APPENDIX 9.6

GEOLOGY AND SOILS REPORTS

APPENDIX 9.6.1

GEOTECHNICAL INVESTIGATION

GEOTECHNICAL INVESTIGATION
TWO PROPOSED
COMMERCIAL/INDUSTRIAL BUILDINGS

Sherman Road, South of Ethanac Road
Menifee, California
for
Core5 Industrial Partners

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

December 4, 2020

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Core5 Industrial Partners
300 Spectrum Center Drive, Suite 880
Irvine, California 92618

Attention: Mr. Jon Kelly
Vice President Development

Project No.: **20G227-1**

Subject: **Geotechnical Investigation**
Two Proposed Commercial/Industrial Buildings
Sherman Road, South of Ethanac Road
Menifee, California

Mr. Kelly:

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.

Pablo Montes Jr.
Staff Engineer

Robert G. Trazo, GE 2655
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
ROBERT TRAZO
No. 2655
GEOTECHNICAL
STATE OF CALIFORNIA

Distribution: (1) Addressee

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1.0 EXECUTIVE SUMMARY

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

Site Preparation

- Initial site preparation should include stripping of the existing moderate to dense native grass and weed growth with some trees. This material should be disposed of offsite. Demolition of the existing residences should include all structures, foundations, slabs, septic systems and utilities that will not remain in place for use with the new development. Debris resulting from the demolition should be disposed of offsite.
- The site is generally underlain moderate strength older alluvium. However, some of the borings encountered artificial fill soils and low strength younger alluvium beginning at the ground surface, and extending to depths of 2 to 3± feet. Some of the near-surface soils also possess a medium expansive potential. Based on their in-situ densities, expansive potential, and unfavorable consolidation characteristics, these materials are not considered suitable, in their present condition, to support the new structures.
- Remedial grading is recommended to be performed within the new building pad areas in order to remove the artificial fill soils in their entirety. The existing soils within the building areas should be overexcavated to a depth of at least 3 feet below existing grade and to a depth of 2 feet below proposed building pad subgrade elevation. Within the foundation influence zones, the overexcavation should extend to a depth of at least 2 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.
- 3,000 lbs/ft^2 maximum allowable soil bearing pressure.
- Reinforcement consisting of at least four (4) No. 5 rebars (2 top and 2 bottom) in strip footings, due to the presence of expansive soils within the new building pad areas. 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: No. 3 bars at 18-inches on-center, in both directions. 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	3½	4	5	5½
Aggregate Base	7	9	11	12	15
Compacted Subgrade	12	12	12	12	12

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	8½
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. 20P397, dated October 29, 2020. The scope of services included a visual site reconnaissance, subsurface exploration, laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slabs, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located on Sherman Road, 335± feet south of Ethanac Road in Menifee, California. The site is bounded to the north by existing single-family residences (SFRs), commercial/industrial buildings and vacant land, to the west by Tumble Road, to the south by an existing concrete-lined channel, and to the east by Dawson Road. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

We were provided with two (2) site plans for the project. Scheme 8 indicates that the site consists of multiple contiguous parcels, which total 72.69± acres in size. Scheme 9 indicates an approximate total area of 69.49± acres. The difference in the two site plans is that the Scheme 9 plan excludes an existing residential lot, located in the southeast region of the overall site. Both of the plans indicate the overall site to be transected by Sherman Road, which trends in a north to south direction. Most of the site is vacant and undeveloped. Ground surface cover throughout the site consists of dense native grass and weed growth. The southeast and southwest regions of the site include ranch-style residential lots, each with one-story single-family residences and detached garages and sheds. The existing structures are of wood-frame and stucco construction, supported on conventional shallow foundations with concrete slab-on-grade floors. Ground surface cover surrounding the residences consists of exposed soil with limited areas of concrete pavements and some medium to large size trees around the perimeters of the properties. As previously noted, Sherman Road, currently not paved, transects the subject site in a north-south direction. The road possesses some scattered debris and trash in addition to several large trees. Several soil berms are located at the northwest corner of the site, near Tumble Road.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth, and visual observations made at the time of the subsurface investigation, the overall site topography slopes downward to the west at a gradient of ½± percent. There is approximately 9 feet of elevation differential across the overall site.

3.2 Proposed Development

SCG was provided with two (2) conceptual site plans (identified as Scheme 8 and Scheme 9) prepared by HPA Architecture. Based on these site plans, two (2) new commercial/industrial buildings, identified as Building 1 and Building 2 will be constructed.

In Scheme 8, Building 1 will be 1,154,160± ft² in size and will be located in the eastern area of the site. Building 2 will be 385,970± ft² in size and will be located in the western area of the site.

In Scheme 9, Building 1 will be 1,075,680± ft² in size and will be located in the eastern area of the site. Building 2 will be 385,970± ft² in size and will be located in the western area of the site.

In both site plans, dock-high doors will be constructed along a portion of the east and west building walls on Building 1 and along a portion of the north building wall on Building 2. The buildings will be surrounded by asphaltic concrete pavements in the parking and drive lanes, Portland cement concrete pavements in the loading dock areas, and limited areas of concrete flatwork and landscape planters throughout. It is expected that the portion of Sherman Road which transects the subject site will be developed with new asphaltic concrete pavements.

Detailed structural information has not been provided. It is assumed that the new buildings will be single-story structures of tilt-up concrete construction, typically supported on conventional shallow foundation systems with concrete slab-on-grade floors. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 4 to 6 kips per linear foot, respectively.

No significant amounts of below grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. Based on the assumed topography, cuts and fills of up to 3 to 5± feet are expected to be necessary to achieve the proposed site grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of nineteen (19) borings advanced to depths of 10 to 25± feet below the existing site grades. All of the borings were logged during drilling by a member of our staff.

All of the borings were advanced with hollow-stem augers by a conventional truck-mounted drilling rig. Representative bulk and relatively undisturbed soil samples were taken during drilling. Relatively undisturbed soil samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. 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 Plans, included as Plate 2A and Plate 2B in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Artificial Fill

Artificial fill soils were encountered at the ground surface at Boring No. B-4, extending to a depth of 3± feet below the existing site grades. The artificial fill soils consist of very dense fine sandy silts. The fill soils generally possess a disturbed appearance, resulting in their classification as artificial fill.

Younger Alluvium

Younger native alluvial soils were encountered at the ground surface at Boring Nos. B-7, B-11, B-17, and B-18, extending to depths of 2 to 3± feet below the existing site grades. The younger alluvium generally consists of medium dense to very dense silty fine to medium sands and very stiff sandy clays.

Older Alluvium

Older alluvial soils were encountered beneath the artificial fill soils at Boring No. B-4, beneath the younger alluvium at Boring Nos. B-7, B-11, B-17, and B-18, and at the ground surface at all of the remaining boring locations, extending to the maximum depth explored of 25± feet below existing site grades. The older alluvial soils generally consist of dense to very dense silty sands, clayey sands, sandy silts, and well graded sands with varying silt and clay content, and very stiff to hard sandy clays, silty clays and clayey silts.

Groundwater

Groundwater was not encountered at any of the boring locations. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 25± feet below existing site grades, at the time of the subsurface investigation.

Recent water level data was obtained from the California Department of Water Resources Water Data Library website, http://wdl.water.ca.gov/. The nearest monitoring well on record is located 4,290± feet northwest of the site. Water level readings within this monitoring well indicate a groundwater level of 62± feet below the ground surface in March 2020.

5.0 LABORATORY TESTING

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

Classification

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

Density and Moisture Content

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

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-6 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

Two (2) representative bulk samples have been tested for their maximum dry densities and optimum moisture contents. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557, and are presented on Plates C-7 and C-8 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 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-8 @ 0 to 5 feet	55	Medium
B-12 @ 0 to 5 feet	40	Low

Soluble Sulfates

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

Sample Identification	Soluble Sulfates (%)	Sulfate Classification
B-5 @ 0 to 5 feet	0.021	Not Applicable (S0)
B-10 @ 0 to 5 feet	0.008	Not Applicable (S0)

Corrosivity Testing

Representative bulk samples of the near-surface soils were submitted to a subcontracted corrosion engineering laboratory to determine if the near-surface soils possess corrosive characteristics with respect to common construction materials. The corrosivity testing included a determination of the electrical resistivity, pH, and chloride and nitrate concentrations of the soils, as well as other tests. The results of the corrosivity testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	Saturated Resistivity (ohm-cm)	pH	Chlorides (mg/kg)	Nitrates (mg/kg)
B-5 @ 0 to 5 feet	1,720	7.1	13	111
B-10 @ 0 to 5 feet	1,880	7.5	17	50

6.0 CONCLUSIONS AND RECOM5MENDATIONS

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 standards in place at the time of this report, the proposed development is expected to be designed in accordance with the requirements of the 2019 edition of the California Building Code (CBC), which was adopted on January 1, 2020. Section 1613.1 of the 2019 CBC states that

"…structures and their supports and attachments shall be designed and constructed to resist the effects of earthquake motions in accordance with Chapters 11, … of ASCE 7."

Section 11.4.8 of ASCE 7-16 states that "it shall be permitted to perform a site response analysis or in Accordance with Section 21.1 and/or a ground motion hazard analysis (GMHA) in accordance with Section 21.2." Therefore, a site-specific GMHAs was performed in accordance with Section 21.2 of ASCE 7-16 to determine the seismic design parameters for the new building at this site.

The site classification was determined using shear wave velocity measurements for the soils present within the upper 100± feet at the subject site. The parameter V100 is defined as the shear- wave velocity of the soil or bedrock material present within the upper 100 feet at the site. The shear-wave velocity was determined by a seismic shear wave survey performed by a licensed geophysicist. The results of the shear-wave survey are included in a report prepared by Terra Geosciences, included in Appendix E of this report. Based on the shear-wave survey performed by Terra Geosciences, the V100 for the site is 1,395.7 feet per second. Table 20.3-1 of ASCE 7-16 indicates that an average shear velocity ranging between 1,200 and 2,500 feet per second corresponds to Site Class C.

Details regarding the performance of the GMHA are presented in the report prepared by Terra Geosciences, in Appendix E of this report. Seismic design parameters computed during this study are tabulated below.

SITE-SPECIFIC SEISMIC DESIGN PARAMETERS BASED ON ASCE 7-10 SECTION 21.2

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.416
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.526
Site Class	---	C
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.412
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.702
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	0.940
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.470

<u>Liquefaction</u>

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 Riverside County GIS website indicates that the subject site is located within a zone of low liquefaction susceptibility. In addition, the soil conditions encountered at the boring locations are not considered to be conducive to liquefaction. These conditions consist of dense, well-graded, granular soils and very stiff to hard cohesive soils extending to depths of 25± feet. In addition, the static groundwater table does not exist within 50± feet of the ground surface. Based on these considerations, liquefaction is not considered to be a design concern for this project.

6.2 Geotechnical Design Considerations

General

The near-surface soils at this site generally consist of a surficial layer of artificial fill soils, extending to a depth of 3± feet, and some zones of low to moderate strength young alluvial soils. These materials are underlain by moderate to high-strength older alluvium. Some of these soils exhibit a medium expansion potential.

The near-surface soils, in their present condition, are not considered suitable to support the foundations and floor slabs of the new structures. Therefore, remedial grading will be necessary within the proposed building areas to remove and replace the upper portion of the existing soils as compacted structural fill.

Settlement

The recommended remedial grading will remove the artificial fill soils and the low-strength 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 possess generally favorable consolidation and collapse characteristics and will not be subject to significant load increases from the foundations of the new structures. Provided that the recommended remedial grading is completed, the post-construction static settlements of the proposed structures are expected to be within tolerable limits.

Expansion

Laboratory testing performed on representative samples of the near surface soils indicates that these materials possess low to medium expansion potentials (EI = 40 and 55). Based on the presence of expansive soils, 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.

Soluble Sulfates

The results of the soluble sulfate testing indicate that the selected samples of the on-site soils contain concentrations of soluble sulfates that correspond to Class S0 with respect to the American Concrete Institute (ACI) Publication 318-14 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building area.

Corrosion Potential

The results of laboratory testing indicate that representative samples of the on-site soils possess saturated resistivity values of 1,720 and 1,880 ohm-cm, and pH values of 7.1 and 7.5. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Sulfides, and redox potential are factors that are also used in the evaluation procedure. We have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH, and moisture content. **Based on these factors, and utilizing the DIPRA procedure, the on-site soils are considered to be highly corrosive to ductile iron pipe. Based on these test results, and since SCG does not practice in the area of corrosion engineering, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.**

Chlorides

Only low levels (13 and 17 mg/kg) of chlorides were detected in the samples submitted for corrosivity testing. In general, soils possessing chloride concentrations in excess of 500 parts per million (ppm) are considered to be corrosive with respect to steel reinforcement within reinforced concrete. Based on the lack of any significant chlorides in the tested sample, the site is considered to have a C1 chloride exposure in accordance with the American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary. Therefore, a specialized concrete mix design for reinforced concrete for protection against chloride exposure is not considered warranted.

Nitrates

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested samples possess nitrate concentrations ranging from 50 to 111 mg/kg. Based on this test result, the on-site soils are considered to be corrosive to copper pipe. **Since SCG does not practice in the area of corrosion engineering, we recommend that the client contact a corrosion engineer to provide recommendations for the protection of copper tubing/pipe in contact with the on-site soils.**

Shrinkage/Subsidence

Removal and recompaction of the near-surface artificial fill and younger alluvial soils, generally located within the upper 3 feet, is estimated to result in an average shrinkage of 5 to 10 percent. Removal and recompaction of the underlying older alluvium is estimated to result in an average shrinkage of 0 to 5 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 and Demolition

Initial site preparation should include stripping of any surficial vegetation and organic soils. Based on conditions encountered at the time of the subsurface exploration, stripping of the moderate to dense native grass, weed, and trees is expected to be necessary. Removal of trees should include the root-masses, with the resulting excavations being backfilled with compacted structural fill. 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.

Demolition of the existing structures and pavements is likely to be necessary in order to facilitate construction at the site. Demolition should include all foundations, floor slabs, utilities, septic systems, and any other subsurface improvements that will not remain in place with the new development. Debris resultant from demolition should be disposed of offsite. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2-inch particle size and incorporated into new structural fills, or it may be crushed and made into CMB if desired.

Treatment of Existing Soils: Building Pads

Remedial grading should be performed within the proposed building pad areas in order to remove the undocumented fill soils and the near-surface alluvium. The recommended remedial grading will also result in moisture conditioning of the near-surface soils, thereby reducing their potential for future heave.

It is recommended that the existing soils be overexcavated to a depth of at least 3 feet below existing grade and to a depth of at least 2 feet below proposed building pad subgrade elevation, whichever is greater. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 2 feet below proposed foundation bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building and foundation perimeters, and to an extent equal to the depth of fill below the new foundations. If the proposed 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 structures. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or loose, porous, overly moist, or low density native soils are encountered at the base of the overexcavation.

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 structural fill soils present within the proposed building areas 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 2 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pad. Any undocumented fill soils within any of these foundation areas should be removed in their entirety. The overexcavation areas should extend at least 5 feet beyond the foundation perimeters, and to an extent equal to the depth of fill below the new foundations. Please note that erection pads are considered to be part of the foundation system. These overexcavation recommendations apply to erection pads also. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils, as discussed for the building areas. The previously excavated soils may then be replaced as compacted structural fill.

If the full lateral extent of overexcavation is not achievable for the proposed walls, foundation elements must be redesigned using a lower bearing pressure. The geotechnical engineer of record should be contacted for recommendations pertaining to this type of condition.

<u>Treatment of Existing Soils: Parking Areas</u>

Based on economic considerations, overexcavation of the undocumented fill soils and 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 fill and younger 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 fill soils and younger 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.

<u>Fill Placement</u>

- 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 Menifee.
- 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 Menifee. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

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

6.4 Construction Considerations

Excavation Considerations

The near surface soils generally consist of silty sands and sandy silts with some zones of clay content. 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 stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

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

Expansive Soils

Based on the results of laboratory testing, the near surface soils have been determined to be very low to medium expansive. 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 Modified Proctor optimum during site grading. All imported fill soils should have 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.

Groundwater

The static groundwater table is considered to exist at a depth in excess of 25± 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 pads will be underlain by structural fill soils extending to depths of at least 2 feet below foundation bearing grade. Based on this subsurface profile, 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: 3,000 lbs/ft^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Four (4) No. 5 rebars (2 top and 2 bottom), due to the presence of potentially expansive soils.
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 24 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slabs.
- 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. 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. Differential movements are expected to occur over a 60-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch.

Lateral Load Resistance

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

- Passive Earth Pressure: 300 lbs/ft^3
- Friction Coefficient: 0.28

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 3,000 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floors of the proposed structures may be constructed as conventional slabs-on-grade supported on newly placed structural fill, extending to a depth of at least 2 feet below finished pad grade. Based on geotechnical considerations, the floor slabs may be designed as follows:

- Minimum slab thickness: 6 inches.

- Modulus of Subgrade Reaction: 100 psi/in.

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

- Slab underlayment: If moisture sensitive floor coverings will be used 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 15 mil 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 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 and in the loading dock areas. 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 on-site soils generally consist of a variety of materials ranging from silty sands to sandy silts and silty clays. **It is recommended that any soils which possess an expansion index greater than 50 be excluded from retaining wall backfill.** Therefore, the soils which will be used as retaining wall backfill are expected to consist primarily of silty sands and sandy silts. Based on their composition, the on-site soils have been assigned a friction angle of 30 degrees.

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

RETAINING WALL DESIGN PARAMETERS

Design Parameter		**Soil Type**
		On-site Silty Sands and Sandy Silts
Internal Friction Angle (ϕ)		30°
Unit Weight		128 bs/ft³
Equivalent Fluid Pressure:	Active Condition (level backfill)	43 lbs/ft³
	Active Condition (2h:1v backfill)	69 lbs/ft³
	At-Rest Condition (level backfill)	64 lbs/ft³

The walls should be designed using a soil-footing coefficient of friction of 0.28 and an equivalent passive pressure of 300 lbs/ft³. 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 2019 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 2 feet below 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. 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. **As discussed above, it is recommended that any material which possesses an expansion index greater than 50 be excluded from the retaining wall backfill.**

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

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

Subsurface Drainage

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

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

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

6.8 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The on-site soils consist of a variety of materials, ranging from silty sands and sandy silts to silty clays and clayey sands. For the purposes of pavement design, the lower strength sandy silts and silty clays will govern the design. Based on their classification, these materials are expected to possess fair to good pavement support characteristics with estimated R-values in the range of 25 to 45. R-value testing was outside the scope of services. 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	3½	4	5	5½
Aggregate Base	7	9	11	12	15
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	8½
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

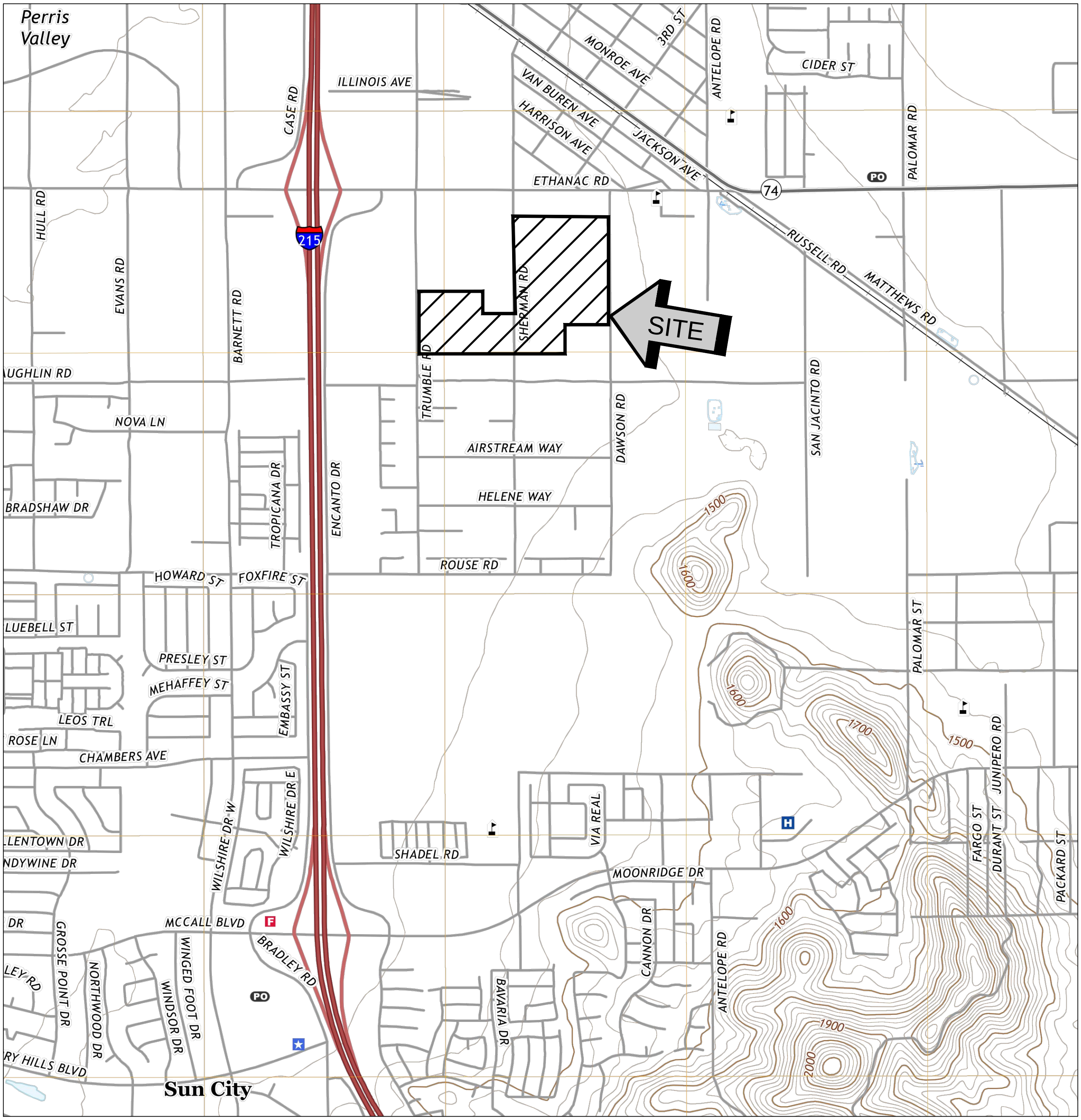
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other 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.

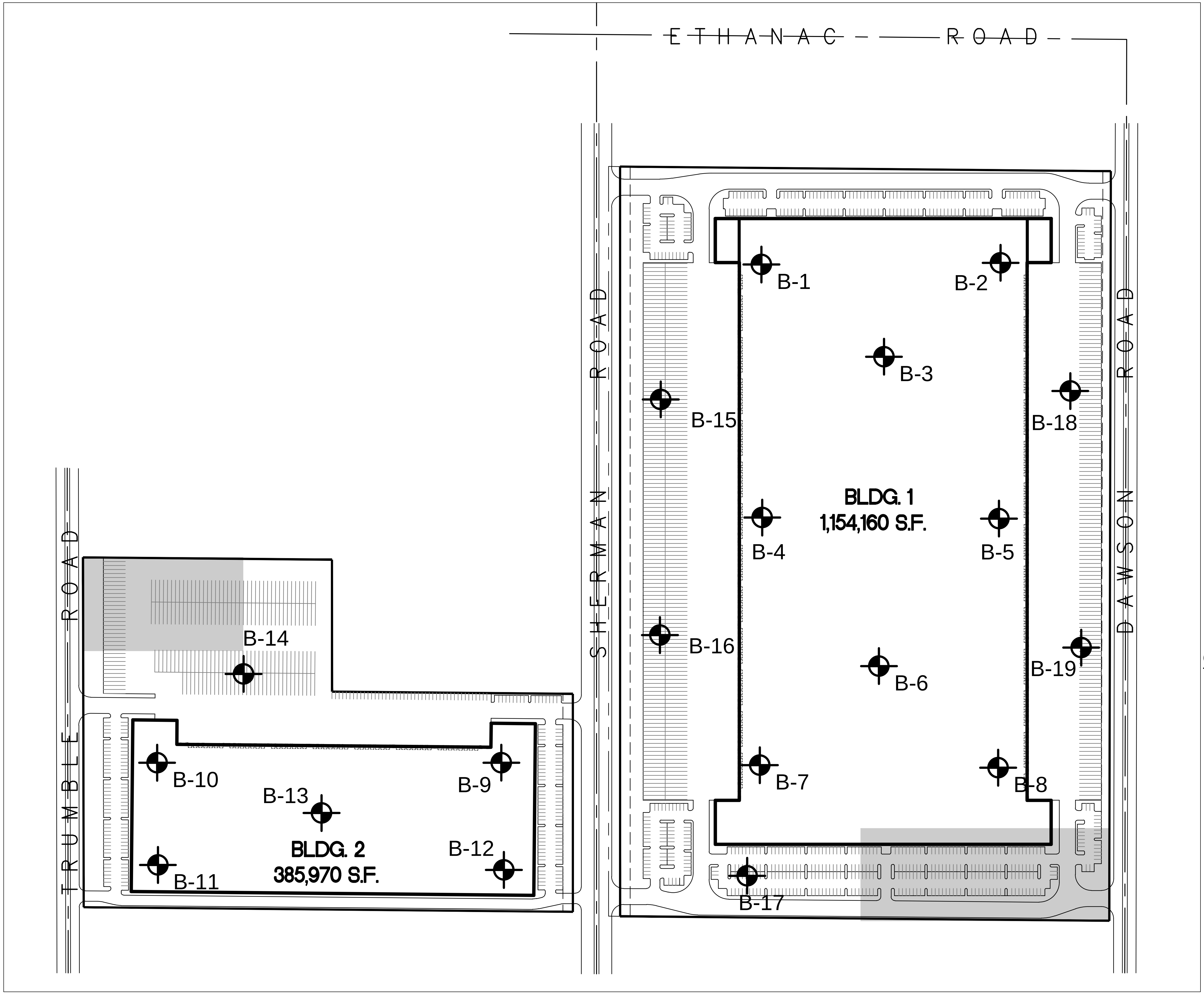
The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAP OF THE ROMOLAND QUADRANGLE, RIVERSIDE COUNTY, CALIFORNIA, 2018

SITE LOCATION MAP	
TWO PROPOSED COMMERCIAL/INDUSTRIAL BUILDINGS	
MENIFEE, CALIFORNIA	
SCALE: 1" = 2000'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAH CHKD: RGT	
SCG PROJECT 20G227-1	
PLATE 1	



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

EXISTING SINGLE FAMILY RESIDENCES TO BE DEMOLISHED

NOTE: SITE PLAN PREPARED BY HPA ACHITECTURE.

BORING LOCATION PLAN - SCHEME 8

TWO PROPOSED COMMERCIAL/INDUSTRIAL BUILDINGS

MENIFEE, CALIFORNIA

SCALE: 1" = 240'

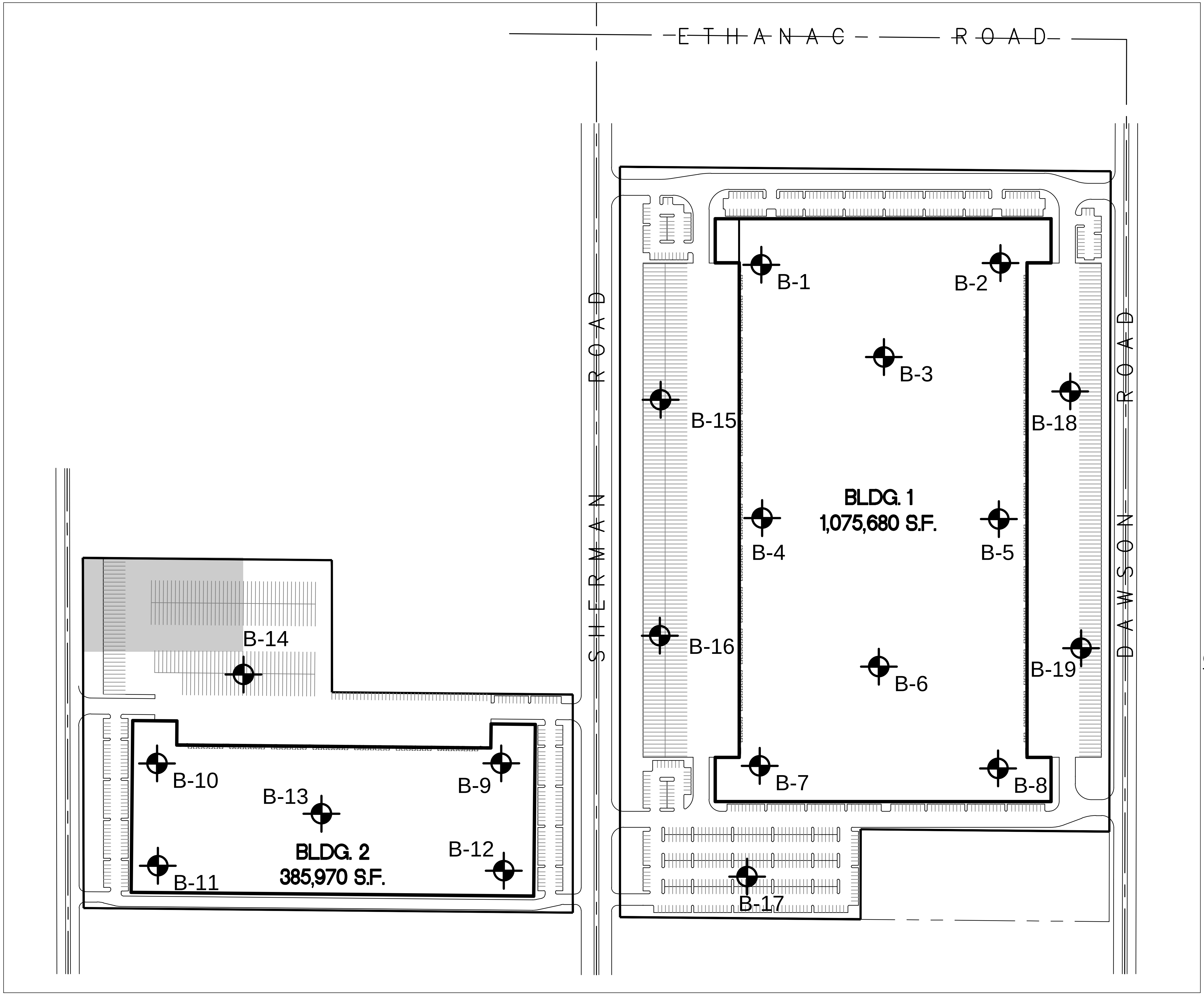
DRAWN: JAH
CHKD: RGT

SCG PROJECT
20G227-1

PLATE 2A

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL



NORTH
SoCalGeo

GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

EXISTING SINGLE FAMILY RESIDENCE TO BE DEMOLISHED

NOTE: SITE PLAN PREPARED BY HPA ARCHITECTURE.

BORING LOCATION PLAN - SCHEME 9

TWO PROPOSED COMMERCIAL/INDUSTRIAL BUILDINGS

MENIFEE, CALIFORNIA

SCALE: 1" = 240'

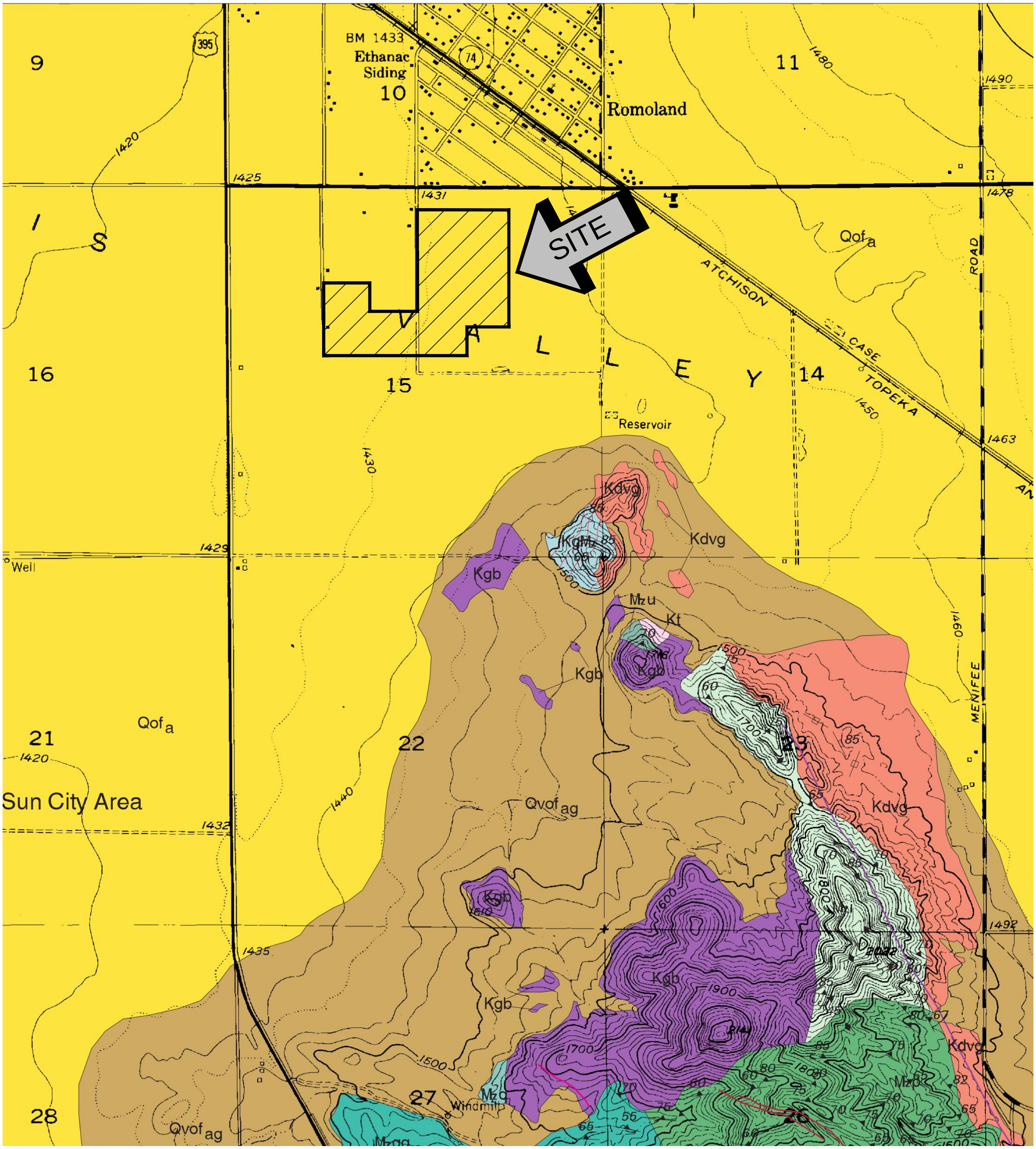
DRAWN: JAH
CHKD: RGT

SCG PROJECT
20G227-1

PLATE 2B

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



DESCRIPTION OF MAP UNITS

Qof — **Old alluvial fan deposits (late to middle Pleistocene)**—Reddish brown, gravel and sand alluvial fan deposits; indurated, commonly slightly dissected. In places includes thin alluvial fan deposits of Holocene age

SOURCE: "GEOLOGIC MAP OF THE ROMOLAND 7.5' QUADRANGLE, RIVERSIDE COUNTY, CALIFORNIA" BY D. M. MORTON.

GEOLOGIC MAP

TWO PROPOSED COMMERCIAL/INDUSTRIAL BUILDINGS

MENIFEE, CALIFORNIA

SCALE: 1" = 2000'

DRAWN: JAH
CHKD: RGT

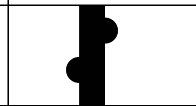
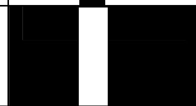
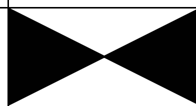
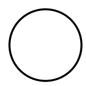
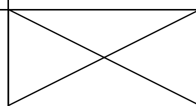
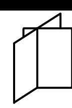
SCG PROJECT
20G227-1

PLATE 3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

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

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

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

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

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

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

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

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

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

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/11/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 19 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (SURFACE ELEVATION: --- MSL)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	67			OLDER ALLUVIUM: Brown Silty fine Sand, trace medium Sand, cemented, dense to very dense-moist	108	11					
	X	94/9"				124	9					
5	X	91/8"			Brown Silty fine Sand to fine Sandy Silt, very dense-moist	120	11					
	X	83/10"				121	12					
10	X	68			Brown Silty fine to Sand, little medium Sand, trace Clay, dense-damp to moist	125	8					
15	X	46				119	10					
20	X	80			Brown Silty fine to medium Sand, very dense-damp to moist	112	8					
25	X	76				119	9					
					Boring Terminated at 25'							

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/11/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	50/6"			OLDER ALLUVIUM: Light Brown Silty fine Sand to fine Sandy Silt, trace medium to coarse Sand, very dense-damp to moist		9					
	X	37			Dark Gray Brown fine Sandy Silt, trace Clay, dense-moist		11					
5	X	91			Dark Brown Silty fine Sand, trace medium to coarse Sand, very dense-moist		10					
10	X	63			Brown fine Sandy Silt, little Clay, very dense-moist		11					
	X	30			Gray Brown fine Sand, little Silt, trace medium Sand, micaceous, dense-moist		10					
15					Light Gray Brown fine to medium Sand, trace Silt, trace coarse Sand, dense-damp		4					
20	X	60			Brown fine Sandy Silt, trace Clay, very dense-moist		13					
25	X	60			Light Gray Brown fine to coarse Sand, trace Silt, very dense-damp		3					
					Boring Terminated at 25'							

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/11/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					OLDER ALLUVIUM: Light Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, dense-dry to damp							
	X	52				128	4					
					Brown Silty fine Sand, little Clay, very dense-damp							
	X	94/9"				119	6					
5					Gray Brown to Dark Brown fine Sandy Silt, little Clay, very dense-moist							
	X	80/9"				113	12					
	X	73/11"				119	12					
					Red Brown Silty fine Sand, very dense-moist							
	X	76				125	9					
10												
					Brown fine Sandy Silt, little Calcareous veining, very dense-very moist							
	X	56					30					
15												
					Boring Terminated at 15'							

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/11/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
	X	84			FILL: Brown fine Sandy Silt, trace Clay, trace fine root fibers, very dense-damp to moist		9					
5	X	54			OLDER ALLUVIUM: Brown Silty fine Sand to fine Sandy Silt, trace fine root fibers, very dense-very moist		17					
	X	91/11"			Brown fine Sandy Clay, little Silt, hard-very moist		16					
10	X	84/11"			Brown Clayey fine Sand, trace Silt, very dense-moist		11					
15	X	94/10"			Brown fine Sandy Silt, trace Clay, very dense-very moist		15					
20	X	54					12					
					Boring Terminated at 20'							

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 20G227-1	DRILLING DATE: 11/11/20	WATER DEPTH: Dry
PROJECT: Two Proposed CI Bldgs	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Menifee, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					<u>OLDER ALLUVIUM:</u> Brown fine Sandy Clay, little Iron oxide staining, hard-moist							
	X	90/10"	4.5			113	13					
					Brown Silty fine to medium Sand, little Clay, little Iron oxide staining, very dense-moist							
	X	94/9"				114	11					
					Dark Brown to Red Brown Silty fine Sand to fine Sandy Silt, medium dense-moist							
5	X	37				117	11					
					Dark Brown fine Sandy Clay, little medium Sand, little Silt, hard-moist							
	X	44	4.5			123	10					
	X	52	4.5			122	12					
10												
					Gray Brown fine Sandy Silt, little Clay, dense-moist							
	X	42					12					
15												
					Gray Brown fine Sand, little Silt, dense-moist							
	X	45					9					
20												

Boring Terminated at 20'

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/12/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					OLDER ALLUVIUM: Light Brown Silty fine Sand, little Clay, very dense-damp to moist							
	X	70/11"					9					
					Brown Silty fine to medium Sand, very dense-damp							
	X	50/5"					7					
5												
					Brown Silty fine Sand to fine Sandy Silt, little Clay, dense to very dense-moist							
	X	74					12					
	X	41					11					
10												
					Brown Silty fine Sand, micaceous, medium dense-moist							
	X	21					11					
15												
					Boring Terminated at 15'							

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-7

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 1/12/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 19 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	90/8"			ALLUVIUM: Light Brown Silty fine Sand, trace fine root fibers, very dense-dry to damp		3					
					OLDER ALLUVIUM: Light Brown Silty Clay, little fine to medium Sand, slightly porous, trace fine root fibers, hard-dry to damp							
5	X	31			Brown Silty fine Sand to fine Sandy Silt, little Clay, dense-moist		10					
	X	66			Brown fine Sandy Clay to Clayey fine Sand, little Silt, very dense/hard-moist		10					
10	X	90/11"			Brown Silty fine Sand, little Clay, very dense-moist		9					
15	X	34			Brown fine Sand, little Silt, dense-moist		8					
20	X	75			Brown fine Sandy Silt, trace Calcareous nodules, very dense-moist		13					
25	X	77					14					

Boring Terminated at 25'

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/11/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					OLDER ALLUVIUM: Red Brown fine Sandy Clay, trace medium Sand, very stiff-moist							
		37	4.5		Brown Clayey fine Sand, medium dense-moist	115	11					EI = 55 @ 0 to 5 feet
		73			Brown Clayey fine to medium Sand, trace coarse Sand, very dense-damp to moist	123	7					
5		50/6"				125	13					
		47	4.0		Brown Silty Clay, trace fine Sand, hard-damp to moist	120	12					
					Brown Clayey fine to coarse Sand, dense to very dense-damp							
		84/10"				121	7					
10												
					Brown Silty fine Sand to fine Sandy Silt, dense-damp to moist							
		35					9					
15												
		48					8					
20												
					Brown fine Sand, trace Silt, very dense-moist							
		50/5"					9					
25												
					Boring Terminated at 25'							

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-9

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/12/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					OLDER ALLUVIUM: Brown Clayey fine to medium Sand, medium dense to very dense-damp to moist							
	X	33					10					
5	X	75					9					
	X	93/11"			Brown fine Sandy Clay, little Silt, little medium Sand, hard-damp		10					
10	X	54			Brown Clayey Silt, little fine Sand, trace Clay, hard-very moist		22					
15	X	25			Brown fine Sandy Silt, trace to little Clay, medium dense to dense-very moist		23					
20	X	45					21					
					Boring Terminated at 20'							

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-10

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/12/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 16 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
		88/11"			<u>OLDER ALLUVIUM:</u> Brown to Red Brown Clayey fine to medium Sand, trace coarse Sand, slighty porous, trace fine root fibers, very dense-damp	124	6					
		45	4.5		Brown Clayey fine Sand to fine Sandy Clay, little Silt, medium dense to dense/hard-damp to moist	127	10					
5		70			Red Brown Clayey fine to coarse Sand, trace Silt, dense-damp to moist	122	7					
		69/10"				122	12					
10		58			Brown fine Sandy Silt, trace medium Sand, little Clay, dense-moist	124	12					
					Red Brown Clayey fine to coarse Sand, dense-moist							
15		32					11					
					Gray Brown fine to medium Sand, trace Silt, very dense-very moist							
20		84/11"					16					

Boring Terminated at 20'

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-11

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/12/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
	X	26			ALLUVIUM: Brown fine Sandy Clay, slightly porous, trace fine root fibers, very stiff-damp		5					
	X	72			OLDER ALLUVIUM: Brown Clayey fine Sand, trace medium Sand, little Silt, very dense-damp to moist		8					
5												
	X	88			Red Brown Silty Clay, little fine to medium Sand, hard-moist		10					
	X	55					10					
10												
					Brown to Gray Brown Clayey fine to coarse Sand, very dense-moist to very moist							
	X	74					9					
15												
	X	72					11					
20					Boring Terminated at 20'							

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-12

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/12/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 16 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					OLDER ALLUVIUM: Brown Silty Clay, very stiff to hard-moist to very moist							
		37	4.5			118	11					EI = 40 @ 0 to 5 feet
		75/10"	4.5			116	16					
					Brown Silty fine Sand, little Clay, very dense-moist to very moist							
5		45			Brown Silty fine Sand to fine Sandy Silt, dense-moist	120	11					
		56			Brown Silty fine Sand, little Clay, trace medium Sand, dense-moist	124	9					
10		92/8"			Brown Clayey fine to coarse Sand, very dense-moist	117	10					
					Brown Silty fine Sand, little Clay, dense-very moist							
15		44					15					
					Gray Brown fine Sandy Silt, dense-very moist							
20		38					18					

Boring Terminated at 20'

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

TEST BORING LOG

PLATE B-12

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-13

JOB NO.: 20G227-1	DRILLING DATE: 11/12/20	WATER DEPTH: Dry
PROJECT: Two Proposed CI Bldgs	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 11 feet
LOCATION: Menifee, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	63			OLDER ALLUVIUM: Brown fine to medium Sandy Clay, hard-damp		7					
5	X	78			Brown Silty fine Sand, little Clay, trace to little medium Sand, very dense-moist		9					
	X	67			Brown fine Sandy Silt, little Clay, very dense-moist		12					
10	X	33			Brown Silty fine Sand, trace Clay, dense-moist to very moist		13					
15	X	78/11"			Brown fine Sandy Silt, very dense-very moist		19					
					Boring Terminated at 15'							

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-14

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/12/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
	X	71/10"			OLDER ALLUVIUM: Brown fine to medium Sandy Clay, hard-damp		8					
	X	69			Light Red Brown Clayey Silt, little fine Sand, hard-moist		14					
5												
	X	60			Brown fine Sandy Silt, little Clay, dense to very dense-moist		12					
	X	44					12					
10					Boring Terminated at 10'							

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-15

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/11/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	74/11"			OLDER ALLUVIUM: Brown fine Sandy Silt, little Clay, trace medium Sand, very dense-damp		6					
5	X	92/11"			Brown Clayey fine Sand, little Silt, very dense-moist		9					
	X	25			Brown Silty fine Sand to fine Sandy Silt, little Clay, medium dense-damp to moist		8					
10	X	70/11"			Brown Clayey fine Sand, little Silt, little Calcareous veining/nodules, very dense-moist to very moist		15					

Boring Terminated at 10'

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-16

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/12/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					OLDER ALLUVIUM: Brown Silty fine Sand, little to some Clay, trace pores, trace fine root fibers, very dense-moist							
	X	67/11"					8					
					Brown Clayey fine Sand, little to some Silt, trace medium Sand, very dense-moist to very moist							
	X	54					11					
5												
	X	87/10"					8					
					Brown fine Sandy Silt, very dense-very moist							
	X	63					14					
10												

Boring Terminated at 10'

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

TEST BORING LOG

PLATE B-16

SOUTHERN CALIFORNIA GEOTECHNICAL

A California Corporation

BORING NO. B-17

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

DRILLING DATE: 11/12/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	23			ALLUVIUM: Brown Silty fine to medium Sand, trace to little Clay, trace medium Sand, medium dense-damp		5					
5	X	62			OLDER ALLUVIUM: Brown Silty fine to coarse Sand, little to some Clay, very dense-moist		10					
	X	86/11"			Brown Silty fine Sand, little Clay, trace medium Sand, very dense-moist		10					
10	X	80			Light Gray Brown fine Sand, trace Silt, trace medium to coarse Sand, very dense-damp to moist		8					

Boring Terminated at 10'

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-18

JOB NO.: 20G227-1	DRILLING DATE: 11/11/20	WATER DEPTH: Dry
PROJECT: Two Proposed CI Bldgs	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 9 feet
LOCATION: Menifee, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	24			ALLUVIUM: Brown Silty fine to medium Sand, trace coarse Sand, medium dense-damp		5					
5	X	86			OLDER ALLUVIUM: Brown Silty fine Sand, little Clay, very dense-moist		10					
	X	40			Brown Clayey fine Sand, little Silt, dense-moist		10					
10	X	70			Brown fine Sandy Silt, little Clay, little medium Sand, very dense-very moist		19					

Boring Terminated at 10'

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

TEST BORING LOG

PLATE B-18

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-19

JOB NO.: 20G227-1
PROJECT: Two Proposed CI Bldgs
LOCATION: Menifee, California

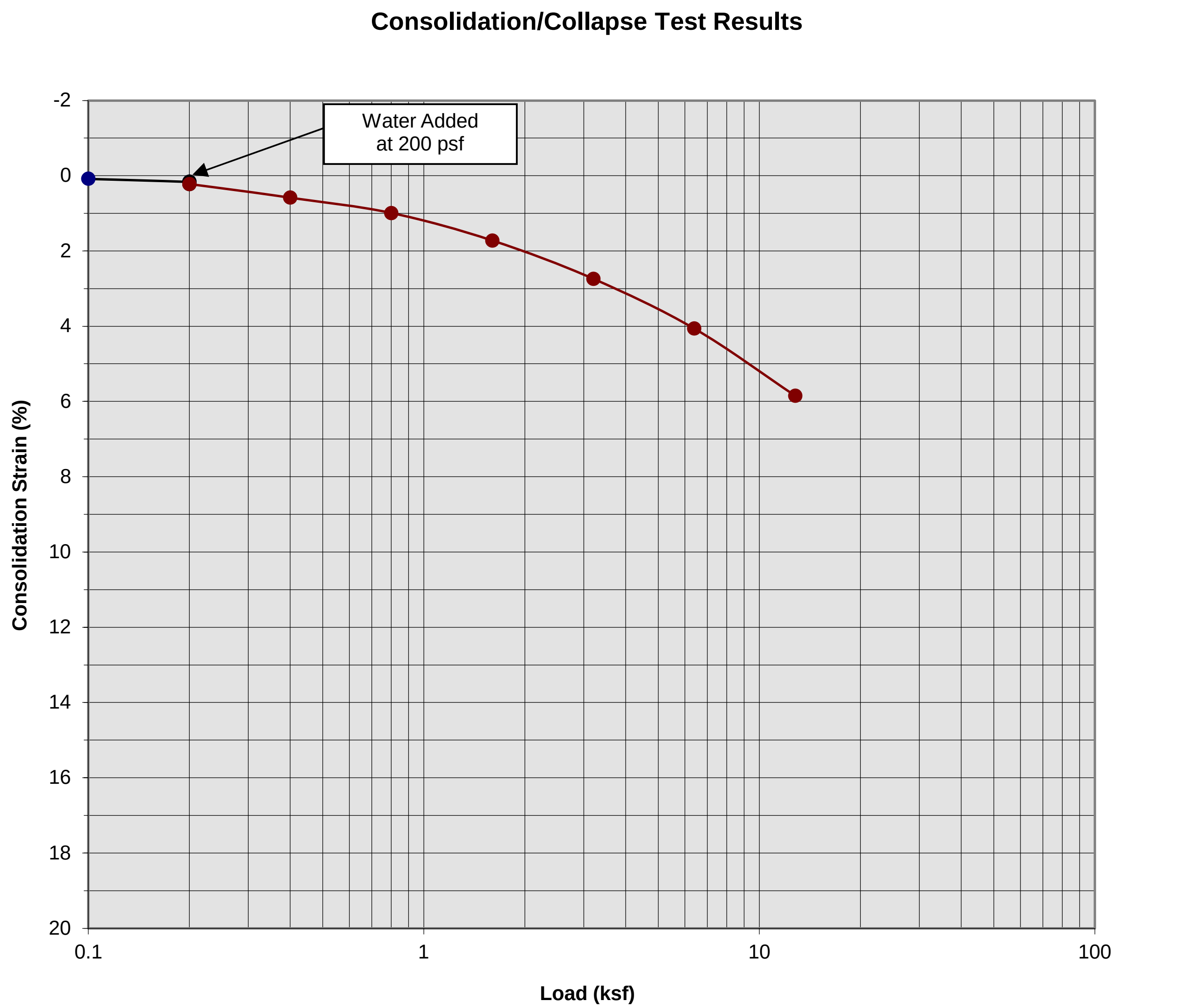
DRILLING DATE: 11/11/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					OLDER ALLUVIUM: Brown Silty fine to medium Sand, trace to little Clay, very dense-damp to moist							
	X	50/6"					11					
	X	84					8					
5												
	X	50/5"					7					
					Brown Silty fine Sand, trace medium Sand, very dense-moist							
	X	76/9"					10					
10												
					Boring Terminated at 10'							

TBL 20G227-1.GPJ SOCALGEO.GDT 12/4/20

APPENDIX C



Classification: Dark Brown to Red Brown Silty fine Sand to fine Sandy Silt

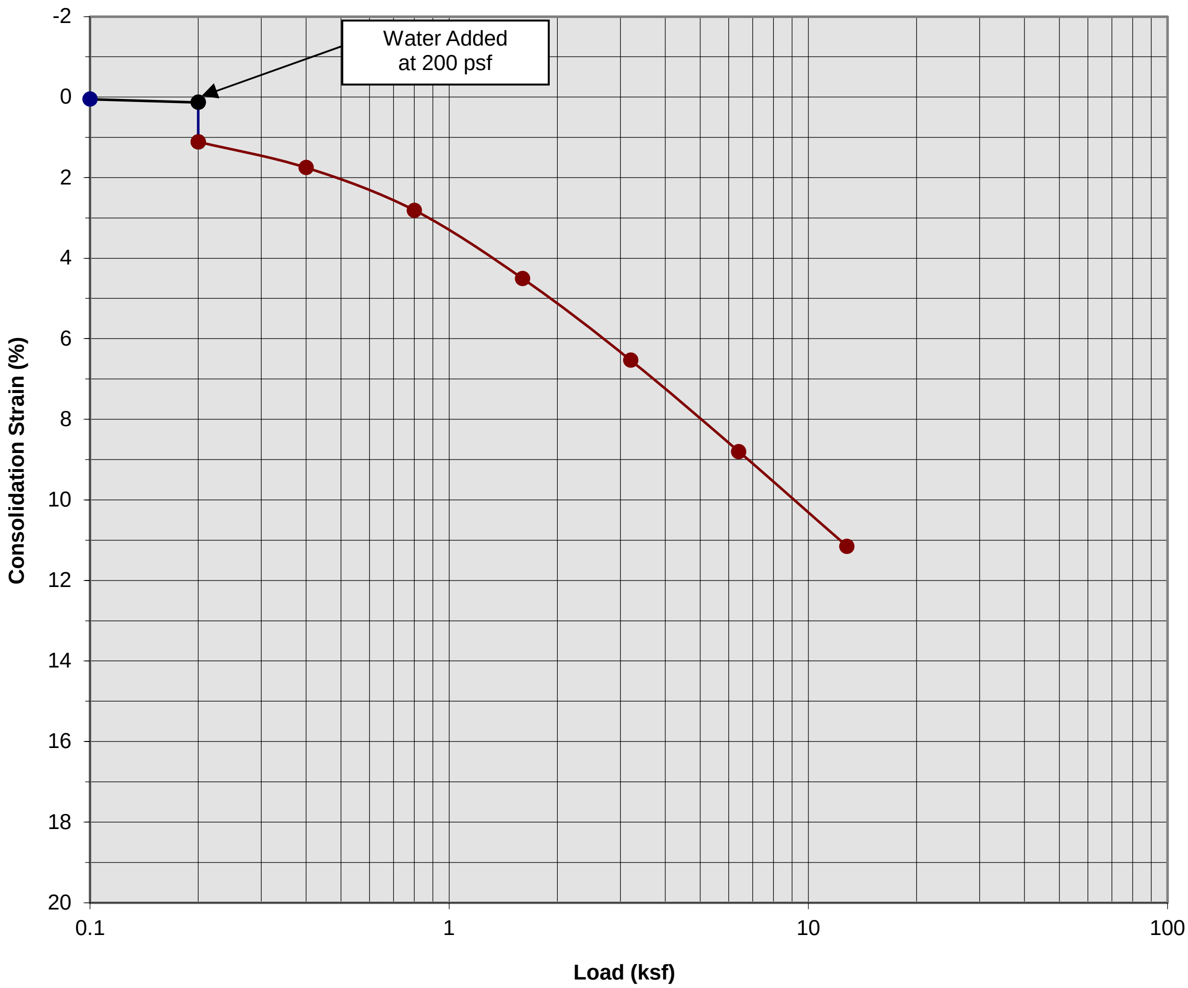
Boring Number:	B-5	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	5 to 6	Initial Dry Density (pcf)	117.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	124.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.06

Two Proposed Commercial/Industrial Buildings
Menifee, California
Project No. 20G227

PLATE C- 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

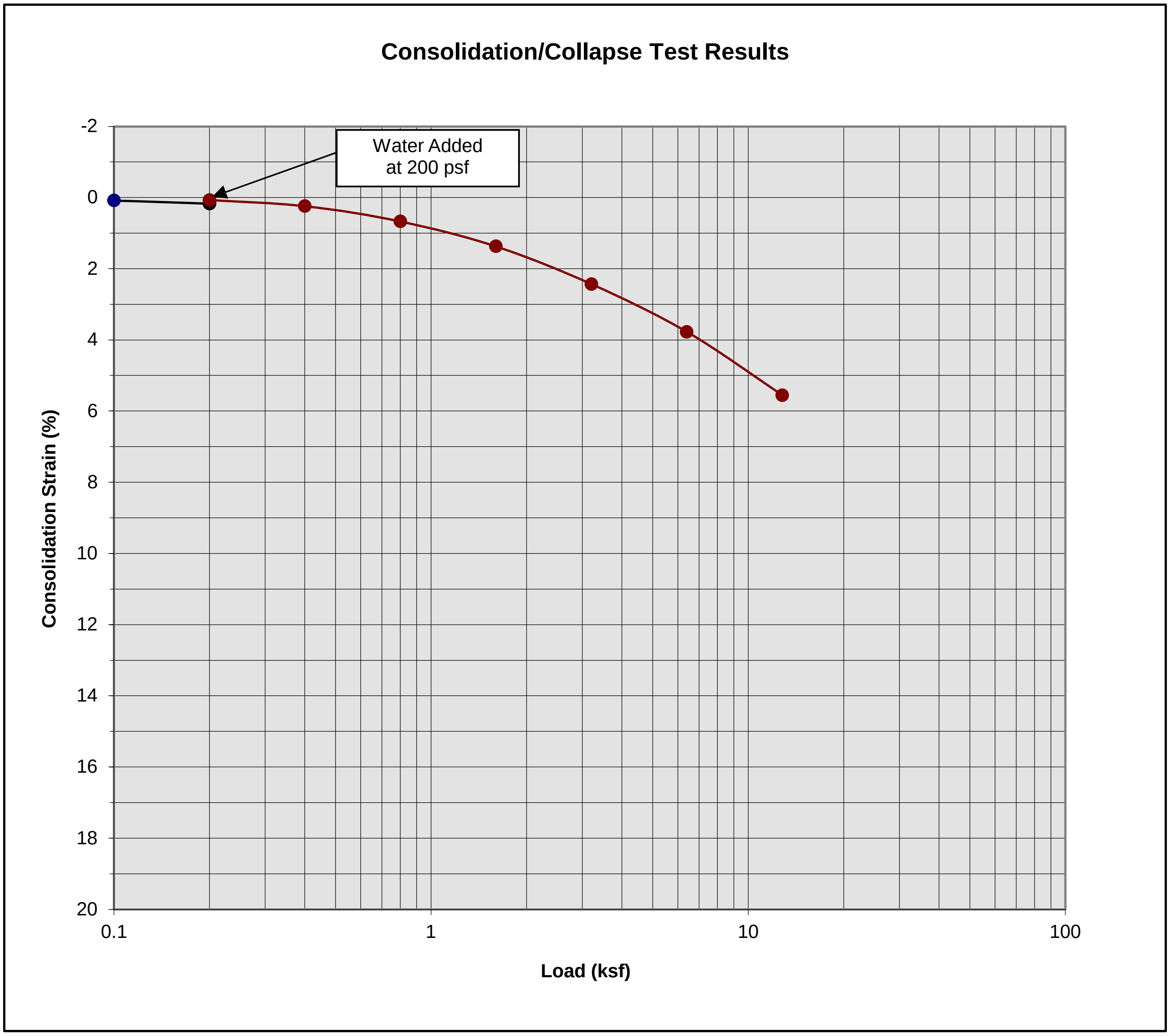


Classification: Brown to Red Brown Clayey fine to medium Sand, trace coarse Sand

Boring Number:	B-10	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	11
Depth (ft)	1 to 2	Initial Dry Density (pcf)	124.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	139.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.98

Two Proposed Commercial/Industrial Buildings
Menifee, California
Project No. 20G227
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

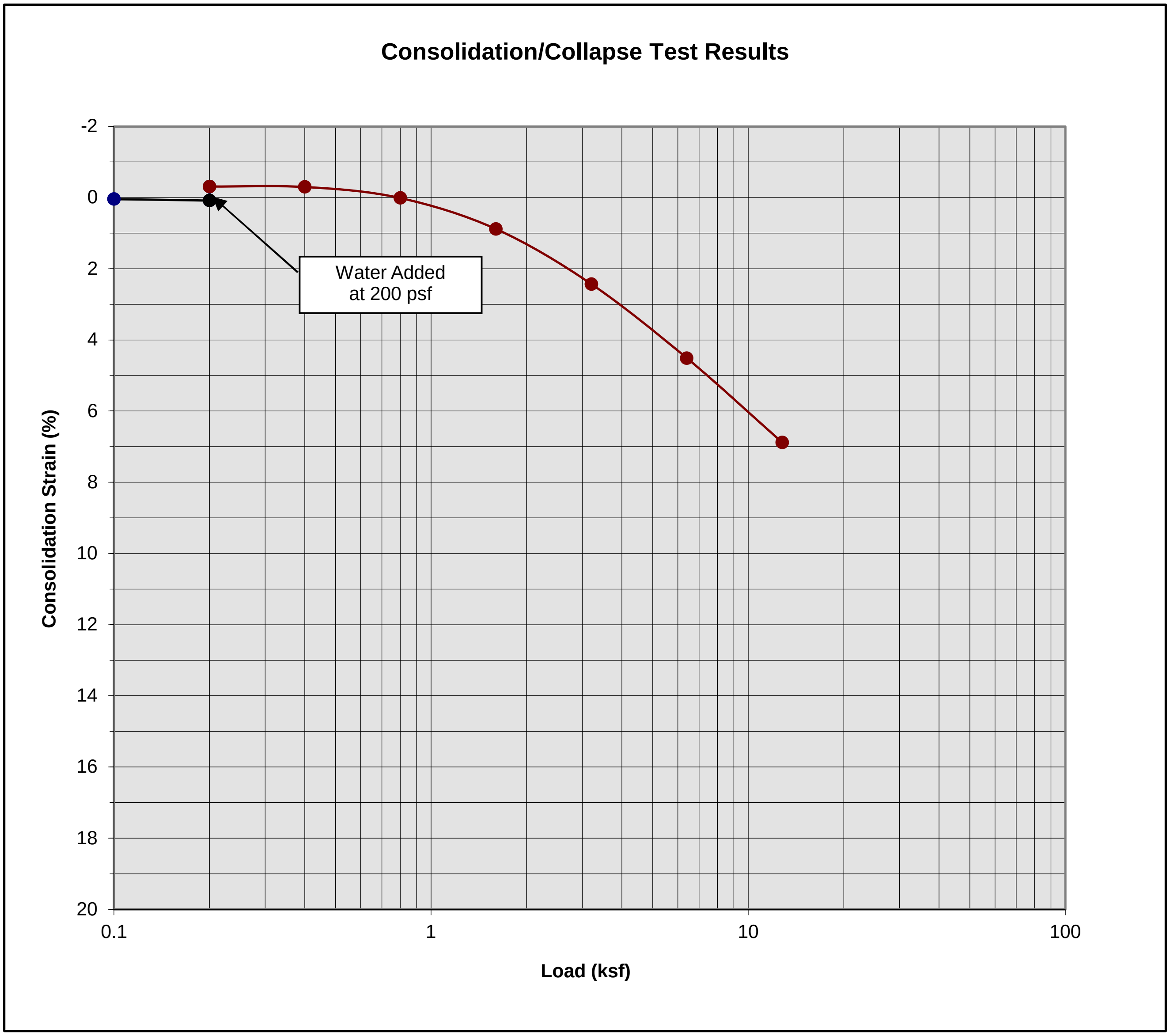


Classification: Brown Clayey fine Sand to fine Sandy Clay, little Silt

Boring Number:	B-10	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	3 to 4	Initial Dry Density (pcf)	127.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	134.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.10

Two Proposed Commercial/Industrial Buildings
Menifee, California
Project No. 20G227
PLATE C- 3

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

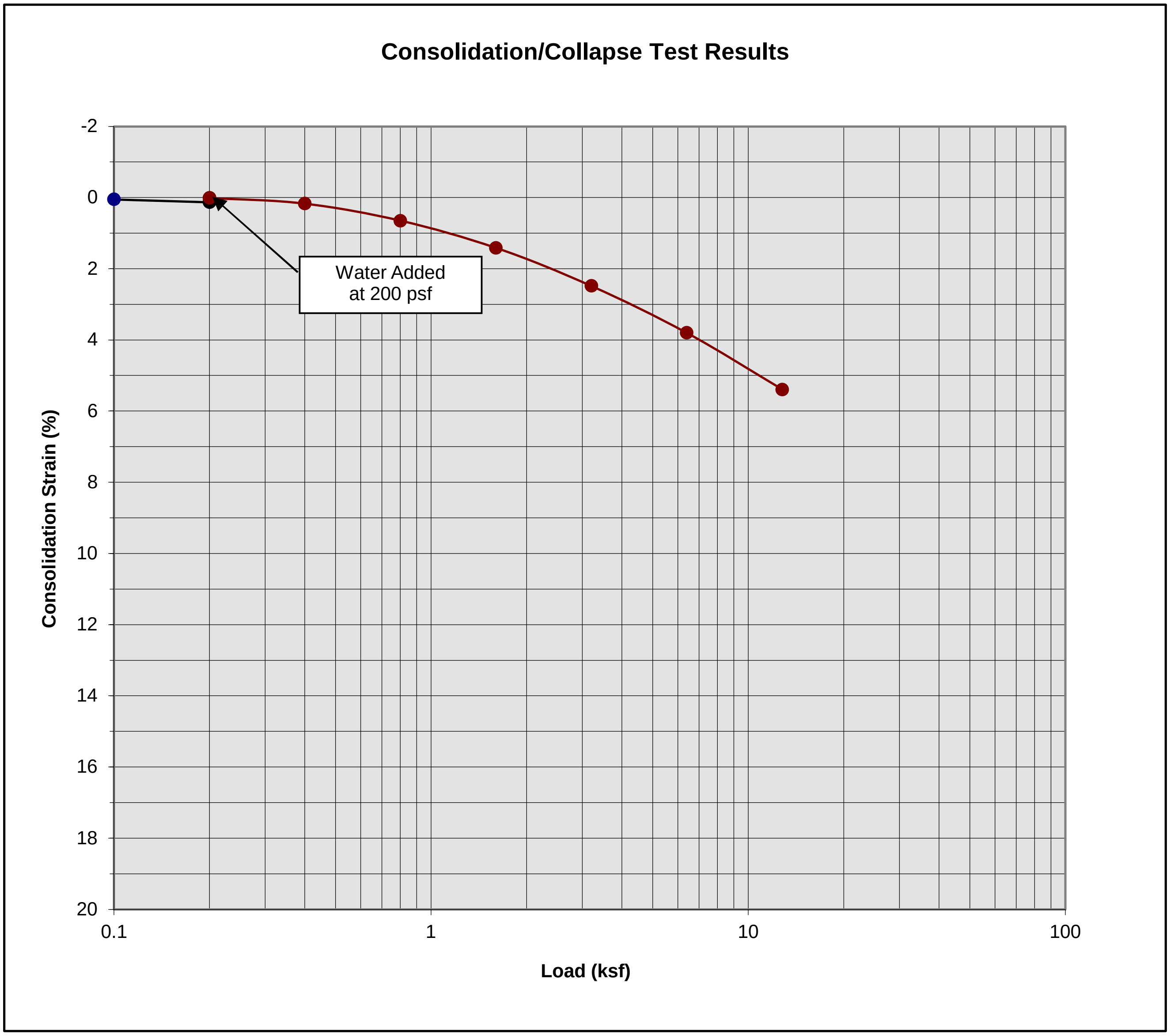


Classification: Brown Silty Clay

Boring Number:	B-12	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	1 to 2	Initial Dry Density (pcf)	118.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	126.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.39

Two Proposed Commercial/Industrial Buildings
Menifee, California
Project No. 20G227
PLATE C-4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

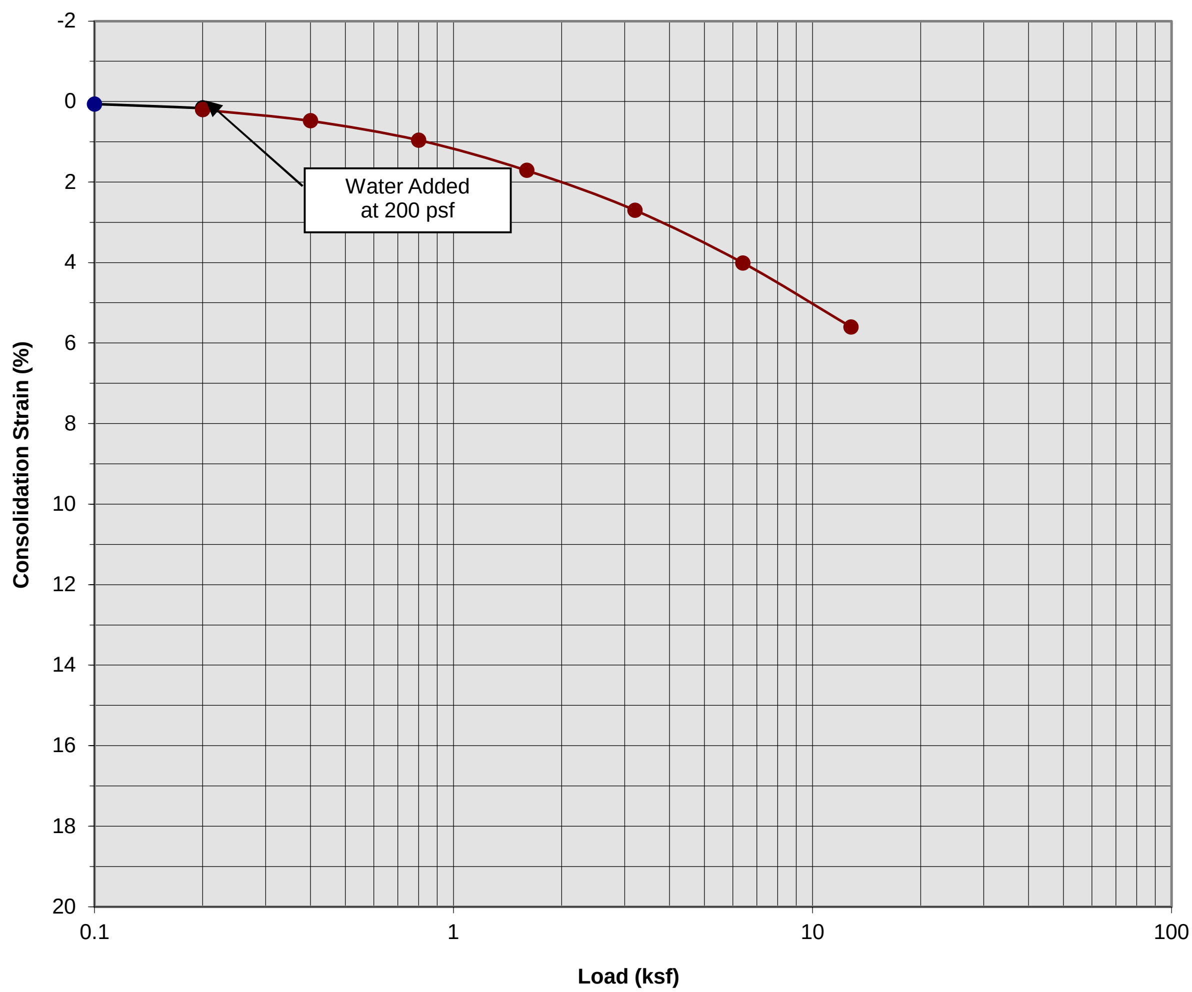


Classification: Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-12	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	5 to 6	Initial Dry Density (pcf)	120.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	126.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.12

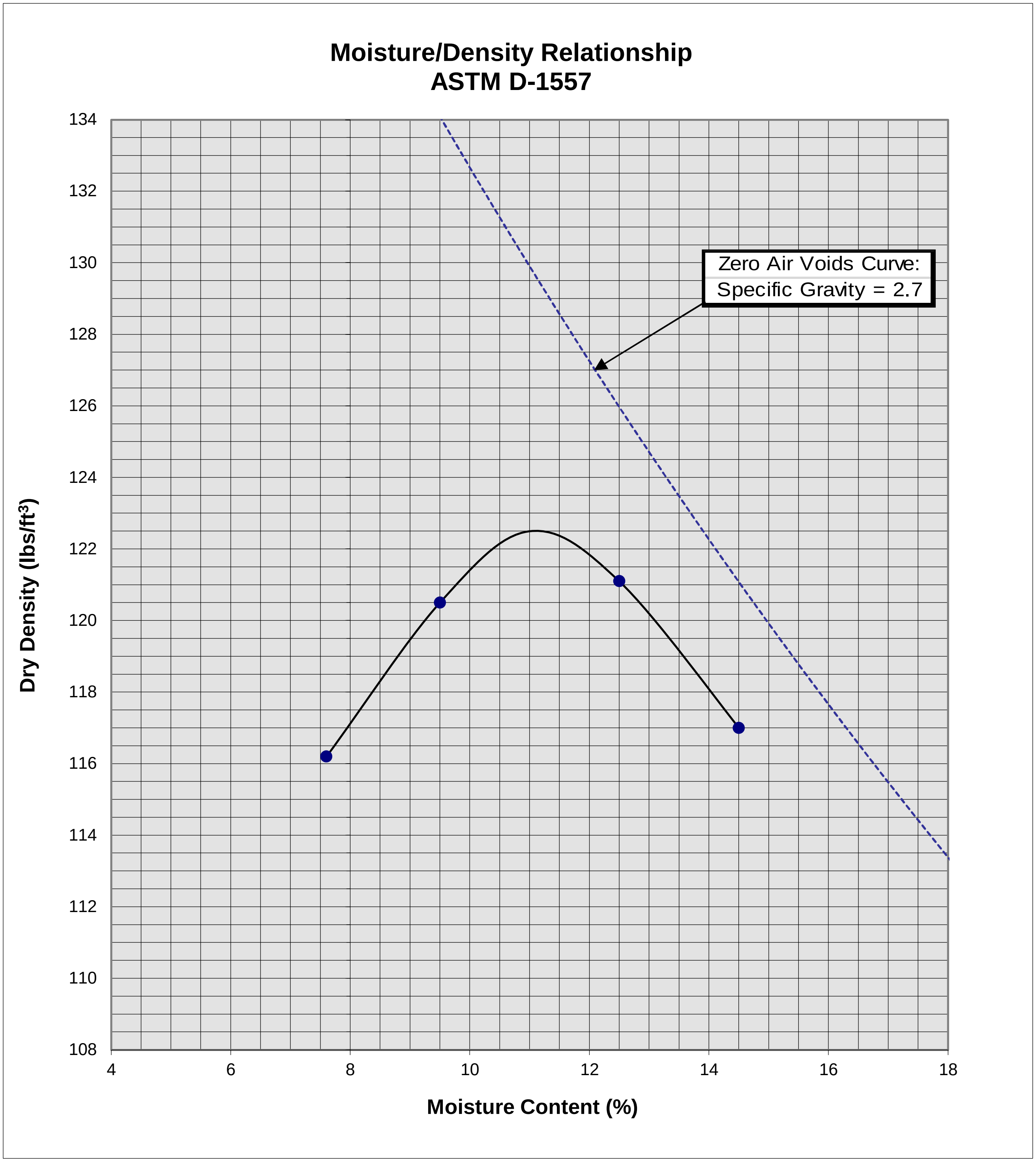
SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



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

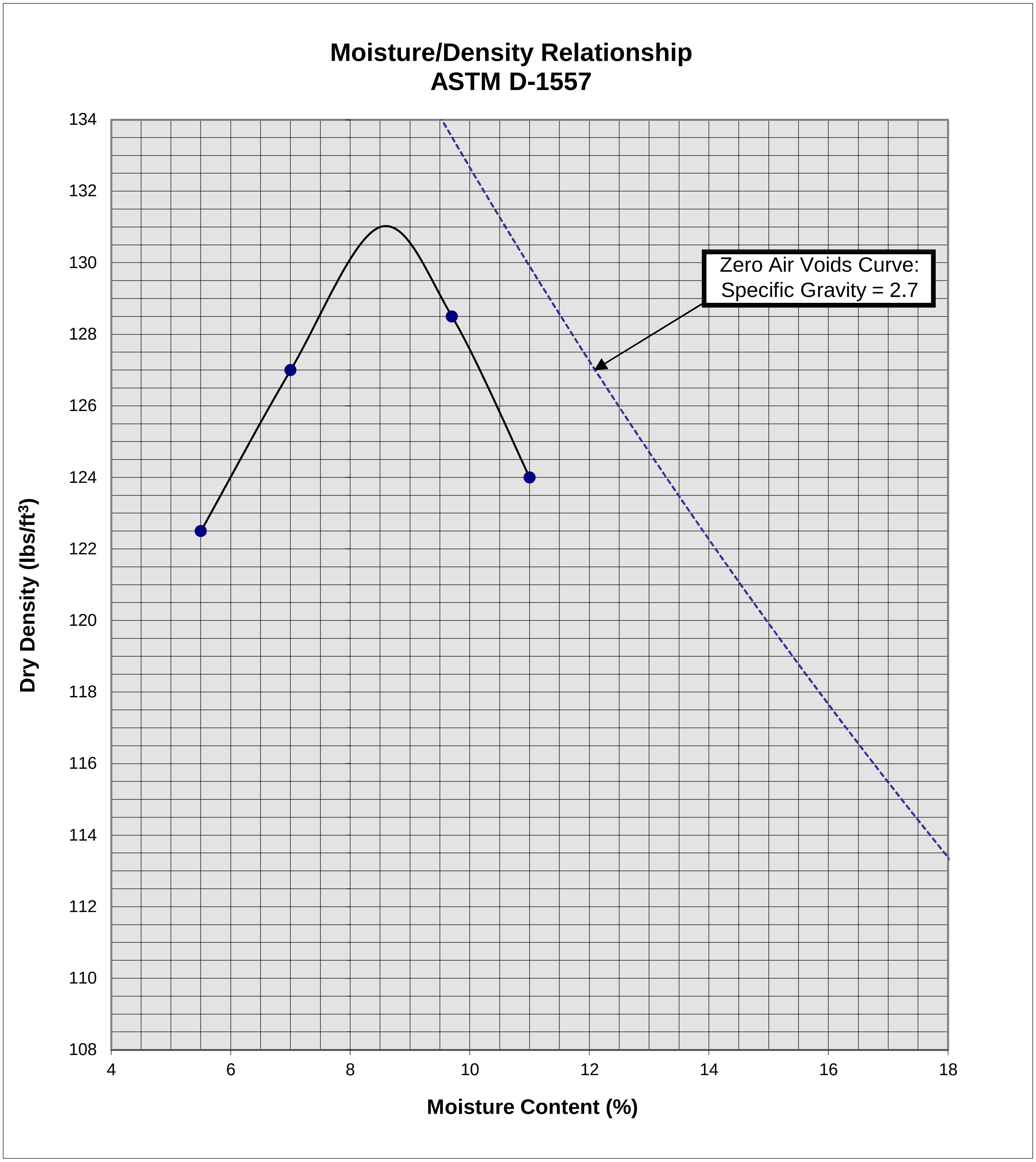
Boring Number:	B-12	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	7 to 8	Initial Dry Density (pcf)	124.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	131.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.04



Soil ID Number	B-5 @ 0-5'
Optimum Moisture (%)	11
Maximum Dry Density (pcf)	122.5
Soil Classification	Brown fine Sandy Clay

Two Proposed Commercial/Industrial Buildings
Menifee, California
Project No. 20G227
PLATE C-7

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-10 @ 0-5'
Optimum Moisture (%)	8.5
Maximum Dry Density (pcf)	131
Soil Classification	Brown Clayey fine to medium Sand, trace coarse Sand

Two Proposed Commercial/Industrial Buildings
Menifee, California
Project No. 20G227

PLATE C-8

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

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

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

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

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

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

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

Site Preparation

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

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

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

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

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

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

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

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

<u>Compacted Fills</u>

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

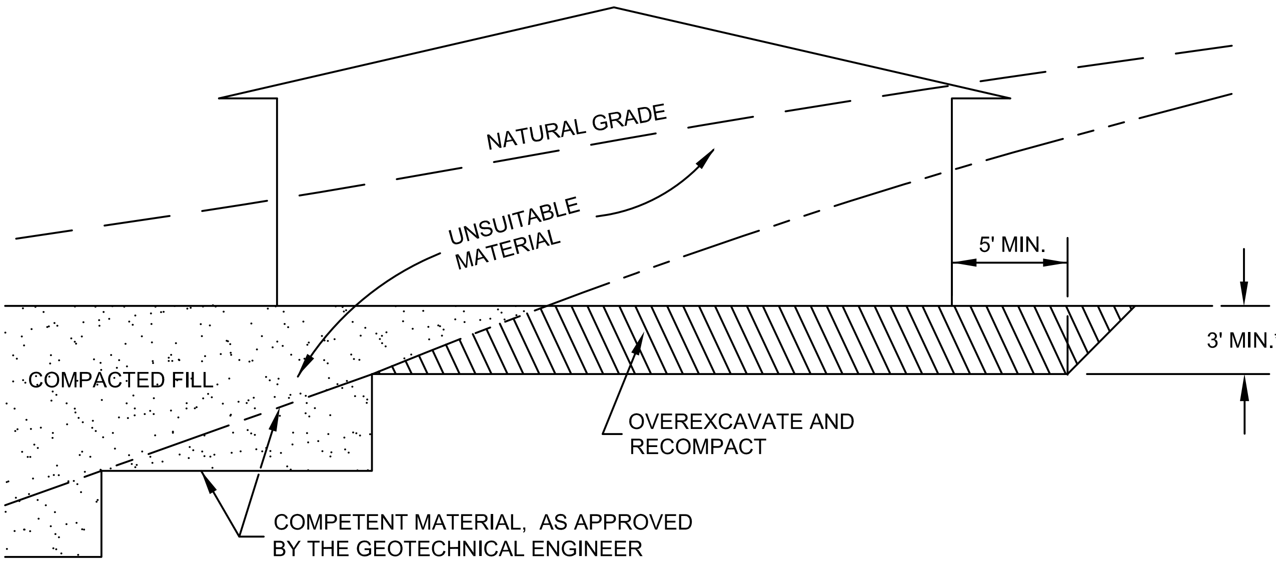
Subdrains

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

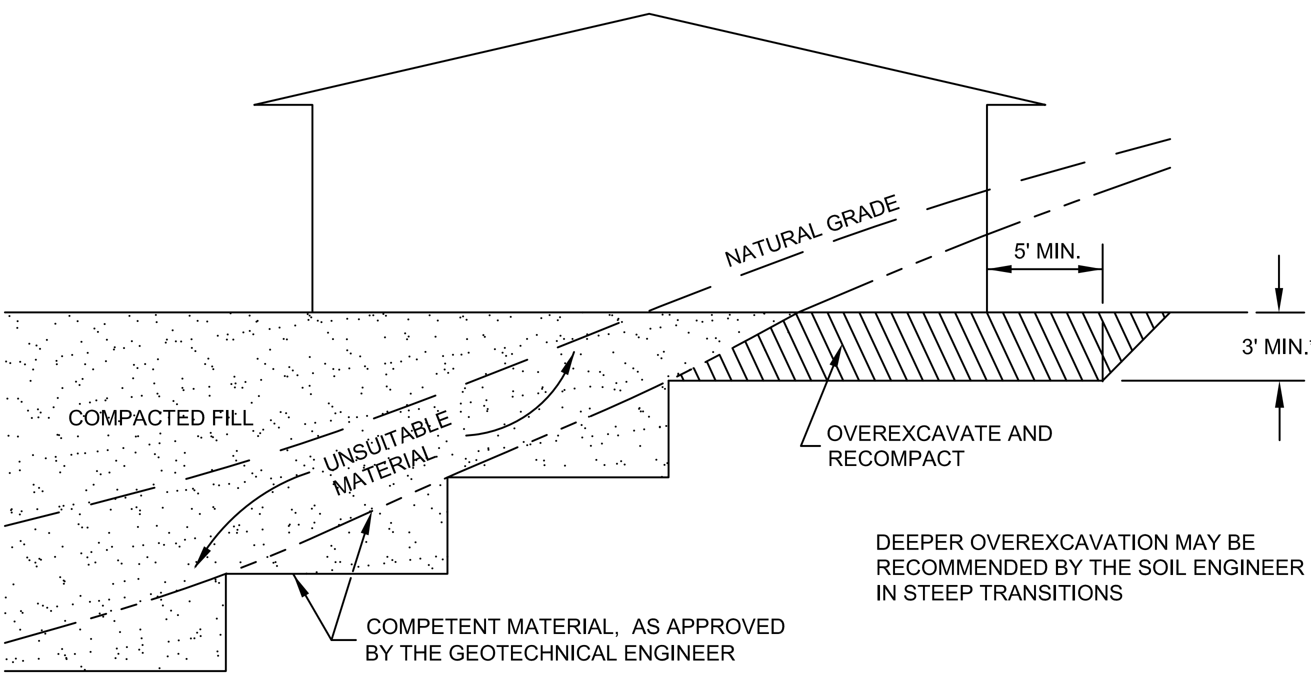
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

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

CUT LOT



CUT/FILL LOT (TRANSITION)

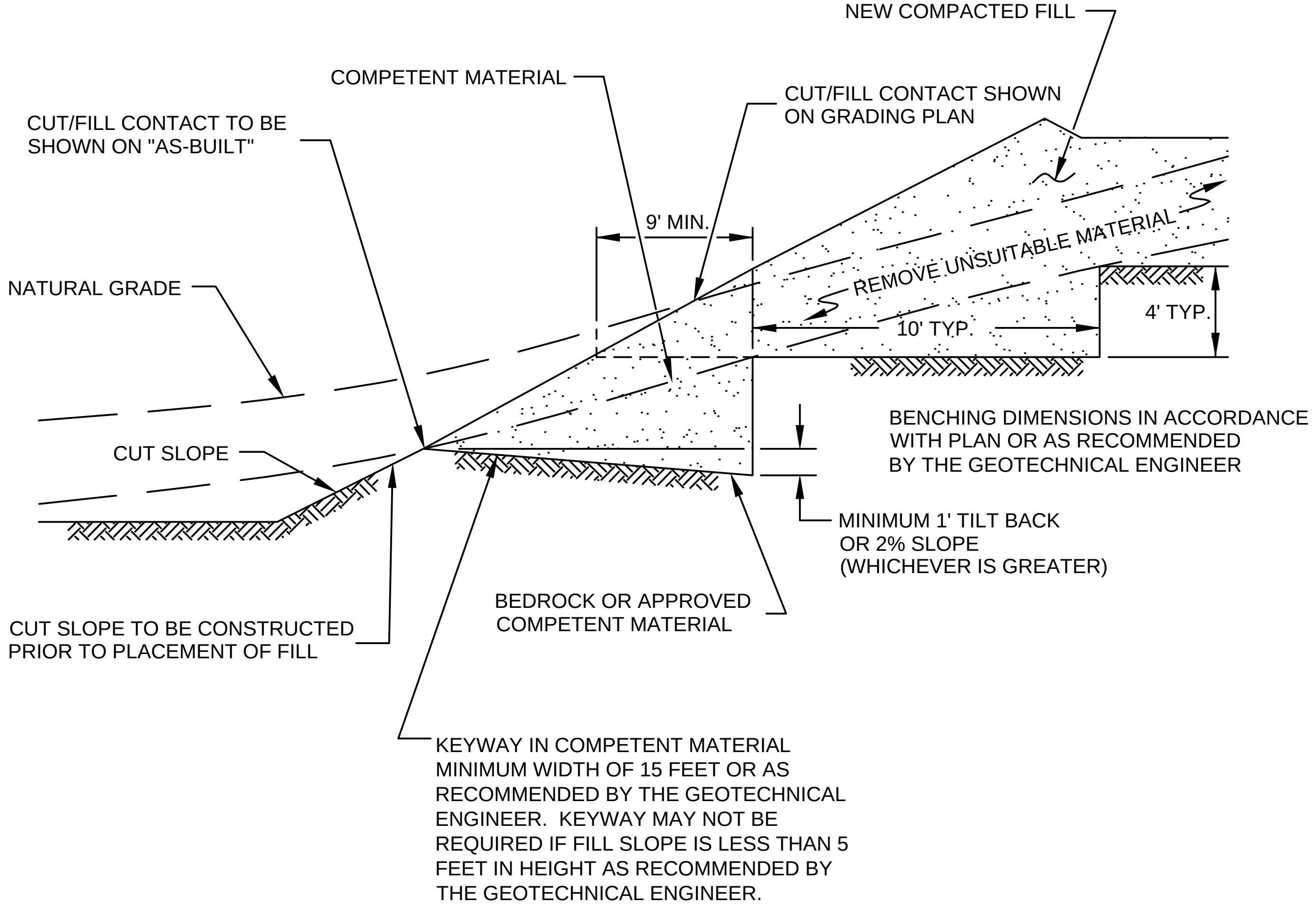


*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION.
ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

TRANSITION LOT DETAIL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-1	



FILL ABOVE CUT SLOPE DETAIL

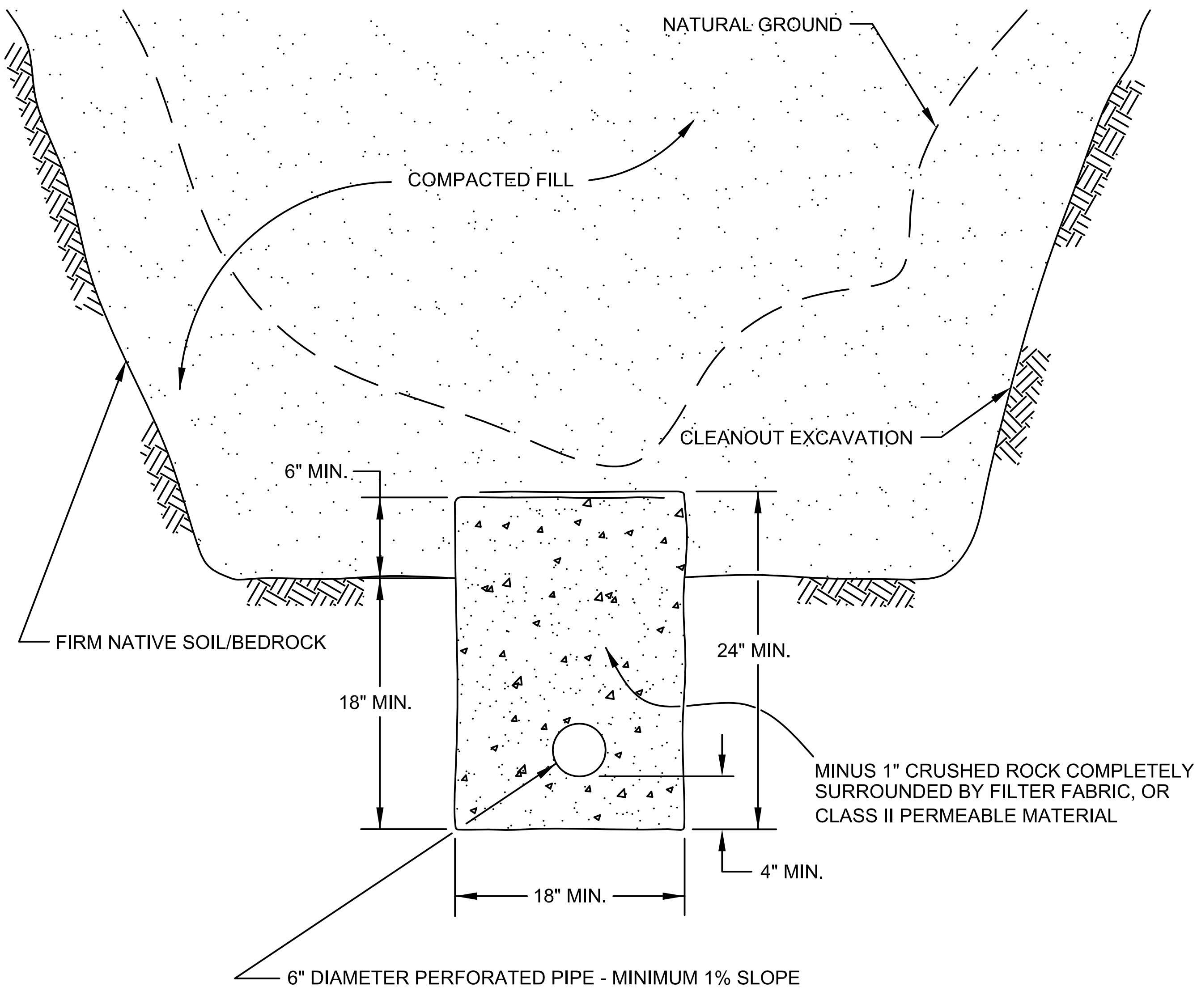
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

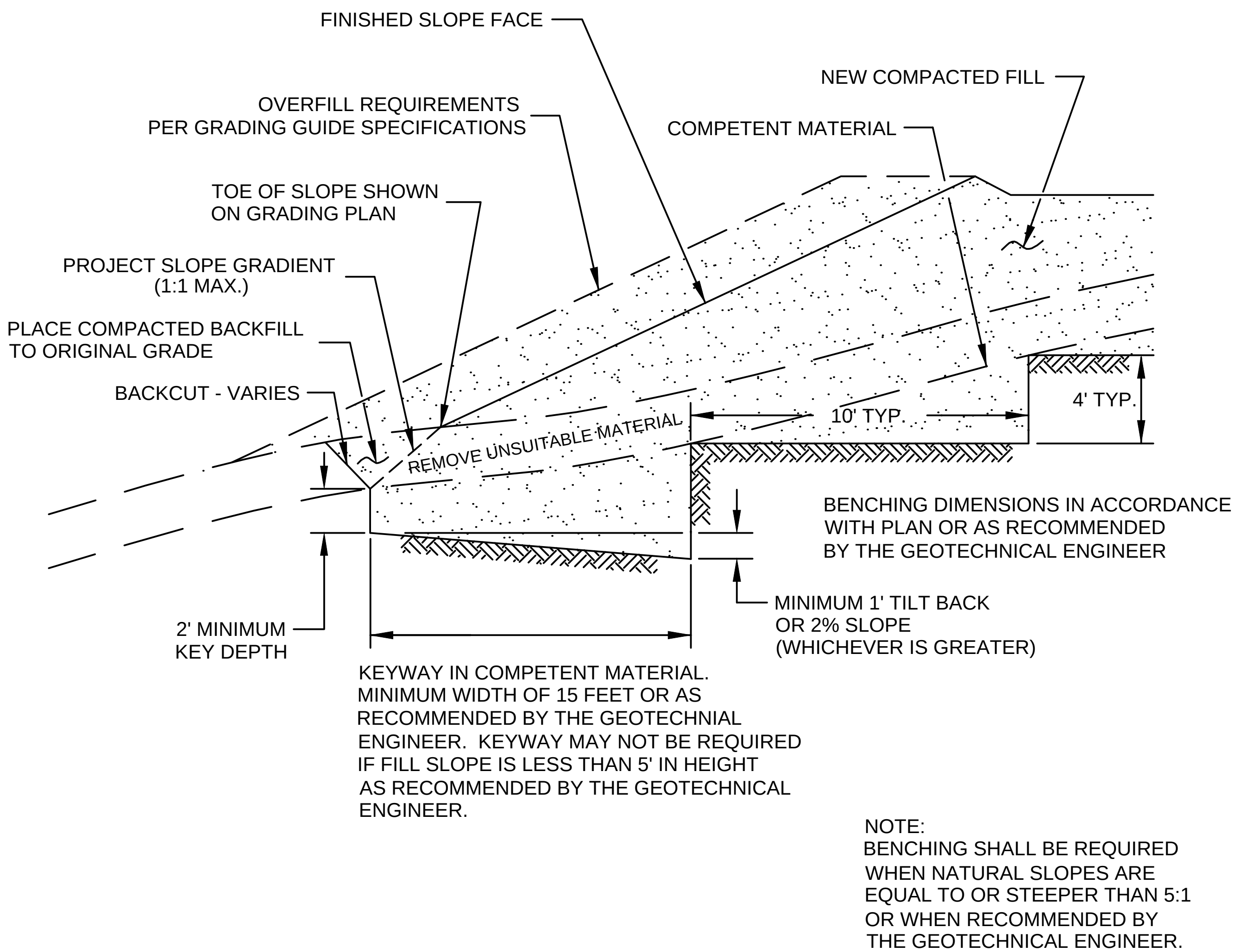
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

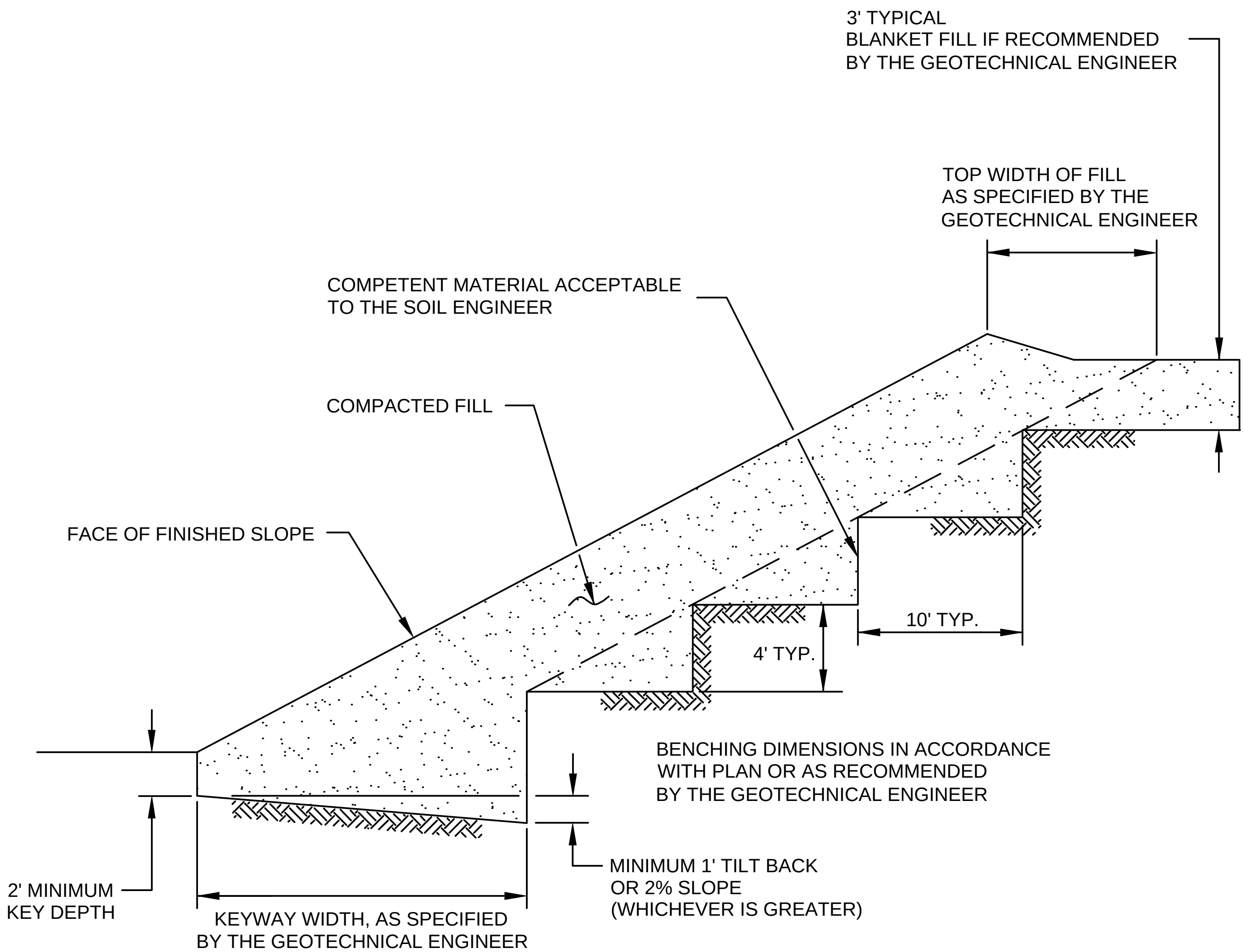
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

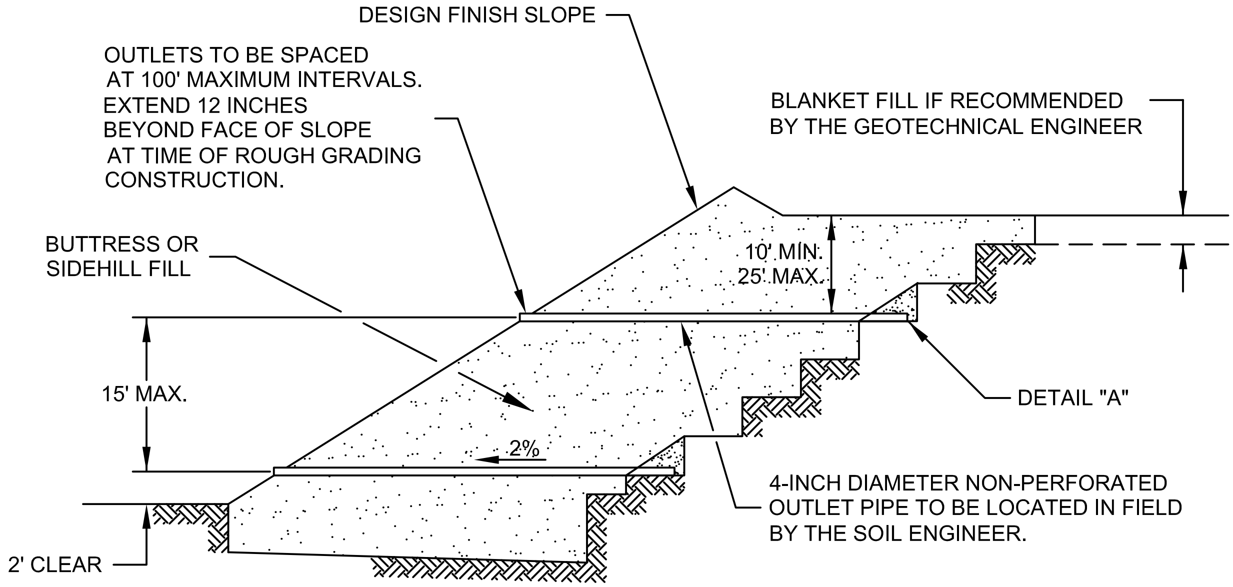
SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-5	



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

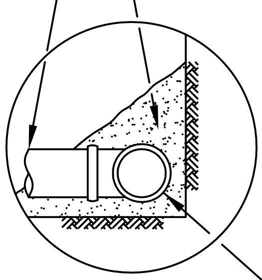
SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

OUTLET PIPE TO BE CONNECTED 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.

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.

DETAIL "A"

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

SLOPE FILL SUBDRAINS

GRADING GUIDE SPECIFICATIONS

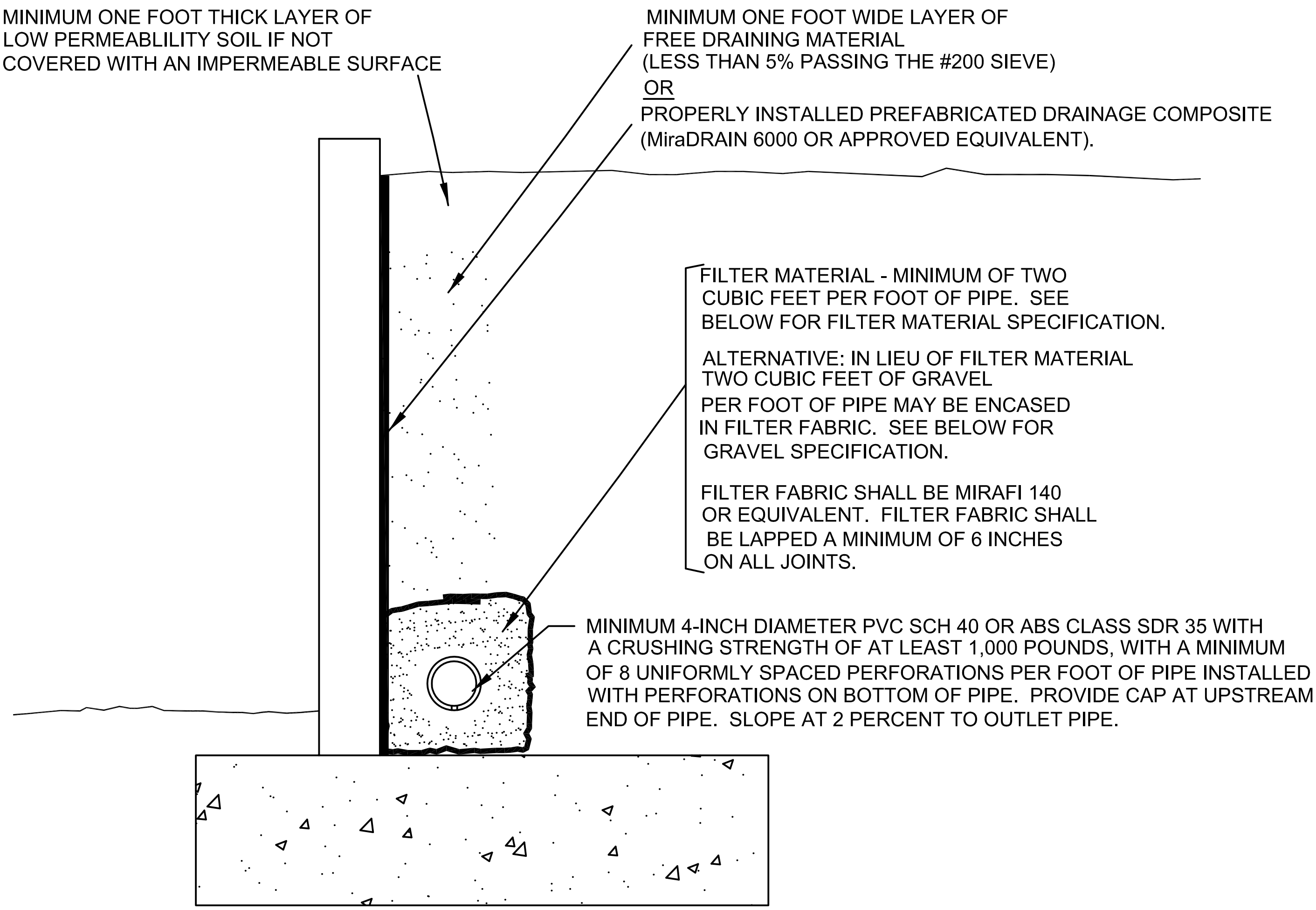
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

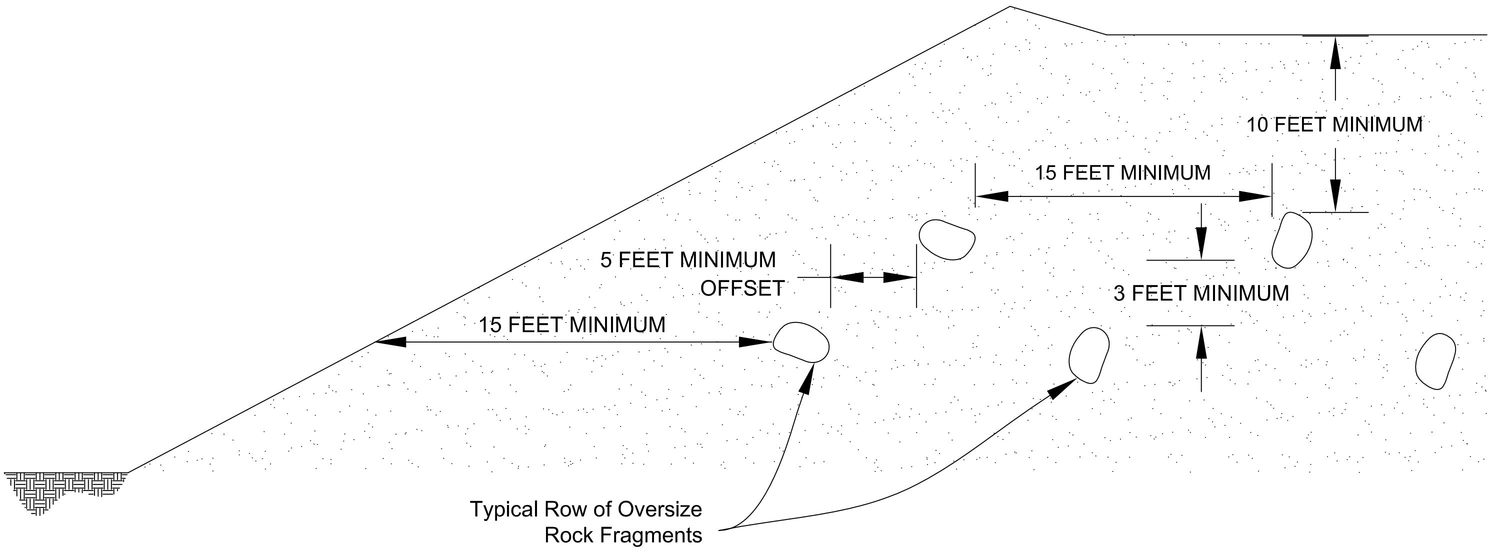
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

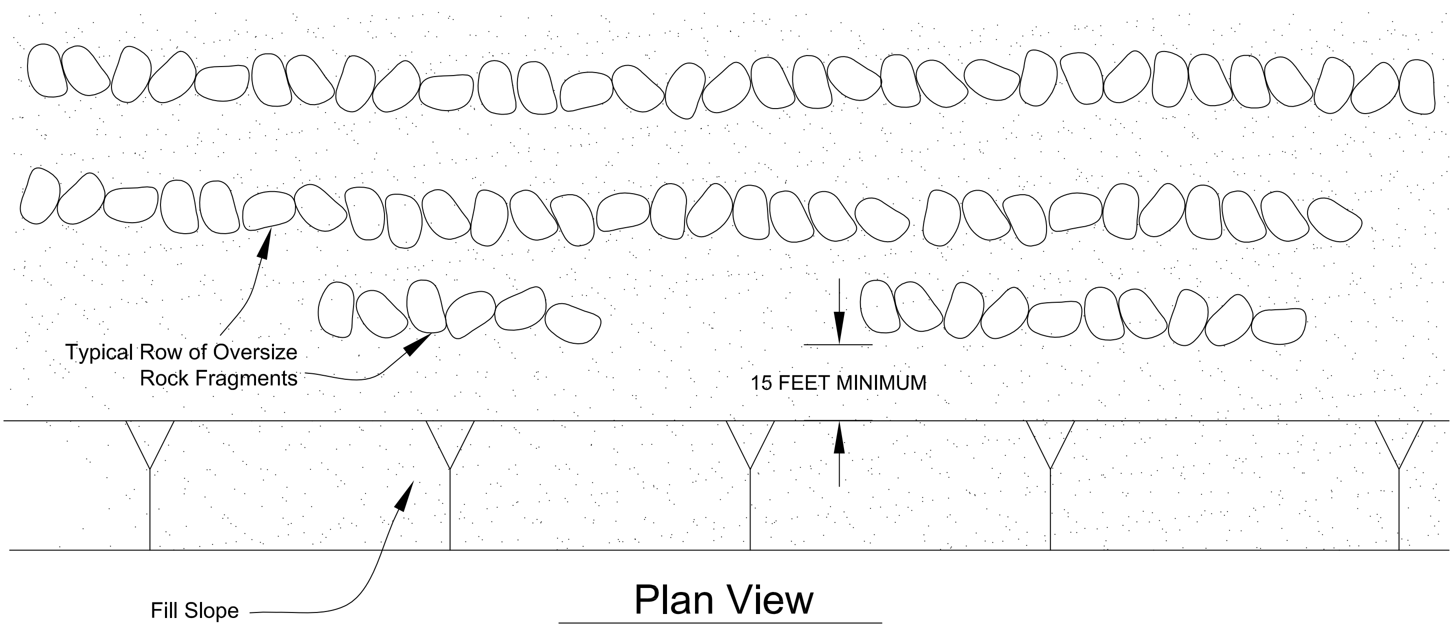
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

SoCalGeo
SOUTHERN
CALIFORNIA
GEOTECHNICAL

APPENDIX E

TERRA GEOSCIENCES

GROUND-MOTION SEISMIC ANALYSIS

PROPOSED COMMERCIAL/INDUSTRIAL DEVELOPMENT

SWC OF DAWSON AND ETHANAC ROADS

MENIFEE, RIVERSIDE COUNTY, CALIFORNIA

Project No. 203539-1

November 24, 2020

Prepared for:

Southern California Geotechnical, Inc.
22885 E. Savi Ranch Parkway, Suite E
Yorba Linda, CA 92887

Southern California Geotechnical, Inc.
22885 E. Savi Ranch Parkway, Suite E
Yorba Linda, CA 92887

Attention: Mr. Dan Nielsen, PE

Regarding: Ground-Motion Seismic Analysis
Proposed Commercial/Industrial Devlopment
SWC of Dawson and Ethanac Roads
Menifee, Riverside County, California SCG
Project No. 20G227

INTRODUCTION

At your request, this firm has prepared a ground-motion seismic analysis report for the proposed commercial/industrial development project, as referenced above. The purpose this study was to evaluate the site-specific ground motion parameters to aid in the seismic design for this project, based on the current 2019 California Building Code (CBC). Our work included performing a seismic shear-wave study for determining the Site Classification and V_{S30} input values for this analysis. The location of the seismic shear-wave survey line is shown on the provided map, as presented on the Seismic Line Location Map (see Plate 1), for reference.

The scope of services provided for this evaluation included the following:

- ***Review of available published and unpublished geologic/seismic data in our files pertinent to the site, including your provided boring logs.***
- ***Performing a seismic surface-wave survey by a licensed State of California Professional Geophysicist that included one traverse for shear-wave velocity analysis purposes.***
- ***Evaluation of the local and regional tectonic setting including performing a site-specific 2019 CBC ground motion analysis.***
- ***Preparation of this report presenting our findings, with respect to the seismic design parameters.***

Accompanying Map and Appendices

Plate 1- Seismic Line Location Map
Appendix A - Shear-Wave Survey
Appendix B - Site Specific Ground Motion Analysis
Appendix C - References

PROJECT SUMMARY

Based on the provided information, we understand that two single-story concrete tilt-up commercial/industrial buildings are proposed to be constructed within the subject property, that are 1,075,680± ft^2 and 385,970± ft^2 in dimension. The buildings will be surrounded by asphaltic concrete pavements in the parking and drive lanes, with Portland cement concrete pavements in the loading dock areas, and limited areas of concrete flatwork and landscape planters throughout. The approximate locations of the proposed buildings are shown as the purple outlines on the Seismic Line Location Map, Plate 1, for reference. For this project, we have performed a field reconnaissance, reviewed pertinent available geologic and geotechnical data in our files, including your provided field boring log (SCG, 2020), along with performing a site-specific seismic shear-wave survey.

To aid in providing applicable data for the site-specific ground motion analysis, a seismic shear-wave survey using the multi-channel analysis of surface waves (MASW) and microtremor array measurements (MAM) methods was performed in order to assess the one-dimensional average shear-wave velocity structure beneath the subject site to a depth of at least 100 feet. This survey line was performed along the southern-central portion of the site, proximal to both proposed buildings (as shown on Plate 1), which provided the necessary survey line length, as well as being representative for the site development. The resultant shear wave velocity (V_S) from this survey line within the upper 100 feet (30 meters) was then used to both determine the Site Classification (ASCE, 2017, Table 20.3-1) of the subject project study area, as well as being used for the V_S input value of the site-specific CBC seismic analysis. The detailed results of this survey are presented within Appendix A for reference.

Geologic mapping of the local area by Morton (2003), indicates that the subject site is mantled by old alluvial fan deposits (late to middle Pleistocene age). These surficial deposits are generally described as being comprised of indurated reddish-brown gravel and sand deposits. Presumably underlying these deposits are progressively older and more indurated alluvial deposits, in turn underlain by Cretaceous age granitic bedrock at depth. Site-specific exploration by Southern California Geotechnical, Inc. (SCG, 2020) revealed that the project development area is generally underlain by interbedded dense to very-dense silty fine sand and fine sandy silt, to a depth of at least 25± feet.

The approximate location of the seismic shear-wave traverse (Seismic Line SW-1) is shown a captured Google™ Earth image (2020) that was provided by Southern California Geotechnical, Inc., as presented on the Seismic Line Location Map, Plate 1. Photographic views of the seismic line traverse have been included within Appendix A for both visual and reference purposes.

SITE-SPECIFIC GROUND MOTION ANALYSIS

As requested, we have performed a site-specific seismic ground motion analysis as discussed above. Geographically, the proposed development project is generally located at Latitude 33.7378 and Longitude -117.1804 (World Geodetic System of 1984). The mapped spectral acceleration parameters, coefficients, and other related seismic parameters, were evaluated using the OSHPD Seismic Design Maps Tool web application (OSHPD, 2020) and the California Building Code criteria (CBC, 2019), with the site-specific ground motion analysis being performed following Section 21 of the ASCE 7-16 Standard (ASCE, 2017).

The results of this site-specific ground motion analysis have been summarized and are tabulated below, with the detailed analysis being presented within Appendix B:

TABLE 1 – SUMMARY OF SITE-SPECIFIC SEISMIC DESIGN PARAMETERS

Factor or Coefficient	Value
S_S	**1.416g**
S_1	**0.526g**
F_a	**1.2**
F_v	**1.47**
S_{DS}	**0.940g**
S_{D1}	**0.470g**
S_{MS}	**1.412g**
S_{M1}	**0.702g**
T_L	**8 Seconds**
MCE_G PGA	**0.64g**
Shear-Wave Velocity (V_{100})	**1,395.7 ft/sec**
Site Classification	**C**
Risk Category	**II**

CLOSURE

Our conclusions and recommendations are based on an interpretation of available existing geologic, geophysical, geotechnical, and seismic data. No subsurface exploration was performed by this firm for this evaluation. We make no warranty, either express or implied. Should conditions be encountered at a later date or more information becomes available that appear to be different than those indicated in this report, we reserve the right to reevaluate our conclusions and recommendations and provide appropriate mitigation measures, if warranted. If this report is not understood, it is the responsibility of the owner, contractor, engineer, and/or governmental agency, etc., to contact this office for further clarification.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Certified Engineering Geologist
CEG 1459

Professional Geophysicist
PGP 1002

SEISMIC LINE LOCATION MAP

Base Map: Provided by client; Seismic shear-wave survey line (SW-1) shown as yellow line, proposed buildings outlined in purple.

APPENDIX A

SHEAR-WAVE SURVEY

SHEAR-WAVE SURVEY

Methodology

The fundamental premise of this survey uses the fact that the Earth is always in motion at various seismic frequencies. These relatively constant vibrations of the Earth's surface are called microtremors, which are very small with respect to amplitude and are generally referred to as background "noise" that contain abundant surface waves. These microtremors are caused by both human activity (i.e., cultural noise, traffic, factories, etc.) and natural phenomenon (i.e., wind, wave motion, rain, atmospheric pressure, etc.) which have now become regarded as useful signal information. Although these signals are generally very weak, the recording, amplification, and processing of these surface waves has greatly improved by the use of technologically improved seismic recording instrumentation and recently developed computer software. For this application, we are mainly concerned with the Rayleigh wave portion of the seismic signals, which is also referred to as "ground roll" since the Rayleigh wave is the dominant component of ground roll.

For the purposes of this study, there are two ways that the surface waves were recorded, one being "active" and the other being "passive." Active means that seismic energy is intentionally generated at a specific location relative to the survey spread and recording begins when the source energy is imparted into the ground (i.e., MASW survey technique). Passive surveying, also called "microtremor surveying," is where the seismograph records ambient background vibrations (i.e., MAM survey technique), with the ideal vibration sources being at a constant level. Longer wavelength surface waves (longer-period and lower-frequency) travel deeper and thus contain more information about deeper velocity structure and are generally obtained with passive survey information. Shorter wavelength (shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure and are generally collected with the use of active sources. For the most part, higher frequency active source surface waves will resolve the shallower velocity structure and lower frequency passive source surface waves will better resolve the deeper velocity structure. Therefore, the combination of both of these surveying techniques provides a more accurate depiction of the subsurface velocity structure.

The assemblage of the data that is gathered from these surface wave surveys results in development of a dispersion curve. Dispersion, or the change in phase velocity of the seismic waves with frequency, is the fundamental property utilized in the analysis of surface wave methods. The fundamental assumption of these survey methods is that the signal wavefront is planar, stable, and isotropic (coming from all directions) making it independent of source locations and for analytical purposes uses the spatial autocorrelation method (SPAC). The SPAC method is based on theories that are able to detect "signals" from background "noise" (Okada, 2003). The shear wave velocity (V_S) can then be calculated by mathematical inversion of the dispersive phase velocity of the surface waves which can be significant in the presence of velocity layering, which is common in the near-surface environment.

Field Procedures

One seismic shear-wave survey traverse was performed within the southern-central portion of the site as approximated on the Seismic Line Location Map, Plate 1. For data collection, the field survey employed a twenty-four channel Geometrics StrataVisor™ NZXP model signal-enhancement refraction seismograph. This survey employed both active (MASW) and passive (MAM) source methods to ensure that both quality shallow and deeper shear-wave velocity information was recorded (Park et al., 2005). Both the MASW and MAM survey lines used the same linear geometry array that consisted of a 184-foot long spread using a series of twenty-four 4.5-Hz geophones that were spaced at regular eight-foot intervals.

For the MASW survey, the ground vibrations were recorded using a one second record length at a sampling rate of 0.5-milliseconds. Two seismic records were obtained using a 30-foot offset from the beginning and end of the survey line utilizing a 16-pound sledge-hammer as the energy source to produce the seismic waves. Each of these shot points used multiple shots (stacking) to improve the signal to noise ratio of the data.

The MAM survey did not require the introduction of artificial seismic sources and only background ambient noise was recorded. The ambient ground vibrations were recorded using a thirty-two second record length at a two-millisecond sampling rate with 25 separate seismic records being obtained for quality control purposes. The seismic-wave forms and associated frequency spectrum that were displayed on the seismograph screen were used to assess the recorded seismic wave data for quality control purposes in the field. The acceptable records were digitally recorded on the in-board seismograph computer and subsequently transferred to a flash drive so that they could be subsequently transferred to our office computer for analysis.

Data Processing

For analysis and presentation of the shear-wave profile and supportive illustrations, this study used the SeisImager/SW™ computer software program developed by Geometrics, Inc. (2009). Both the active (MASW) and passive (MAM) survey results were combined for this analysis (Park et al., 2005). The combined results maximize the resolution and overall depth range in order to obtain one high resolution V_s curve over the entire sampled depth range. These methods economically and efficiently estimate one-dimensional subsurface shear-wave velocities using data collected from standard primary-wave (P-wave) refraction surveys, however, it should be noted that surface waves by their physical nature cannot resolve relatively abrupt or small-scale velocity anomalies.

Processing of the data proceeded by calculating the dispersion curve from the input data which subsequently created an initial shear-wave model based on the observed data. This initial model was then inverted in order to converge on the best fit of the initial model and the observed data, creating the final shear-wave model (Seismic Line SW-1) as presented within this appendix.

Data Analysis

Data acquisition went very smoothly and the quality was considered to be good. Analysis revealed that the average shear-wave velocity ("weighted average") in the upper 100 feet of the subject survey area is **1,395.7** feet per second as shown on the shear-wave model for Seismic Line SW-1, as presented within this appendix. This average velocity classifies the underlying soils to that of Site Class "**C**" (Very Dense Soil and Soft Rock), which has a velocity range from 1,200 to 2,500 ft/sec (ASCE, 2017; Table 20.3-1).

The "weighted average" velocity is computed from a formula that is used by the ASCE (2017; Section 20.4, Equation 20.4-1) to determine the average shear-wave velocity for the upper 100 feet of the subsurface (V100).

$$\mathbf{Vs = 100/[(d1/v1) + (d2/v2) + ...+ (dn/vn)]}$$

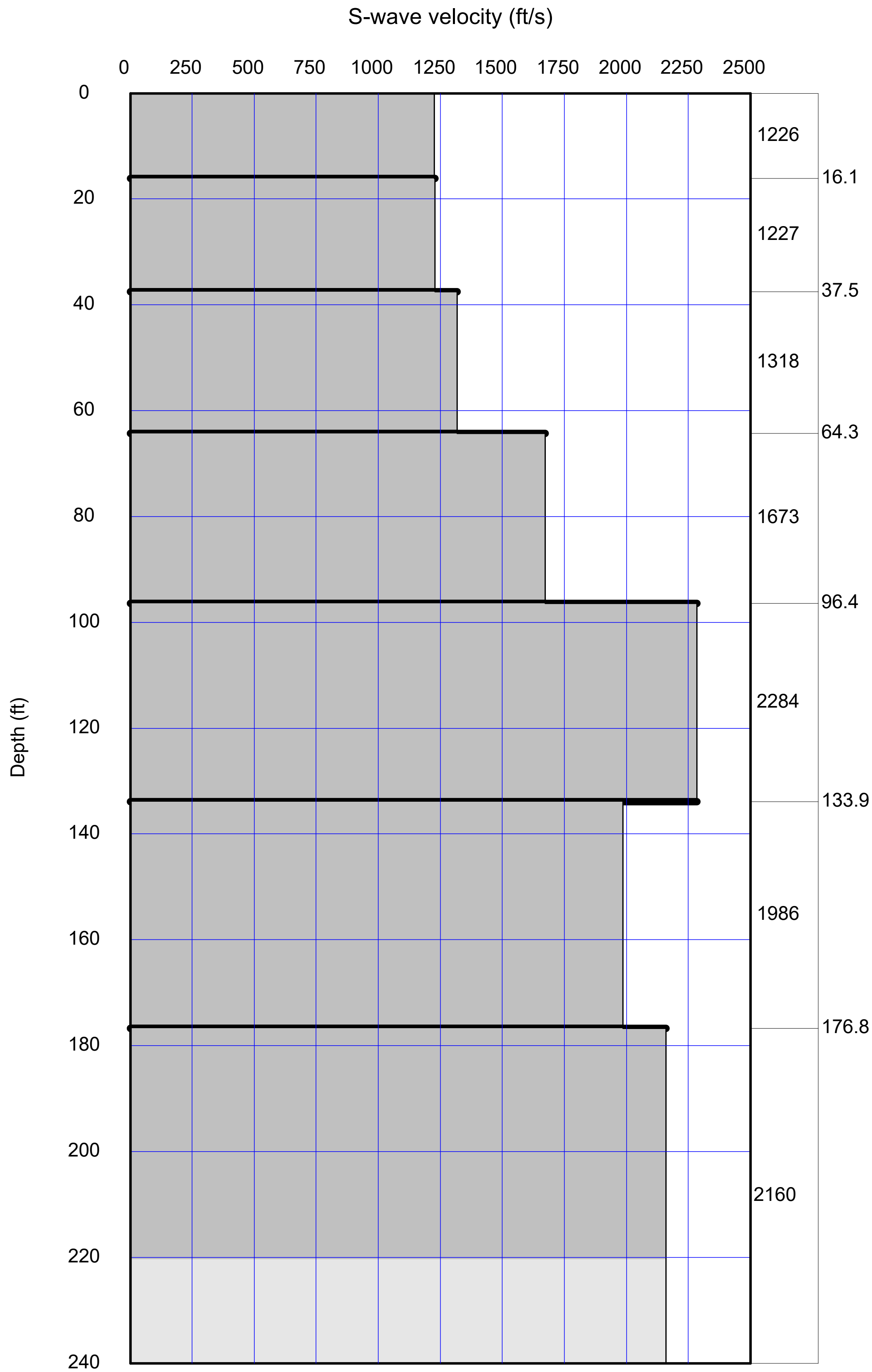
Where d1, d2, d3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n. The detailed shear-wave model displays these calculated layer boundaries/depths and associated velocities (feet/second) for the 220-foot profile where locally measured. The constrained data is represented by the dark-gray shading on the shear-wave model. The associated Dispersion Curves (for both the active and passive methods) which show the data quality and picks, along with the resultant combined dispersion curve model, are also included within this appendix, for reference purposes.

Limitations

This survey was performed using "state of the art" geophysical equipment, techniques, and computer software. We make no warranty, either expressed or implied. It should be understood that when using these theoretical geophysical principles and techniques, sources of error are possible in both the data obtained and in the interpretation. Compared with traditional borehole shear-wave surveys of which use vertical body waves, the sources of error (if present) using horizontal surface waves for this project are not believed to be greater than 15 percent. It is also important to understand that the fundamental limitation for seismic surveys is known as nonuniqueness, wherein a specific seismic data set does not provide sufficient information to determine a single "true" earth model. Therefore, the interpretation of any seismic data set uses "best-fit" approximations along with the geologic models that appear to be most reasonable for the local area being surveyed.

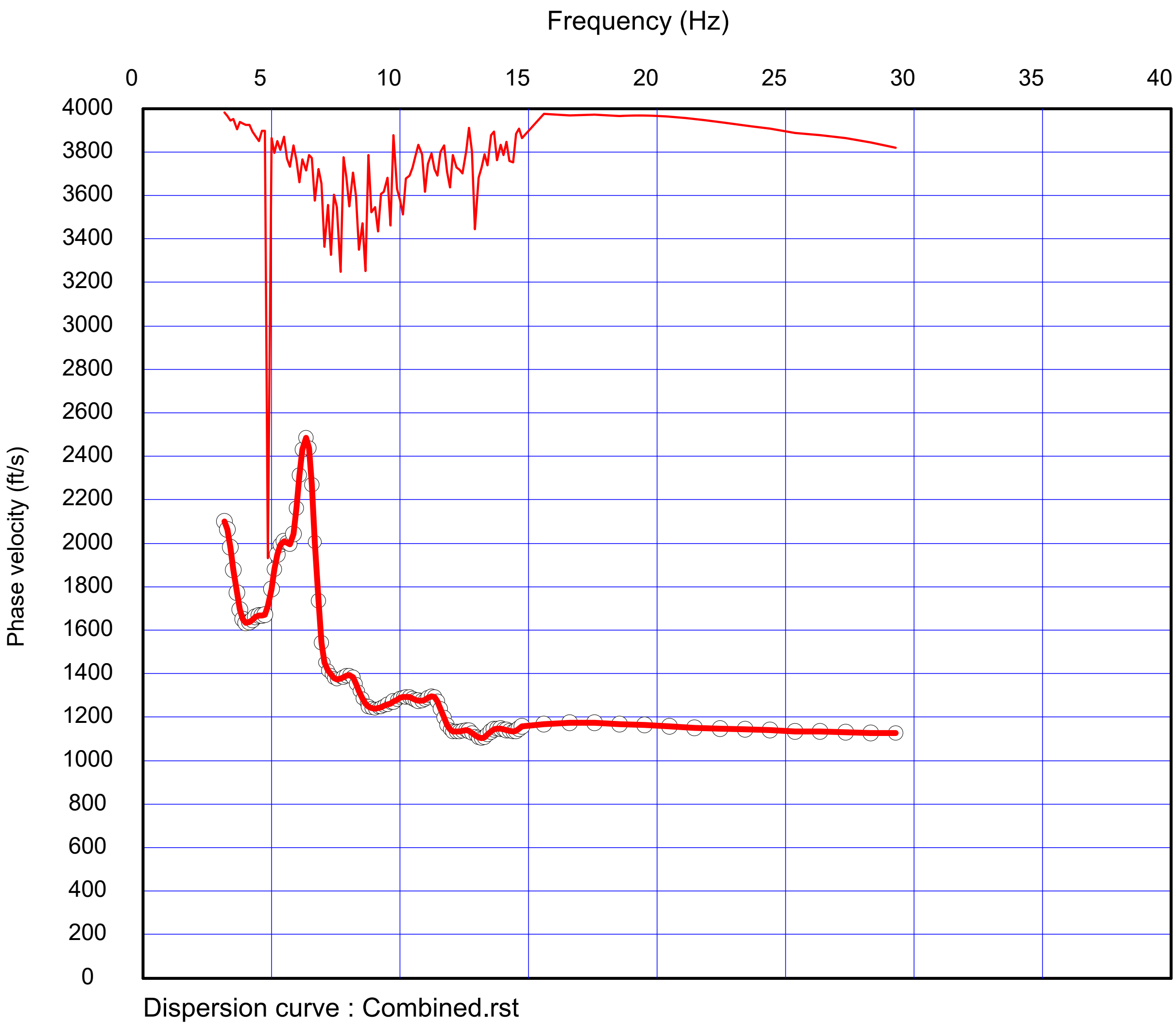
SEISMIC LINE SW-1

SHEAR-WAVE MODEL



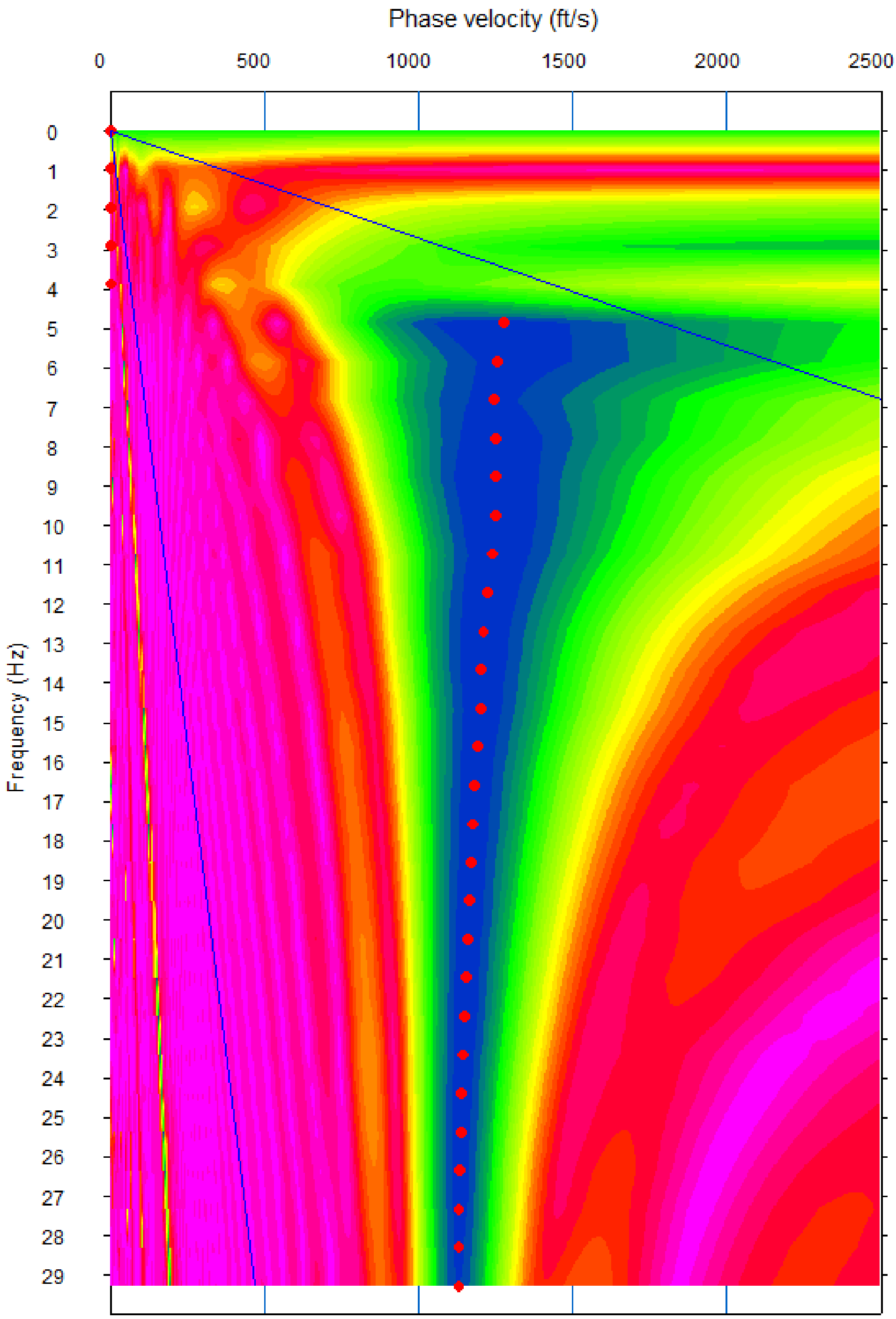
S-wave velocity model (inverted) : Final.rst **Average Vs 100ft = 1395.7 ft/sec**

SHEAR-WAVE MODEL SW-1



COMBINED DISPERSION CURVE

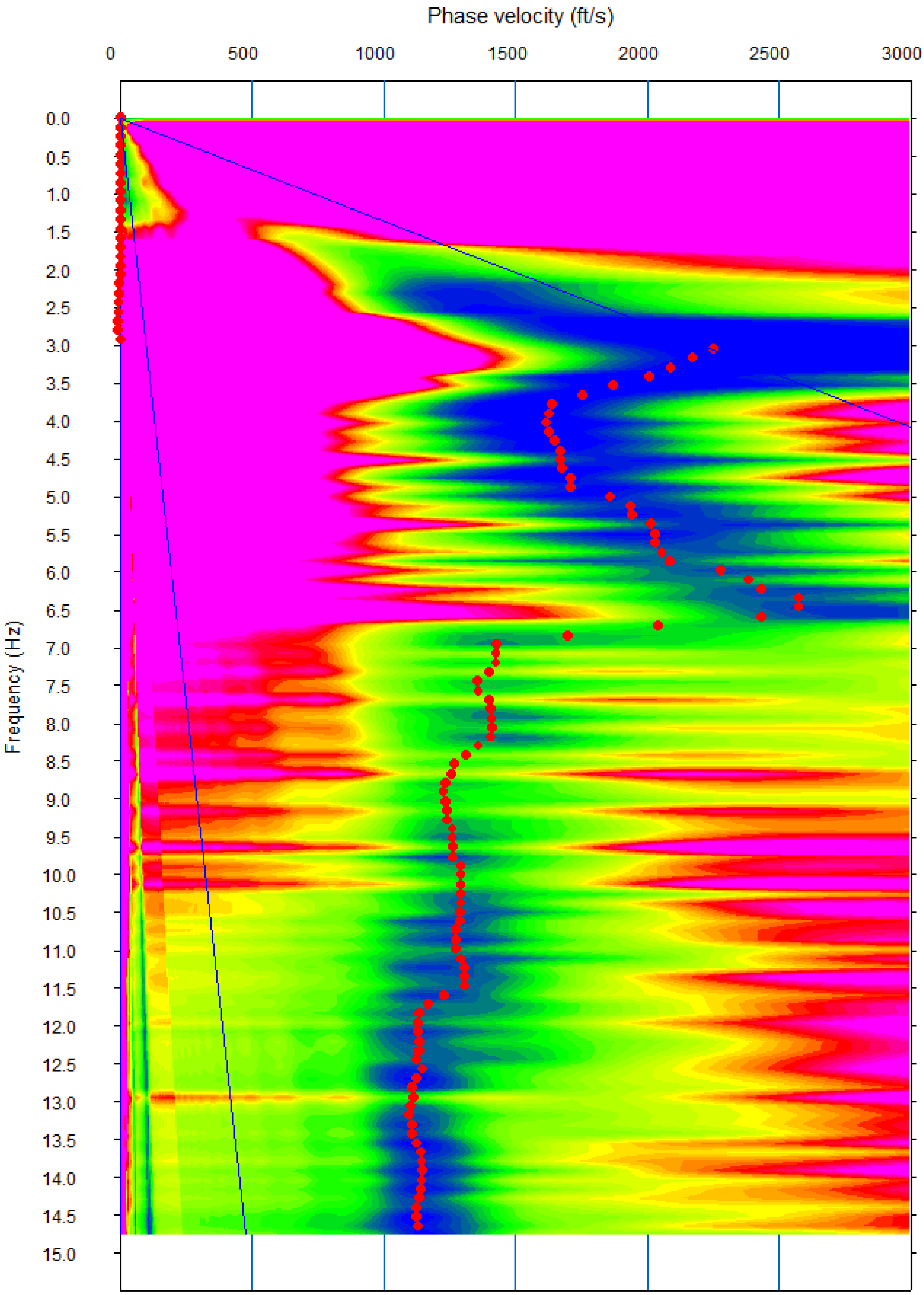
SEISMIC LINE SW-1



Dispersion Cure: Active.dat

ACTIVE DISPERSION CURVE

SEISMIC LINE SW-1



Dispersion Curve: Passive.dat

PASSIVE DISPERSION CURVE

SHEAR-WAVE SURVEY LINE PHOTOGRAPHS

View looking south along Seismic Line SW-1.

View looking north along Seismic Line SW-1.

APPENDIX B

SITE-SPECIFIC GROUND MOTION ANALYSIS

SITE-SPECIFIC GROUND MOTION ANALYSIS

A detailed summary of the site-specific ground motion analysis, which follows Section 21 of the ASCE Standard 7-16 (2017) and the 2019 California Building Code is presented below, with the Seismic Design Parameters Summary included within this appendix following the summary text.

- ***Mapped Spectral Acceleration Parameters (CBC 1613.2.1)***-

  Based on maps prepared by the U.S.G.S (Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for the 0.2 and 1-second Spectral Response Acceleration (5% of Critical Damping; Site Class B/C), a value of **1.416g** for the 0.2 second period (S_S) and **0.526** for the 1.0 second period (S_1) was calculated (ASCE 7-16 Figures 22-1, 22-2 and CBC 1613A.2.1).

- ***Site Classification (CBC 1613.2.2 & ASCE 7-16 Chapter 20)***-

  Based on the site-specific measured shear-wave value of 425.4 m/sec (1,395.7 feet/second), the soil profile type used should be Site Class **"C."** This Class is defined as having the upper 30 meters (100 feet) of the subsurface being underlain by very dense soil and soft rock with average shear-wave velocities of 360 to 760 meters/second (1,200 to 2,500 feet/second), as detailed within Appendix A.

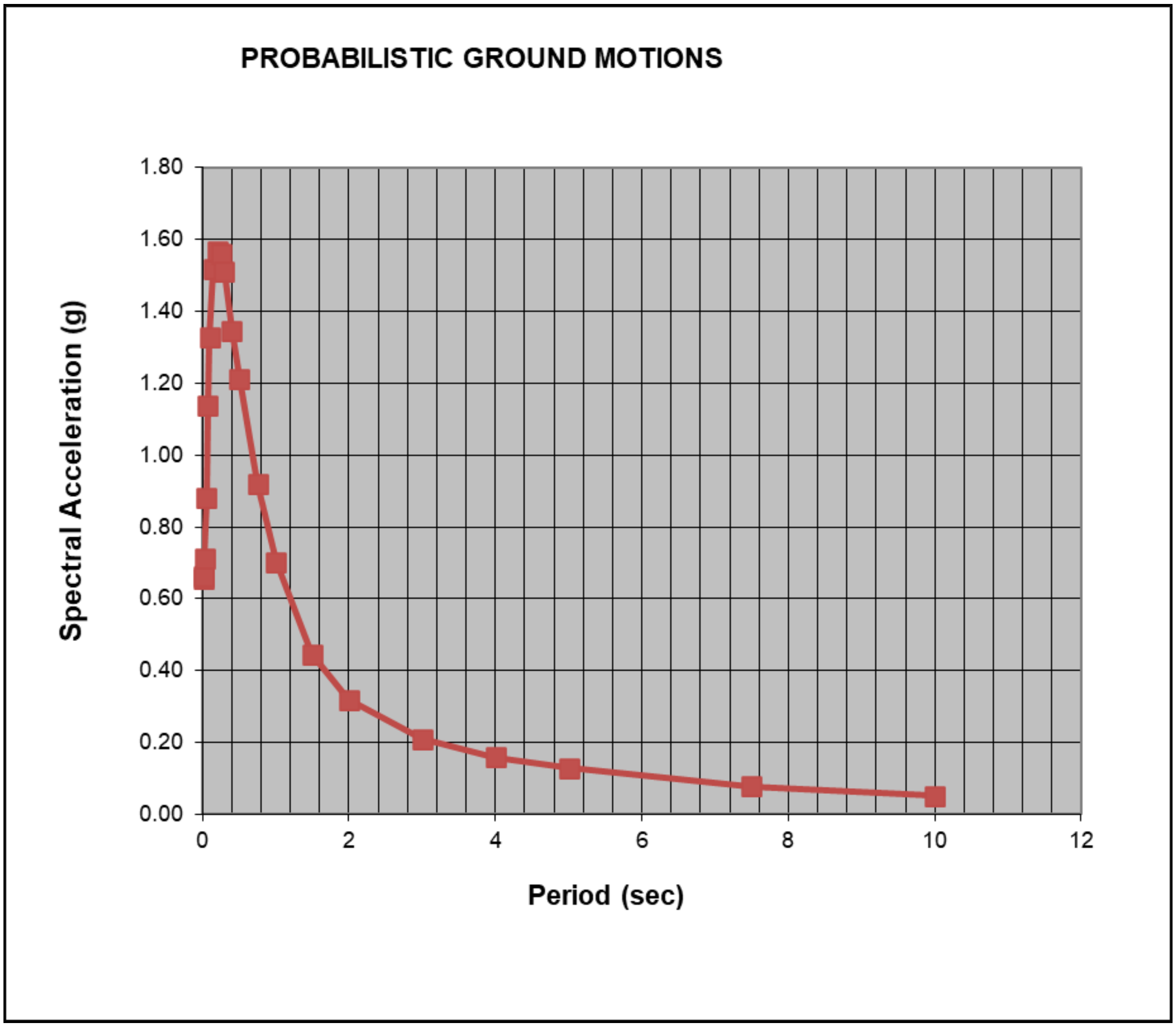
- ***Site Coefficients (CBC 1613.2.3)***-

  Based on CBC Tables 1613.2.3(1) and 1613A.2.3(2), the site coefficient $\mathbf{F_a = 1.2}$ and $\mathbf{F_v = 1.47}$, respectively.

- ***Probabilistic (MCE_R) Ground Motions (ASCE 7 Section 21.2.1.1)***-

  Per Section 21.2.1.1 (**Method 1**), the probabilistic MCE spectral accelerations shall be taken as the spectral response accelerations in the direction of maximum response represented by a five percent damped acceleration response spectrum that is expected to achieve a one percent probability of collapse within a 50-year period.

  The probabilistic analysis included the use of the Open Seismic Hazard Analysis (OpenSHA). The selected Earthquake Rupture Forecast (ERF) was UCERF3 along with a Probability of Exceedance of 2% in 50 Years. The average of four Next Generation Attenuation West-2 Relations (2014 NGA) were utilized to produce a response spectrum. These included Chiou & Youngs (2014), Abrahamsom et al. (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Campbell & Bozorgnia (2014). The Probabilistic Risk Targeted Response Spectrum was determined as the product of the ordinates of the probabilistic response spectrum and the applicable risk coefficient (C_R). These values were then modified to produce a spectrum based upon the maximum rotated components of ground motion. The resulting MCE_R Response Spectrum is indicated below:



- ***Deterministic Spectral Response Analyses (ASCE 7 Section 21.2.2)***-

  The deterministic MCE_R response acceleration at each period shall be calculated as an 84th-percentile 5 percent damped spectral response acceleration in the direction of maximum horizontal response computed at that period. The largest such acceleration calculated for the characteristic earthquakes on all known active faults within the region shall be used. Analyses were conducted using the average of four Next Generation Attenuation West-2 Relations (2014 NGA), including Chiou & Youngs (2014), Abrahamsom et al. (2014), Boore et al. (2014), and Campbell & Bozorgnia (2014).

  Based on our review of the Fault Section Database within the Uniform California Earthquake Rupture Forecast (UCERF 3; Field et al., 2013), discussions with the California Geologic Survey (CGS), and based on the length and maximum magnitude of each of the segments of the Elsinore Fault Zone (which includes the five individual fault segments), the largest moment magnitude (M_W) used for this fault is 7.8, considering a cascading event along the entire fault zone (design fault).

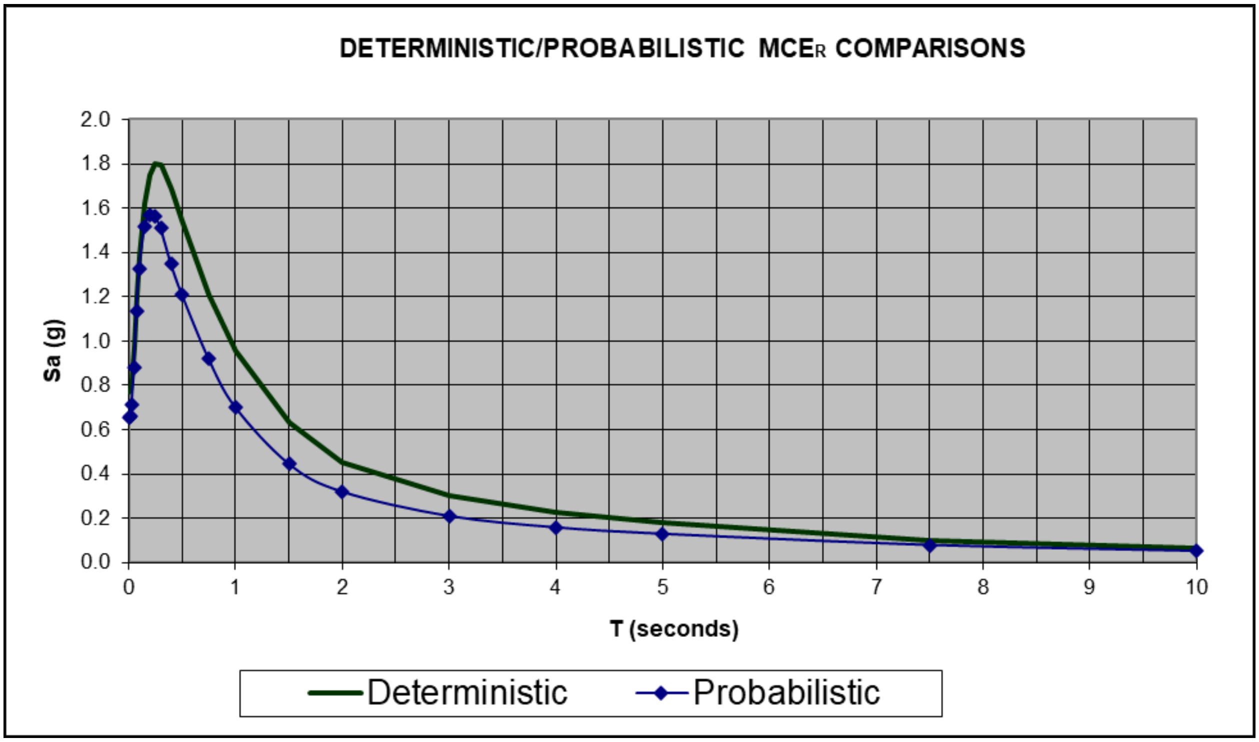
- **Site Specific MCE_R (ASCE 7 Section 21.2.3)**-

  The site-specific MCE_R spectral response acceleration at any period, S_{aM}, shall be taken as the lesser of the spectral response accelerations from the probabilistic ground motions of Section 21.2.1 and the deterministic ground motions of Section 21.2.2. The deterministic ground motions were compared with the probabilistic ground motions that were determined in accordance with Section 21.2.1. These results are tabulated below:

Comparison of Deterministic MCE_R Values with Probabilistic MCE_R Values - Section 21.2.3

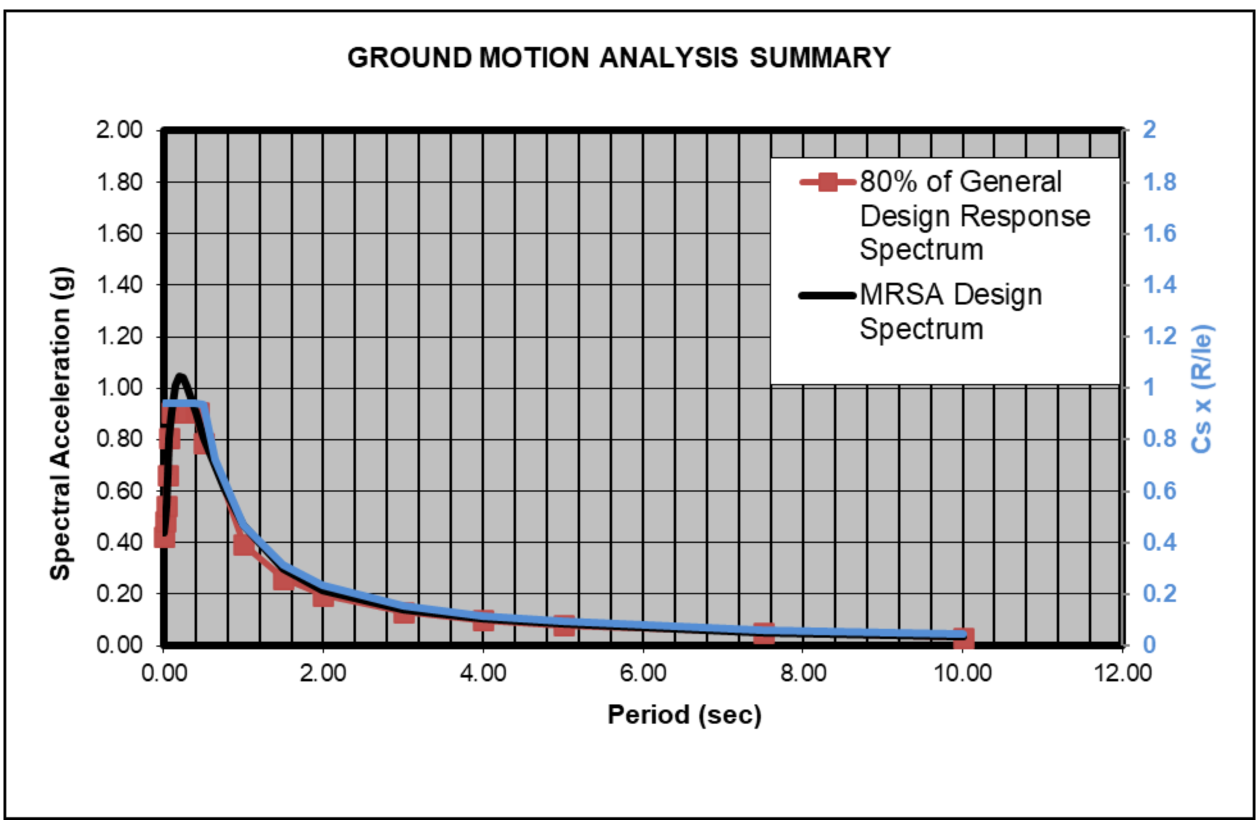
Period T	Deterministic MCE_R	Probabilistic MCE_R	Lower Value (Site Specific MCE_R)	Governing Method
0.010	0.77	0.66	0.66	Probabilistic Governs
0.020	0.78	0.66	0.66	Probabilistic Governs
0.030	0.82	0.71	0.71	Probabilistic Governs
0.050	0.97	0.88	0.88	Probabilistic Governs
0.075	1.20	1.14	1.14	Probabilistic Governs
0.100	1.38	1.33	1.33	Probabilistic Governs
0.150	1.62	1.52	1.52	Probabilistic Governs
0.200	1.75	1.57	1.57	Probabilistic Governs
0.250	1.80	1.56	1.56	Probabilistic Governs
0.300	1.80	1.51	1.51	Probabilistic Governs
0.400	1.68	1.35	1.35	Probabilistic Governs
0.500	1.55	1.21	1.21	Probabilistic Governs
0.750	1.21	0.92	0.92	Probabilistic Governs
1.000	0.95	0.70	0.70	Probabilistic Governs
1.500	0.63	0.45	0.45	Probabilistic Governs
2.000	0.45	0.32	0.32	Probabilistic Governs
3.000	0.30	0.21	0.21	Probabilistic Governs
4.000	0.23	0.16	0.16	Probabilistic Governs
5.000	0.18	0.13	0.13	Probabilistic Governs
7.500	0.10	0.08	0.08	Probabilistic Governs
10.000	0.06	0.05	0.05	Probabilistic Governs

These comparisons are plotted in the following diagram:



- ***Design Response Spectrum (ASCE 7 Section 21.3)***-

In accordance with Section 21.3, the Design Response Spectrum was developed by the following equation: $S_a = 2/3S_{aM}$, where S_{aM} is the MCE_R spectral response acceleration obtained from Section 21.1 or 21.2. The design spectral response acceleration shall not be taken less than 80 percent of S_a. These are plotted and compared with 80% of the CBC Spectrum values in the following diagram:



- **Design Acceleration Parameters (ASCE 7 Section 21.4)**-

  Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall obtained from the site-specific spectra at a period of 0.2 s, except that it shall not be taken less than 90 percent of the peak spectral acceleration, S_a, at any period larger than 0.2 s. The parameter S_{D1} shall be taken as the greater of the products of Sa * T for periods between 1 and 5 seconds. The parameters S_{MS}, and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 80 percent of the values determined in accordance with Section 11.4.4 for S_{MS}, and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1}.

- **Site Specific Design Parameters** -

  For the 0.2 second period (S_{DS}), a value of 0.94g was computed, based upon the average spectral accelerations. The maximum average acceleration for any period exceeding 0.2 seconds was 1.05g occurring at T=0.20 seconds. This was multiplied by 0.9 to produce a value of 0.94g making this the applicable value. A value of 0.47g was calculated for S_{D1} at a period of 1 second (ASCE 7-16, 21.4). For the MCE_R 0.2 second period, a value of 1.412g (S_{MS}) was computed, along with a value of 0.702g (S_{M1}) for the MCE_R 1.0 second period was also calculated (ASCE 7-16, 21.2.3).

- **Site-Specific MCE_G Peak Ground Accelerations (ASCE 7 Section 21.5)**-

  The probabilistic geometric mean peak ground acceleration (2 percent probability of exceedance within a 50-year period) was calculated as 0.64g. The deterministic geometric mean peak ground acceleration (largest 84^{th} percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region) was calculated as 0.70g. The site-specific MCE_G peak ground acceleration was calculated to be **0.64g**, which was determined by using the lesser of the probabilistic (0.64g) or the deterministic (0.70g) geometric mean peak ground accelerations, but not taken as less than 80 percent of PGA_M (i.e., 0.60g x 0.80 = 0.48g).

  .

SEISMIC DESIGN PARAMETERS SUMMARY

Project:	Ethanac and Dawson Roads Project	Lattitude:	33.7378
Project #:	203539-1	Longitude:	-117.1804
Date:	11/24/20		

CALIFORNIA BUILDING CODE CHAPTER 16/ASCE7-16

Mapped Acceleration Parameters per ASCE 7-16, Chapter 22

S_s=	1.416	Figure 22-1
S_1=	0.526	Figure 22-2

Site Class per Table 20.3-1

Site Class=	C - Very Dense Soil and Soft Rock

Site Coefficients per ASCE 7-16 CHAPTER 11

F_a=	1.2	Table 11.4-1	=	1.20	For Site Specific Analysis per ASCE7-16 21.3
F_v=	1.47	Table 11.4-2	=	1.40	For Site Specific Analysis per ASCE7-16 21.3

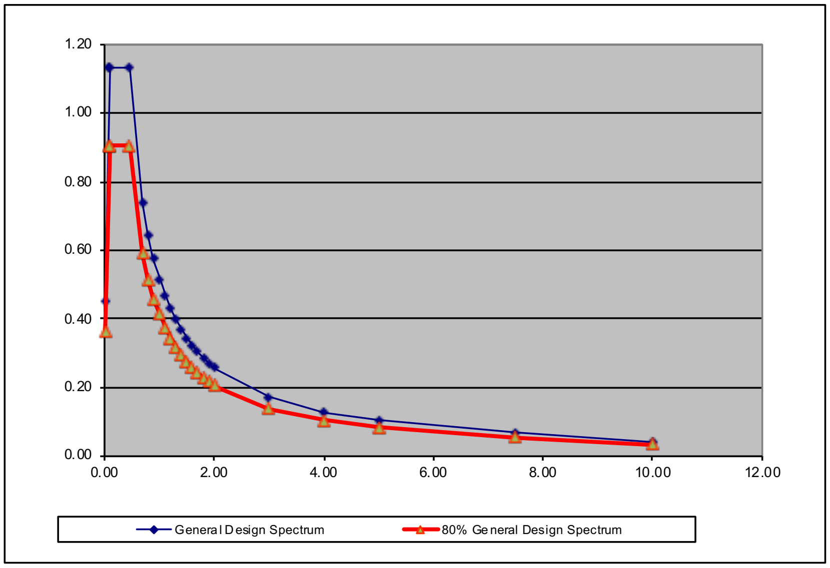
Mapped Design Spectral Response Acceleration Parameters

S_{MS}=	1.6992	Equation 11.4-1	1.6992	For Site Specific Analysis per ASCE7-16 21.3
S_{M1}=	0.775	Equation 11.4-2	0.736	For Site Specific Analysis per ASCE7-16 21.3

T_0=	0.091	sec	
T_S=	0.456	sec	
T_L=	8	sec	From Fig 22-12
PGA	0.5	g	
F_{PGA}=	1.2		From Table 11.8-1
C_{RS}=	0.935		Figure 22-17
C_{R1}=	0.918		Figure 22-18

S_{DS}=	1.133	Equation 11.4-3
S_{D1}=	0.517	Equation 11.4-4

Period (T)	Sa (ASCE7-16 - 11.4.6)	80% General Design Spectrum
0.01	0.45	0.363
0.09	1.13	0.906
0.09	1.13	0.906
0.46	1.13	0.906
0.70	0.74	0.591
0.80	0.65	0.517
0.90	0.57	0.459
1.00	0.52	0.414
1.10	0.47	0.376
1.20	0.43	0.345
1.30	0.40	0.318
1.40	0.37	0.295
1.50	0.34	0.276
1.60	0.32	0.258
1.70	0.30	0.243
1.80	0.29	0.230
1.90	0.27	0.218
2.00	0.26	0.207
3.00	0.17	0.138
4.00	0.13	0.103
5.00	0.10	0.083
7.50	0.07	0.055
10.00	0.04	0.033



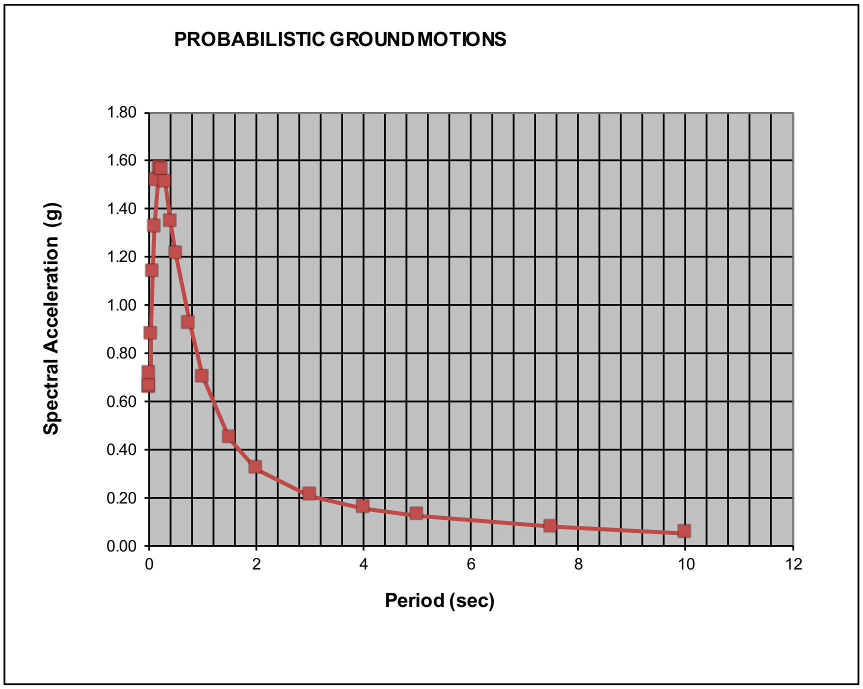
ASCE 7-16 - RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION ANALYSIS

Use Maximum Rotated Horizontal Component?* (Y/N) y

Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014) , Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships
Earthquake Rupture Forecast - UCERF3 Single Branch ERF, Fault Model 3.1

PROBABILISTIC MCER per 21.2.1.1 **Method 1**
Risk Coefficients taken from Figures 22-18 and 22-19 of ASCE 7-16
OpenSHA data
2% Probability Of Exceedance in 50 years
Maximum Rotated Horizontal Component determined per ASCE7-16 Ssection 21.2

T	Sa 2% in 50	MCER
0.01	0.70	0.66
0.02	0.71	0.66
0.03	0.76	0.71
0.05	0.94	0.88
0.08	1.22	1.14
0.10	1.42	1.33
0.15	1.62	1.52
0.20	1.68	1.57
0.25	1.67	1.56
0.30	1.62	1.51
0.40	1.45	1.35
0.50	1.31	1.21
0.75	1.00	0.92
1.00	0.76	0.70
1.50	0.49	0.45
2.00	0.35	0.32
3.00	0.23	0.21
4.00	0.17	0.16
5.00	0.14	0.13
7.50	0.09	0.08
10.00	0.06	0.05



S_s=	1.68	1.57
S_1=	0.76	0.70
PGA	0.64 g	

Risk Coefficients:			
C_{RS}	0.935	Figure 22-18	Get from Mapped Values
C_{R1}	0.918	Figure 22-19	
Fa=	1.2	Table 11.4-1	Per ASCE7-16 - 21.2.3
Is $Sa_{(max)}$<1.2XFa?		NO	If "YES", Probabilistic Spectrum prevails

DETERMINISTIC MCE per 21.2.2

Input Parameters		Elsinore Fault Zone
Fault		
M	= Moment magnitude	7.8
R_{RUP}	= Closest distance to coseismic rupture (km)	15.1
R_{JB}	= Closest distance to surface projection of coseismic rupture (km)	15.1
Rx	= Horizontal distance to top edge of rupture measured perpendicular to strike (km)	15.1
U	= Unspecified Faulting Flag (Boore et.al.)	0
F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust	0
F_{NM}	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique and thrust; 1 for normal and normal-oblique	0
F_{HW}	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08	0
Z_{TOR}	= Depth to top of coseismic rupture (km)	0
δ	= Average dip of rupture plane (degrees)	90
V_{S30}	= Average shear-wave velocity in top 30m of site profile	425.4
$F_{Measured}$		1
$Z_{1.0}$	= Depth to Shear Wave Velocity of 1.0 km/sec (km)	0.05
$Z_{2.5}$	= Depth to Shear Wave Velocity of 2.5 km/sec (km)	0.35
Site Class		C
W (km)	= Fault rupture width (km)	15
F_{AS}	= 0 for mainshock; 1 for aftershock	0
σ	=Standard Deviation	1

Deterministic Summary - Section 21.2.2 (Supplement 1)

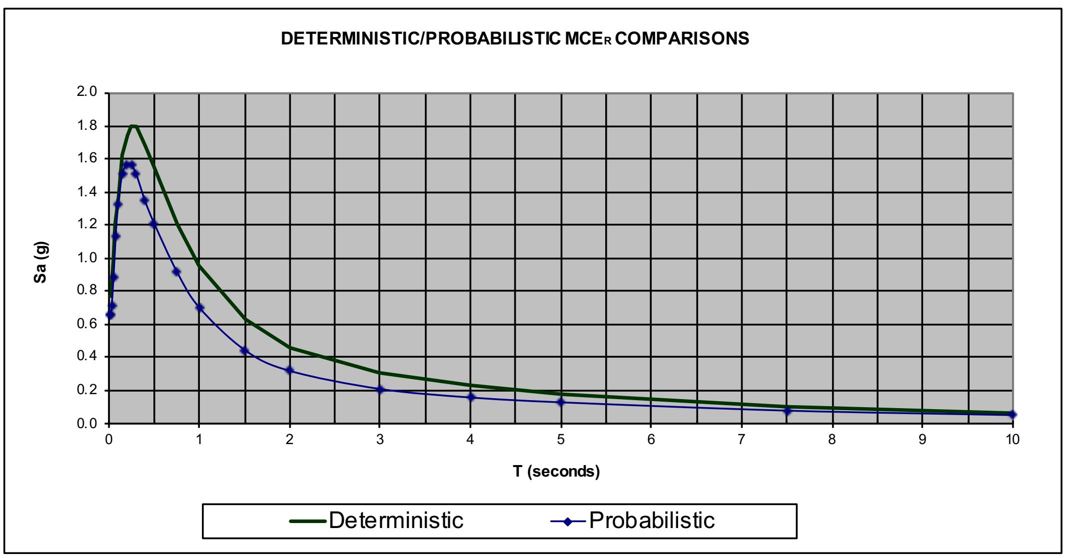
T	Median S_a (Average)	*Corrected* S_a (per ASCE7-16)*	*Scaled $S_{a(Average)}$*
0.010	0.54	0.59	0.77
0.020	0.54	0.60	0.78
0.030	0.58	0.63	0.82
0.050	0.68	0.75	0.97
0.075	0.84	0.93	1.20
0.100	0.97	1.07	1.38
0.150	1.14	1.25	1.62
0.200	1.22	1.35	1.75
0.250	1.25	1.39	1.80
0.300	1.23	1.39	1.80
0.400	1.13	1.30	1.68
0.500	1.02	1.19	1.55
0.750	0.75	0.93	1.21
1.000	0.57	0.74	0.95
1.500	0.37	0.49	0.63
2.000	0.26	0.35	0.45
3.000	0.17	0.23	0.30
4.000	0.12	0.18	0.23
5.000	0.09	0.14	0.18
7.500	0.05	0.08	0.10
10.000	0.03	0.05	0.06
PGA	0.54		0.70 g
Max Sa=	1.39		
Fa =	1.20	Per ASCE7-16 21.2.2	
1.5XFa=	1.8		
Scaling Factor=	1.30		

* Correction is the adjustment for Maximum Rotated Value if Applicable

SITE SPECIFIC MCE_R - Compare Deterministic MCE_R Values (S_a) with Probabilistic MCE_R Values (S_a) per 21.2.3

Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014) , Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships

Period T	Deterministic MCE_R	Probabilistic MCE_R	Lower Value (Site Specific MCE_R)	Governing Method
0.010	0.77	0.66	0.66	Probabilistic Governs
0.020	0.78	0.66	0.66	Probabilistic Governs
0.030	0.82	0.71	0.71	Probabilistic Governs
0.050	0.97	0.88	0.88	Probabilistic Governs
0.075	1.20	1.14	1.14	Probabilistic Governs
0.100	1.38	1.33	1.33	Probabilistic Governs
0.150	1.62	1.52	1.52	Probabilistic Governs
0.200	1.75	1.57	1.57	Probabilistic Governs
0.250	1.80	1.56	1.56	Probabilistic Governs
0.300	1.80	1.51	1.51	Probabilistic Governs
0.400	1.68	1.35	1.35	Probabilistic Governs
0.500	1.55	1.21	1.21	Probabilistic Governs
0.750	1.21	0.92	0.92	Probabilistic Governs
1.000	0.95	0.70	0.70	Probabilistic Governs
1.500	0.63	0.45	0.45	Probabilistic Governs
2.000	0.45	0.32	0.32	Probabilistic Governs
3.000	0.30	0.21	0.21	Probabilistic Governs
4.000	0.23	0.16	0.16	Probabilistic Governs
5.000	0.18	0.13	0.13	Probabilistic Governs
7.500	0.10	0.08	0.08	Probabilistic Governs
10.000	0.06	0.05	0.05	Probabilistic Governs



DESIGN RESPONSE SPECTRUM per Section 21.3

DESIGN ACCELERATION PARAMETERS per Section 21.4 (MRSA)

Period	2/3*MCE_R	80% General Design Response Spectrum (per ASCE 7-16 Figure 11.4-1)	Design Response Spectrum	TXSa
0.01	0.44	0.42	0.44	
0.02	0.44	0.48	0.48	
0.03	0.47	0.54	0.54	
0.05	0.59	0.66	0.66	
0.08	0.76	0.81	0.81	
0.10	0.89	0.91	0.91	
0.15	1.01	0.91	1.01	
0.20	1.05	0.91	1.05	
0.25	1.04	0.91	1.04	
0.30	1.01	0.91	1.01	
0.40	0.90	0.91	0.91	
0.50	0.81	0.79	0.81	
0.75	0.61	0.52	0.61	
1.00	0.47	0.39	0.47	0.47
1.50	0.30	0.26	0.30	0.45
2.00	0.21	0.20	0.21	0.43
3.00	0.14	0.13	0.14	0.42
4.00	0.11	0.10	0.11	0.42
5.00	0.09	0.08	0.09	0.43
7.50	0.05	0.05	0.05	
10.00	0.03	0.03	0.03	

Highest value of S_a for any period exceeding 0.2 sec.= 1.05

90%of Highest Value = 0.94

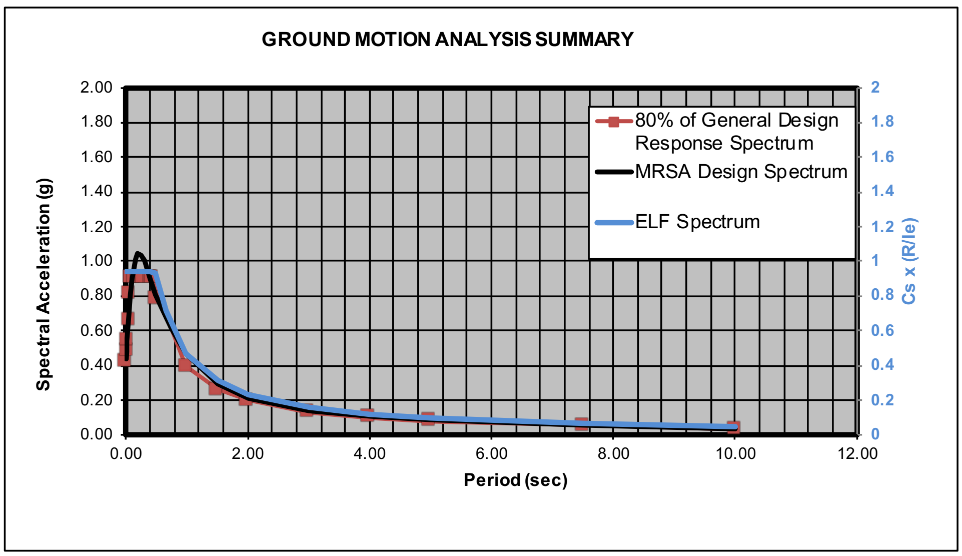
Max Tsa from T=1s-2s = 0.47

S_{DS}=	0.94
S_{D1}=	0.47
T_s =	0.50

S_{MS}=	1.412
S_{M1}=	0.702

PGA Determination:

Site Coefficient F_{PGA}=	1.2	
Mapped PGA=	0.50	Figure 22-7
PGA_M =	0.60	g
Deterministic PGA =	0.70	g
Probabilistic PGA =	0.64	g
Lesser of Deterministic/Probabilistic =	0.64	g
80% of PGA_{M}=	0.48	g
MCE_G PGA=	0.64	g



APPENDIX C

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REFERENCES

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APPENDIX 9.6.2

PALEONTOLOGICAL OVERVIEW

January 5, 2022

Kari Cano
Kimley-Horn
3880 Lemon Street, Suite 420
Riverside, California 92501

Subject: Paleontological Overview for Plot Plan No. 2019-005, Menifee, Riverside County, California (BCR Consulting Project No. KIM2114)

Dear Kari:

BCR Consulting LLC (BCR Consulting) is pleased to present the following Paleontological Overview for Plot Plan No. 2019-005 in the City of Menifee (City), Riverside County, California. CEQA provides guidance relative to significant impacts on paleontological resources, indicating that a project would have a significant impact on paleontological resources if it disturbs or destroys a unique paleontological resource or site or unique geologic feature. Section 5097.5 of the California Public Resources Code specifies that any unauthorized removal of paleontological remains is a misdemeanor. Further, California Penal Code Section 622.5 sets the penalties for damage or removal of paleontological resources. CEQA documentation prepared for projects would be required to analyze paleontological resources as a condition of the CEQA process to disclose potential impacts. A paleontological overview completed by professional paleontologists from the Western Science Center is summarized below and provided as Attachment A.

> The geologic units underlying the project area are mapped entirely as alluvial fan deposits dating to the late to middle Pleistocene (Morton, Bovard & Morton, 2003). Pleistocene alluvial units are considered to be of high paleontological sensitivity, and while the Western Science Center does not have localities within the project area, we do have numerous localities in similarly mapped units throughout the region. Riverside County Pleistocene sediments are well documented to contain abundant fossil material including those associated with mastodon (Mammut pacificus), mammoth (Mammuthus columbi), ancient horse (Equus sp.), camel (Camelops hesternus), sabertooth cat (Smilodon fatalis) and many more.
>
> Any fossil specimens recovered from the Menifee Commerce Center Project would be scientifically significant. Excavation activity associated with the development of the project area would impact the paleontologically sensitive Pleistocene units, and it is the recommendation of the Western Science Center that a paleontological resource mitigation program be put in place to monitor, salvage, and curate any recovered fossils from the study area.

Based on these results, the following mitigation recommendations have been developed. Prior to issuance of grading permits, the applicant should retain a qualified paleontologist to create and implement a Paleontological Resource Mitigation Program (PRIMP). The project paleontologist would review the grading plan and conduct any pre-construction work necessary to render appropriate monitoring and mitigation requirements, to be documented in the PRIMP. The PRIMP would be submitted to the City prior to issuance of a grading permit. Information contained in the PRIMP would minimally include:

1. Description of the project site and proposed grading operations
2. Description of the level of monitoring required for earth-moving activities

3. Identification and qualifications of the paleontological monitor to be employed during earth moving
4. Identification of personnel with authority to temporarily halt or divert grading to allow recovery of large specimens
5. Direction for fossil discoveries to be reported to the developer and the City
6. Means and methods to be employed by the paleontological monitor to quickly salvage fossils to minimize construction delays
7. Sampling methods for sediments that are likely to contain small fossil remains, if any.
8. Procedures and protocol for collecting and processing of samples and specimens, as necessary
9. Fossil identification and curation procedures
10. Identification of the repository to receive fossil material
11. All pertinent maps and exhibits
12. Procedures for reporting of findings
13. Acknowledgement of the developer for content of the PRIMP and acceptance of financial responsibility for monitoring, reporting, and curation.

If you have any questions or comments regarding this paleontological overview, please contact me at 909-525-7078 or david.brunzell@yahoo.com.

Sincerely,

BCR Consulting LLC

David Brunzell, M.A./RPA
Principal Investigator/Archaeologist

Attachment A: Paleontological Overview

WESTERN SCIENCE CENTER

BCR Consulting LLC
David Brunzell
505 West 8th Street
Claremont, CA 91711

November 9, 2021

Dear Mr. Brunzell,

This letter presents the results of a record search conducted for the Menifee Commerce Center Project in the city of Menifee, Riverside County, California. The project site is located on north of McLaughlin Road, south of California Highway 74, east of Barnett Road and west of Palomar Road in Section 9, 10, 11 and 15 of Township 5 South, and Range 3 West, on the *Romoland, CA* USGS 7.5-minute quadrangle.

The geologic units underlying the project area are mapped entirely as alluvial fan deposits dating to the late to middle Pleistocene (Morton, Bovard & Morton, 2003). Pleistocene alluvial units are considered to be of high paleontological sensitivity, and while the Western Science Center does not have localities within the project area, we do have numerous localities in similarly mapped units throughout the region. Riverside County Pleistocene sediments are well documented to contain abundant fossil material including those associated with mastodon (*Mammut pacificus*), mammoth (*Mammuthus columbi*), ancient horse (*Equus sp.*), camel (*Camelops hesternus*), sabertooth cat (*Smilodon fatalis)* and many more.

Any fossil specimens recovered from the Menifee Commerce Center Project would be scientifically significant. Excavation activity associated with the development of the project area would impact the paleontologically sensitive Pleistocene units, and it is the recommendation of the Western Science Center that a paleontological resource mitigation program be put in place to monitor, salvage, and curate any recovered fossils from the study area.

If you have any questions, or would like further information, please feel free to contact me at dradford@westerncentermuseum.org

Sincerely,

Darla Radford
Collections Manager

APPENDIX 9.7

GREENHOUSE GAS EMISSIONS REPORT