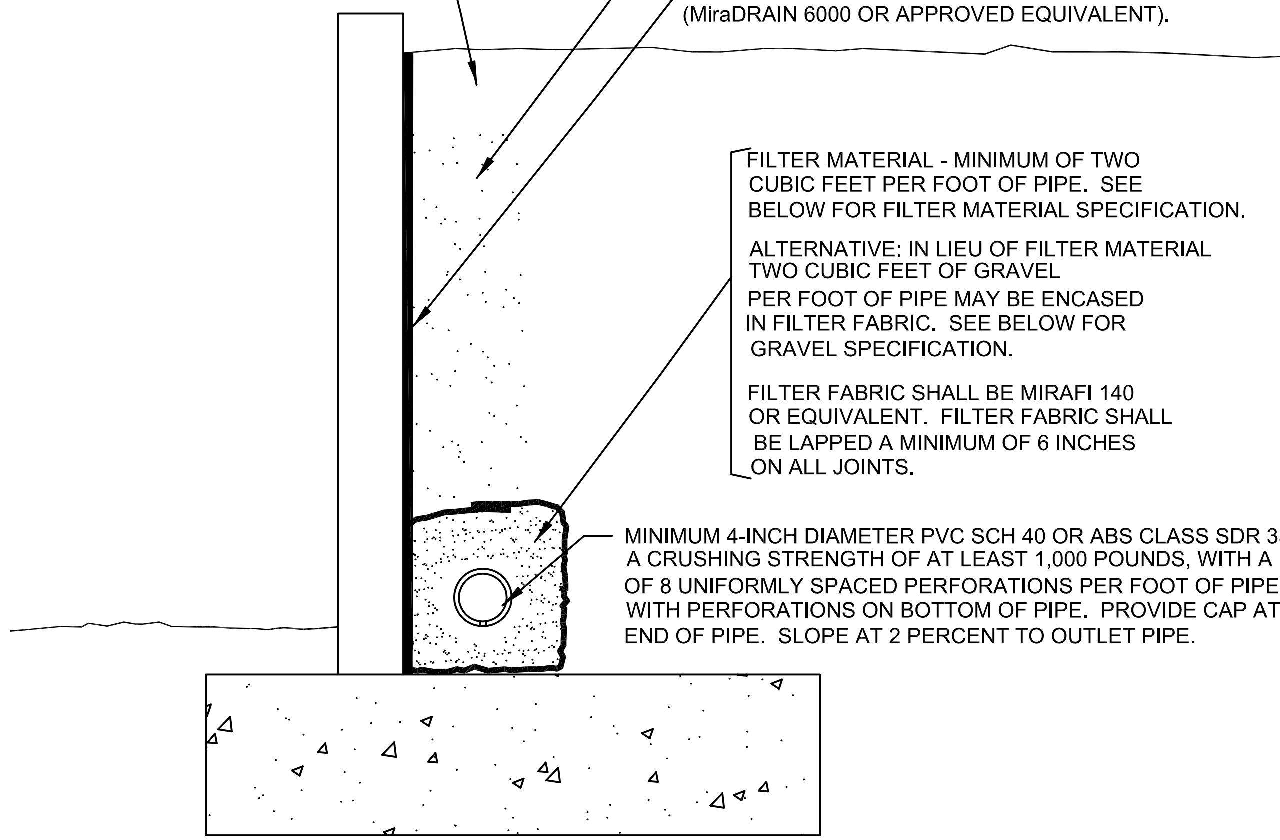
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

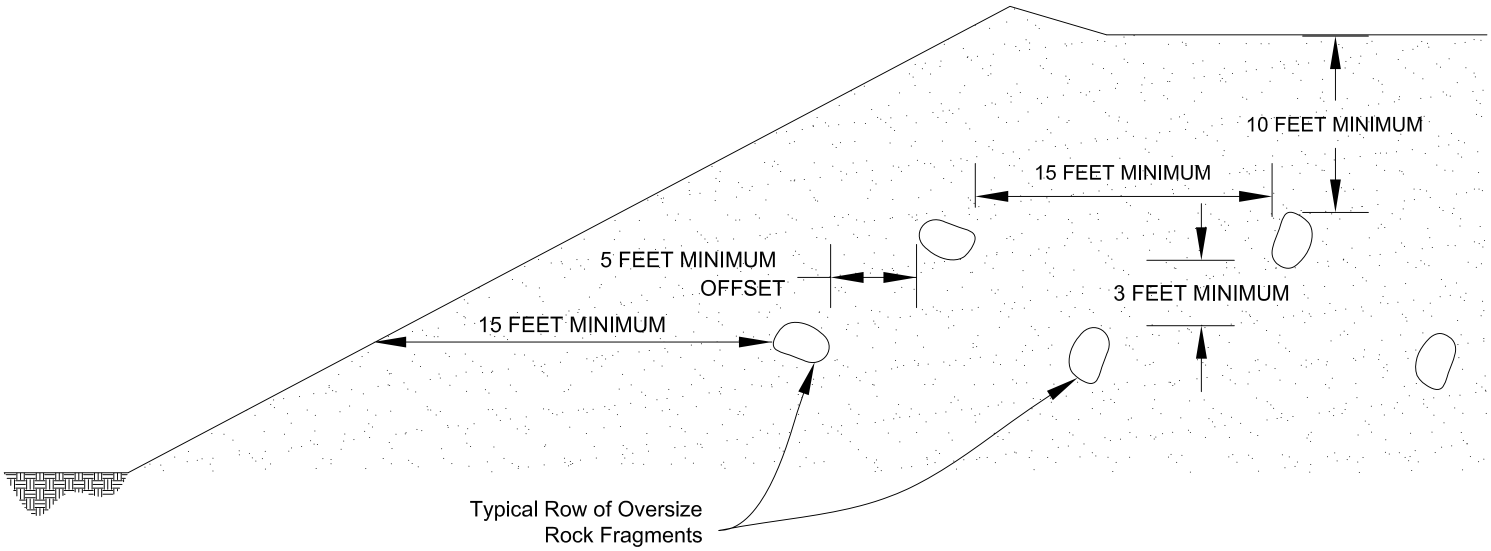
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

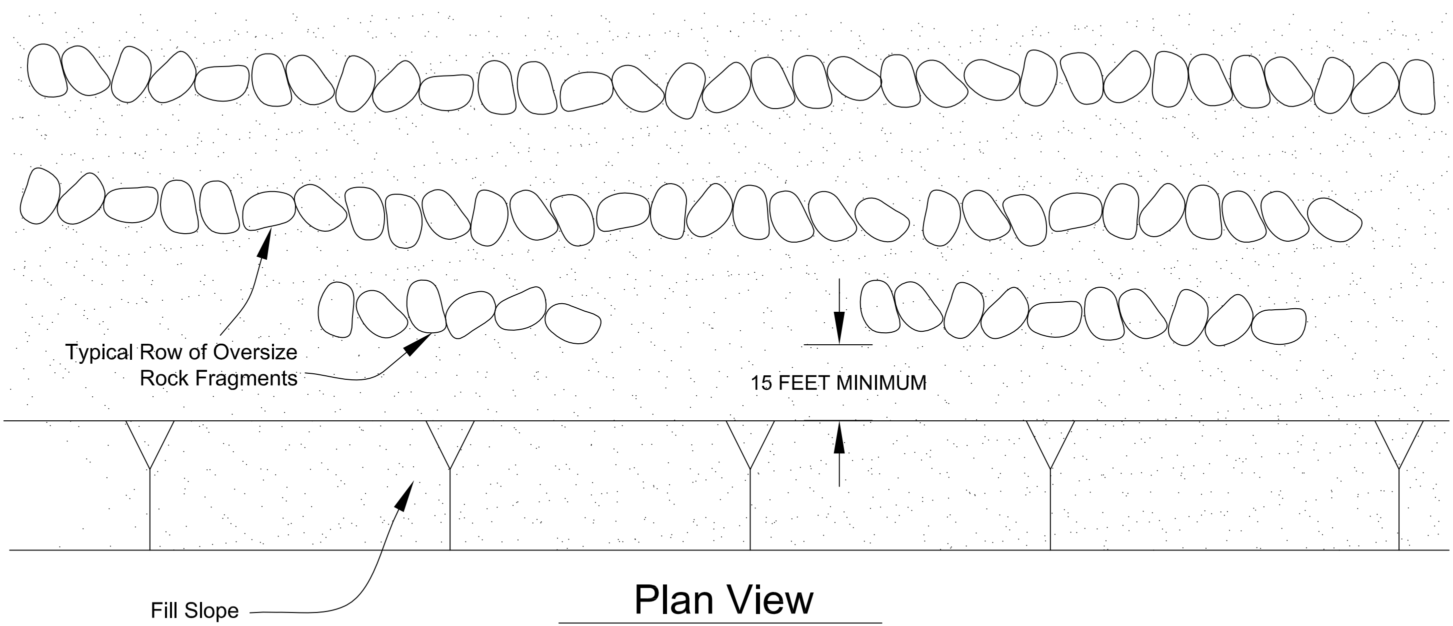
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

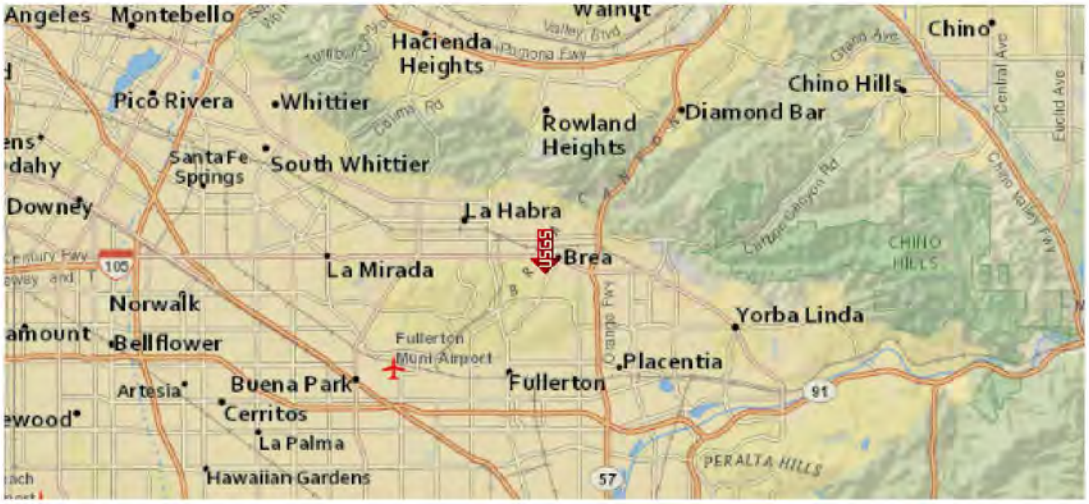
PLACEMENT OF OVERSIZED MATERIAL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: PM CHKD: GKM	
PLATE D-8	

APPENDIX E

USGS Design Maps Summary Report

User–Specified Input

Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	33.91941°N, 117.90633°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

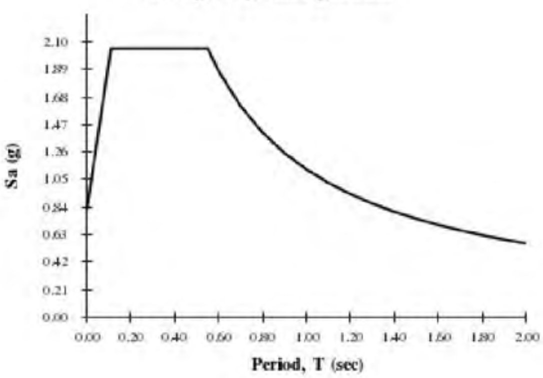


USGS–Provided Output

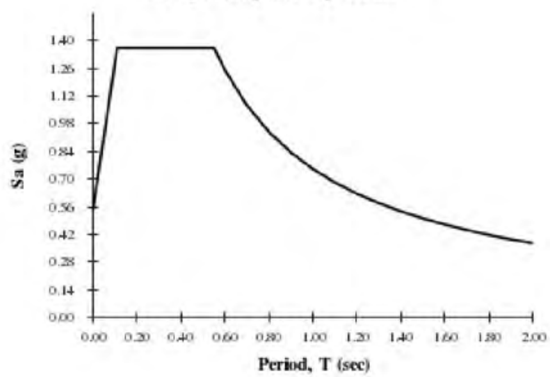
$S_S =$ 2.047 g	$S_{MS} =$ 2.047 g	$S_{DS} =$ 1.364 g
$S_1 =$ 0.751 g	$S_{M1} =$ 1.127 g	$S_{D1} =$ 0.751 g

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 RESIDENTIAL DEVELOPMENT

BREA, CALIFORNIA

DRAWN: JLH
CHKD: RGT

SCG PROJECT
18G118-1

PLATE E-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

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

From Figure 22-7 [4]	PGA = 0.795
Equation (11.8–1):	$PGA_M = F_{PGA}PGA = 1.000 \times 0.795 = 0.795$ g

Table 11.8–1: Site Coefficient F_{PGA}

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

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

For Site Class = D and PGA = 0.795 g, F_{PGA} = 1.000

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

From Figure 22-17 [5]	C_{RS} = 0.945
From Figure 22-18 [6]	C_{R1} = 0.955

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

MCE PEAK GROUND ACCELERATION

PROPOSED RESIDENTIAL DEVELOPMENT

BREA, CALIFORNIA

DRAWN: JLH
CHKD: RGT

SCG PROJECT
18G118-1

PLATE E-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Proposed Multi Unit Res Dev	MCE$_G$ Design Acceleration	0.795 (g)
Project Location	Brea, CA	Design Magnitude	6.71
Project Number	18G118	Historic High Depth to Groundwater	10 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	27 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-2		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	KS	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.71)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	10	5		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	600	600	600	0.99	1.03	1.07	N/A	N/A	N/A	N/A	Above Water Table
14.5	10	15	12.5		120		1.3	1.05	1.1	1.31	0.85	0.0	0.0	1500	1344	1500	0.96	1.03	1.02	N/A	N/A	N/A	N/A	Above Basement Bottom
14.5	15	17	16	22	120		1.3	1.05	1.3	1.03	0.85	34.3	34.3	1920	1546	1920	0.94	1.35	1.08	0.96	1.39	0.60	2.30	Non-Liquefiable
19.5	17	22	19.5	26	120		1.3	1.05	1.3	0.97	0.95	42.6	42.6	2340	1747	2340	0.92	1.35	1.05	2.00	2.00	0.64	3.13	Non-Liquefiable
24.5	22	27	24.5	24	120		1.3	1.05	1.3	0.90	0.95	36.4	36.4	2940	2035	2940	0.89	1.35	1.01	1.52	2.00	0.67	3.00	Non-Liquefiable
29.5	27	32	29.5	15	120	31	1.3	1.05	1.19	0.83	0.95	19.2	24.6	3540	2323	3384	0.86	1.20	0.98	0.28	0.33	0.68	0.49	**Liquefiable**
34.5	32	37	34.5	32	120		1.3	1.05	1.3	0.87	1	49.7	49.7	4140	2611	3672	0.83	1.35	0.94	2.00	2.00	0.68	2.93	Non-Liquefiable
39.5	37	42	39.5	42	120		1.3	1.05	1.3	0.91	1	67.8	67.8	4740	2899	3960	0.80	1.35	0.9	2.00	2.00	0.68	2.95	Non-Liquefiable
44.5	42	47	44.5	44	120		1.3	1.05	1.3	0.91	1	71.0	71.0	5340	3187	4248	0.77	1.35	0.88	2.00	2.00	0.67	2.99	Non-Liquefiable
49.5	47	50	48.5	22	120	14	1.3	1.05	1.3	0.77	1	30.2	33.1	5820	3418	4478	0.75	1.35	0.88	0.77	0.92	0.66	1.40	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 Multi Unit Res Dev
Project Location	Brea, CA
Project Number	18G118
Engineer	DWN

Boring No. B-2

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines cont	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	10	5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	10.00		0.000		0.00	Above Water Table
14.5	10	15	12.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	5.00		0.000		0.00	Above Basement Bottom
14.5	15	17	16	34.3	0.0	34.3	2.30	0.02	-0.38	0.00	2.00		0.000		0.00	Non-Liquefiable
19.5	17	22	19.5	42.6	0.0	42.6	3.13	0.00	-1.00	0.00	5.00		0.000		0.00	Non-Liquefiable
24.5	22	27	24.5	36.4	0.0	36.4	3.00	0.02	-0.54	0.00	5.00		0.000		0.00	Non-Liquefiable
29.5	27	32	29.5	19.2	5.4	24.6	0.49	0.09	0.26	0.09	5.00		0.019		1.16	**Liquefiable**
34.5	32	37	34.5	49.7	0.0	49.7	2.93	0.00	-1.56	0.00	5.00		0.000		0.00	Non-Liquefiable
39.5	37	42	39.5	67.8	0.0	67.8	2.95	0.00	-3.10	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	71.0	0.0	71.0	2.99	0.00	-3.38	0.00	5.00		0.000		0.00	Non-Liquefiable
49.5	47	50	48.5	30.2	2.9	33.1	1.40	0.03	-0.30	0.02	3.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				1.16	

Notes:

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

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

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

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

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

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

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

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

LIQUEFACTION EVALUATION

Project Name	Proposed Multi Unit Res Dev	MCE_G Design Acceleration	0.795 (g)
Project Location	Brea, CA	Design Magnitude	6.71
Project Number	18G118	Historic High Depth to Groundwater	10 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	27 (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=6.71)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	10	5		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	600	600	600	0.99	1.03	1.07	N/A	N/A	N/A	N/A	Above Water Table
14.5	10	15	12.5		120		1.3	1.05	1.1	1.31	0.85	0.0	0.0	1500	1344	1500	0.96	1.03	1.02	N/A	N/A	N/A	N/A	Above Basement Bottom
14.5	15	17	16	20	120		1.3	1.05	1.3	1.04	0.85	31.2	31.2	1920	1546	1920	0.94	1.31	1.07	0.57	0.80	0.60	1.33	Non-Liquefiable
19.5	17	22	19.5	14	120	36	1.3	1.05	1.21	0.96	0.95	21.2	26.7	2340	1747	2340	0.92	1.23	1.03	0.34	0.43	0.64	0.67	**Liquefiable**
24.5	22	27	24.5	14	120	32	1.3	1.05	1.19	0.88	0.95	18.9	24.3	2940	2035	2940	0.89	1.20	1	0.27	0.33	0.67	0.50	**Liquefiable**
29.5	27	32	29.5	18	120	9	1.3	1.05	1.24	0.83	0.95	23.9	24.7	3540	2323	3384	0.86	1.20	0.98	0.28	0.33	0.68	0.49	**Liquefiable**
34.5	32	37	34.5	26	120		1.3	1.05	1.3	0.85	1	39.0	39.0	4140	2611	3672	0.83	1.35	0.94	2.00	2.00	0.68	2.93	Non-Liquefiable
39.5	37	42	39.5	33	120		1.3	1.05	1.3	0.86	1	50.4	50.4	4740	2899	3960	0.80	1.35	0.9	2.00	2.00	0.68	2.95	Non-Liquefiable
44.5	42	47	44.5	38	120		1.3	1.05	1.3	0.87	1	58.9	58.9	5340	3187	4248	0.77	1.35	0.88	2.00	2.00	0.67	2.99	Non-Liquefiable
49.5	47	50	48.5	39	120		1.3	1.05	1.3	0.87	1	60.1	60.1	5820	3418	4478	0.75	1.35	0.86	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 Multi Unit Res Dev
Project Location	Brea, CA
Project Number	18G118
Engineer	DWN

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 cont	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	10	5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	10.00		0.000		0.00	Above Water Table
14.5	10	15	12.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	5.00		0.000		0.00	Above Basement Bottom
14.5	15	17	16	31.2	0.0	31.2	1.33	0.04	-0.17	0.02	2.00		0.004		0.08	Non-Liquefiable
19.5	17	22	19.5	21.2	5.5	26.7	0.67	0.07	0.13	0.07	5.00		0.016		0.96	**Liquefiable**
24.5	22	27	24.5	18.9	5.4	24.3	0.50	0.10	0.27	0.10	5.00		0.019		1.17	**Liquefiable**
29.5	27	32	29.5	23.9	0.7	24.7	0.49	0.09	0.25	0.09	5.00		0.019		1.15	**Liquefiable**
34.5	32	37	34.5	39.0	0.0	39.0	2.93	0.01	-0.73	0.00	5.00		0.000		0.00	**Non-Liquefiable**
39.5	37	42	39.5	50.4	0.0	50.4	2.95	0.00	-1.62	0.00	5.00		0.000		0.00	**Non-Liquefiable**
44.5	42	47	44.5	58.9	0.0	58.9	2.99	0.00	-2.33	0.00	5.00		0.000		0.00	Non-Liquefiable
49.5	47	50	48.5	60.1	0.0	60.1	3.04	0.00	-2.43	0.00	3.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				3.37	

Notes:

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

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

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

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

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

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

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

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

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