GEOTECHNICAL INVESTIGATION
RIDER 2 – PROPOSED
COMMERCIAL/INDUSTRIAL BUILDING

NEC Redlands Avenue at Rider Street
Perris, California
for
IDI Gazeley

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

A California Corporation

November 22, 2017
September 9, 2019 (Revised)

IDI Gazeley
8 Corporate Park, Suite 300-34
Irvine, California 92606

Attention: Mr. Stephen Hollis

Project No.:   **17G199-1R**

Subject: **Geotechnical Investigation**
               Rider 2 – Proposed Commercial/Industrial Building
               NEC Redlands Avenue at Rider Street
               Perris, California

Dear Mr. Hollis:

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.

Gregory K. Mitchell, GE 2364
Principal Engineer

Robert G. Trazo, GE 2655
Principal Engineer

Distribution: (1) Addressee

TABLE OF CONTENTS

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
- The subject site is located within an area of moderate liquefaction susceptibility.
- Our site-specific liquefaction evaluation included two borings extended to depths of 50± feet. Liquefiable soils were encountered within one 5-foot stratum at one of these boring locations.
- The potential liquefaction induced settlement is approximately 1.6± inches.
- Based on the estimated magnitude of the differential settlements, the proposed structure may be supported on shallow foundations. Additional design considerations related to the potentially liquefiable soils are presented within the text of this report.

Site Preparation
- Initial site preparation should include stripping of the existing crop stubble as well as any existing native grass and weed growth.
- The near-surface soils generally consist of low to medium expansive native alluvium which possesses a moderate potential for consolidation/collapse. Therefore, remedial grading is recommended to remove the upper portion of the near-surface native alluvium and replace these soils as compacted structural fill. The recommended remedial grading will reduce potential differential settlements by replacing collapsible/compressible soils as compacted structural fill.
- The proposed building area should be overexcavated to a depth of at least 4 feet below existing grade and to a depth of 4 feet below proposed building pad subgrade elevation. Within the foundation influence zones, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade. The overexcavation should extend horizontally at least 5 feet beyond the building and foundation perimeters.
- After the overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be removed. Resulting subgrade should then be scarified to a depth of 12 inches and moisture conditioned to 2 to 4 percent above optimum. The previously excavated soils may then be replaced as compacted structural fill. All structural fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.
- The new pavement 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.
- 2,500 lbs/ft^2 maximum allowable soil bearing pressure.
- Reinforcement consisting of at least six (6) No. 5 rebars (3 top and 3 bottom) in strip footings, due to the presence of medium expansive soils and minor amounts of liquefaction-induced settlement. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade, 6 inches thick.
- Modulus of Subgrade Reaction: k = 100 psi/in.
- Minimum slab reinforcement: Reinforcement of the floor slab should consist of No. 3 bars at 18-inches on center in both directions due to the presence of medium expansive soils and minor amounts of liquefaction-induced settlement. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

Pavements

ASPHALT PAVEMENTS (R=25)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	4	4½	5	6
Aggregate Base	7	8	11	12	14
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R=30)				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	7	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 17P381, dated October 10, 2017. 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 this 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 at the northeast corner of Redlands Avenue and Rider Street in Perris, California. The site is bounded to the north by an agricultural field, to the west by Redlands Avenue, to the south by Rider Street and to the east by a flood control channel. 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 trapezoidal-shaped parcel that is approximately 38.3 acres in size. The site was most recently utilized as an agricultural field. Current groundcover consists of crop stubble and some small native shrubs.

Detailed topographic information was obtained from a conceptual site plan prepared by Albert A. Webb Associates (Webb), the project civil engineer. This plan indicates that the overall site topography generally slopes downward to the east-southeast at an estimated gradient of less than 1 percent. The maximum site elevation is 1445± feet mean sea level (msl) located in the northwestern corner of the subject site, and the minimum site elevation is 1441± feet msl in the southeastern corner of the subject site.

3.2 Proposed Development

The site plan provided to our office by the client indicates that the new development be developed with one (1) new commercial/industrial building. The building will be located in the center of the site and will be 820,676± ft² in size. The building will be constructed in a cross-dock configuration with loading docks along both the north and south sides of the building. It is expected that the building will be surrounded by asphaltic concrete pavements for parking and drive lanes and Portland cement concrete pavements in the loading dock areas. Landscape planters and concrete flatwork are expected to be included throughout the site. A detention basin will be located in the southeastern corner of the site.

Detailed structural information has not been provided. It is assumed that the building will be a single-story structure of tilt-up concrete construction, typically supported on conventional shallow foundations with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 4 to 7 kips per linear foot, respectively.

Based on the conceptual site plan prepared by Webb, fills of 2 to 4± feet will be necessary to achieve the proposed pad grade. No significant amounts of below grade construction, such as basements or crawl spaces, are expected to be included in the proposed development.

SOUTHERN
CALIFORNIA
GEOTECHNICAL
SoCalGeo

Rider 2 – Perris, CA
Project No. 17G199-1R
Page 4

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of twelve (12) borings, advanced to depths of 5 to 50± 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

Alluvium

Native alluvial soils were encountered at the ground surface at all twelve (12) of the boring locations. The alluvium varies widely in composition and strength, generally consisting of stiff to very stiff silty clays and clayey silts as well as loose to dense silty sands and fine sandy silts. These interbedded layers of sands, silts and clays, generally extend to at least the maximum depth explored of 50± feet.

The alluvial soils generally become more dense and more stiff with depth. Boring Nos. B-1 and B-8, both of which were drilled to depths of 50± feet, encountered medium dense to very dense clayey sands and silty sands as well as very stiff to hard silty clays and clayey silts below depths of 15 to 20± feet.

Most of the alluvial soils possess elevated moisture contents. However, the elevated moisture contents appear to be primarily due to the minerology of the soils, as many of these soils possessed damp to moist apparent moisture contents.

SOUTHERN
CALIFORNIA
GEOTECHNICAL

Rider 2 – Perris, CA
Project No. 17G199-1R
Page 5

Groundwater

Free water was encountered during drilling of Boring Nos. B-1 and B-8 at a depth of 33± 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 a depth of 33± feet below existing site grades at the time of the subsurface investigation.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the groundwater depths in this area is the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. Several monitoring wells are located within a mile radius of the subject site with high groundwater level readings ranging from 26 to 108± feet from the ground surface. Therefore, the high groundwater depth of 26± feet (February 2012) reported in a monitoring well located 0.75 miles east of the subject site is considered to be conservative with respect to the recent site conditions.

SOUTHERN
CALIFORNIA
GEOTECHNICAL
SoCalGeo

Rider 2 – Perris, CA
Project No. 17G199-1R
Page 6

5.0 LABORATORY TESTING

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

Classification

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

In-situ Density and Moisture Content

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

Consolidation

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

Maximum Dry Density and Optimum Moisture Content

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

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829. 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

SOUTHERN
CALIFORNIA
GEOTECHNICAL

SoCalGeo

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-3 @ 0 to 5 feet	68	Medium
B-6 @ 0 to 5 feet	34	Low

Soluble Sulfates

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

Sample Identification	Soluble Sulfates (%)	Sulfate Classification
B-3 @ 0 to 5 feet	0.006	Negligible
B-6 @ 0 to 5 feet	0.006	Negligible

Grain Size Analysis

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

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on selected samples of various soil strata encountered at the site. This test is used to determine the Liquid Limit and Plastic Limit of the soil. The Plasticity Index is the difference between the two limits. Plasticity Index is a general indicator of the expansive potential of the soil, with higher numbers indicating higher expansive potential. Soils with a PI greater than 25 are considered to have a high plasticity, and a high expansion potential. Soils with a PI greater than 18 are not considered to be susceptible to liquefaction when the moisture content of the soil is less than 80 percent of the liquid limit. The results of the Atterberg Limits testing are presented on the boring logs.

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. Furthermore, SCG did not identify any evidence of faulting during the geotechnical investigation. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

Based on information provided by the EIR consultant, we understand that the proposed development is expected to be designed in accordance with the requirements of the 2019 edition of the California Building Code (CBC), which becomes effective January 1, 2020. 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 2019 CBC Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool, a web-based software application available at the website www.seismicmaps.org. This software application calculates seismic design parameters in accordance with several building code reference documents, including ASCE 7-16, upon which the 2019 CBC is based. The application utilizes a database of risk-targeted maximum considered earthquake (MCE_R) site accelerations at 0.01-degree intervals for each of the code documents.

The 2019 CBC requires that a site-specific ground motion study be performed in accordance with Section 11.4.8 of ASCE 7-16 for Site Class D sites with a mapped S_1 value greater than 0.2. However, Section 11.4.8 of ASCE 7-16 also indicates an exception from the requirement for a site-specific ground motion hazard analysis for certain structures on Site Class D sites. The commentary for Section 11 of ASCE 7-16 (Page 534 of Section C11 of ASCE 7-16) indicates that "In general, this exception effectively limits the requirements for site-specific hazard analysis to very tall and or flexible structures at Site Class D sites." **Based on our understanding of the proposed development, the seismic design parameters presented below were calculated assuming that the exception in Section 11.8.4 applies to the proposed structure(s) at this site. However, the structural engineer should verify that this exception is applicable to the proposed structures.** Based on the exception, the spectral response accelerations presented below were calculated using the site coefficients (F_a and F_v) from Tables 1613.2.3(1) and 1613.2.3(2) presented in Section 16.4.4 of the 2019 CBC.

The table below was created using data obtained from the application. The output generated from this program is included as Plate E-1 in Appendix E of this report. Based on this output, the following parameters may be utilized for the subject site:

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped MCE_R Acceleration at 0.2 sec Period	S_S	1.500
Mapped MCE_R Acceleration at 1.0 sec Period	S_1	0.576
Site Class	---	D*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.500
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.993
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.000
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.662

*The 2019 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 11.4.8 of ASCE 7-16. However, Section 20.3.1 of ASCE 7-16 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 (F_a and F_v) for Site Class D, assuming that the fundamental period of the structure is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. Therefore, if the proposed structure has a fundamental period greater than 0.5 seconds, a site-specific seismic hazards analysis would be required and additional subsurface exploration would be necessary.

It should be noted that the site coefficient F_v and the parameters S_{M1} and S_{D1} were not included in the underlined SEAOC/OSHPD Seismic Design Maps Tool output for the 2019 CBC. We calculated these parameters-based Section 16.4.4 of the 2019 CBC using the value of S_1 obtained from the Seismic Design Maps Tool.

Ground Motion Parameters

For the purposes of the liquefaction analysis performed for this study, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2019 CBC. The peak ground acceleration (PGA_M) was determined in accordance with Section 11.8.3 of ASCE 7-16. 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 7-16. The web-based software application SEAOC/OSHPD Seismic Design Maps Tool (described in the previous section) was used to determine PGA_M, based on ASCE 7-16 as the building code reference document. A portion of the program output is included as Plate E-1 in Appendix E of this report. As indicated on Plate E-1, the PGA_M for this site is 0.55g. 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 7.09, based on the peak ground acceleration and Site Class D soils.

Liquefaction

The Riverside County GIS website indicates that the subject site is located within a zone of high liquefaction susceptibility. Based on this mapping, the scope of this investigation included additional subsurface exploration, laboratory testing, and engineering analysis 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 $(N_1)_{60\text{-cs}}$,

SOUTHERN
CALIFORNIA
GEOTECHNICAL

Rider 2 – Perris, CA
Project No. 17G199-1R
Page 11

adjusted for fines content. The factor of safety against liquefaction is defined as CRR/CSR. Based on Special Publication 117A, a factor of safety of at least 1.3 is required in order to demonstrate that a given soil stratum is non-liquefiable. Additionally, in accordance with Special Publication 117A, clayey soils which do not meet the criteria for liquefiable soils defined by Bray and Sancio (2006), loose soils with a plasticity index (PI) less than 12 and moisture content greater than 85% of the liquid limit, are considered to be insusceptible to liquefaction. Non-sensitive soils with a PI greater than 18 are also considered non-liquefiable.

As part of the liquefaction evaluation, Boring Nos. B-1 and B-8 were extended to depths of 50± feet. The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report, using the data obtained from these borings. The liquefaction potential of the site was analyzed utilizing a PGA_M of 0.55g for a magnitude 7.09 seismic event.

The historic high groundwater depth was obtained from the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/, which indicates a historic high groundwater depth in the vicinity of the subject site of approximately 26 feet. Free water was encountered during the drilling of Boring No. B-1 and B-8 at a depth of 33 feet. Therefore, the historic high groundwater table was conservatively assumed to exist at a depth of 26± 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

Potentially liquefiable soils were encountered one of the 50-foot deep boring locations. None of the soil strata encountered at Boring No. B-1 are subject to liquefaction during the design seismic event. One (1) stratum of soils encountered at Boring No. B-8 was identified to be potentially liquefiable. This stratum exists at a depth of 32 to 37± feet. The remaining soil strata encountered below the historic high groundwater table either possess adequate factors of safety, or are considered non-liquefiable due to their cohesive characteristics and the results of the Atterberg limits testing with respect to the requirements of Special Publication 117A. Settlement analyses were conducted for the potentially liquefiable strata. The results of the settlement analyses indicate the following total deformations:

- Boring No. B-1:   0 inches
- Boring No. B-8:   1.59 inches

Based on the results of the settlement analyses, differential settlements are expected to be on the order of 1.5± inches or less. The estimated differential settlement can be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of approximately 0.001 inches per inch.

Based on our understanding of the proposed development, it is considered feasible to support the proposed structure on shallow foundations. Such a foundation system can be designed to

resist the effects of the anticipated differential settlements, to the extent that the structures would not catastrophically fail. Designing the proposed structure to remain completely undamaged during a seismic event that could occur once every 2475 years (the code-specified return period used in the liquefaction analysis) is not considered to be economically feasible. Based on this understanding, the use of shallow foundation systems is considered to be the most economical means of supporting the proposed structure.

In order to support the proposed structure 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 re-leveling, 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 the extent of liquefiable soils encountered at this site. The post-liquefaction damage that could occur within the building proposed for this site will also be typical of similar buildings in the vicinity of this project. However, if the owner determines that this level of potential damage is not acceptable, other geotechnical and structural options are available, including the use of ground improvement or mat foundations.

6.2 Geotechnical Design Considerations

General

The subsurface conditions encountered at the boring locations generally consist of variable strength native alluvium. The results of laboratory testing indicate that the near surface alluvium (within the upper 5± feet) possesses a potential for moderate collapse when exposed to moisture infiltration as well as excessive consolidation when exposed to load increases in the range of those that will be exerted by the new foundations. By visual examination, the majority of the near surface samples also possess calcareous nodules and veining throughout, and appear to be weakly cemented. Cemented soils with low relative densities are generally prone to settlement due to collapse when inundated with water. Based on these conditions, remedial grading will be necessary within the proposed building area to provide a subgrade suitable for support of the new foundations and floor slab. The remedial grading will also serve to create more uniform support characteristics across any cut/fill transitions.

Settlement

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

SOUTHERN
CALIFORNIA
GEOTECHNICAL

Rider 2 – Perris, CA
Project No. 17G199-1R
Page 13

Expansion

Laboratory testing performed on representative samples of the near surface soils indicates that these materials possess a low to medium expansion potential (EI = 34 and 68). Based on the presence of expansive soils at this site, care should be given to proper moisture conditioning of all building pad subgrade soils to a moisture content of 2 to 4 percent above the ASTM D-1557 optimum during site grading. In addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintaining moisture content of these soils at 2 to 4 percent above the optimum moisture content. This will require the contractor to frequently moisture condition these soils throughout the grading process, unless grading occurs during a period of relatively wet weather.

Shrinkage/Subsidence

Removal and recompaction of the near-surface native fill soils is estimated to result in an average shrinkage of 5 to 10 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 0.10 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

It is recommended that we be provided with copies of the grading and foundation plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

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

Site Stripping

Initial site preparation should include stripping of any surficial vegetation and organic soils. Based on conditions encountered at the time of the subsurface exploration, minor stripping of the crop

SOUTHERN
CALIFORNIA
GEOTECHNICAL
SoCalGeo

Rider 2 – Perris, CA
Project No. 17G199-1R
Page 14

stubble and native grass and weed growth is expected to be necessary. These materials should be disposed of offsite. 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 encountered materials.

<u>Treatment of Existing Soils: Building Pad</u>

Remedial grading should be performed within the proposed building pad area in order to remove the existing potentially compressible/collapsible native alluvium. It is recommended that the overexcavation extend to a depth of at least 4 feet below existing grade and to a depth of at least 4 feet below proposed grade, whichever is greater. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade.

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

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

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. Concrete and asphalt debris that is crushed to a 3 to 6-inch particle size may also be feasible to use as a subgrade stabilization material. 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.

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 2 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The building pad areas may then be raised to grade with previously excavated soils or imported, structural fill. All

SOUTHERN
CALIFORNIA
GEOTECHNICAL
SoCalGeo

structural fill soils present within the proposed building area should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of proposed retaining and non-retaining site walls should be overexcavated to a depth of at least 3 feet below foundation bearing grade and replaced as compacted structural fill. Any undocumented fill soils should also be removed from the retaining wall areas. 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: Parking Areas

Based on economic considerations, overexcavation of the surficial alluvial soils in the new parking areas is not considered warranted, with the exception of areas where lower strength or unstable soils are identified by the geotechnical engineer during grading.

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

The grading recommendations presented above for the proposed parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within the proposed parking areas. The grading recommendations presented above do not completely mitigate the extent of existing collapsible and compressible alluvium in the parking areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking 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.

Treatment of Existing Soils: Flatwork Areas

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

Fill Placement

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

Imported Structural Fill

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

Utility Trench Backfill

In general, all utility trench backfill 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 Perris. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

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

6.4 Construction Considerations

Excavation Considerations

The near surface soils generally consist of low to moderate strength clayey silts and sandy clays. These materials may be subject to minor caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 1.5h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation

SOUTHERN
CALIFORNIA
GEOTECHNICAL
SoCalGeo

Rider 2 – Perris, CA
Project No. 17G199-1R
Page 17

stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Expansive Soils

Based on the results of laboratory testing, some of the near surface soils have been determined to be medium expansive. Therefore, care should be given to proper moisture conditioning of all building pad subgrade soils to a moisture content of 2 to 4 percent above the Modified Proctor optimum during site grading. All imported fill soils should have very low expansive characteristics. In addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintain moisture content of these soils at 2 to 4 percent above the Modified Proctor optimum. This will require the contractor to frequently moisture condition these soils throughout the grading process, unless grading occurs during a period of relatively wet weather.

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

- Ponding and areas of low flow gradients in unpaved walkways, grass and planter areas should be avoided. In general, minimum drainage gradients of 2 percent should be maintained in unpaved areas.
- Bare soil within five feet of proposed structures should be sloped at a minimum five percent gradient away from the structure (about three inches of fall in five feet), or the same area could be paved with a minimum surface gradient of one percent. Pavement is preferable.
- Decorative gravel ground cover tends to provide a reservoir for surface water and may hide areas of ponding or poor drainage. Decorative gravel is, therefore, not recommended and should not be utilized for landscaping unless equipped with a subsurface drainage system designed by a licensed landscape architect.
- Positive drainage devices, such as graded swales, paved ditches, and catch basins should be installed at appropriate locations within the area of proposed development.
- Concrete walks and flatwork should not obstruct the free flow of surface water to the appropriate drainage devices.
- Area drains should be recessed below grade to allow free flow of water into the drain. Concrete or brick flatwork joints should be sealed with mortar or flexible mastic.
- Gutter and downspout systems should be installed to capture all discharge from roof areas. Downspouts should discharge directly into a pipe or paved surface system to be conveyed offsite.
- Enclosed planters adjoining, or in close proximity to proposed structures, should be sealed at the bottom and provided with subsurface collection systems and outlet pipes.
- Depressed planters should be raised with soil to promote runoff (minimum drainage gradient two percent or five percent, see above), and/or equipped with area drains to eliminate ponding.
- Drainage outfall locations should be selected to avoid erosion of slopes and/or properly armored to prevent erosion of graded surfaces. No drainage should be directed over or towards adjoining slopes.

- All drainage devices should be maintained on a regular basis, including frequent observations during the rainy season to keep the drains free of leaves, soil and other debris.
- Landscape irrigation should conform to the recommendations of the landscape architect and should be performed judiciously to preclude either soaking or excessive drying of the foundation soils. This should entail regular watering during the drier portions of the year and little or no irrigation during the rainy season. Automatic sprinkler systems should, therefore, be switched to manual operation during the rainy season. Good irrigation practice typically requires frequent application of limited quantities of water that are sufficient to sustain plant growth, but do not excessively wet the soils. Ponding and/or run-off of irrigation water are indications of excessive watering.

Other provisions, as determined by the landscape architect or civil engineer, may also be appropriate.

Groundwater

The static groundwater table is considered to exist at a depth of 33± feet below existing grade. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by structural fill soils extending to depths of at least 3 feet below foundation bearing grade. Based on this subsurface profile, and based on the design considerations presented in Section 6.1 of this report, the proposed structure may be supported on conventional shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

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

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

- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom), due to the presence of expansive soils and minor amounts of liquefaction-induced settlement.

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

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

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is

based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements, as discussed in Section 6.1. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

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

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

Estimated Foundation Settlements

Post-construction total and differential static 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: 275 lbs/ft^3
- Friction Coefficient: 0.28

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 soils. The maximum allowable passive pressure is 2,500 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the **Site Grading Recommendations** section of this report. Based on the anticipated grading which will occur at this site, and based on the design considerations presented in Section 6.1 of this report, the floor of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, extending to a depth of at least 4 feet below finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.

- Modulus of Subgrade Reaction: 100 lbs/in^3.

- Minimum slab reinforcement: No. 3 bars at 18-inches on-center, in both directions, due to presence of medium expansive soils and potentially liquefiable soils, at this site. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading, and the potential liquefaction induced settlements.

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

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

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

6.7 Exterior Flatwork Design and Construction

Subgrades which will support new exterior slabs-on-grade for sidewalks, patios, and other concrete flatwork, should be prepared in accordance with the recommendations contained in the **Grading Recommendations** section of this report. Based on geotechnical considerations, exterior slabs on grade may be designed as follows:

- Minimum slab thickness: 4½ inches.

- Minimum slab reinforcement: No. 3 bars at 18 inches on center, in both directions.

- The flatwork at building entry areas should be structurally connected to the perimeter foundation that is recommended to span across the door opening. This recommendation is designed to reduce the potential for differential movement at this joint.

- Moisture condition the slab subgrade soils to at least 2 to 4 percent of optimum moisture content, to a depth of at least 12 inches. Adequate moisture conditioning should be verified by the geotechnical engineer 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.

- Control joints should be provided at a maximum spacing of 8 feet on center in two directions for slabs and at 6 feet on center for sidewalks. Control joints are intended to direct cracking. Minor cracking of exterior concrete slabs on grade should be expected.

Expansion or felt joints should be used at the interface of exterior slabs on grade and any fixed structures to permit relative movement.

6.8 Retaining Wall Design and Construction

Although not indicated on the site plan, some small (less than 6 feet in height) retaining walls may be required to facilitate the new site grades. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. The following parameters assume that only the on-site soils will be utilized for retaining wall backfill. The near surface soils generally consist of clayey silts and sandy clays. Based on their composition, the on-site soils have been assigned a friction angle of 28 degrees. It is recommended that the medium expansive soils be excluded from use as retaining wall backfill, where possible.

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 Clayey Sands and Sandy Clays
Internal Friction Angle (ϕ)		28°
Unit Weight		115 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	42 lbs/ft^3
	Active Condition (2h:1v backfill)	73 lbs/ft^3
	At-Rest Condition (level backfill)	61 lbs/ft^3

The walls should be designed using a soil-footing coefficient of friction of 0.28 and an equivalent passive pressure of 275 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

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

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

Seismic Lateral Earth Pressures

In accordance with the 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.

Retaining Wall Foundation Design

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

Backfill Material

On-site soils may be used to backfill the retaining walls, provided that they are low expansive (EI < 50). All backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

It is recommended that a minimum 1 foot thick layer of free-draining granular material (less than 5 percent passing the No. 200 sieve) be placed against the face of the retaining walls. This material should extend from the top of the retaining wall footing to within 1 foot of the ground surface on the back side of the retaining wall. This material should be approved by the geotechnical engineer. In lieu of the 1 foot thick layer of free-draining material, a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls, may be used. If the layer of free-draining material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The layer of free draining granular material should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

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

Subsurface Drainage

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

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

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

6.9 Pavement Design Parameters

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

SOUTHERN
CALIFORNIA
GEOTECHNICAL
SoCalGeo

Rider 2 – Perris, CA
Project No. 17G199-1R
Page 24

designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near surface soils generally consist of clayey silts and silty clays. These soils are generally considered to possess poor to fair pavement support characteristics with an estimated R-values of 25 to 35. The subsequent pavement design is therefore based upon an assumed R-value of 25. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

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

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

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

ASPHALT PAVEMENTS (R=25)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	4	4½	5	6
Aggregate Base	7	8	11	12	14
Compacted Subgrade	12	12	12	12	12

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

Portland Cement Concrete

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

PORTLAND CEMENT CONCRETE PAVEMENTS (R=25)				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	7	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. Any reinforcement within the PCC pavements should be determined by the project structural engineer. 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

SITE LOCATION MAP

RIDER 2 - PROPOSED COMMERCIAL/INDUSTRIAL BUILDING
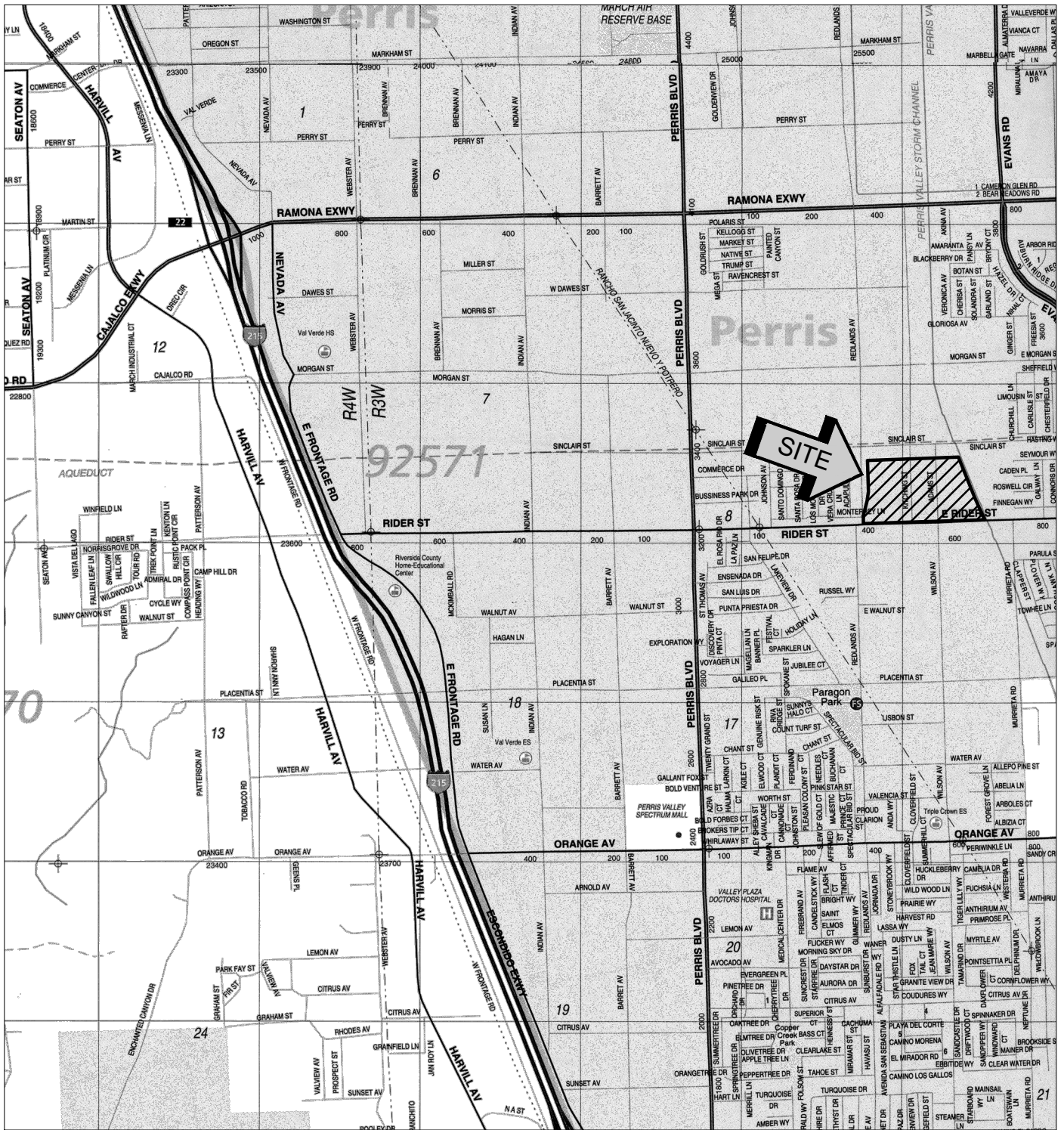
PERRIS, CALIFORNIA

SOURCE: RIVERSIDE COUNTY
THOMAS GUIDE, 2013

SCALE: 1" = 2400'

DRAWN: SAM
CHKD: GKM

SCG PROJECT
17G199-1

PLATE 1

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL

REDLANDS AVE

PERRIS VALLEY STORM CHANNEL

B-9

B-10

B-1

B-2

B-3

BUILDING AREA
820,676 S.F.

B-5

B-4

B-7

B-8

B-6

B-11

B-12

RIDER STREET
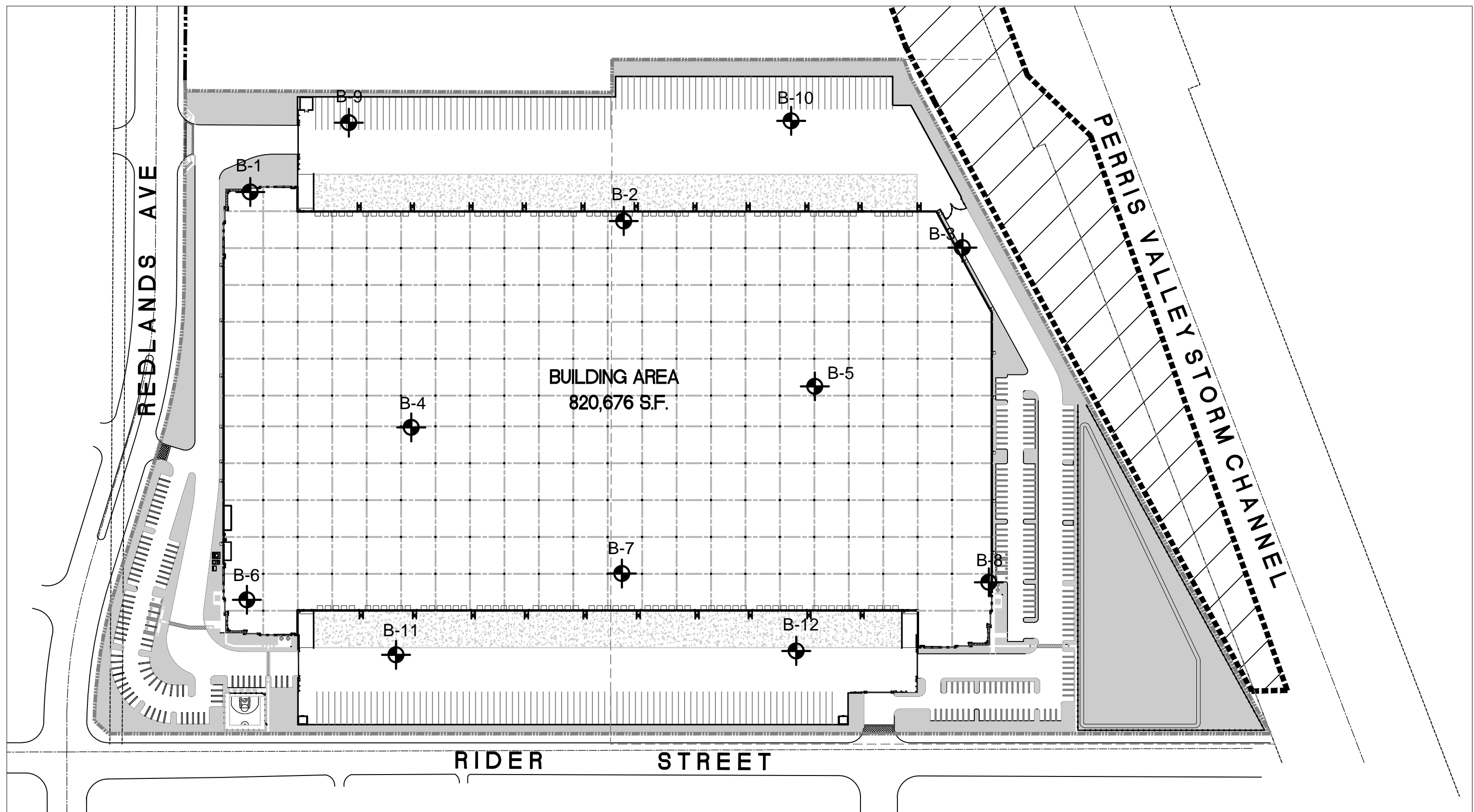
GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

NORTH
SoCalGeo
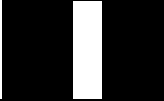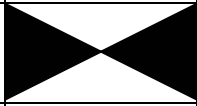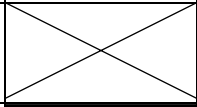
NOTE: BASE MAP PREPARED BY HPA ARCHITECTS, INC.

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

JOB NO.: 17G199	DRILLING DATE: 11/3/17	WATER DEPTH: 33 feet
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 33 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

FIELD RESULTS / LABORATORY RESULTS

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1443.5 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	⊠	40			ALUVIUM: Light Brown Silty fine Sand, trace Clay, trace fine root fibers, dense-damp to moist		8					
	⊠	32	2.5		Gray Brown Clayey Silt, trace fine Sand, trace calcareous veining, slightly porous, hard-moist to very moist		19					
5	⊠	17			Gray Brown fine Sandy Silt, trace Clay, some calcareous nodules and veining, medium dense-moist		14					
	⊠	21	2.5		Gray Brown Clayey Silt to Silty Clay, trace fine Sand, abundant calcareous nodules and veining, very stiff-very moist		25					
10												
	⊠	19	4.0				17					
15												
	⊠	28			Brown Clayey fine Sand, little medium Sand, trace calcareous nodules, medium dense-damp to moist		15					
20												
	⊠	60					8					
25												
	⊠	32					15					
30												
	⊠	36			@ 33 feet, Water encountered during drilling		12					

TBL 17G199.GPJ SOCALGEO.GDT 11/22/17

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

JOB NO.: 17G199	DRILLING DATE: 11/3/17	WATER DEPTH: 33 feet
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 33 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Brown Clayey fine Sand, little medium Sand, trace calcareous nodules, medium dense-damp to moist							
40	X	19	3.0		Brown Clayey Silt to Silty Clay, some fine Sand, very stiff-very moist		15			57		
45	X	25			Gray Brown Silty fine Sand, trace medium Sand, medium dense-wet		16			38		
50	X	43			Brown Silty fine to medium Sand, dense-wet		16					
					Boring Terminated at 50'							

TEST BORING LOG　　　　　　　　　　　　　　　　　**PLATE B-1b**

JOB NO.: 17G199	DRILLING DATE: 11/2/17	WATER DEPTH: Dry
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 8 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

FIELD RESULTS / LABORATORY RESULTS

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION — SURFACE ELEVATION: 1443 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	⊠	7			ALUVIUM: Light Brown Silty fine Sand, trace Clay, trace fine root fibers, loose-damp		5					
	⊠	16	3.5		Light Gray Clayey Silt, trace fine Sand, slightly porous, some calcareous veining and nodules, stiff to very stiff-moist to very moist		19					
5	⊠	13	1.0				16					
	⊠	10	1.0		Dark Gray Brown Clayey Silt, stiff-moist		13					
10												
	⊠	28			Brown fine Sandy Clay, very stiff-moist		13					
15												
	⊠	37			Brown fine Sandy Silt, dense-damp to moist		13					
20												

Boring Terminated at 20'

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

JOB NO.: 17G199	DRILLING DATE: 11/2/17	WATER DEPTH: Dry
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 12 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1442 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
		36	4.5+		ALLUVIUM: Light Gray Brown Clayey Silt, trace fine root fibers, very stiff-damp to moist	108	9					EI = 68 @ 0 to 5'
		34	2.5		Dark Gray Silty fine Sand, medium dense-damp to moist	89	11					
5		38	4.5+		Light Gray Silty Clay, abundant calcareous veining and nodules, hard-moist	109	15					
		26	4.5+		Brown fine Sandy Clay, very stiff to hard-moist	110	15					
10		30	4.5+			120	12					
15		38			Brown fine Sandy Silt, medium dense-damp to moist		13					
20		29			Brown Silty fine Sand, trace Clay nodules, medium dense-damp to moist		13					
25		46			Brown fine Sandy Silt, some Clay, dense-moist		13					
					Boring Terminated at 25'							

TBL 17G199.GPJ SOCALGEO.GDT 11/22/17

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
SoCalGeo
A California Corporation

JOB NO.: 17G199	DRILLING DATE: 11/2/17	WATER DEPTH: Dry
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 8 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

FIELD RESULTS / LABORATORY RESULTS

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION — SURFACE ELEVATION: 1442.5 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	⊠	34			ALLUVIUM: Light Brown Clayey Silt, trace fine Sand, trace fine root fibers, very stiff-damp	82	7					
	⊠	26	3.0		Gray Brown Clayey Silt to Silty Clay, abundant calcareous veining and nodules, very stiff-very moist	80	22					
5	⊠	26					23					Disturbed sample
	⊠	15	3.0			84	25					
10	⊠	34			Gray Brown Clayey fine Sand, abundant calcareous nodules, porous, medium dense-damp	115	7					
					Brown fine Sandy Clay, calcareous nodules, very stiff-damp							
15	⊠	25	2.0				10					

Boring Terminated at 15'

TBL 17G199.GPJ SOCALGEO.GDT 11/22/17

TEST BORING LOG   **PLATE B-4**

SOUTHERN CALIFORNIA GEOTECHNICAL
SoCalGeo
A California Corporation

BORING NO.
B-5

JOB NO.: 17G199	DRILLING DATE: 11/3/17	WATER DEPTH: Dry
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 8 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

FIELD RESULTS / LABORATORY RESULTS

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1442 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	▣	24			ALLUVIUM: Brown fine Sandy Silt, trace Clay, trace calcareous veining and nodules, medium dense-damp	100	9					
	▣	44				103	6					
5	▣	35	4.5+		Light Brown Silty Clay to Clayey Silt, trace fine Sand, abundant calcareous veining and nodules, very stiff-damp to moist	108	12					
	▣	31	1.5		@ 7 to 8 feet, very moist	85	26					
10	▣	35	4.5+			110	18					
	⊠	20			Light Brown Clayey fine Sand, trace medium Sand, medium dense-moist		13					
15												

Boring Terminated at 15'

TBL 17G199.GPJ SOCALGEO.GDT 11/22/17

TEST BORING LOG

PLATE B-5

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

JOB NO.: 17G199	DRILLING DATE: 11/2/17	WATER DEPTH: Dry
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 15 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1443 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	⊠	41	4.0		ALLUIVUM: Light Gray Clayey Silt, slightly porous, abundant calcareous veining and nodules, very stiff-damp	88	8					EI = 34 @ 0 to 5'
	⊠	36	4.5+			104	9					
5	⊠	67			Gray Brown Silty fine Sand, dense-damp	113	5					
	⊠	36	3.5		Light Brown Clayey Silt, abundant calcareous nodules, very stiff-moist	102	14					
10	⊠	26	3.0			99	17					
15	⊠	15			Brown Silty fine Sand, medium dense-damp to moist		7					
20	⊠	29			Light Brown fine to medium Sand, trace Silt, some coarse Sand, medium dense-damp		4					
25	⊠	35			Brown Clayey fine Sand, dense-moist		12					
					Boring Terminated at 25'							

TEST BORING LOG

PLATE B-6

SOUTHERN CALIFORNIA GEOTECHNICAL
SoCalGeo
A California Corporation

JOB NO.: 17G199	DRILLING DATE: 11/2/17	WATER DEPTH: Dry
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 12 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: 1442 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
		20			ALLUVIUM: Light Gray Brown Clayey Silt, trace fine Sand, trace to some calcareous nodules, very stiff-very moist		24					
		22					21					
5												
		27			Light Brown Silty fine Sand, trace Clay, some calcareous nodules, medium dense-damp to moist		8					
		31	4.0		Gray Brown Silty Clay, abundant calcareous nodules and veining, hard-moist		14					
10												
		13			Brown Clayey fine Sand, medium dense-moist		14					
15												
		42	4.5+		Light Brown fine Sandy Clay, hard-moist		14					
20												
					Boring Terminated at 20'							

TEST BORING LOG

PLATE B-7

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

BORING NO. B-8

JOB NO.: 17G199	DRILLING DATE: 11/2/17	WATER DEPTH: 33 feet
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 33 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

FIELD RESULTS | | | | LABORATORY RESULTS

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1441.5 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	⊠	17	4.5+		ALLUVIUM: Light Gray Brown Clayey Silt, trace fine root fibers, slightly porous, very stiff-damp		9					
5	⊠	12	2.0		Light Gray Clayey Silt to Silty Clay, abundant calcareous nodules and veining, stiff to very stiff-moist		17					
	⊠	16	3.5				13					
10	⊠	22	3.5		Brown fine Sandy Clay, very stiff-moist		14					
15	⊠	41	2.0		Brown Silty fine to medium Sand, dense-damp to moist		7					
20	⊠	25			Brown Silty fine Sand, medium dense-damp to moist		11					
25	⊠	39			Brown fine Sandy Silt, trace Clay, dense-moist		15					
30	⊠	39			Brown Silty fine Sand, dense-moist		13					
	⊠	10			@ 33 to 35', loose to medium dense-wet @ 33 feet, Water encountered during drilling		18			44		

TBL 17G199.GPJ SOCALGEO.GDT 11/22/17

TEST BORING LOG

PLATE B-8a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

SoCalGeo

JOB NO.: 17G199	DRILLING DATE: 11/2/17	WATER DEPTH: 33 feet
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 33 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

FIELD RESULTS					LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Brown Silty fine Sand, dense-moist						
					Brown fine to medium Sandy Clay, hard-very moist						
40	X	49					14				
45	X	52	4.0		Gray Brown Clayey Silt, trace fine Sand, hard-very moist		18				
					Brown fine Sandy Silt, dense-wet						
50	X	37					23				
					Boring Terminated at 50'						

TEST BORING LOG

PLATE B-8b

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation
SoCalGeo

JOB NO.: 17G199	DRILLING DATE: 11/3/17	WATER DEPTH: Dry
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 3 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

FIELD RESULTS | | | | | DESCRIPTION | LABORATORY RESULTS

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					SURFACE ELEVATION: 1444 feet MSL							
	⊠	6			ALLUVIUM: Light Brown fine Sandy Silt, trace Clay, fine root fibers, loose-moist		13					
	⊠	10	2.5		Light Brown Clayey Silt, some calcareous nodules, stiff-very moist		19					
5												
					Boring Terminated at 5'							

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 17G199	DRILLING DATE: 11/3/17	WATER DEPTH: Dry
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 3.5 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

FIELD RESULTS

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1443 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	⊠	23			ALLUVIUM: Light Brown fine Sandy Silt, trace Clay, some calcareous nodules, medium dense-moist		13					
	⊠	14			Light Brown Clayey Silt, calcareous nodules and veining, stiff-very moist		23					
5												

Boring Terminated at 5'

TBL 17G199.GPJ SOCALGEO.GDT 11/22/17

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL

SoCalGeo

A California Corporation

JOB NO.: 17G199	DRILLING DATE: 11/2/17	WATER DEPTH: Dry
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 3.5 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	**LABORATORY RESULTS**						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: 1442 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	⊠	18			ALLUVIUM: Gray Brown Clayey Silt, trace to little calcareous nodules and veining, very stiff-very moist		24					
	⊠	22					25					
5												
					Boring Terminated at 5'							

TEST BORING LOG   **PLATE B-11**

SOUTHERN CALIFORNIA GEOTECHNICAL

SoCalGeo

A California Corporation

JOB NO.: 17G199	DRILLING DATE: 11/2/17	WATER DEPTH: Dry
PROJECT: Rider 2	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 3.5 feet
LOCATION: Perris, California	LOGGED BY: Jason Hiskey	READING TAKEN: At Completion

FIELD RESULTS / LABORATORY RESULTS

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: 1442 feet MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	⊠	20			ALUVIUM: Light Brown fine Sandy Silt, trace Clay, trace fine root fibers, medium dense-damp		6					
	⊠	27			Gray Brown Clayey Silt to Silty Clay, abundant calcareous nodules, very stiff-moist		16					
5					Boring Terminated at 5'							

TBL 17G199.GPJ SOCALGEO.GDT 11/22/17

TEST BORING LOG

APPENDIX C

Consolidation/Collapse Test Results

Water Added at 1600 psf

Y-axis: **Consolidation Strain (%)** (0 to 20)
X-axis: **Load (ksf)** (0.1 to 100)
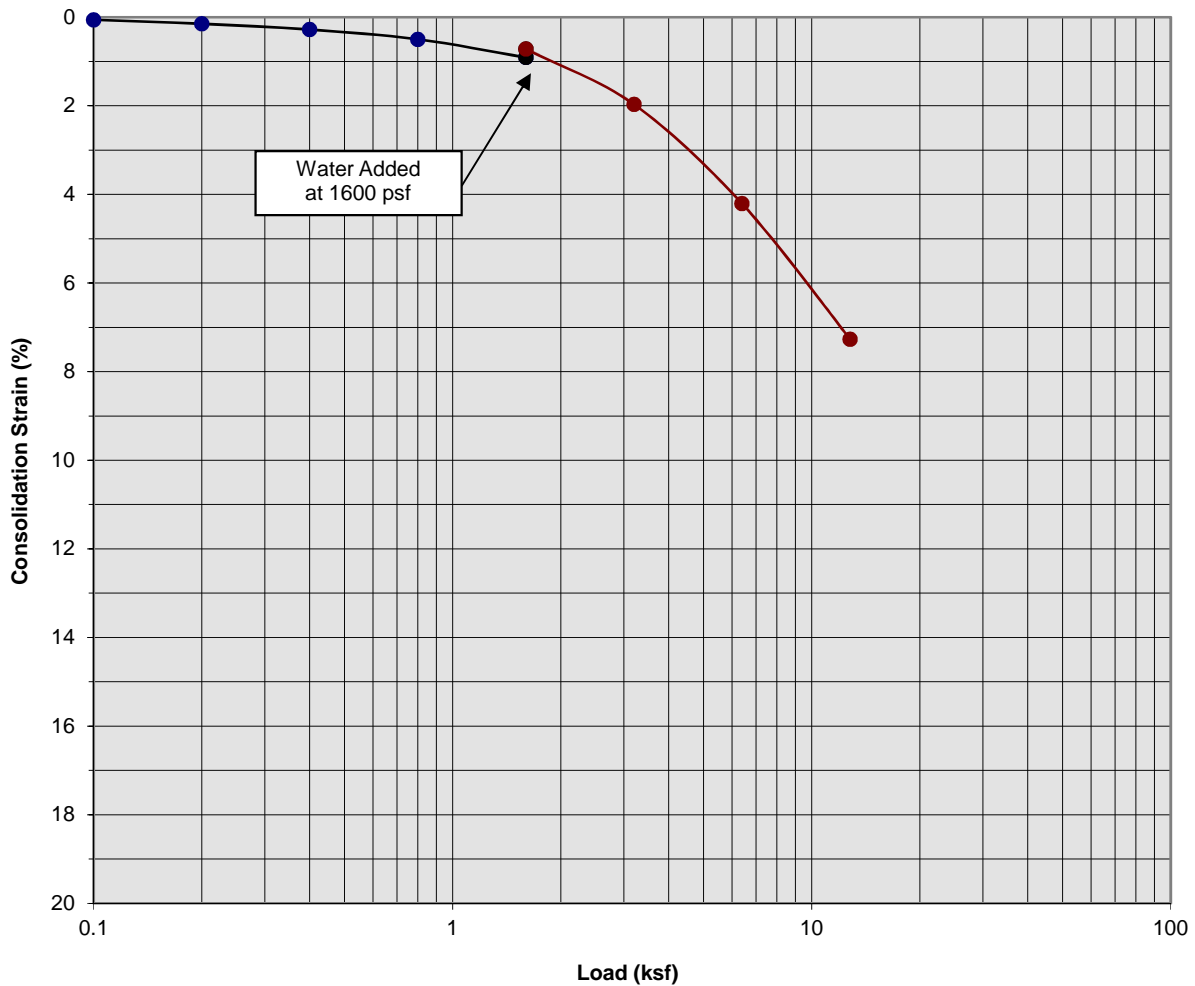
Classification: Light Gray Brown Clayey Silt

Boring Number:	B-3	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	1 to 2	Initial Dry Density (pcf)	106.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.19

Rider 2
Perris, California
Project No. 17G199
PLATE C- 1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

Chart: Consolidation Strain (%) vs Load (ksf). Annotation: "Water Added at 1600 psf"
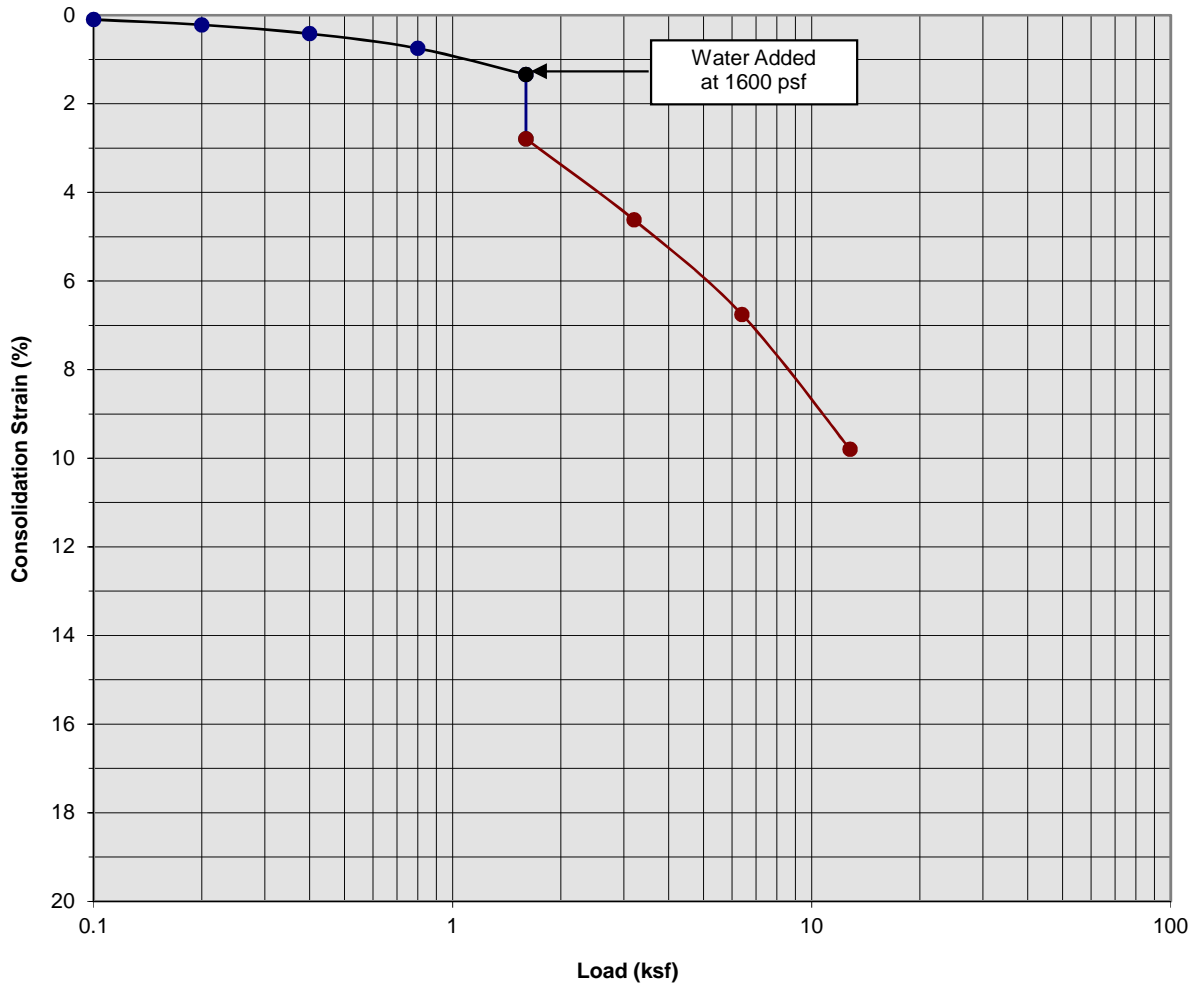
Classification: Dark Gray Silty fine Sand

Boring Number:	B-3	Initial Moisture Content (%)	12
Sample Number:	---	Final Moisture Content (%)	27
Depth (ft)	3 to 4	Initial Dry Density (pcf)	90.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	100.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.45

Rider 2
Perris, California
Project No. 17G199
PLATE C- 2

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

Water Added at 1600 psf

Y-axis: Consolidation Strain (%)
X-axis: Load (ksf)
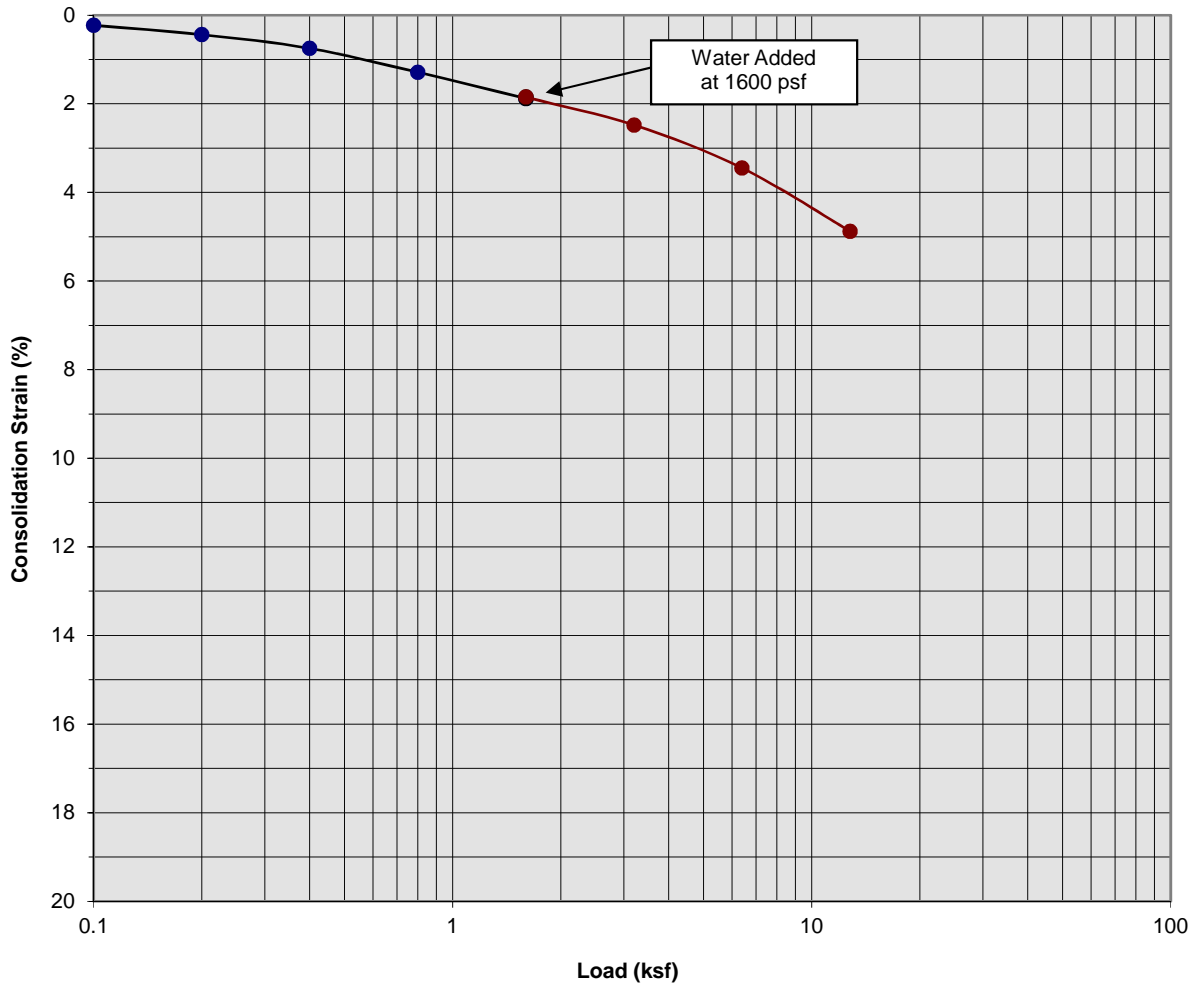
Classification: Light Gray Silty Clay

Boring Number:	B-3	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	5 to 6	Initial Dry Density (pcf)	108.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	114.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.03

Rider 2
Perris, California
Project No. 17G199
PLATE C- 3

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

Water Added at 1600 psf

Consolidation Strain (%)

Load (ksf)
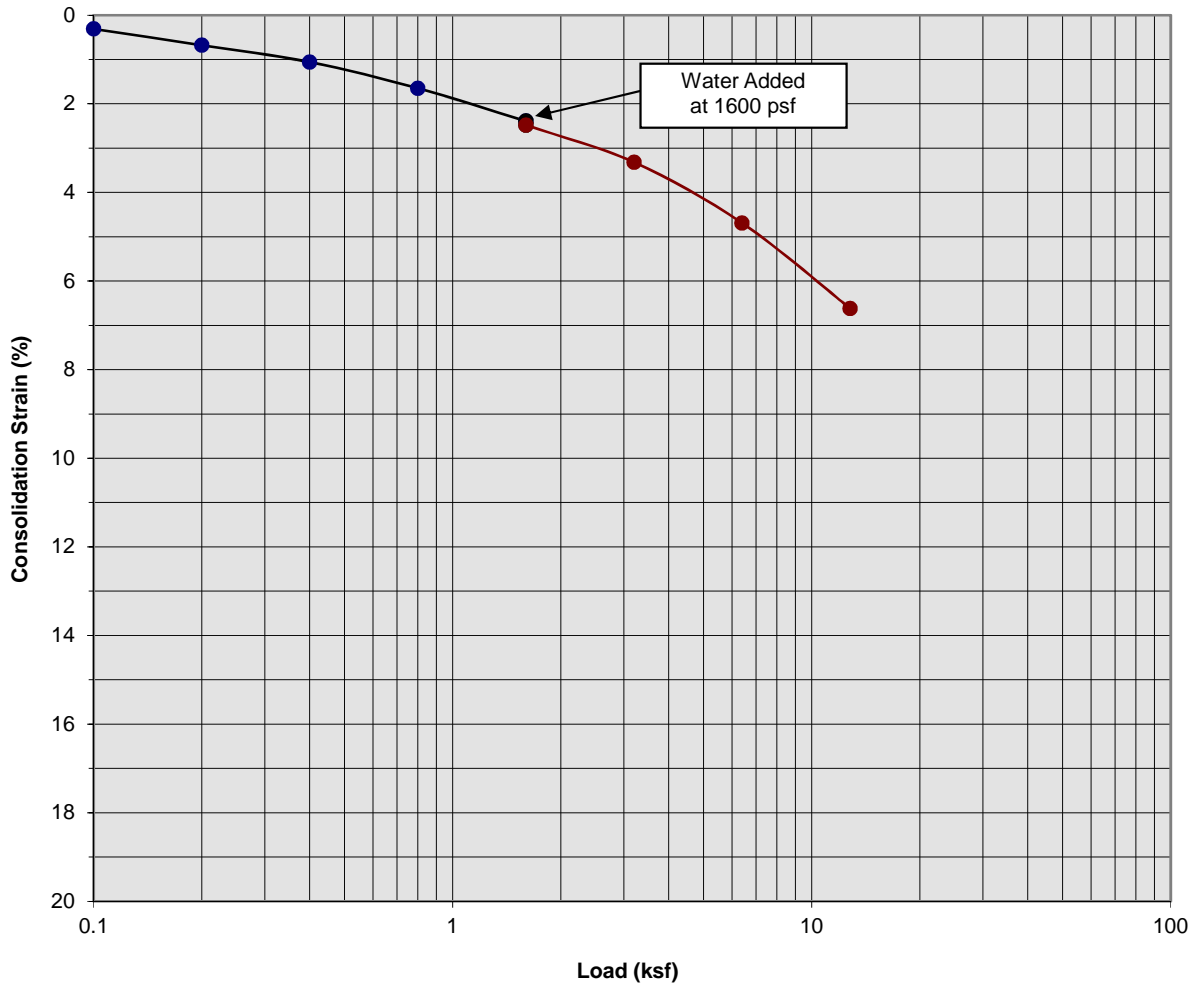
Classification: Brown fine Sandy Clay

Boring Number:	B-3	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	7 to 8	Initial Dry Density (pcf)	109.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	117.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.09

Rider 2
Perris, California
Project No. 17G199
PLATE C- 4

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

Water Added at 1600 psf

Y-axis: Consolidation Strain (%)
X-axis: Load (ksf)
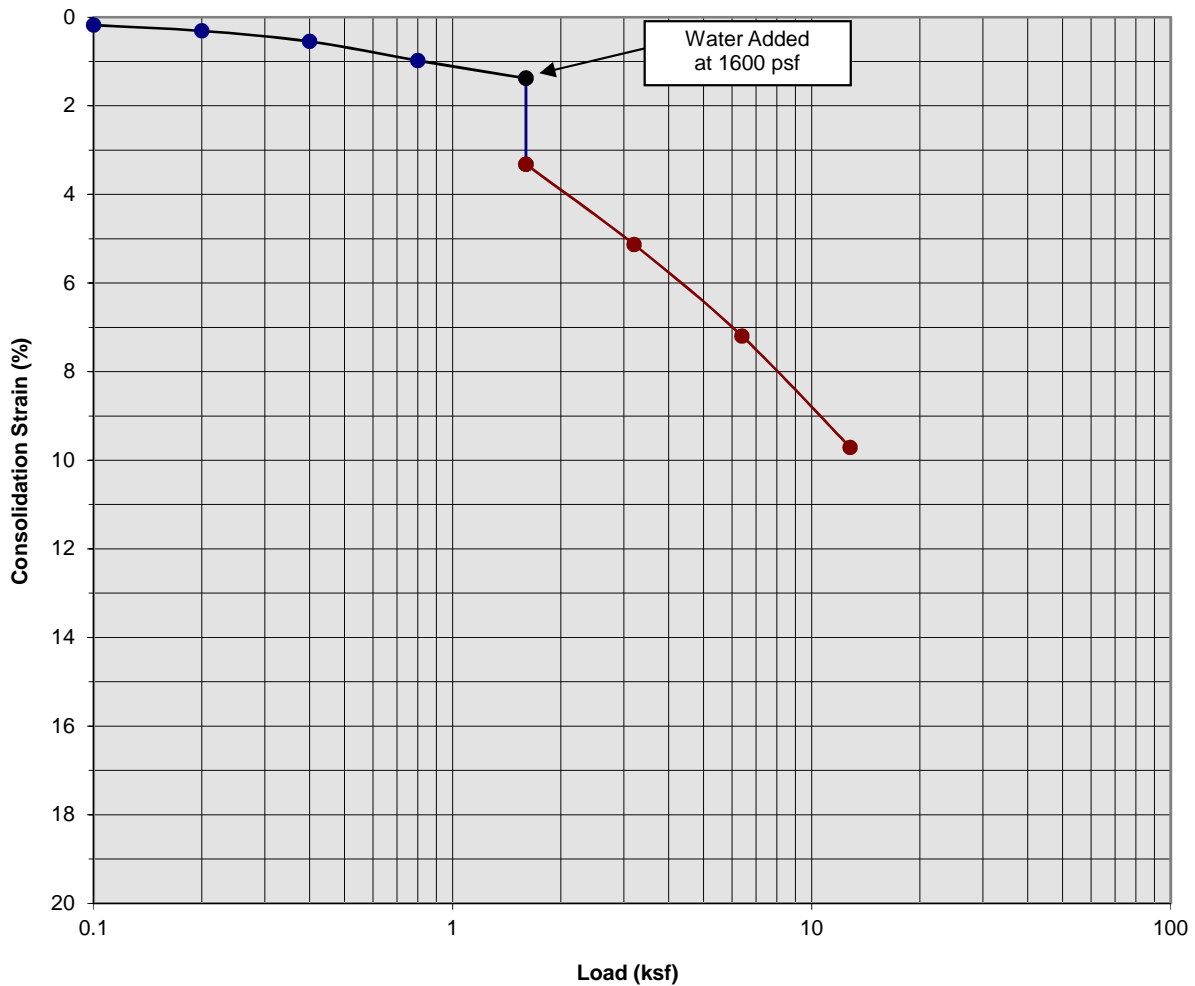
Classification: Light Gray Clayey Silt

Boring Number:	B-6		Initial Moisture Content (%)	8
Sample Number:	---		Final Moisture Content (%)	19
Depth (ft)	3 to 4		Initial Dry Density (pcf)	104.4
Specimen Diameter (in)	2.4		Final Dry Density (pcf)	115.3
Specimen Thickness (in)	1.0		Percent Collapse (%)	1.94

Rider 2
Perris, California
Project No. 17G199
PLATE C- 5

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

Consolidation Strain (%) vs Load (ksf)

Water Added at 1600 psf
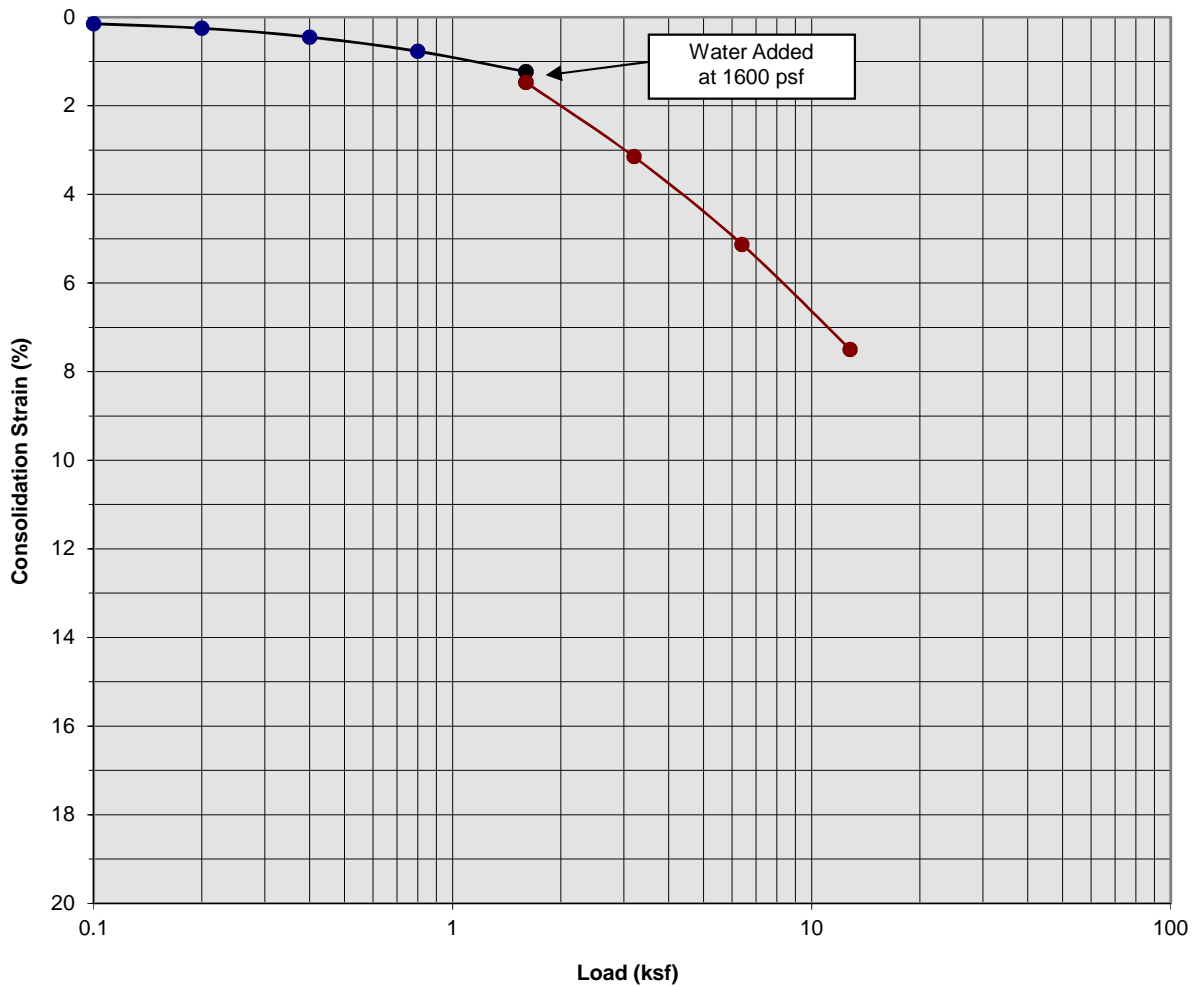
Classification: Gray Brown Silty fine Sand

Boring Number:	B-6	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	5 to 6	Initial Dry Density (pcf)	114.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	123.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.24

Rider 2
Perris, California
Project No. 17G199
PLATE C- 6

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

Water Added
at 1600 psf

Consolidation Strain (%)

Load (ksf)
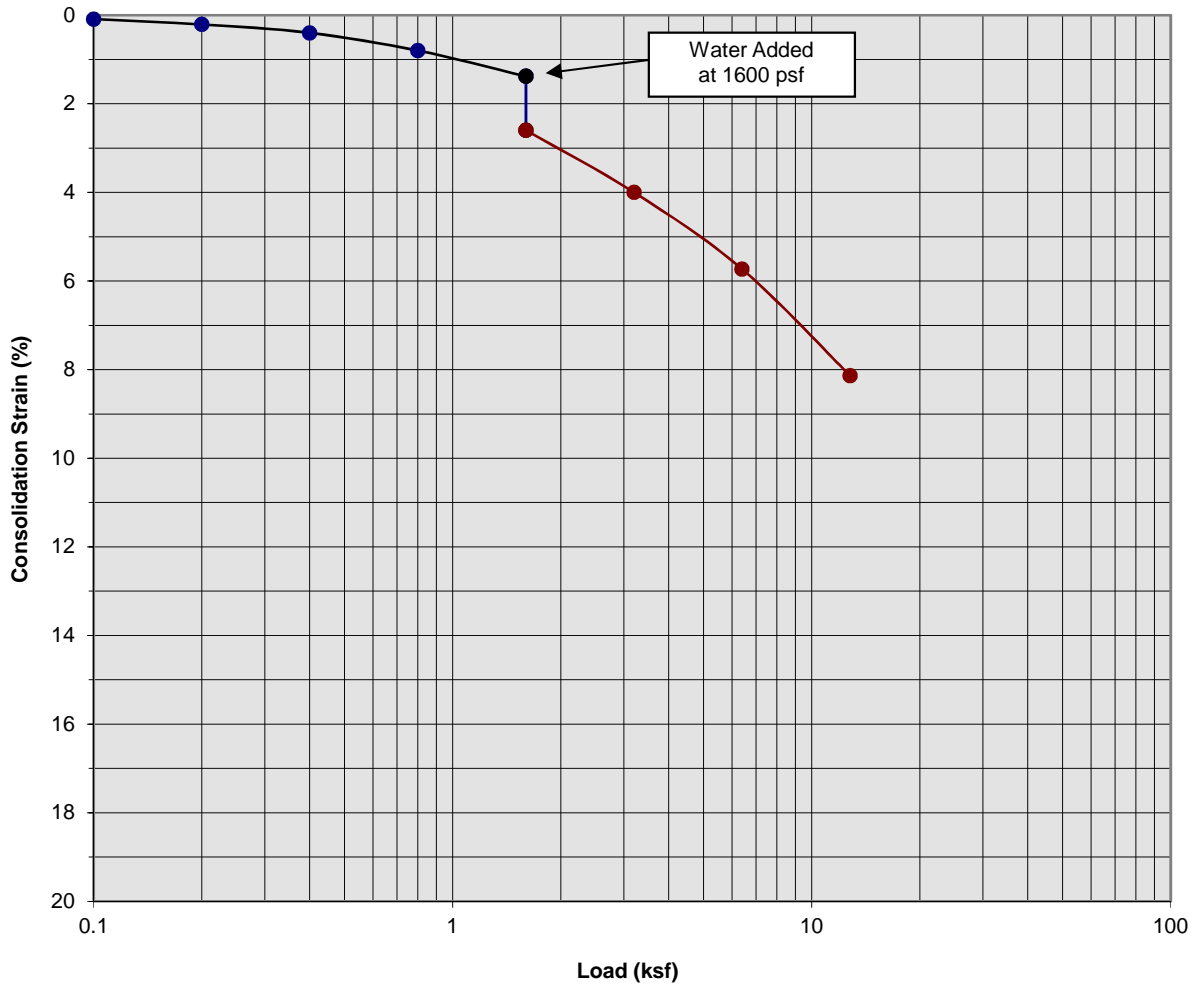
Classification: Light Brown Clayey Silt

Boring Number:	B-6	Initial Moisture Content (%)	14
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	7 to 8	Initial Dry Density (pcf)	102.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.22

Rider 2
Perris, California
Project No. 17G199
PLATE C- 7

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

Water Added at 1600 psf

Y-axis: Consolidation Strain (%)
X-axis: Load (ksf)
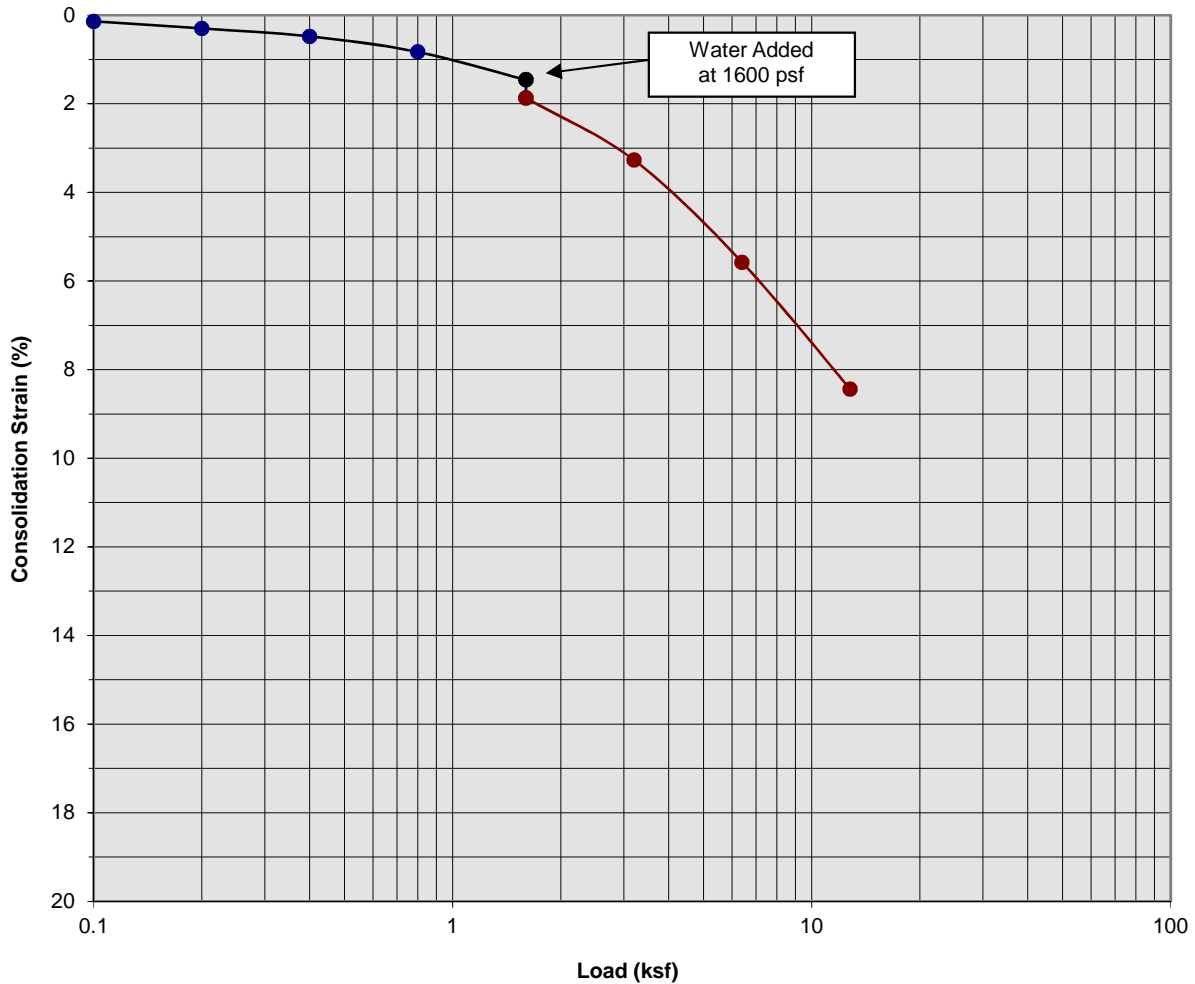
Classification: Light Brown Clayey Silt

Boring Number:	B-6	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	24
Depth (ft)	9 to 10	Initial Dry Density (pcf)	99.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.41

Rider 2
Perris, California
Project No. 17G199
PLATE C- 8

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

Moisture/Density Relationship
ASTM D-1557
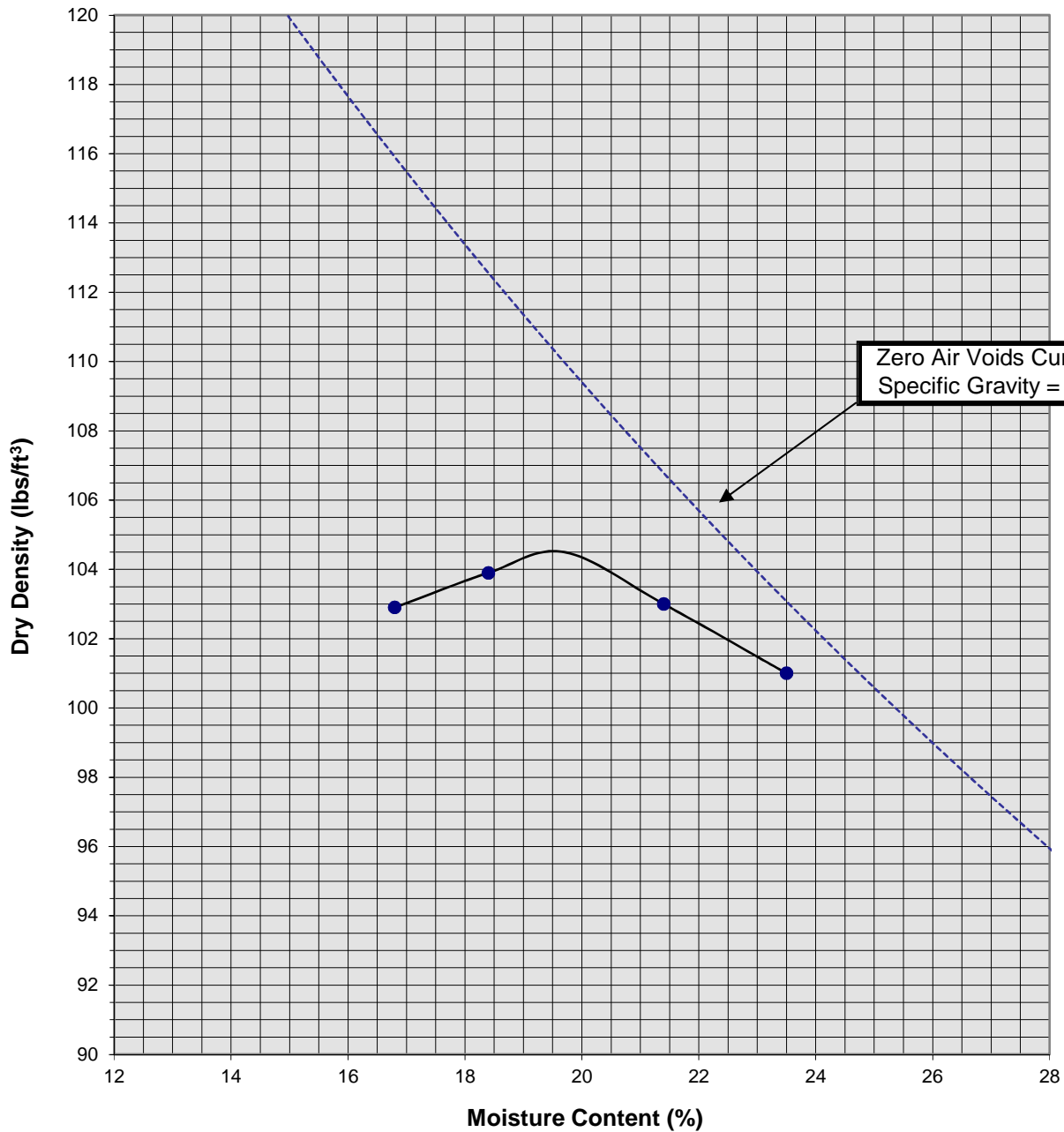
Zero Air Voids Curve:
Specific Gravity = 2.7

Dry Density (lbs/ft³)

Moisture Content (%)

Soil ID Number	B-4 @ 0 to 5'
Optimum Moisture (%)	19.5
Maximum Dry Density (pcf)	104.5
Soil Classification	Brown Clayey Silt, trace fine Sand

Rider 2
Perris, California
Project No. 17G199
PLATE C-9

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

APPENDIX

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.

<u>General</u>

- 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.

<u>Site Preparation</u>

- 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.

Compacted Fills

- 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

NATURAL GRADE

UNSUITABLE
MATERIAL

5' MIN.

3' MIN.*

COMPACTED FILL

OVEREXCAVATE AND
RECOMPACT

COMPETENT MATERIAL, AS APPROVED
BY THE GEOTECHNICAL ENGINEER

CUT/FILL LOT (TRANSITION)

NATURAL GRADE

5' MIN.

3' MIN.*

COMPACTED FILL

UNSUITABLE
MATERIAL

OVEREXCAVATE AND
RECOMPACT

COMPETENT MATERIAL, AS APPROVED
BY THE GEOTECHNICAL ENGINEER

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	SoCalGeo	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM		
PLATE D-1		

NEW COMPACTED FILL

COMPETENT MATERIAL

CUT/FILL CONTACT SHOWN
ON GRADING PLAN

CUT/FILL CONTACT TO BE
SHOWN ON "AS-BUILT"

9' MIN.

REMOVE UNSUITABLE MATERIAL

NATURAL GRADE

10' TYP.

4' TYP.

CUT SLOPE

BENCHING DIMENSIONS IN ACCORDANCE
WITH PLAN OR AS RECOMMENDED
BY THE GEOTECHNICAL ENGINEER

MINIMUM 1' TILT BACK
OR 2% SLOPE
(WHICHEVER IS GREATER)

BEDROCK OR APPROVED
COMPETENT MATERIAL

CUT SLOPE TO BE CONSTRUCTED
PRIOR TO PLACEMENT OF FILL

KEYWAY IN COMPETENT MATERIAL
MINIMUM WIDTH OF 15 FEET OR AS
RECOMMENDED BY THE GEOTECHNICAL
ENGINEER. KEYWAY MAY NOT BE
REQUIRED IF FILL SLOPE IS LESS THAN 5
FEET IN HEIGHT AS RECOMMENDED BY
THE GEOTECHNICAL ENGINEER.

FILL ABOVE CUT SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL

PLATE D-2

NATURAL GROUND

COMPACTED FILL

CLEANOUT EXCAVATION

6" MIN.

FIRM NATIVE SOIL/BEDROCK

18" MIN.

24" MIN.

MINUS 1" CRUSHED ROCK COMPLETELY
SURROUNDED BY FILTER FABRIC, OR
CLASS II PERMEABLE MATERIAL

4" MIN.

18" MIN.

6" DIAMETER PERFORATED PIPE - MINIMUM 1% SLOPE

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

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL

PLATE D-3

FINISHED SLOPE FACE

NEW COMPACTED FILL

OVERFILL REQUIREMENTS
PER GRADING GUIDE SPECIFICATIONS

COMPETENT MATERIAL

TOE OF SLOPE SHOWN
ON GRADING PLAN

PROJECT SLOPE GRADIENT
(1:1 MAX.)

PLACE COMPACTED BACKFILL
TO ORIGINAL GRADE

BACKCUT - VARIES

REMOVE UNSUITABLE MATERIAL

10' TYP.

4' TYP.

BENCHING DIMENSIONS IN ACCORDANCE
WITH PLAN OR AS RECOMMENDED
BY THE GEOTECHNICAL ENGINEER

MINIMUM 1' TILT BACK
OR 2% SLOPE
(WHICHEVER IS GREATER)

2' MINIMUM
KEY DEPTH

KEYWAY IN COMPETENT MATERIAL.
MINIMUM WIDTH OF 15 FEET OR AS
RECOMMENDED BY THE GEOTECHNIAL
ENGINEER. KEYWAY MAY NOT BE REQUIRED
IF FILL SLOPE IS LESS THAN 5' IN HEIGHT
AS RECOMMENDED BY THE GEOTECHNICAL
ENGINEER.

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	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
PLATE D-4	

3' TYPICAL
BLANKET FILL IF RECOMMENDED
BY THE GEOTECHNICAL ENGINEER

TOP WIDTH OF FILL
AS SPECIFIED BY THE
GEOTECHNICAL ENGINEER

COMPETENT MATERIAL ACCEPTABLE
TO THE SOIL ENGINEER

COMPACTED FILL

FACE OF FINISHED SLOPE

10' TYP.

4' TYP.

BENCHING DIMENSIONS IN ACCORDANCE
WITH PLAN OR AS RECOMMENDED
BY THE GEOTECHNICAL ENGINEER

MINIMUM 1' TILT BACK
OR 2% SLOPE
(WHICHEVER IS GREATER)

2' MINIMUM
KEY DEPTH

KEYWAY WIDTH, AS SPECIFIED
BY THE GEOTECHNICAL ENGINEER

STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL

PLATE D-5

DESIGN FINISH SLOPE

OUTLETS TO BE SPACED
AT 100' MAXIMUM INTERVALS.
EXTEND 12 INCHES
BEYOND FACE OF SLOPE
AT TIME OF ROUGH GRADING
CONSTRUCTION.

BLANKET FILL IF RECOMMENDED
BY THE GEOTECHNICAL ENGINEER

BUTTRESS OR
SIDEHILL FILL

10' MIN.
25' MAX.

15' MAX.

DETAIL "A"

2%

4-INCH DIAMETER NON-PERFORATED
OUTLET PIPE TO BE LOCATED IN FIELD
BY THE SOIL ENGINEER.

2' CLEAR

"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	

OUTLET PIPE TO BE CON-
NECTED TO SUBDRAIN PIPE
WITH TEE OR ELBOW
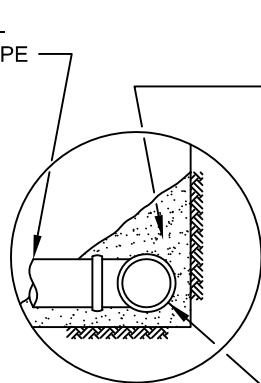
FILTER MATERIAL - MINIMUM OF FIVE
CUBIC FEET PER FOOT OF PIPE. SEE
ABOVE FOR FILTER MATERIAL SPECIFICATION.

ALTERNATIVE: IN LIEU OF FILTER MATERIAL
FIVE CUBIC FEET OF GRAVEL
PER FOOT OF PIPE MAY BE ENCASED
IN FILTER FABRIC. SEE ABOVE FOR
GRAVEL SPECIFICATION.

FILTER FABRIC SHALL BE MIRAFI 140
OR EQUIVALENT. FILTER FABRIC SHALL
BE LAPPED A MINIMUM OF 12 INCHES
ON ALL JOINTS.

DETAIL "A"

MINIMUM 4-INCH DIAMETER PVC SCH 40 OR ABS CLASS SDR 35 WITH
A CRUSHING STRENGTH OF AT LEAST 1,000 POUNDS, WITH A MINIMUM
OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED
WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM
END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

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

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL

PLATE D-6

MINIMUM ONE FOOT THICK LAYER OF
LOW PERMEABLILITY SOIL IF NOT
COVERED WITH AN IMPERMEABLE SURFACE

MINIMUM ONE FOOT WIDE LAYER OF
FREE DRAINING MATERIAL
(LESS THAN 5% PASSING THE #200 SIEVE)
OR
PROPERLY INSTALLED PREFABRICATED DRAINAGE COMPOSITE
(MiraDRAIN 6000 OR APPROVED EQUIVALENT).

FILTER MATERIAL - MINIMUM OF TWO
CUBIC FEET PER FOOT OF PIPE. SEE
BELOW FOR FILTER MATERIAL SPECIFICATION.

ALTERNATIVE: IN LIEU OF FILTER MATERIAL
TWO CUBIC FEET OF GRAVEL
PER FOOT OF PIPE MAY BE ENCASED
IN FILTER FABRIC. SEE BELOW FOR
GRAVEL SPECIFICATION.

FILTER FABRIC SHALL BE MIRAFI 140
OR EQUIVALENT. FILTER FABRIC SHALL
BE LAPPED A MINIMUM OF 6 INCHES
ON ALL JOINTS.

MINIMUM 4-INCH DIAMETER PVC SCH 40 OR ABS CLASS SDR 35 WITH
A CRUSHING STRENGTH OF AT LEAST 1,000 POUNDS, WITH A MINIMUM
OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED
WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM
END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

"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	SoCalGeo · SOUTHERN CALIFORNIA GEOTECHNICAL
PLATE D-7	

Section View

10 FEET MINIMUM

15 FEET MINIMUM

5 FEET MINIMUM OFFSET

3 FEET MINIMUM

15 FEET MINIMUM

Typical Row of Oversize Rock Fragments

Plan View

Typical Row of Oversize Rock Fragments

15 FEET MINIMUM

Fill Slope

PLACEMENT OF OVERSIZED MATERIAL
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE		
DRAWN: PM		
CHKD: GKM	SoCalGeo	SOUTHERN CALIFORNIA GEOTECHNICAL
PLATE D-8		

APPENDIX E

Rider 2

Latitude, Longitude: 33.83137, -117.21511
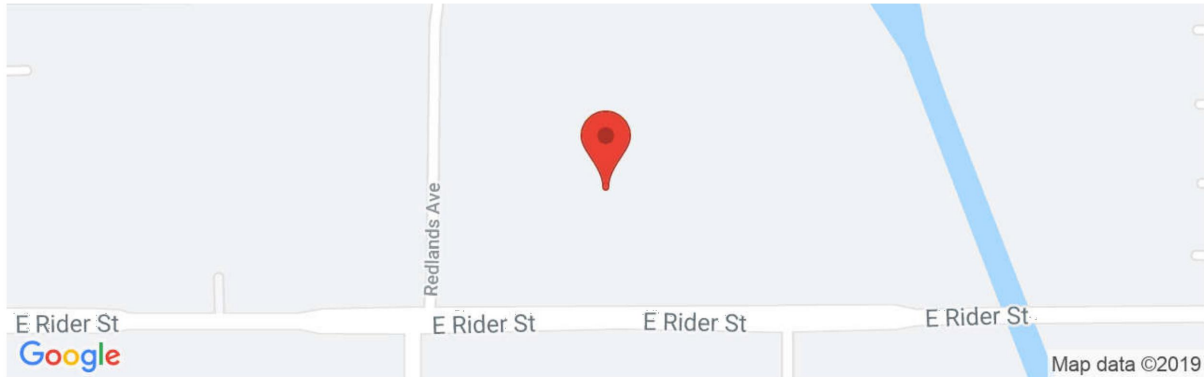
Date	8/23/2019, 2:21:38 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.5	MCE$_R$ ground motion. (for 0.2 second period)
S_1	0.576	MCE$_R$ ground motion. (for 1.0s period)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.5	MCE$_G$ peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA$_M$	0.55	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.536	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.647	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.576	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.632	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.932	Mapped value of the risk coefficient at short periods

SOURCE: SEAOC/OSHPD Seismic Design Maps Tool
<https://seismicmaps.org/>

SEISMIC DESIGN PARAMETERS
RIDER 2 - PROPOSED COMMERCIAL/INDUSTRIAL BUILDING
PERRIS, CALIFORNIA

DRAWN: PM	
CHKD: GKM	SoCalGeo
SCG PROJECT 17G199-1R	SOUTHERN CALIFORNIA GEOTECHNICAL
PLATE E-1	

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Rider 2
Project Location	Perris, CA
Project Number	17G199
Engineer	DWN

Boring No.	B-1

MCE$_G$ Design Acceleration	0.550	(g)
Design Magnitude	7.09	
Historic High Depth to Groundwater	26	(ft)
Depth to Groundwater at Time of Drilling	33	(ft)
Borehole Diameter	6	(in)

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	KS	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.09)	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	28	14		120		1.3	1.05	1.1	1.20	0.75	0.0	0.0	1680	1680	1680	0.96	1.01	1.01	0.06	0.06	N/A	N/A	Above Water Table
29.5	28	32	30	32	120		1.3	1.05	1.3	0.87	0.95	47.1	47.1	3600	3350	3600	0.88	1.17	0.86	2.00	2.00	0.34	6.48	Nonliquefiable
34.5	32	37	34.5	36	120		1.3	1.05	1.3	0.87	1	55.7	55.7	4140	3610	4046	0.86	1.17	0.84	2.00	1.97	0.35	6.14	Nonliquefiable
39.5	37	42	39.5	19	120	57	1.3	1.05	1.25	0.77	1	25.1	30.7	4740	3898	4334	0.83	1.15	0.87	0.53	0.53	0.36	1.62	Nonliquefiable
44.5	42	47	44.5	25	120	38	1.3	1.05	1.3	0.80	1	35.3	40.8	5340	4186	4622	0.81	1.17	0.8	2.00	1.87	0.37	5.58	Nonliquefiable
49.5	47	50	48.5	42	120		1.3	1.05	1.3	0.87	1	65.0	65.0	5820	4416	4853	0.78	1.17	0.78	2.00	1.83	0.37	5.44	Nonliquefiable

Notes:

(1) Energy Correction for N$_{90}$ of automatic hammer to standard N$_{60}$

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Rider 2
Project Location	Perris, CA
Project Number	17G199
Engineer	DWN

Boring No. **B-1**

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines cont	$(N_1)_{60-cs}$	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	28	14	0.0	0.0	0.0	N/A	0.50	0.95	0.00	28.00		0.000		0.00	Above Water Table
29.5	28	32	30	47.1	0.0	47.1	6.48	0.00	-1.35	0.00	4.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	55.7	0.0	55.7	6.14	0.00	-2.06	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	25.1	5.6	30.7	1.62	0.04	-0.14	0.01	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	35.3	5.6	40.8	5.58	0.01	-0.87	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	65.0	0.0	65.0	5.44	0.00	-2.86	0.00	3.00		0.000		0.00	Nonliquefiable

	Total Deformation (in)	0.00

Notes:

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

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

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

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

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

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

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

(8) 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

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.09)	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	28	14		120		1.3	1.05	1.1	1.20	0.75	0.0	0.0	1680	1680	1680	0.96	1.01	1.01	0.06	0.06	N/A	N/A	Above Water Table
29.5	28	32	30	39	120		1.3	1.05	1.3	0.90	0.95	59.4	59.4	3600	3350	3600	0.88	1.17	0.86	2.00	2.00	0.34	6.48	Nonliquefiable
34.5	32	37	34.5	10	120	44	1.3	1.05	1.11	0.74	1	11.2	16.8	4140	3610	4046	0.86	1.05	0.94	0.17	0.17	0.35	0.53	**Liquefiable**
39.5	37	42	39.5	49	120		1.3	1.05	1.3	0.94	1	81.5	81.5	4740	3898	4334	0.83	1.17	0.82	2.00	1.92	0.36	5.82	Nonliquefiable
44.5	42	47	44.5	52	120		1.3	1.05	1.3	0.95	1	87.7	87.7	5340	4186	4622	0.81	1.17	0.8	2.00	1.87	0.37	5.58	Nonliquefiable
49.5	47	50	48.5	37	120		1.3	1.05	1.3	0.84	1	54.9	54.9	5820	4416	4853	0.78	1.17	0.78	2.00	1.83	0.37	5.44	Nonliquefiable

Notes:

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

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name: Rider 2
Project Location: Perris, CA
Project Number: 17G199
Engineer: DWN

Boring No.: B-8

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	$(N_1)_{60-cs}$	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	28	14	0.0	0.0	0.0	N/A	0.50	0.95	0.00	28.00		0.000		0.00	Above Water Table
29.5	28	32	30	59.4	0.0	59.4	6.48	0.00	-2.37	0.00	4.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	11.2	5.6	16.8	0.53	0.23	0.68	0.23	5.00		0.026		1.59	**Liquefiable**
39.5	37	42	39.5	81.5	0.0	81.5	5.82	0.00	-4.33	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	87.7	0.0	87.7	5.58	0.00	-4.91	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	54.9	0.0	54.9	5.44	0.00	-2.00	0.00	3.00		0.000		0.00	Nonliquefiable

Total Deformation (in): 1.59

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)