

APPENDIX F1

GEOTECHNICAL INVESTIGATION

**UPDATE GEOTECHNICAL
INVESTIGATION
HERITAGE VALLEY BUSINESS CENTER -
BUILDING 2**

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



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

August 4, 2025

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**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Attention: Mr. Jon Kelly
Vice President Development

Project No.: **25G144-1R**

Subject: **Update Geotechnical Investigation**
Heritage Valley Business Center – Building 2
Trumble Road, South of Ethanac Road
Menifee, California

Mr. Kelly:

In accordance with your request, this report provided the results of the geotechnical investigation performed 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.
Project Engineer

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Principal Engineer

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.

Geotechnical Design Considerations

- Artificial fill soils were encountered at some of the boring locations, extending to depths of 2½ to 4½± feet. The existing fill soils are considered to represent undocumented fill.
- In the present condition, the near-surface alluvial soils within the upper 3 to 4± feet possess a moderate potential for settlement when exposed to load increases in the range of those that will be exerted by the new foundations.
- The undocumented fill soils and the upper-portion of the alluvial soils generally possess varying strengths and unfavorable consolidation/collapse characteristics. These soils, in their present condition, are not considered suitable for support of the foundation loads of the new structure.
- Remedial grading will be necessary which includes overexcavation and recompaction of the undocumented fill soils and the upper portion of the alluvial soils.

Site Preparation

- Initial site preparation should include removal of all vegetation, including tree root masses and any organic topsoil.
- Remedial grading is recommended within the proposed building pad area to remove the undocumented fill soils, which extend to depths of 2½ to 4½± feet at the boring locations, in their entirety. At a minimum, the building pad area should be overexcavated to a depth of at least 4 feet below existing grade and to a depth of at least 3 feet below proposed pad grade, whichever is greater. Overexcavation within the foundation areas is recommended to extend to a depth of at least 3 feet below proposed foundation bearing grade.
- After overexcavation has been completed, the subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated. The resulting subgrade should then be scarified to a depth of 12 inches, moisture conditioned or air dried to 2 to 4 percent above optimum, and recompact to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.
- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, moisture conditioned and recompact to at least 90 percent of the ASTM D-1557 maximum dry density.

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,500 lbs/ft² maximum allowable soil bearing pressure.
- Reinforcement consisting of at least four (4) No. 5 rebars (2 top and 2 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade: minimum 6 inches thick.

- Modulus of Subgrade Reaction: $k = 100$ psi/in.
- Reinforcement is not expected to be necessary for geotechnical considerations. The actual thickness and reinforcement of the floor slab should be determined by the structural engineer.

Pavement Design Recommendations

ASPHALT PAVEMENTS (R=30)					
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	6	8	10	11	13
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R=30)				
Materials	Thickness (inches)			
	Autos and Light Truck Traffic (TI = 5.0 to 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 25P135, dated May 1, 2025. The scope of services included review of the previous subsurface explorations, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. 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 overall is the Heritage Valley Business Center, located on the east side of Trumble Road, approximately 600 feet south of Ethanac Road in Menifee, California. The overall site is bounded to the north by a vacant field, to the east by Sherman Road, to the west by Trumble Road, and to the south by a concrete-lined channel. The general location of the site is illustrated on the Site Location Map, included as Plate 1 in Appendix A of this report.

The overall site is comprised of several contiguous parcels, totaling $41.2\pm$ acres in size. The majority of the site is vacant and undeveloped. Ground surface cover consist of moderate to heavy native grass and weed growth. There are three (3) ranch-style lots are located in the western region of the site, along Trumble Road. Two of these lots appear to developed as nurseries. The third lot appears to be utilized for commercial use, with trailer storage throughout the majority of the lot. Several single-story structures, ranging between 600 to $3,000\pm$ ft² in size, are located within these residential lots. The existing structures are of wood-frame and steel-frame construction, presumably supported on conventional shallow foundations, with concrete slab-of-grade floors. Ground surface cover within these lots consists of exposed soil with areas of aggregate base. There are several medium to large trees and heavy vegetation within, and surrounding, the residential lots. The parcels north and south of the residential lots are vacant and undeveloped. Ground surface cover within these lots consists of exposed soils with moderate to heavy native grass and weed growth. Several soil stockpiles, measuring less than $5\pm$ in height are located in the south-central region of the site. Along the western region of the site, adjacent to Sherman Road, the northern-most parcel is approximately 11.3 acres in size and is occupied by an aggregate (sand and gravel) supply company. The facility includes a single-story residence, located in the northeast region of the site. Ground surface cover throughout this lot consists of exposed soil with areas of aggregate base.

Topographic information was not available at the time of this report. The site topography generally slopes downward to the southwest at a gradient of less than 1 percent. There is approximately $5\pm$ feet of elevation differential across the site.

3.2 Proposed Development

Based on the site plan provided by the client, the site will be developed as part of the Heritage Valley Business Center, and will include one building, identified as Building 2, approximately $829,755\pm$ ft² in size. The new building will be located in the central region of the site. Loading docks will be constructed along portions of the east and west building walls. The new building will be surrounded by Portland cement concrete pavements in the truck court areas, with asphaltic concrete pavements in the automobile parking and drive areas. Concrete flatwork and landscaped planters will be utilized throughout the site.

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

No significant amounts of below grade construction, such as crawl spaces or new basements, are expected to be included in the proposed development. Based on the existing topography, cuts and fills of 2 to 4± feet are expected to be necessary to achieve the proposed site grades.

3.3 Previous Studies

Southern California Geotechnical, Inc. (SCG) previously conducted three (3) separate geotechnical investigations within the subject site. The previous reports are identified as follows:

Geotechnical Investigation, Two Commercial/Industrial Buildings, Sherman Road, South of Ethanac Road, Perris, California, prepared by SCG for Core5 Industrial Partners, SCG Project No. 20G227-1, dated December 4, 2020.

Geotechnical Investigation, Proposed Menifee Commerce Center – Phase III, Sherman Road, North of McLaughlin Road, Perris, California, prepared by SCG for Core5 Industrial Partners, SCG Project No. 22G173-1, dated June 3, 2022.

Geotechnical Investigation, Proposed Menifee Commerce Center – Phase IV, Trumble Road, North of McLaughlin Road, Perris, California, prepared by SCG for Core5 Industrial Partners, SCG Project No. 22G206-1, dated July 27, 2022.

The previous subsurface exploration conducted for the overall project consisted of seventeen (17) borings advanced to depths of 15 to 25± feet below the existing site grades. The locations of the previous borings are illustrated on Plate 2 of this report. In addition, the borings logs and relevant laboratory data are included in Appendixes B and C, respectively

Artificial fill soils were encountered at the ground surface at some of the boring locations, extending to depths of 2½ to 4½ ± feet below ground surface. The fill soils generally consisted of medium dense to very dense silty fine sands, clayey fine sands, and fine sandy silts, with varying medium to coarse sand and clay content. Older alluvium was encountered beneath the artificial fill soils or at the ground surface at all of the boring locations, extending to at least the maximum depth explored of 25± feet below ground surface. The older alluvium generally consisted of medium dense to very dense silty fine to coarse sands, clayey fine to coarse sands, and very stiff to hard silty clays, fine sandy clays, and clayey silts with varying medium to coarse sand content.

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 at the time of the subsurface exploration.

4.0 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. The 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 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

Representative bulk samples were tested for their maximum dry densities and optimum moisture contents. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented in Appendix G of this report. This test is 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 (EI)

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:

<u>Sample Identification</u>	<u>Expansion Index</u>	<u>Expansive Potential</u>
B-1 @ 0 to 5 feet (SCG Project No. 22G206-1)	50	Low
B-4 @ 0 to 5 feet (SCG Project No. 22G173-1)	36	Low
B-12 @ 0 to 5 feet (SCG Project No. 20G227-1)	40	Low

Soluble Sulfates

Representative samples of the near-surface soil obtained from the previous geotechnical investigations 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 performed in the previous study and the current investigation are presented below, and are discussed further in a subsequent section of this report.

<u>Sample Identification</u>	<u>Soluble Sulfates (%)</u>	<u>ACI Classification</u>
B-1 @ 0 to 5 feet (SCG Project No. 22G206-1)	0.010	Not Applicable (S0)
B-3 @ 0 to 5 feet (SCG Project No. 22G206-1)	0.011	Not Applicable (S0)
B-4 @ 0 to 5 feet (SCG Project No. 22G173-1)	0.034	Not Applicable (S0)
B-5 @ 0 to 5 feet (SCG Project No. 22G173-1)	0.027	Not Applicable (S0)
B-10 @ 0 to 5 feet (SCG Project No. 20G227-1)	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, chloride, and nitrate concentrations of the soils, as well as other tests. The results of some of these tests are presented below.

<u>Sample Identification</u>	<u>Saturated Resistivity (ohm-cm)</u>	<u>pH</u>	<u>Chlorides (mg/kg)</u>	<u>Nitrates (mg/kg)</u>
B-1 @ 0 to 5 feet (SCG Project No. 22G206-1)	3,015	8.8	38.1	3.4
B-3 @ 0 to 5 feet (SCG Project No. 22G206-1)	3,417	8.6	48.5	1.8
B-4 @ 0 to 5 feet (SCG Project No. 22G173-1)	1,005	7.8	142.4	220.6
B-5 @ 0 to 5 feet (SCG Project No. 22G173-1)	1,072	8.6	96.3	160.3

<u>Sample Identification</u>	<u>Saturated Resistivity (ohm- cm)</u>	<u>pH</u>	<u>Chlorides (mg/kg)</u>	<u>Nitrates (mg/kg)</u>
B-10 @ 0 to 5 feet (SCG Project No. 20G227-1)	1,880	7.5	17	50

5.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

5.1 Seismic Design Considerations

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

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. In addition, our review of the Riverside County RCIT GIS website indicates that the site is not located within a Riverside County 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.

The potential for other geologic hazards such as seismically induced settlement, lateral spreading, tsunamis, inundation, seiches, flooding, and subsidence affecting the site is considered low.

Seismic Design Parameters

The 2022 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. In general, the seismic design parameters for small, relatively rigid structures have not changed with the 2022 CBC. Section 1613.1 of the 2022 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 in accordance with Section 21.2." Therefore, a site-specific ground motion hazard analysis 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 V_{100} 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 the report prepared by Terra Geosciences, included in Appendix E of the referenced report. Based on the shear-wave survey performed by Terra Geosciences, the V_{100} 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 ground motion hazard analysis are presented in the report prepared by Terra Geosciences, in Appendix E of the referenced report. Seismic design parameters computed during this study are tabulated below.

SITE-SPECIFIC SEISMIC DESIGN PARAMETERS BASED ON ASCE 7-16 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

Liquefaction

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 subsurface conditions encountered at the boring locations are not considered to be conducive to liquefaction. These conditions consist of moderate to high strength older native alluvial soils and no evidence of a long-term groundwater table within 25 feet of the ground surface. In addition, research of available well data indicates that the groundwater depths in the area of the site are more than 62 feet below grade. Based on these considerations, liquefaction is not considered to be a design concern for this project.

5.2 Geotechnical Design Considerations

General

Some of the borings encountered artificial fill materials, extending to depths of 2½ to 4½± feet. Based on their strength characteristics and a lack of documentation regarding the placement and compaction of the existing fill materials, these soils are considered to consist of undocumented fill. In addition, the near-surface alluvial soils within the upper 3 to 4± feet possess a potential for moderate settlement when exposed to load increases in the range of those that will be exerted by the new foundations. The undocumented fill soils and the near-surface native alluvium are not considered to be suitable for the support of the foundation and floor slab loads of the proposed structure. Based on these conditions, remedial grading is considered warranted within the proposed building area to completely remove the existing artificial fill soils and the upper portion of the near-surface native alluvium and replace these 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 indicate that these materials possess a low expansion potential ($EI = 36, 40$ and 50). Based on the presence of potentially expansive soils at this site, care should be given to proper moisture conditioning

the building pad subgrade soils to a moisture content of 2 to 4 percent above the ASTM D-1557 optimum during site grading. It is recommended that additional expansion index testing be conducted at the completion of rough grading to verify the expansion potential of the as-graded building pad.

Soluble Sulfates

The results of the soluble sulfate testing indicate that the selected samples of the on-site soils contain sulfate concentrations that correspond to Class S0 with respect to the American Concrete Institute (ACI) Publication 318-05 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 the tested samples of the on-site soils possess saturated resistivities ranging from of 1,005 to 3,417 ohm-cm, and pH values ranging from 7.5 to 8.8. 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. Resistivity and pH are two of the five factors that enter into the evaluation procedure. Redox potential, relative soil moisture content and sulfides are also included. Although sulfide testing was not part of the scope of services for this project, 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 moderately corrosive to ductile iron pipe. Therefore, polyethylene protection is expected to be required for cast iron or ductile iron pipes.** It should be noted that SCG does not practice in the field of corrosion engineering.

Relatively low concentrations (17 to 142.4 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 samples, 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 present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested samples possess nitrate concentrations ranging from 1.8 to 220.6 mg/kg. **Based on these test results, 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 shrinkage estimate is based on the results of dry density testing performed on small-diameter samples of the existing soils taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1 feet. This estimate may be used for grading in areas that are underlain by native alluvial soils.

These estimates are based on previous experience and the subsurface conditions encountered at the boring locations. The actual amount of subsidence is expected to be variable and will be 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

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

5.3 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 heavy 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 1-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 Pad

In order to support the foundation loads of the new structure, remedial grading shall be performed within the proposed building pad area. Remedial grading is the overexcavation and recompaction of the existing undocumented fill soils and a portion of the near-surface native alluvium. Remedial grading should be performed within the proposed building pad area to overexcavate the existing undocumented fill soils and a portion of the near-surface native alluvium. The undocumented fill soils extend to depths of 2½ to 4½± feet at the boring locations within the building area. The soils within the proposed building pad area should also be overexcavated to a depth of 4 feet below existing grade and to a depth of at least 3 feet below proposed building pad subgrade elevation. The proposed foundation influence zones within the industrial building should be overexcavated to a depth of at least 3 feet below proposed foundation bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building and foundation perimeters, and to an extent equal to the depth of fill placed below the foundation bearing grade, whichever is greater. If the proposed structure incorporates any exterior columns (such as for a canopy or overhang) the area of overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the building area should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if fill materials or loose, porous, 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 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 area may then be raised to grade with previously excavated soils or imported structural fill.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining walls and site walls should be overexcavated to a depth of 3 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pad. Any undocumented fill soils or disturbed native alluvium within any of these foundation areas should be removed in their entirety. The overexcavation areas should extend at least 3 feet beyond the foundation perimeters, and to an extent equal to the depth of fill below the new foundations. Any erection pads for tilt-up concrete walls are considered to be part of the foundation system. Therefore, these overexcavation recommendations are applicable to erection pads. The overexcavation

subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning to within 2 to 4 percent above the optimum moisture content, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

If the full lateral recommended remedial grading cannot be completed for the proposed retaining walls and site walls located along property lines, the foundations for those walls should be designed using a reduced allowable bearing pressure. Furthermore, the contractor should take necessary precautions to protect the adjacent improvements during rough grading. Specialized grading techniques, such as A-B-C slot cuts, will likely be required during remedial grading. The geotechnical engineer of record should be contacted if additional recommendations, such as shoring design recommendations, are required during grading.

Treatment of Existing Soils: Parking and Drive Areas

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

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

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

Treatment of Existing Soils: Flatwork Areas

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

As noted previously, the subject site is underlain by low expansive soils. Support of new flatwork on low expansive soils carries a minor risk with respect to flatwork movement and potential distress. This report provides recommendations for moisture conditioning and additional steel reinforcement in the flatwork areas in order to minimize the potential effects of the expansive soils. However, if additional protection is desired, the client should consider the placement of a 1 to 2-foot thick layer of non-expansive soil beneath all flatwork.

Fill Placement

- Fill soils should be placed in thin ($6\pm$ inches), near-horizontal lifts, moisture conditioned (or air dried) 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 2022 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 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 (horizontal to vertical) 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.

Any soils used to backfill voids around subsurface utility structures, such as manholes or vaults, should be placed as compacted structural fill. If it is not practical to place compacted fill in these areas, then such void spaces may be backfilled with lean concrete slurry. Uncompacted pea gravel

or sand is not recommended for backfilling these voids since these materials have a potential to settle and thereby cause distress of pavements placed around these subterranean structures.

5.4 Construction Considerations

Excavation Considerations

The near surface soils generally consist of moderate strength silty to clayey fine sands and fine sandy silts with minor clay content. These upper 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 2h:1v for sands and 1.5h:1v for moderate strength clays, or the Cal-OSHA excavation guidelines for the type of soil encountered. Deeper excavations may require some form of external stabilization such as shoring or bracing or flattened/stepped excavations. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. Excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Expansive Soils

The near-surface soils within the subject site have been determined to possess a low expansion potential. Therefore, care should be given to proper moisture conditioning of all 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 ($EI < 50$) characteristics. **In addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintain the 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.**

Moisture Sensitive Subgrade Soils

Some 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.

If the construction schedule dictates that site grading will occur during a period of wet weather, allowances should be made for costs and delays associated with drying the on-site soils or import of a drier, less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pad area.

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.

5.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by structural fill soils used to replace any undocumented fill soils and a portion of the near-surface alluvial soils. These new structural fill soils are expected to extend to a depth of at least 3 feet below proposed foundation bearing grade, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. 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: 2,500 lbs/ft².
- Maximum, net allowable soil bearing pressure: 1,500 lbs/ft² if the full recommended lateral extent of remedial grading cannot be achieved.
- 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 low expansive and potentially liquefiable soils.
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations. 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 to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill or suitable native alluvium (where reduced bearing pressures are utilized), with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

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

Estimated Foundation Settlements

Post-construction total and differential static settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively, under static conditions. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch.

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/ft³
- Friction Coefficient: 0.30

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill soils. The maximum allowable passive pressure is 3,000 lbs/ft².

5.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, and based on the design considerations presented in Section 6.1 of this report, the floor of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, extending to a depth of at least 3 feet below finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.
- Modulus of Subgrade Reaction: 100 psi/in.

- Minimum slab reinforcement: Not required for geotechnical considerations. 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 a 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.

5.7 Exterior Flatwork Design and Construction

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

- Minimum slab thickness: 4½ inches.
- Minimum slab reinforcement: No. 3 bars at 18 inches on center, in both directions.
- The flatwork at building entry areas should be structurally connected to the perimeter foundation that is recommended to span across the door opening. This recommendation is designed to reduce the potential for differential movement at this joint.

- Moisture condition the slab subgrade soils to at least 2 to 4 percent of optimum moisture content, to a depth of at least 12 inches. Adequate moisture conditioning should be verified by the geotechnical engineer 24 hours prior to concrete placement.
- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.
- Control joints should be provided at a maximum spacing of 8 feet on center in two directions for slabs and at 6 feet on center for sidewalks. Control joints are intended to direct cracking. Minor cracking of exterior concrete slabs on grade should be expected.

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

5.8 Retaining Wall Design and Construction

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

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. The following parameters assume that only the on-site soils will be utilized for retaining wall backfill. The near-surface soils vary in composition and include silty fine sands, fine sandy silts, clayey sands and sandy clays. Based on their composition, the on-site soils are expected to possess a friction angle of 30 degrees when compacted to at least 90 percent of the ASTM D-1557 maximum dry density. It is recommended that the clayey soils be excluded from use as retaining wall backfill.

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, Sandy Silts, Clayey Sands
Internal Friction Angle (ϕ)		29°
Unit Weight		133 lbs/ft ³
Equivalent Fluid Pressure:	46 lbs/ft ³	43 lbs/ft ³
	77 lbs/ft ³	70 lbs/ft ³
	69 lbs/ft ³	65 lbs/ft ³

The walls should be designed using a soil-footing coefficient of friction of 0.30 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 2022 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 underlain by at least 3 feet of newly placed structural fill. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

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

It is recommended that a minimum 1-foot thick layer of free-draining granular material (less than 5 percent passing the No. 200 sieve) be placed against the face of the retaining walls. This material should extend from the top of the retaining wall footing to within 1 foot of the ground

surface on the back side of the retaining wall. This material should be approved by the geotechnical engineer. In lieu of the 1-foot thick layer of free-draining material, a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls, may be used. If the layer of free-draining material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The layer of free draining granular material should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557-91). 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 2-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 10-foot on-center spacing. Alternatively, 4-inch diameter holes at an approximate 20-foot on-center spacing can be used for this type of drainage system. In addition, 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. The actual design of this type of system should be determined by the civil engineer to verify that the drainage system possesses the adequate capacity and slope for its intended use.

Weep holes or a footing drain will not be required for building stem walls.

5.9 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface soils generally consist of silty sands, clayey sands, and sandy clays. Based on their classification, these materials are expected to possess fair pavement support characteristics with estimated R-values in the range of 30 to 40. Since R-value testing was outside the scope of services for this project, the subsequent pavement design is therefore based upon an assumed R-value of 30. 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=30)					
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	6	8	10	11	13
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=30)				
Materials	Thickness (inches)			
	Autos and Light Truck Traffic (TI = 5.0 to 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

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

6.0 GENERAL COMMENTS

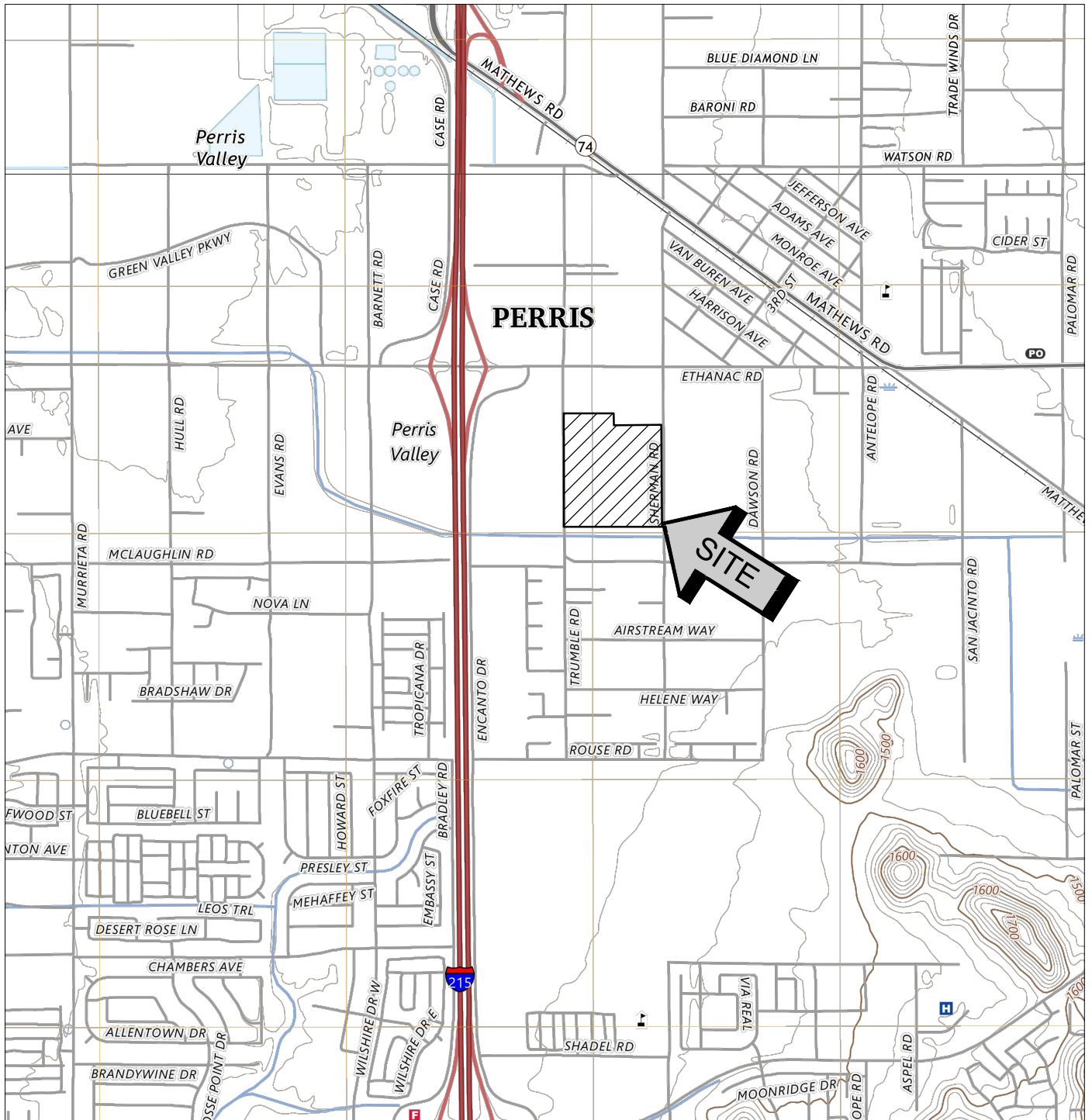
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other 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 MAPS OF THE
ROMOLAND QUADRANGLES, RIVERSIDE COUNTY,
CALIFORNIA, 2021 AND 2023.



SITE LOCATION PLAN

HERITAGE VALLEY BUSINESS PARK - BUILDING 2

MENIFEE, CALIFORNIA

SCALE: 1" = 2000'

DRAWN: PM

CHKD: RGT

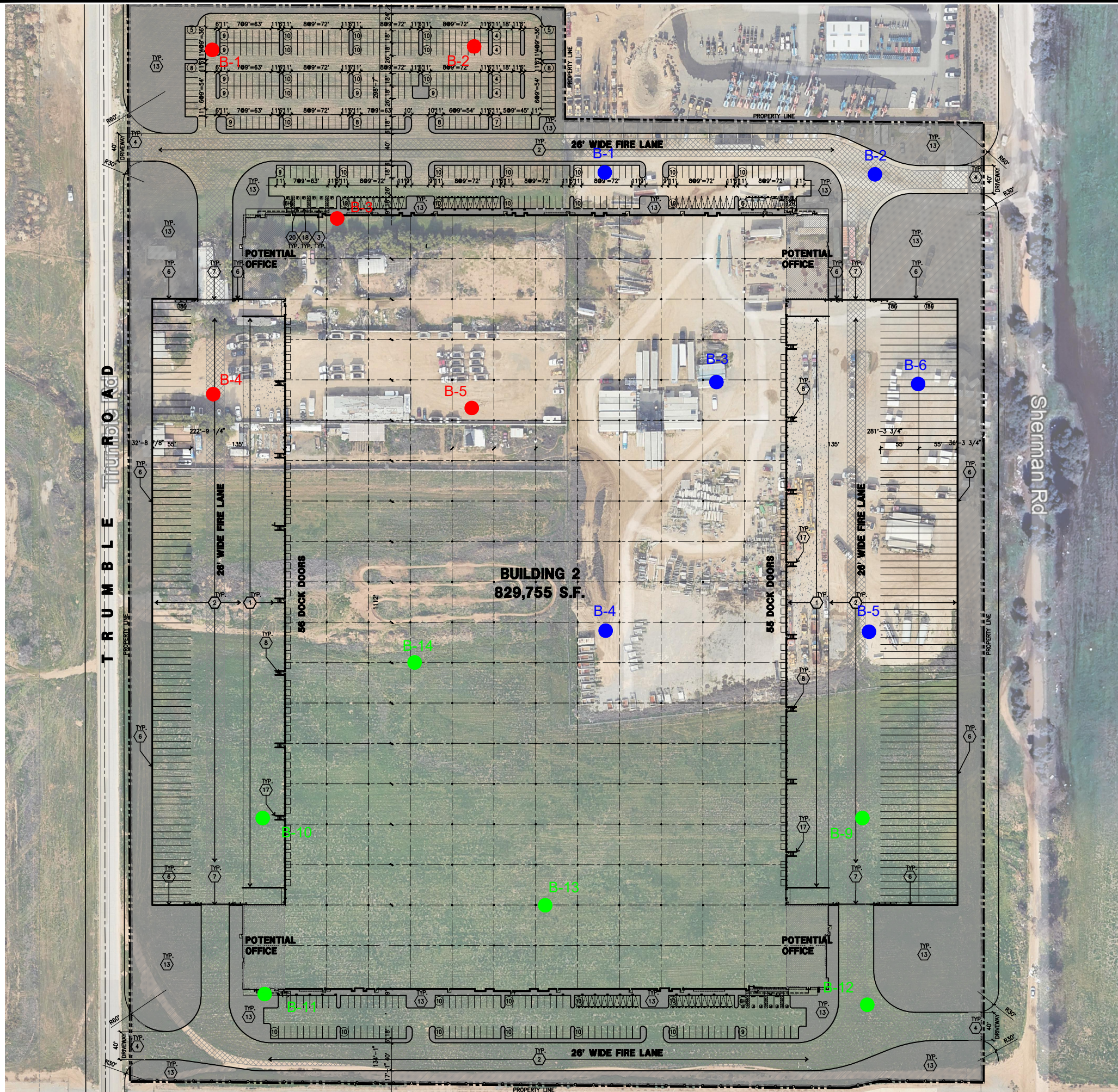
SCG PROJECT

25G144-1

PLATE 1



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**



GEOTECHNICAL LEGEND

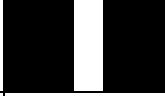
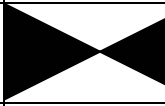
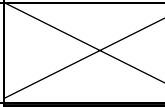
- PREVIOUS BORING LOCATION (SCG PROJECT NO. 20G227-1)
- PREVIOUS BORING LOCATION (SCG PROJECT NO. 22G173-1)
- PREVIOUS BORING LOCATION (SCG PROJECT NO. 22G206-1)

NOTE: BASE SITE PLAN PROVIDED BY THE CLIENT.

BORING LOCATION PLAN	
HERITAGE VALLEY BUSINESS CENTER - BUILDING 2	
MENIFEE, CALIFORNIA	
SCALE: 1" = 150'	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: PM	
CHKD: RGT	
SCG PROJECT 25G144-1	
PLATE 2	

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/ft³.

MOISTURE CONTENT:

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

LIQUID LIMIT:

The moisture content above which a soil behaves as a liquid.

PLASTIC LIMIT:

The moisture content above which a soil behaves as a plastic.

PASSING #200 SIEVE:

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

UNCONFINED SHEAR:

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

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS



JOB NO.: 20G227-1	DRILLING DATE: 11/12/20	WATER DEPTH: Dry
PROJECT: Two Proposed CI Bldgs	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 14 feet
LOCATION: Menifee, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

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

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



JOB NO.: 20G227-1	DRILLING DATE: 11/12/20	WATER DEPTH: Dry
PROJECT: Two Proposed CI Bldgs	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 16 feet
LOCATION: Menifee, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
5 												

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



JOB NO.: 20G227-1	DRILLING DATE: 11/12/20	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				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS	
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)		
					SURFACE ELEVATION: --- MSL								
5		26			<u>ALLUVIUM</u> : Brown fine Sandy Clay, slightly porous, trace fine root fibers, very stiff-damp		5						
		72			<u>OLDER ALLUVIUM</u> : Brown Clayey fine Sand, trace medium Sand, little Silt, very dense-damp to moist		8						
		88			Red Brown Silty Clay, little fine to medium Sand, hard-moist		10						
		55					10						
10					Brown to Gray Brown Clayey fine to coarse Sand, very dense-moist to very moist		9						
15		74											
													
20		72					11						
					Boring Terminated at 20'								

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



JOB NO.: 20G227-1	DRILLING DATE: 11/12/20	WATER DEPTH: Dry
PROJECT: Two Proposed CI Bldgs	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 16 feet
LOCATION: Menifee, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
5 <												

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



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				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
5 <												

TBL_20G227-1.GPJ_SOCALGEO.GDT 12/4/20



JOB NO.: 20G227-1	DRILLING DATE: 11/12/20	WATER DEPTH: Dry
PROJECT: Two Proposed CI Bldgs	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 8 feet
LOCATION: Menifee, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					SURFACE ELEVATION: --- MSL							
5 <												

TBL_20G227-1.GPJ_SOCALGEO.GDT 12/4/20



JOB NO.: 22G173-1	DRILLING DATE: 4/27/22	WATER DEPTH: Dry
PROJECT: Menifee Commerce Center Phase III	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 19 feet
LOCATION: Menifee, California	LOGGED BY: Joey Hernandez	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
5	X	87/10"			OLDER ALLUVIUM: Light Orange Brown Silty fine Sand, little medium to coarse Sand, trace Calcareous nodules, very dense-damp to moist		12					
	X	85				7						
	X	37			Light Brown Clayey fine to coarse Sand, little Silt, little Calcareous nodules, dense to very dense-moist		10					
	X	87				12						
	X	34			Light Brown fine to coarse Sand, little Clay, trace Silt, little Calcareous nodules, dense-moist		10					
15	X											
	X	49		Light Brown fine to coarse Sand, little Silt, dense-moist		10						
20	X											
	X	36				8						
25	X											
Boring Terminated at 25'												

TBL 22G173-1.GPJ SOCALGEO.GDT 6/1/22



JOB NO.: 22G173-1	DRILLING DATE: 4/27/22	WATER DEPTH: Dry
PROJECT: Menifee Commerce Center Phase III	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 15 feet
LOCATION: Menifee, California	LOGGED BY: Joey Hernandez	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
		70			FILL: Dark Brown Clayey fine to coarse Sand, little Silt, dense-moist	121	10					
		70			OLDER ALLUVIUM: Brown Clayey fine Sand, little Silt, trace medium to coarse Sand, slightly porous, trace Iron oxide staining, dense to very dense-damp to moist	121	9					
5		50/5"					118	7				
		77			Light Brown fine Sandy Silt, trace to little Clay, dense to very dense-moist	118	11					
10		69				124	9					
		33			Light Brown fine to coarse Sand, little Clay, little Calcareous nodules, dense to very dense-moist		9					
15												
		61					10					
20												
Boring Terminated at 20'												

TBL 22G173-1.GPJ, SOCALGEO.GDT 6/1/22



JOB NO.: 22G173-1				DRILLING DATE: 4/27/22				WATER DEPTH: Dry				
PROJECT: Menifee Commerce Center Phase III				DRILLING METHOD: Hollow Stem Auger				CAVE DEPTH: 9 feet				
LOCATION: Menifee, California				LOGGED BY: Joey Hernandez				READING TAKEN: At Completion				
FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
5		43	4.5		FILL: Dark Brown Clayey Silt, little fine to coarse Sand, very stiff-damp	121	7					
		23	4.5		OLDER ALLUVIUM: Dark Brown fine Sandy Clay, little Silt, trace Calcareous nodules, very stiff-very moist	119	14					
		40			Brown fine Sandy Silt, trace Clay, trace medium Sand, medium dense-moist	115	10					
		60			Brown Clayey fine Sand, little Silt, dense-moist	121	9					
		50/5"			Light Brown fine Sandy Silt, trace Clay, very dense-moist	120	9					
		50/5"			Brown Silty fine to coarse Sand, little fine to coarse Gravel, very dense-moist	120	10					
15					Boring Terminated at 15'							

TBL 22G173-1.GPJ SOCALGEO.GDT 6/1/22



JOB NO.: 22G173-1	DRILLING DATE: 4/27/22	WATER DEPTH: Dry
PROJECT: Menifee Commerce Center Phase III	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 13 feet
LOCATION: Menifee, California	LOGGED BY: Joey Hernandez	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
5		37	89/9"		OLDER ALLUVIUM: Dark Brown Clayey fine to medium Sand, little Silt, little fine to coarse Gravel, little Calcareous nodules, medium dense-damp	124	10				EI = 36 @ 0-5'	
		70			Light Brown Silty fine Sand, trace medium to coarse Sand, trace Iron oxide staining, trace Calcareous nodules, dense-moist	113	9					
					Light Brown fine Sandy Silt, trace to little Clay, very dense-moist to very moist	121	9					
		77				125	11					
		82				124	13					
10												
15		54					11					
		50		Light Brown Clayey fine to medium Sand, very dense-moist		10						
20					Boring Terminated at 20'							

TBL 22G173-1.GPJ SOCALGEO.GDT 6/1/22



JOB NO.: 22G173-1	DRILLING DATE: 4/27/22	WATER DEPTH: Dry
PROJECT: Menifee Commerce Center Phase III	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 21 feet
LOCATION: Menifee, California	LOGGED BY: Joey Hernandez	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
5 												

TBL 22G173-1.GPJ, SOCALGEO.GDT 6/1/22



JOB NO.: 22G173-1	DRILLING DATE: 4/27/22	WATER DEPTH: Dry
PROJECT: Menifee Commerce Center Phase III	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Menifee, California	LOGGED BY: Joey Hernandez	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
5		17			FILL: Gray Brown Silty fine to coarse Sand, little fine to coarse Gravel, medium dense-damp		4					
		29	4.5		OLDER ALLUVIUM: Brown fine Sandy Clay, little Silt, very stiff-very moist		14					
		33			Brown Silty fine Sand, trace Clay, dense-moist		10					
		50			Brown Clayey fine to coarse Sand, little Silt, little Calcareous nodules, very dense-moist		8					
		42	4.5		Gray Brown fine to medium Sandy Clay, trace coarse Sand, hard-very moist		16					
15		54			Gray Brown Clayey fine to coarse Sand, little Silt, very dense-moist		10					
20					Boring Terminated at 20'							

TBL 22G173-1.GPJ SOCALGEO.GDT 6/1/22



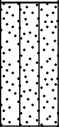
JOB NO.: 22G206-1	DRILLING DATE: 6/23/22	WATER DEPTH: Dry
PROJECT: Menifee Commerce Center Phase IV	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 21 feet
LOCATION: Menifee, California	LOGGED BY: Joey Hernandez	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
5		70/10"	4.5		OLDER ALLUVIUM: Dark Brown Silty Clay, trace fine to medium Sand, hard-damp to moist	125	10				EI = 50 @ 0-5'	
		87/11"			Red Brown Clayey fine Sand, trace Silt, trace medium to coarse Sand, slightly cemented, dense to very dense-moist	125	9					
		50				126	8					
		75			Red Brown Silty fine Sand, trace to little medium to coarse Sand, little Iron Oxide staining, very dense-moist	130	10					
		50/5"				120	9					
		43			Light Gray Clayey fine to medium Sand, little Silt, trace coarse Sand, dense-moist		11					
20		42		Light Brown fine to coarse Sand, little Clay, trace Silt, dense-moist		9						
		44				9						
25					Boring Terminated at 25'							

TBL 22G206-1.GPJ SoCalGeo.GDT 7/27/22



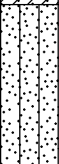
JOB NO.: 22G206-1	DRILLING DATE: 6/23/22	WATER DEPTH: Dry
PROJECT: Menifee Commerce Center Phase IV	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Menifee, California	LOGGED BY: Joey Hernandez	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
5		26			FILL: Light Brown Silty fine Sand, trace medium to coarse Sand, medium dense-damp		4					
		20			OLDER ALLUVIUM: Light Red Brown fine to coarse Sand, little Clay, trace Silt, highly weathered, medium dense-damp to moist		7					
		25					8					
		22			Dark Gray Brown Clayey fine to coarse Sand, little Silt, slightly weathered, medium dense-moist		8					
		26					10					
15												
20		37			@ 18½ feet, dense		11					
	Boring Terminated at 20'											

TBL 22G206-1.GPJ SOCALGEO.GDT 7/27/22



JOB NO.: 22G206-1	DRILLING DATE: 6/23/22	WATER DEPTH: Dry
PROJECT: Menifee Commerce Center Phase IV	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 12 feet
LOCATION: Menifee, California	LOGGED BY: Joey Hernandez	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
5		40	4.5		OLDER ALLUVIUM: Brown Clayey fine Sand, trace medium to coarse Sand, trace fine root fibers, slightly porous, medium dense-damp	114	5					
		50/5"			Gray Brown Silty fine Sand, trace medium Sand, slightly laminated, very dense-damp to moist	116	7					
		50/5"				118	8					
		50/5"				120	9					
		67				122	12					
10												
15		52			Gray Brown fine to coarse Sandy Clay, slightly weathered, hard-very moist	118	13					
					Boring Terminated at 15'							

TBL 22G206-1.GPJ SOCALGEO.GDT 7/27/22



JOB NO.: 22G206-1	DRILLING DATE: 6/23/22	WATER DEPTH: Dry
PROJECT: Menifee Commerce Center Phase IV	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Menifee, California	LOGGED BY: Joey Hernandez	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: --- MSL												
5 <												

TBL 22G206-1.GPJ SOCALGEO.GDT 7/27/22

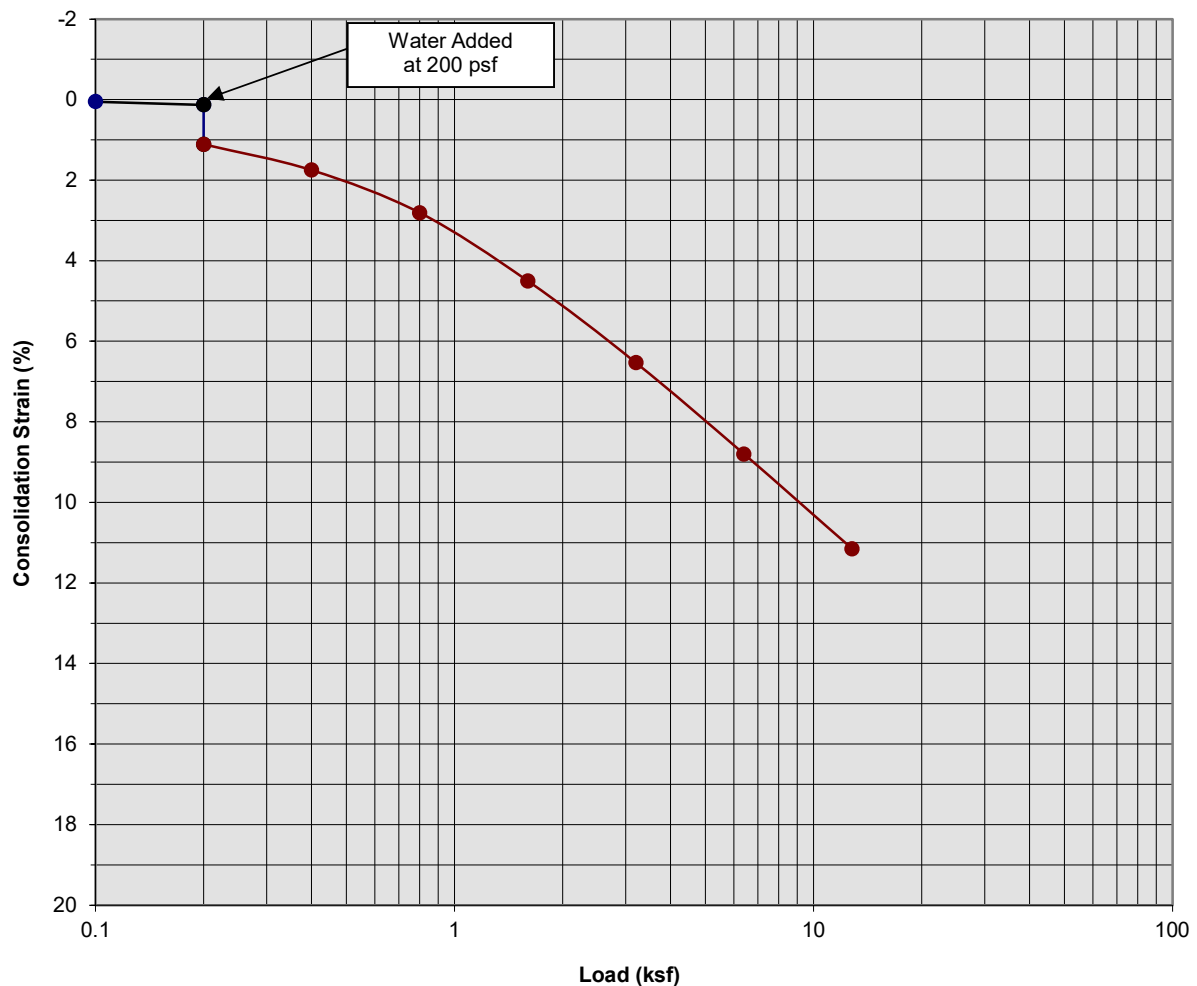


JOB NO.: 22G206-1		DRILLING DATE: 6/23/22		WATER DEPTH: Dry								
PROJECT: Menifee Commerce Center Phase IV		DRILLING METHOD: Hollow Stem Auger		CAVE DEPTH: 21 feet								
LOCATION: Menifee, California		LOGGED BY: Joey Hernandez		READING TAKEN: At Completion								
FIELD RESULTS				LABORATORY RESULTS								
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
SURFACE ELEVATION: --- MSL												
					FILL: Brown fine Sandy Silt, trace medium Sand, trace Clay, paper and plastic debris, loose to medium dense-damp	98	6					
						104	6					
5					OLDER ALLUVIUM: Brown fine Sandy Clay, little Silt, trace medium Sand, very stiff-moist	120	11					
					Red Brown Clayey fine Sand, dense-moist	127	11					
10					Brown Silty Clay, trace fine to medium Sand, very stiff-moist	123	10					
					Brown Clayey fine to coarse Sand, trace Silt, moderately weathered, slightly cemented, dense to very dense-moist		9					
15							10					
20												
25							10					
Boring Terminated at 25'												

TBL 22G206-1.GPJ SOCALGEO.GDT 7/27/22

APPENDIX

Consolidation/Collapse Test Results



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

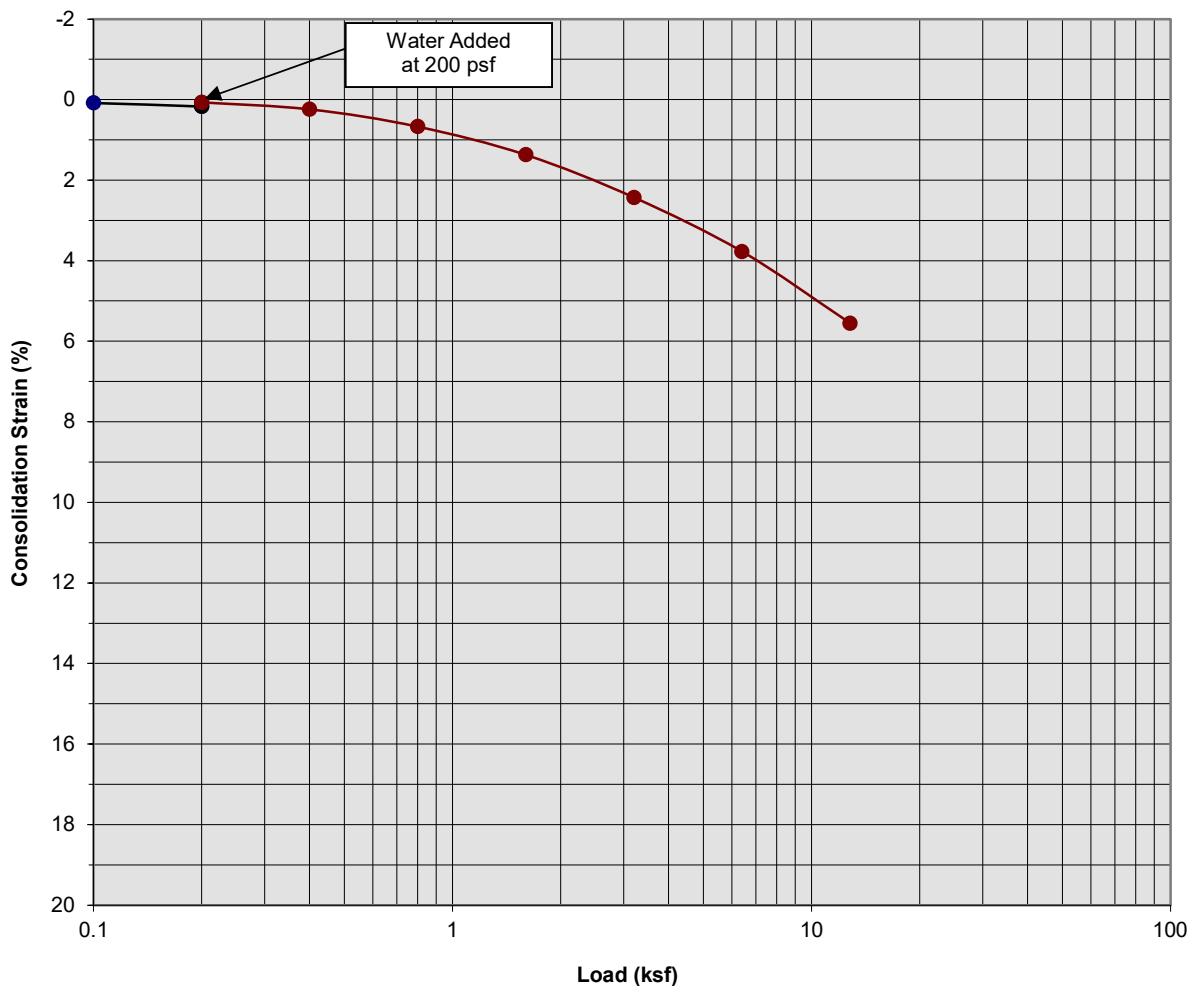
Boring Number:	B-10	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	11
Depth (ft)	1 to 2	Initial Dry Density (pcf)	124.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	139.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.98

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



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



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

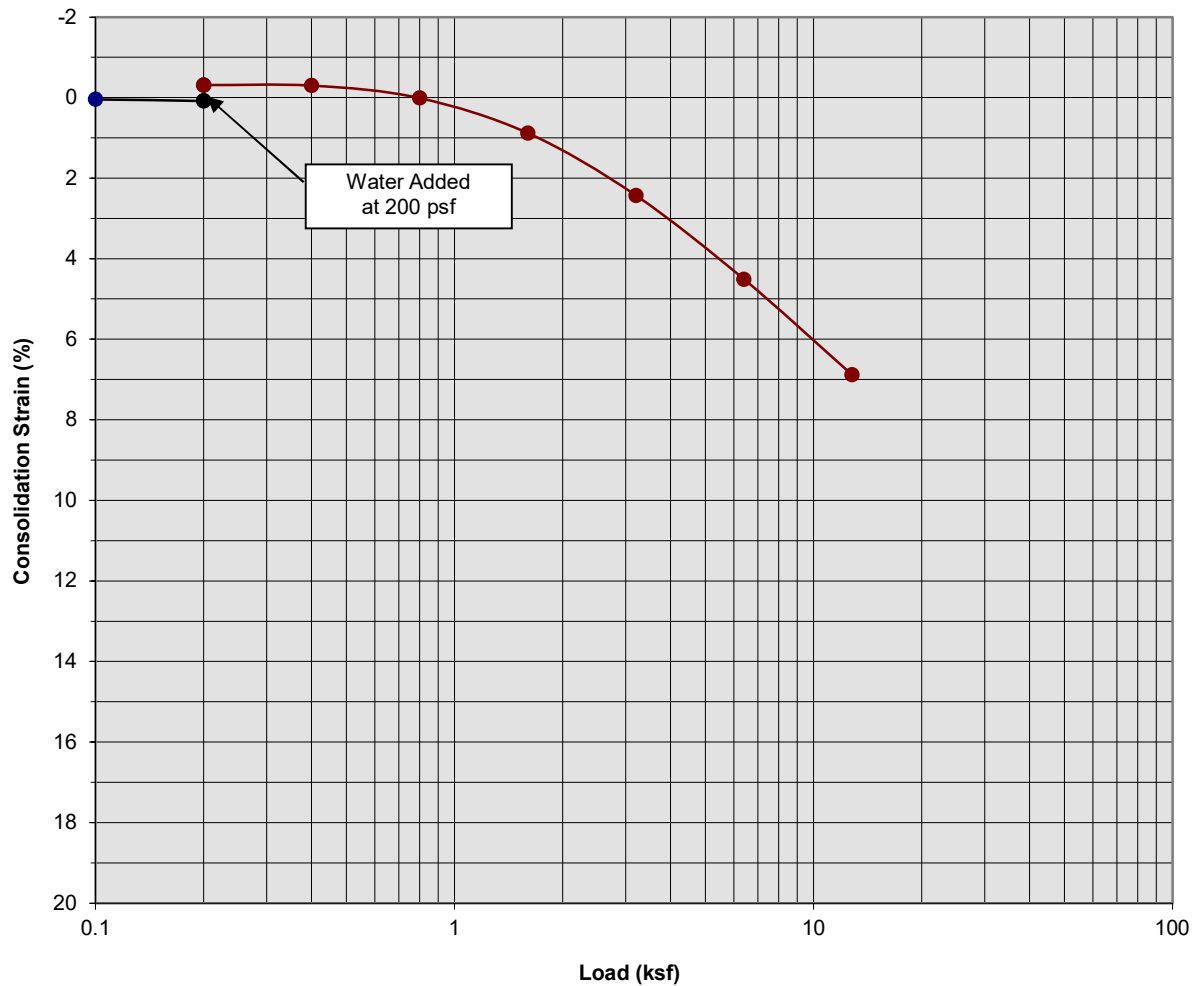
Boring Number:	B-10	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	3 to 4	Initial Dry Density (pcf)	127.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	134.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.10

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



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown Silty Clay

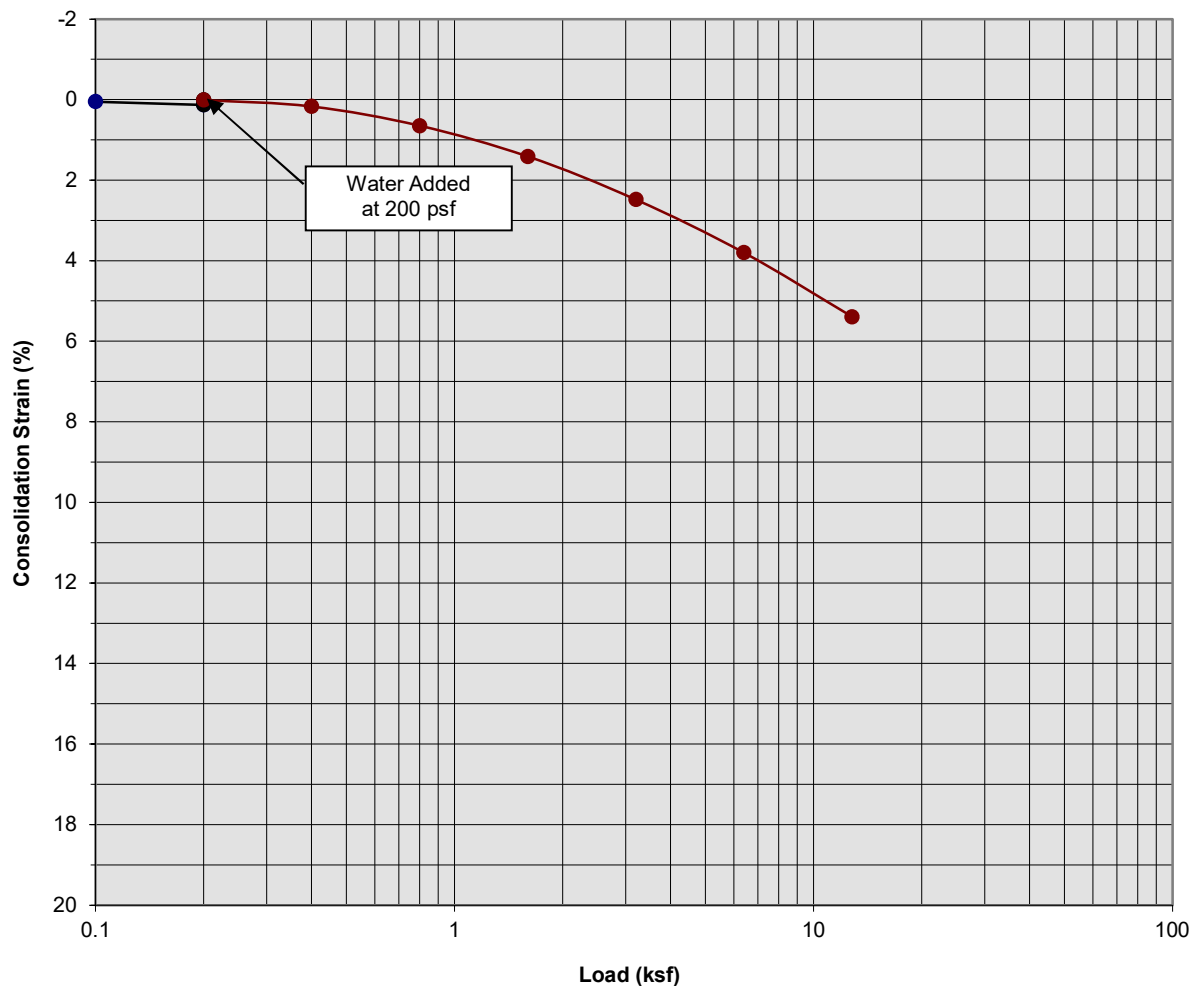
Boring Number:	B-12	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	1 to 2	Initial Dry Density (pcf)	118.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	126.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.39

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



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown Silty fine Sand to fine Sandy Silt

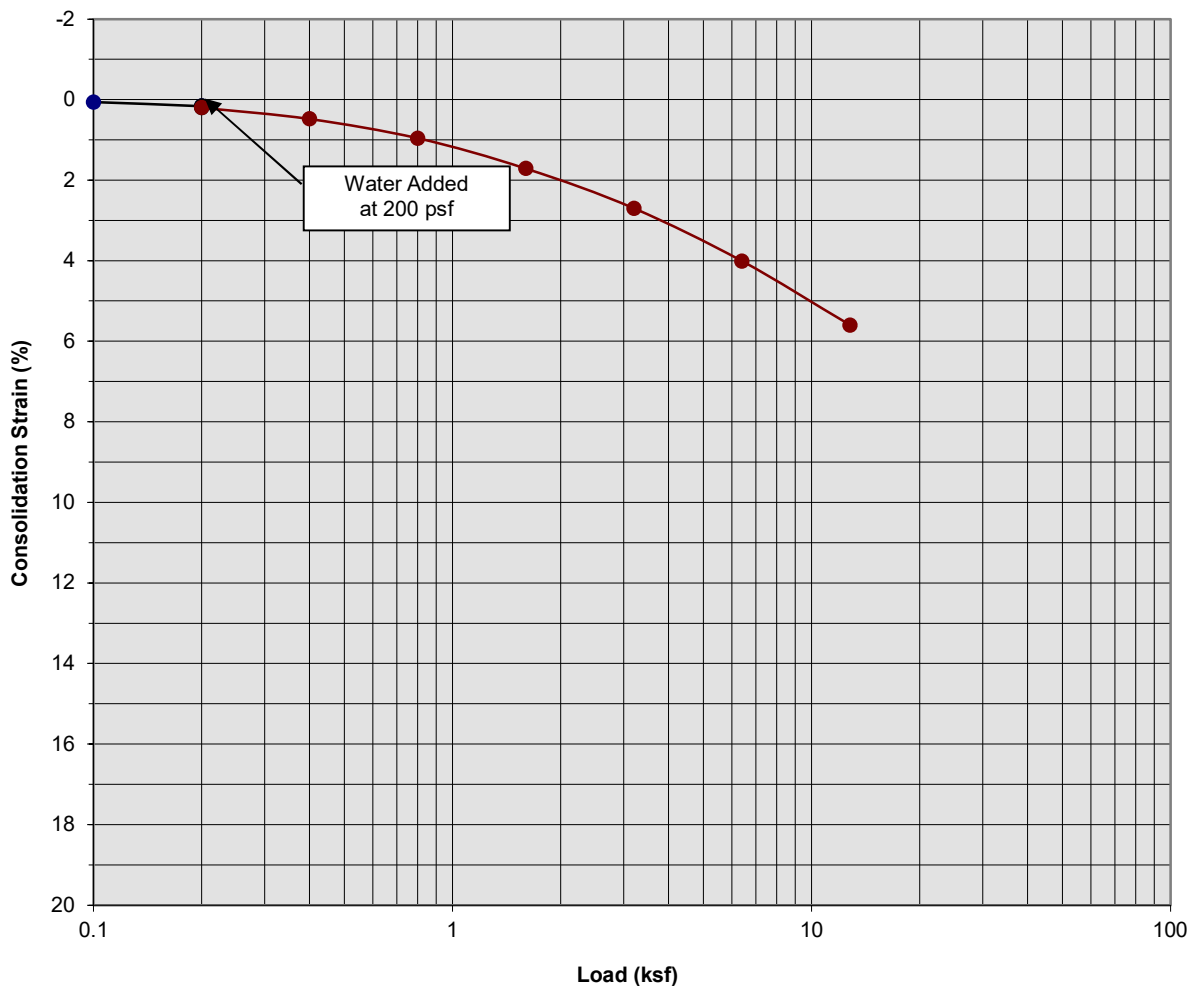
Boring Number:	B-12	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	5 to 6	Initial Dry Density (pcf)	120.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	126.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.12

Two Proposed Commercial/Industrial Buildings
Menifee, California
Project No. 20G227
PLATE C-5



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



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

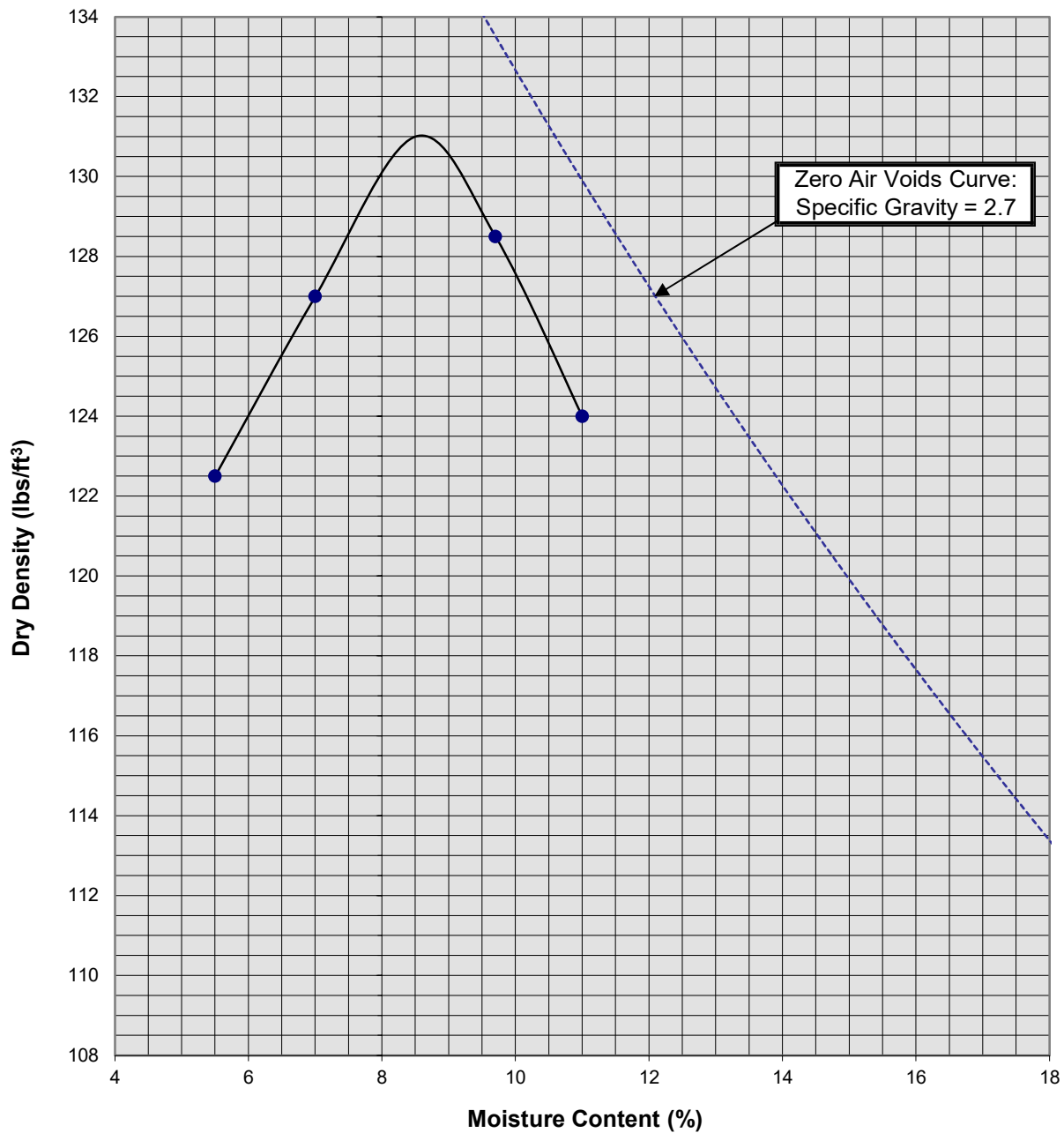
Boring Number:	B-12	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	7 to 8	Initial Dry Density (pcf)	124.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	131.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.04

Two Proposed Commercial/Industrial Buildings
Menifee, California
Project No. 20G227
PLATE C-6



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Moisture/Density Relationship ASTM D-1557



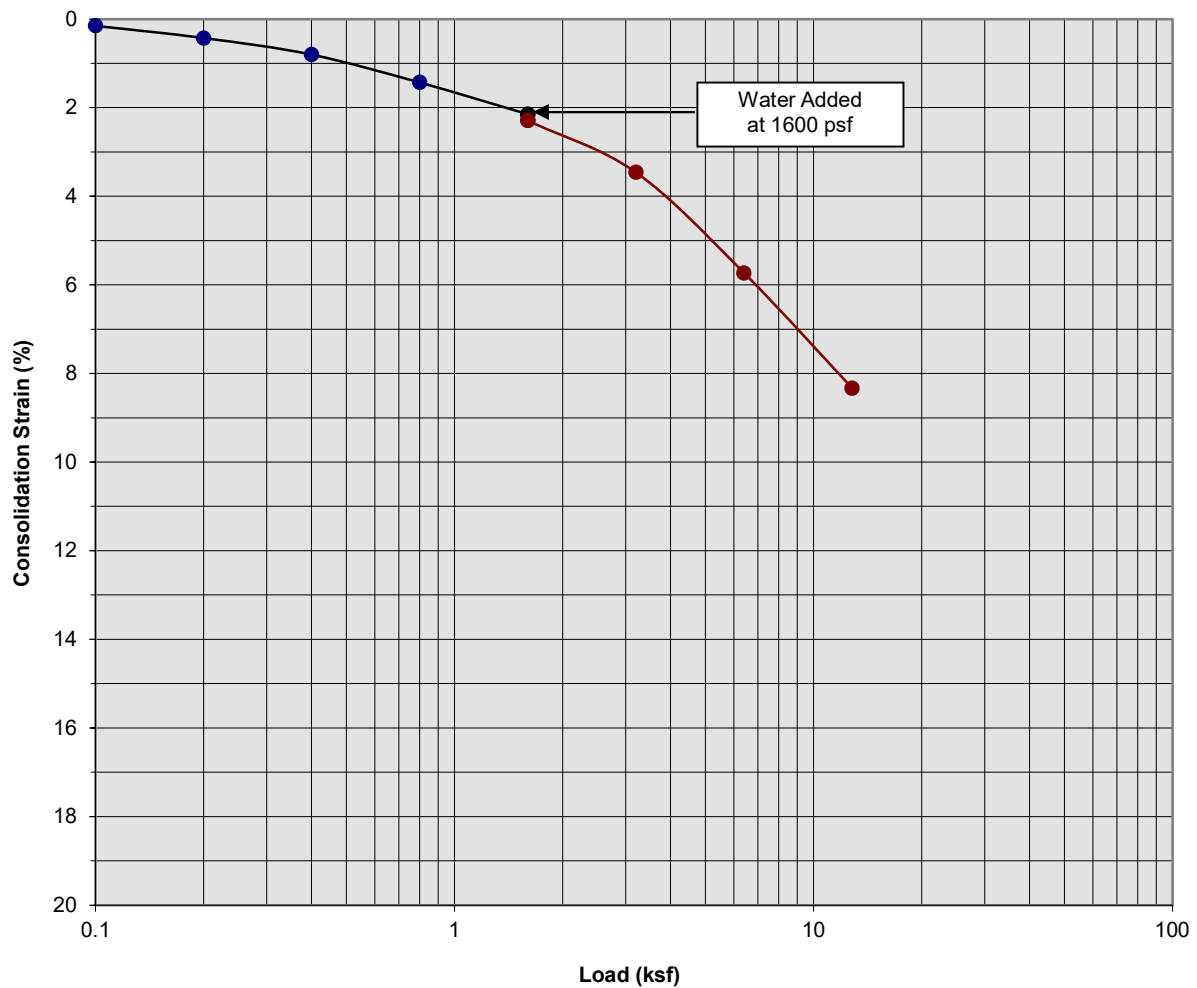
Soil ID Number	B-10 @ 0-5'
Optimum Moisture (%)	8.5
Maximum Dry Density (pcf)	131
Soil Classification	Brown Clayey fine to medium Sand, trace coarse Sand

Two Proposed Commercial/Industrial Buildings
Menifee, California
Project No. 20G227
PLATE C-8



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Dark Brown fine Sandy Clay, little Silt

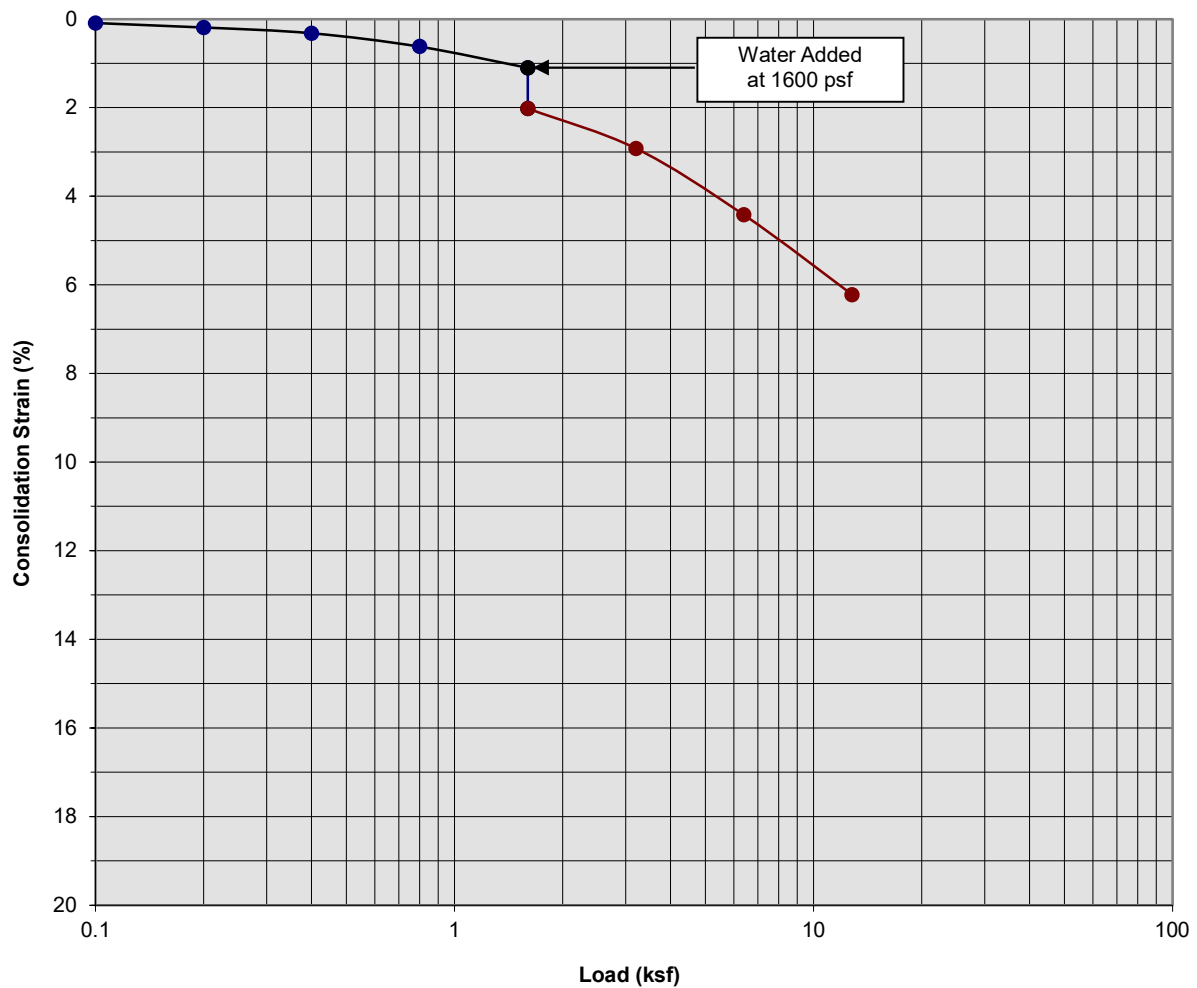
Boring Number:	B-3	Initial Moisture Content (%)	14
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	3 to 4	Initial Dry Density (pcf)	119.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	129.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.14

Menifee Commerce Center - Phase III
Menifee, California
Project No. 22G173
PLATE C-1



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown fine Sandy Silt, trace Clay, trace medium Sand

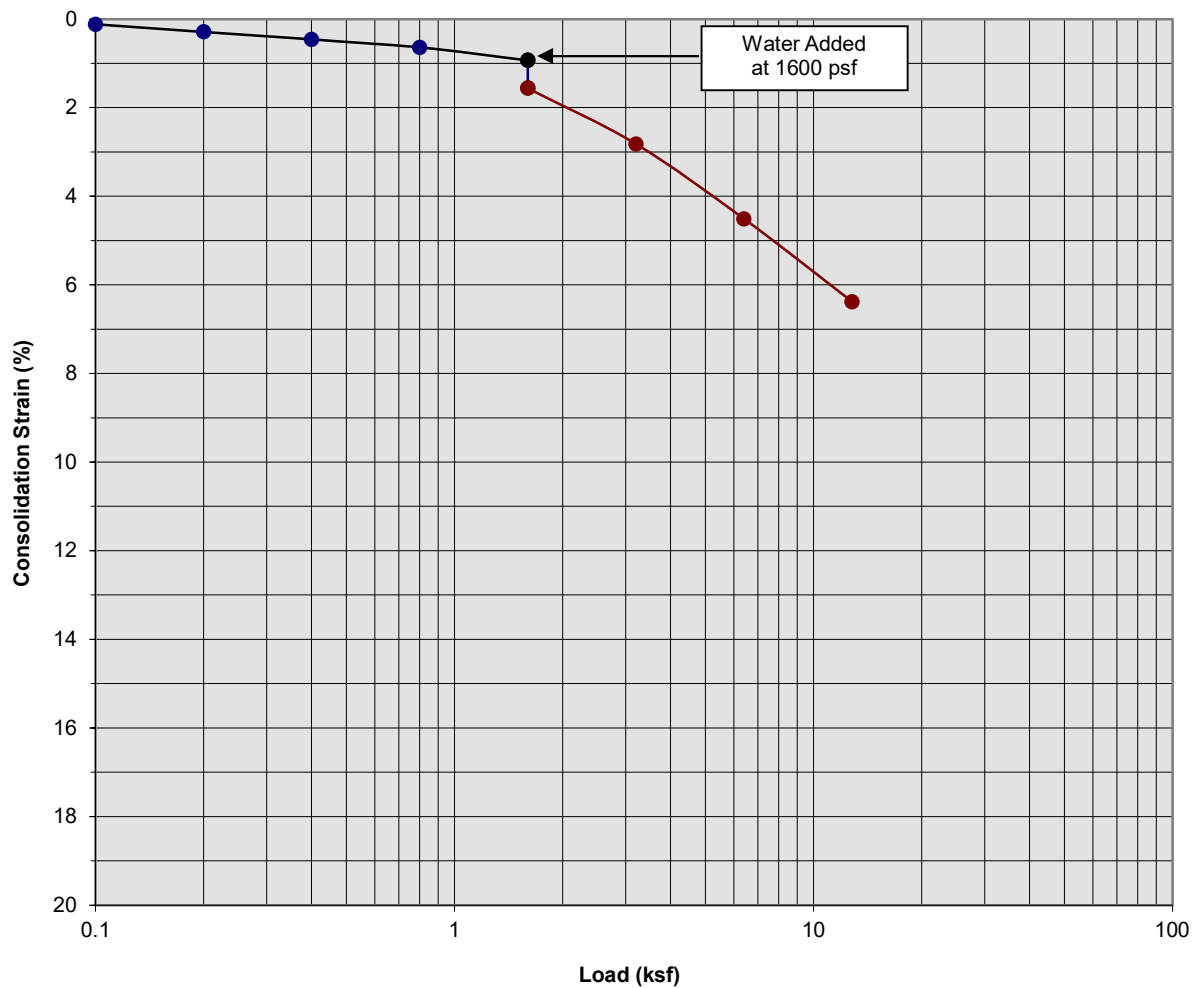
Boring Number:	B-3	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	5 to 6	Initial Dry Density (pcf)	115.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	122.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.92

Menifee Commerce Center - Phase III
Menifee, California
Project No. 22G173
PLATE C-2



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown Clayey fine Sand, little Silt

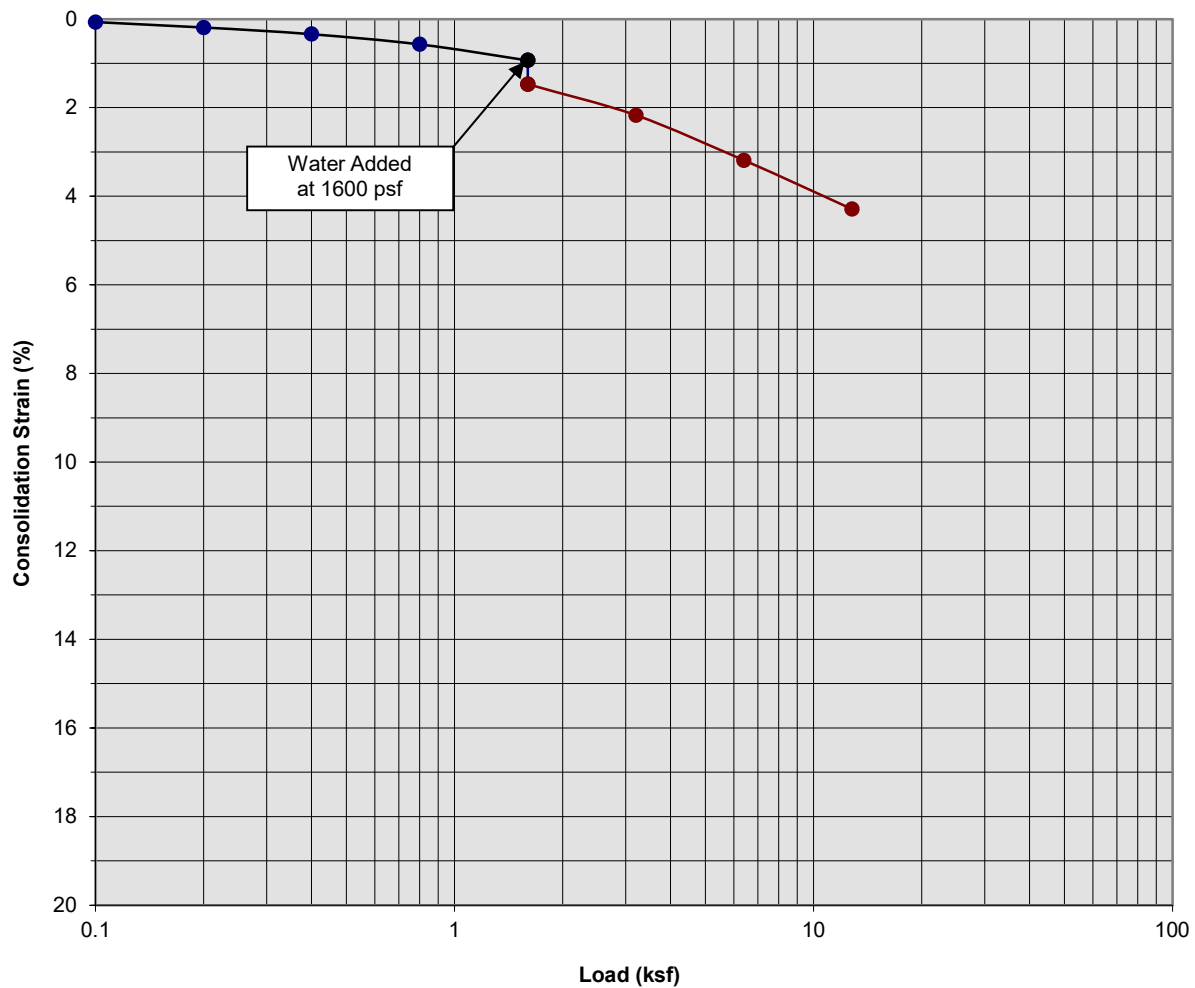
Boring Number:	B-3	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	7 to 8	Initial Dry Density (pcf)	121.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	129.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.63

Menifee Commerce Center - Phase III
Menifee, California
Project No. 22G173
PLATE C-3



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Light Brown fine Sandy Silt, trace Clay

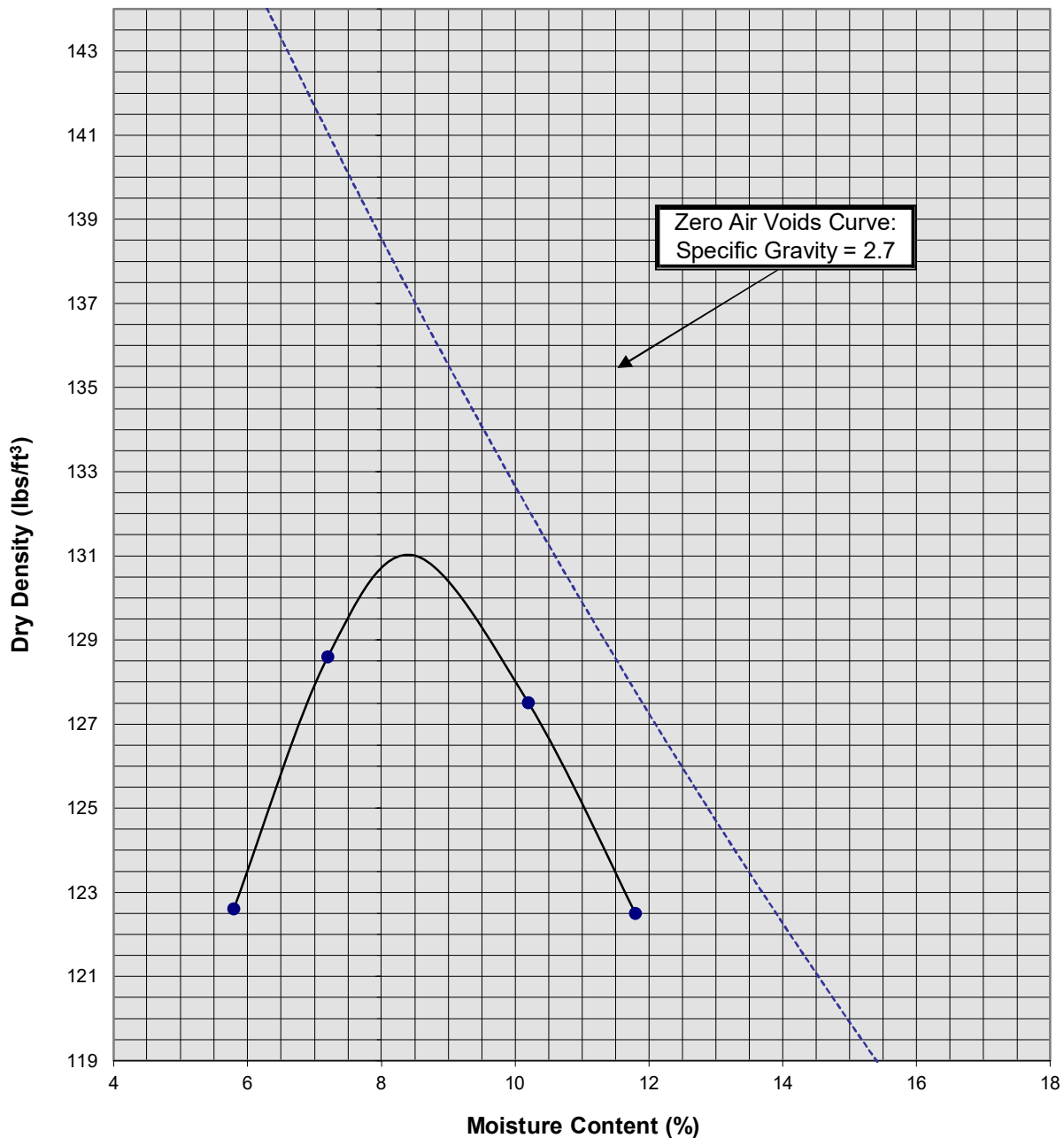
Boring Number:	B-3	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	9 to 10	Initial Dry Density (pcf)	120.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	125.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.54

Menifee Commerce Center - Phase III
Menifee, California
Project No. 22G173
PLATE C-4



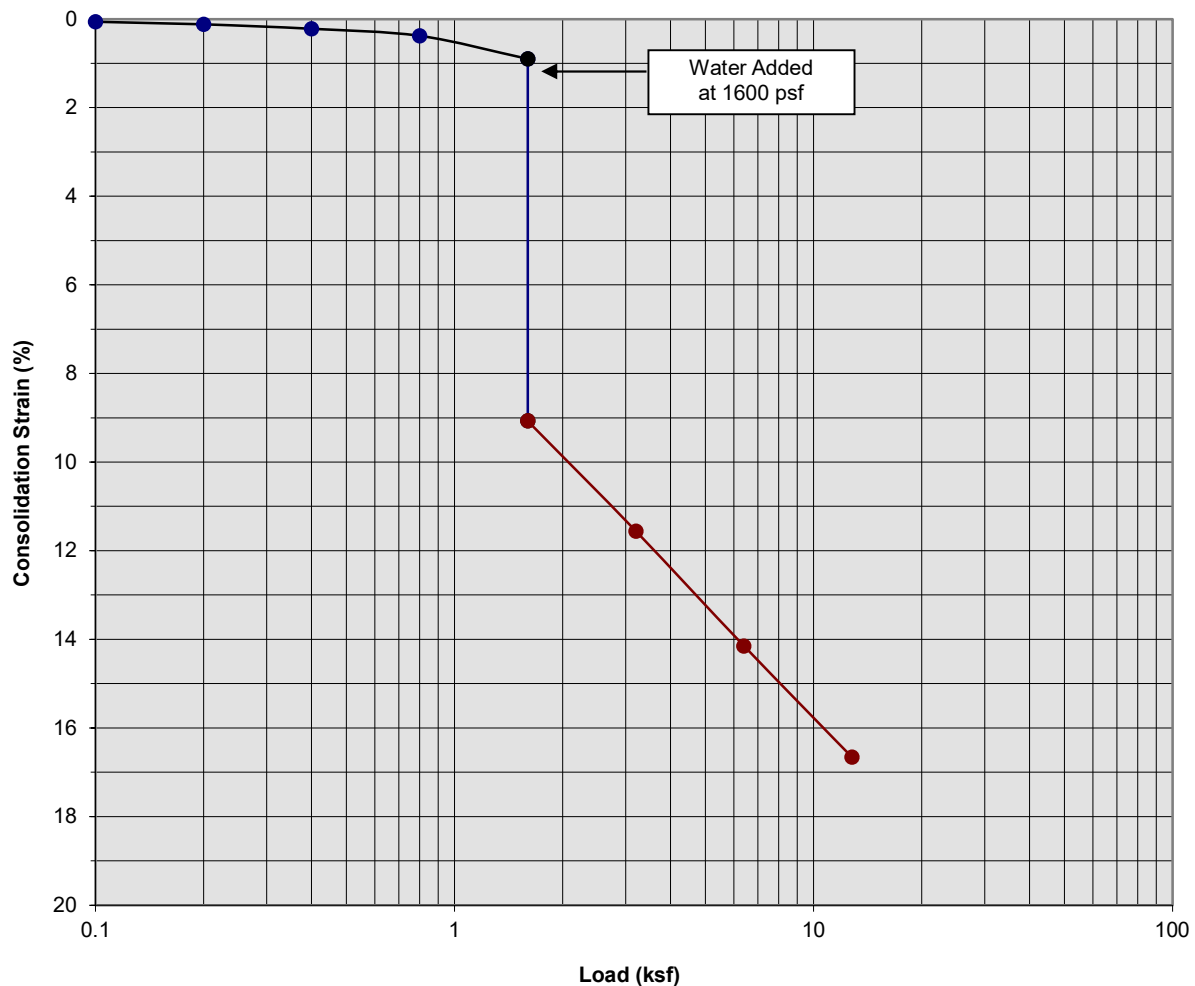
**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Moisture/Density Relationship ASTM D-1557



Soil ID Number	B-5 @ 0 to 5'
Optimum Moisture (%)	8.5
Maximum Dry Density (pcf)	131
Soil Classification	Light Gray Brown Silty fine to coarse Sand

Consolidation/Collapse Test Results



Classification: FILL: Brown fine Sandy Silt, trace medium Sand, trace Clay

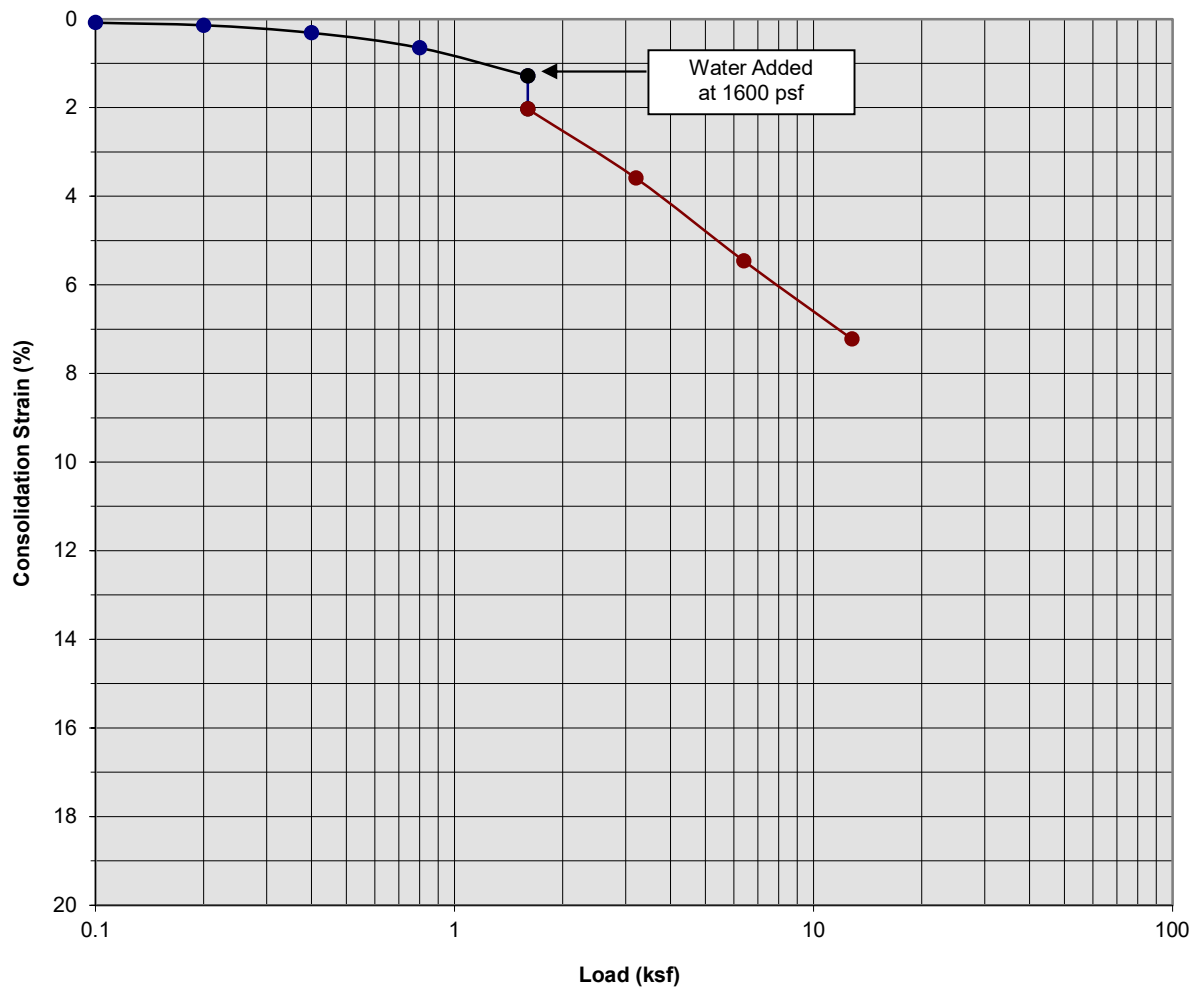
Boring Number:	B-5	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	3 to 4	Initial Dry Density (pcf)	104.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	124.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	8.17

Menifee Commerce Center Phase IV
Menifee, California
Project No. 22G206-1
PLATE C- 1



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown fine Sandy Clay, little Silt, trace medium Sand

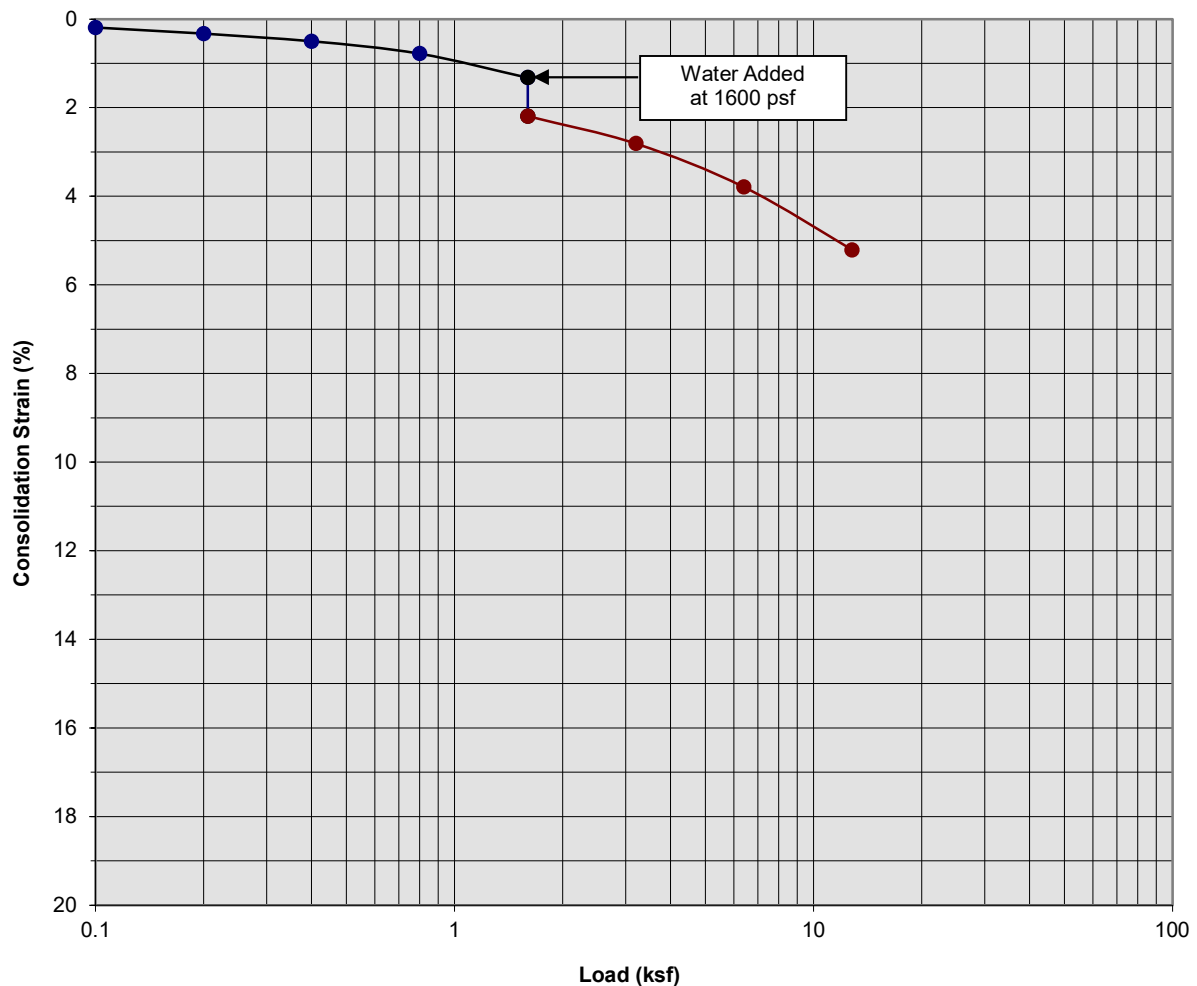
Boring Number:	B-5	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	5 to 6	Initial Dry Density (pcf)	120.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	129.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.75

Menifee Commerce Center Phase IV
Menifee, California
Project No. 22G206-1
PLATE C- 2



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Red Brown Clayey fine Sand

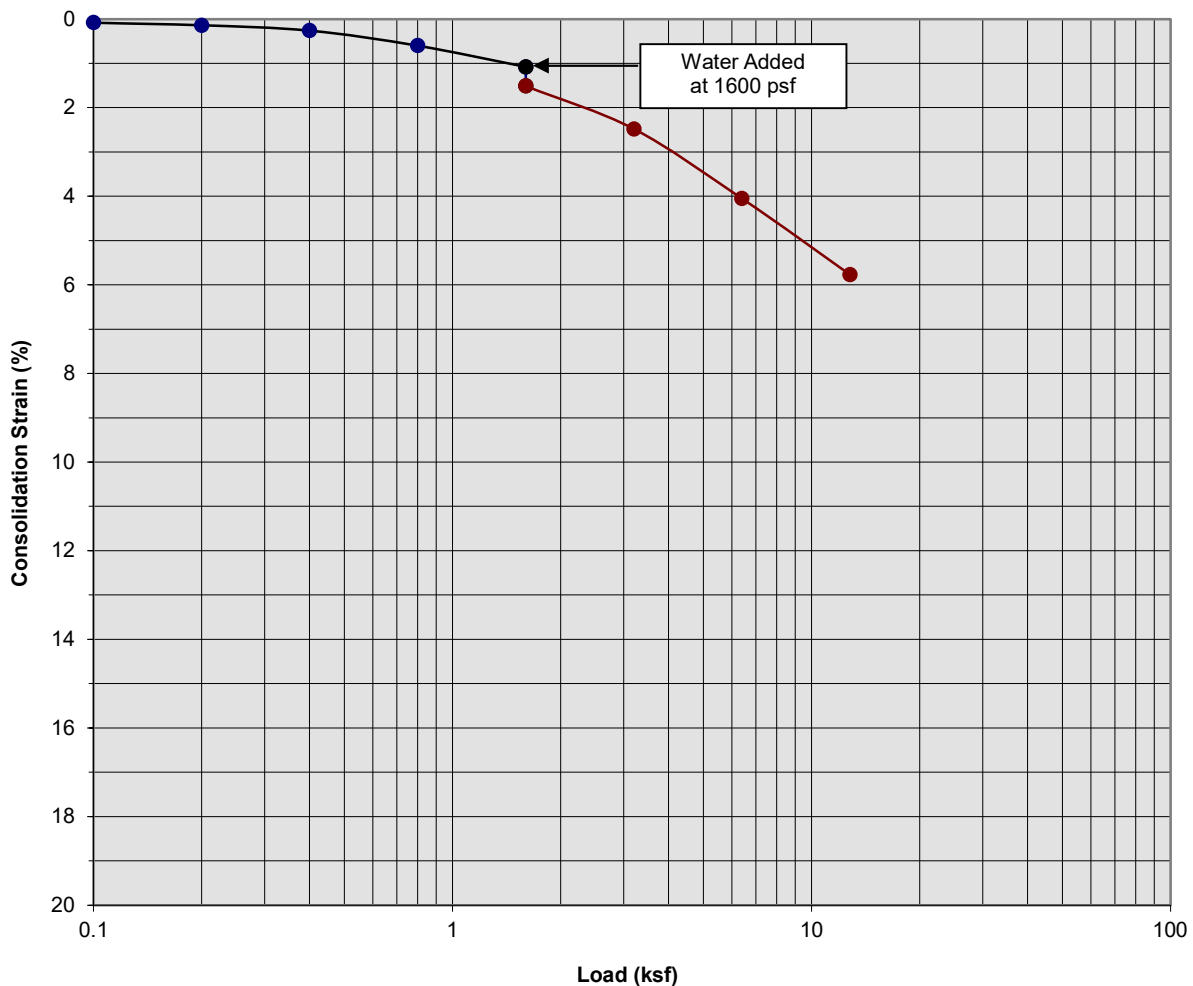
Boring Number:	B-5	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	7 to 8	Initial Dry Density (pcf)	127.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	133.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.87

Menifee Commerce Center Phase IV
Menifee, California
Project No. 22G206
PLATE C- 3



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown Silty Clay, trace fine to medium Sand

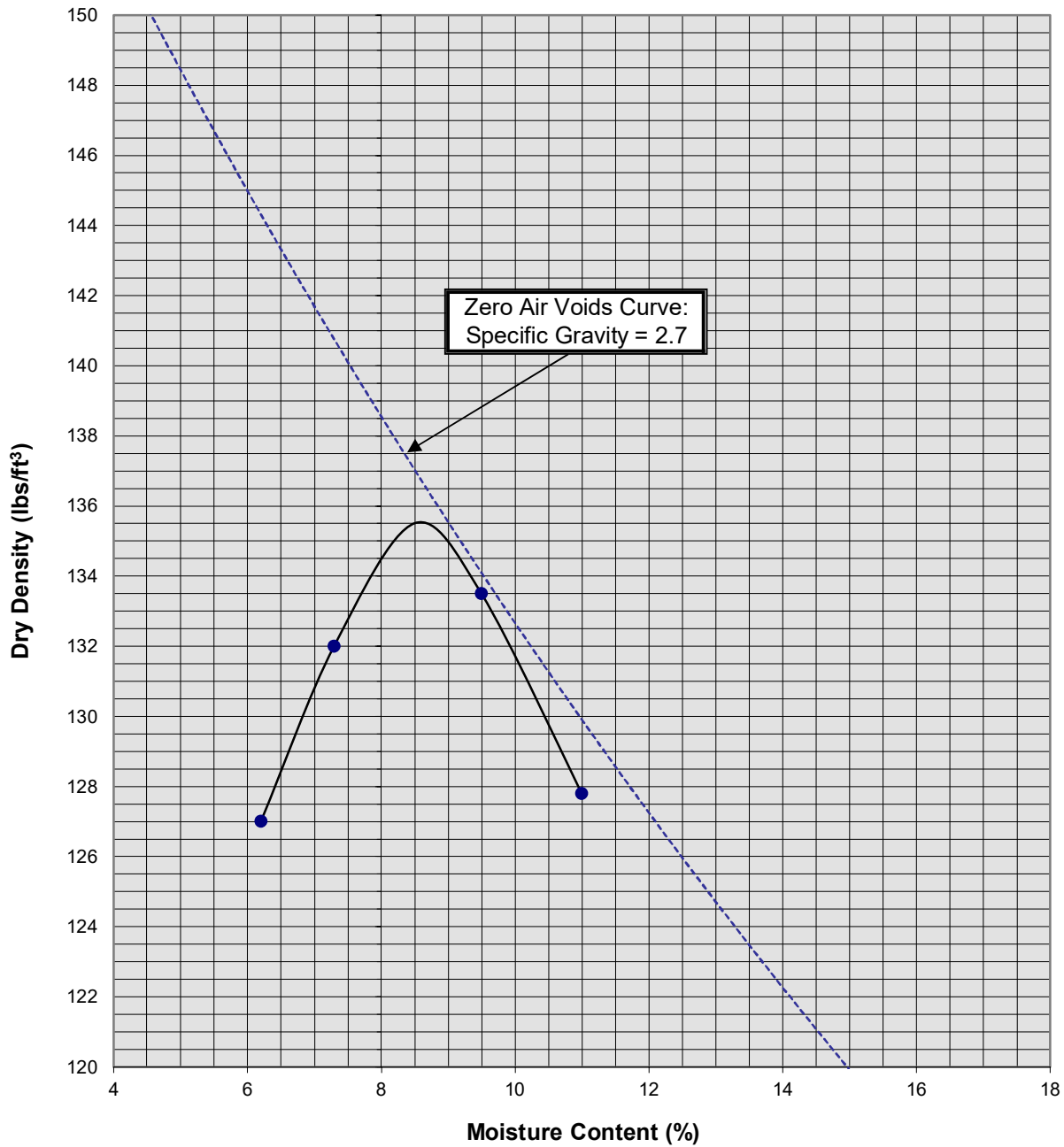
Boring Number:	B-5	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	9 to 10	Initial Dry Density (pcf)	123.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	130.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.43

Menifee Commerce Center Phase IV
Menifee, California
Project No. 22G206
PLATE C- 4



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Moisture/Density Relationship ASTM D-1557



Soil ID Number	B-3 @ 0-5'
Optimum Moisture (%)	8.5
Maximum Dry Density (pcf)	136.5
Soil Classification	Brown Clayey fine Sand, trace medium to coarse Sand

Meniffee Commerce Center Phase IV
Meniffee, CA
Project No. 22G206-1
PLATE C-5



APPENDIX

GRADING GUIDE SPECIFICATIONS

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

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.

Compacted Fills

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated," and shall be very low to non-expansive with a maximum expansion index (EI) of 20. 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 1/2 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

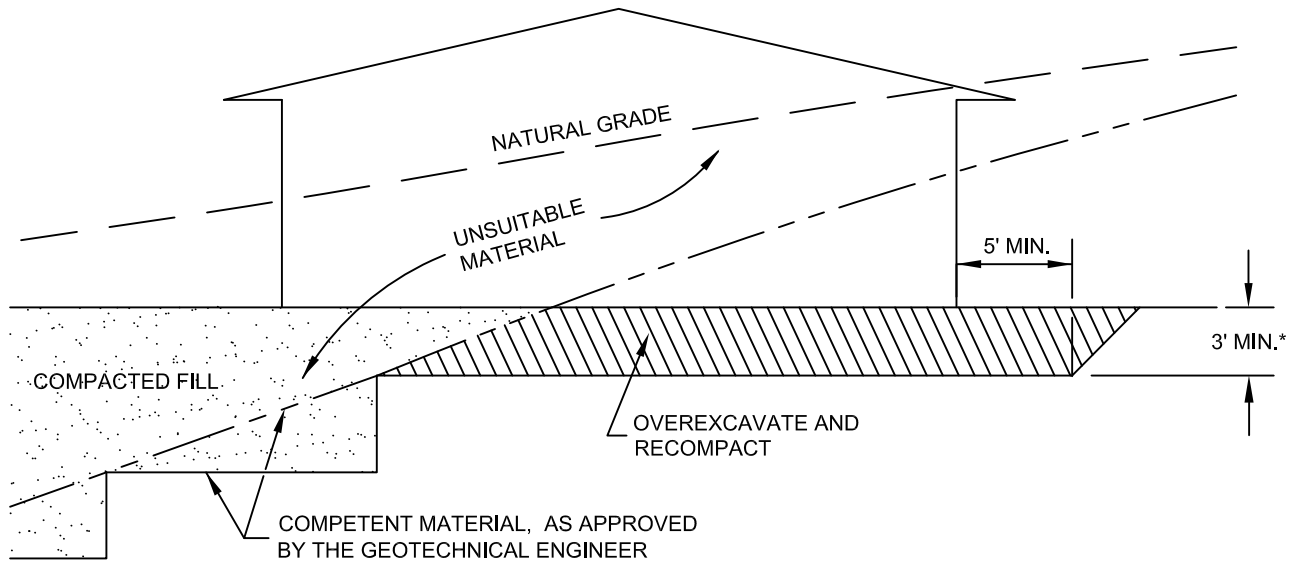
- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.
-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

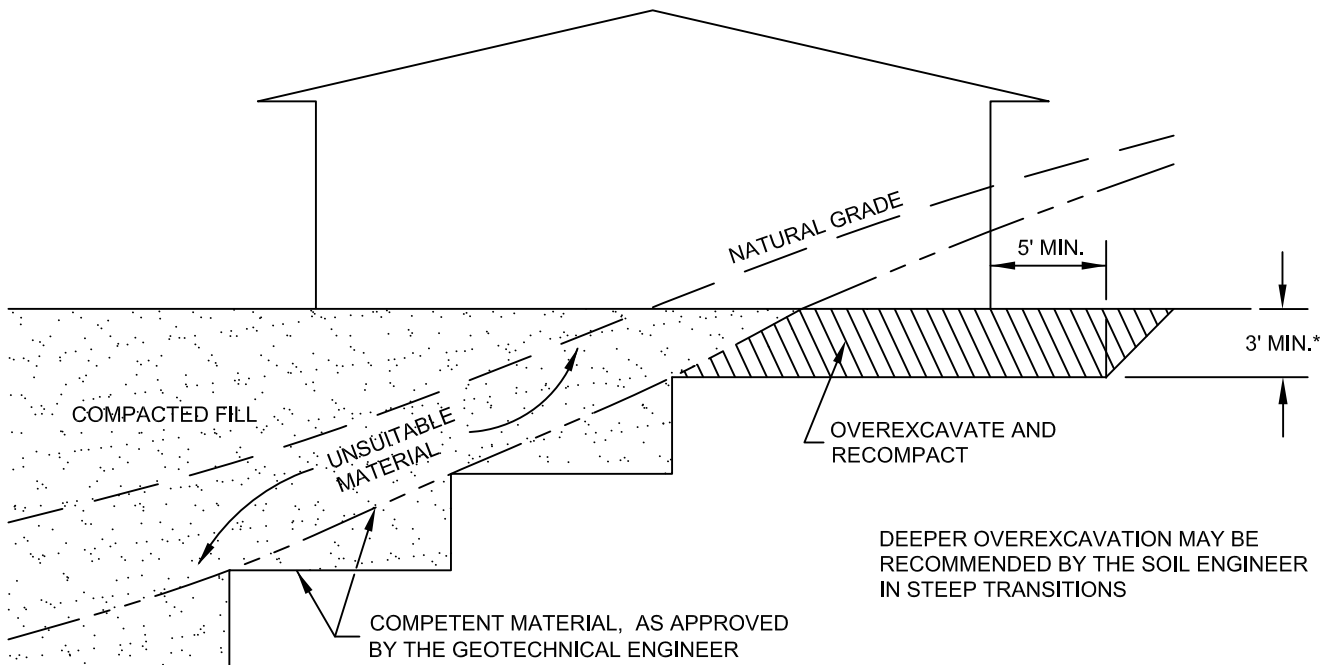
Subdrains

- 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 $\frac{3}{4}$ -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

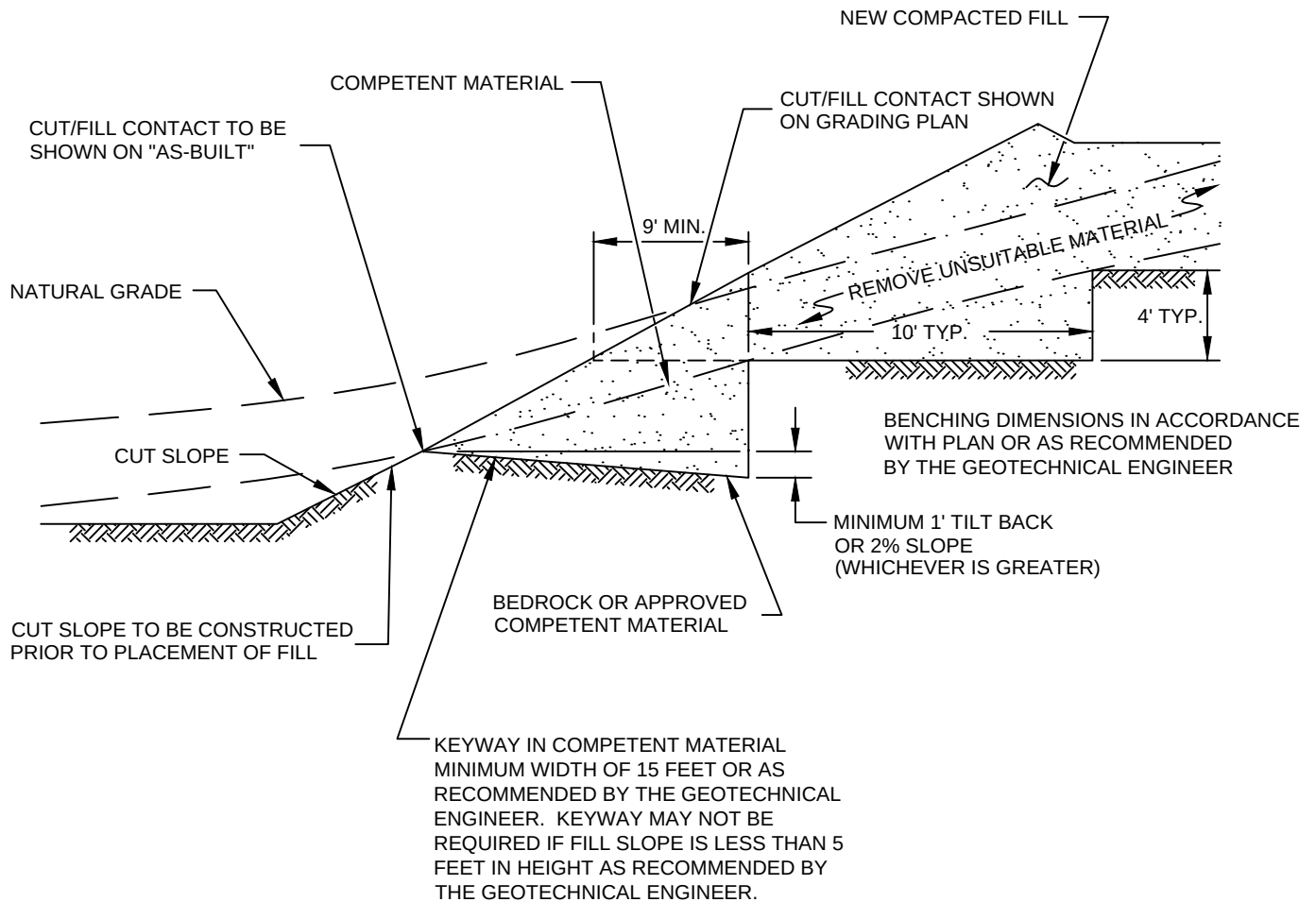
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**



FILL ABOVE CUT SLOPE DETAIL
GRADING GUIDE SPECIFICATIONS

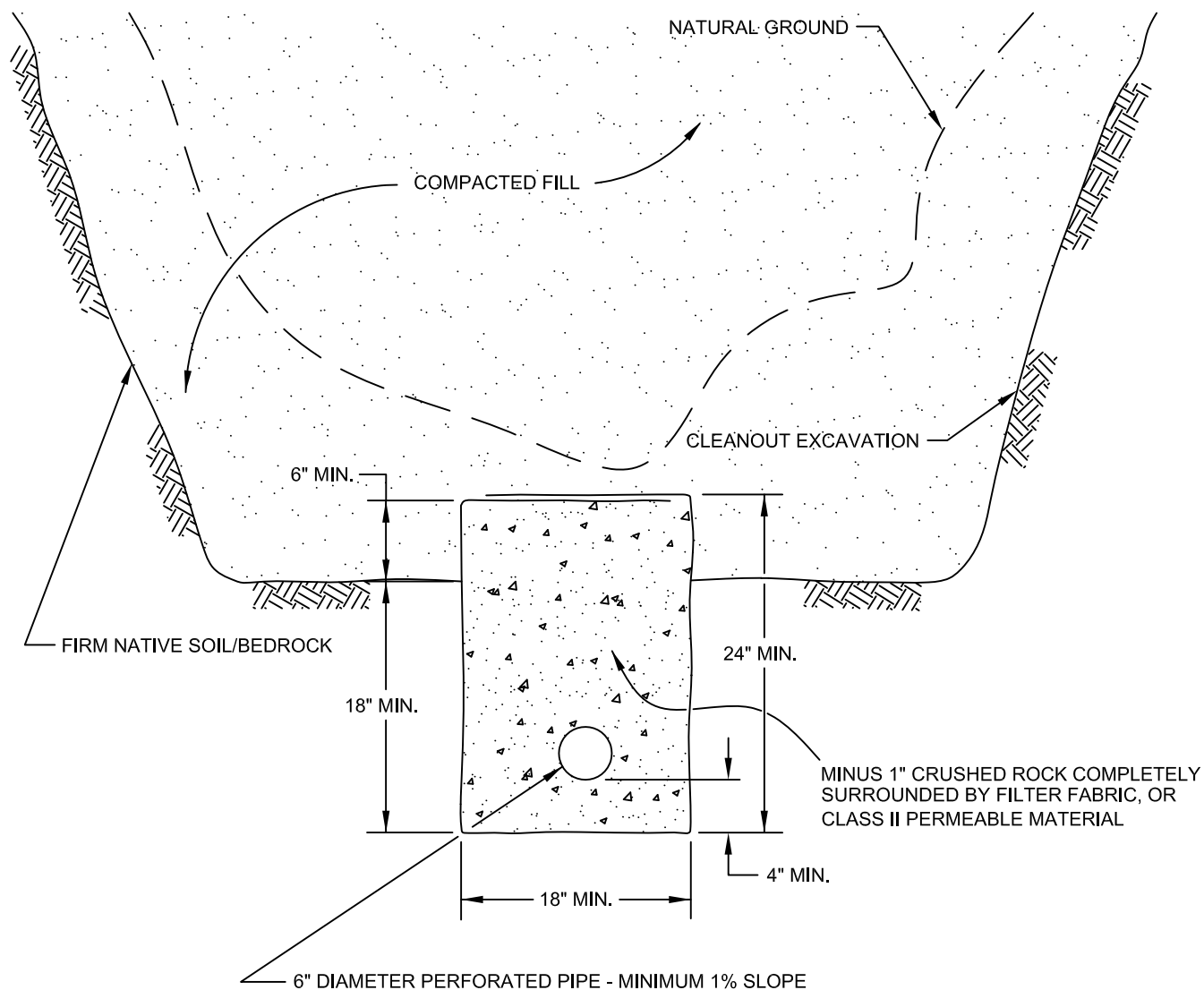
NOT TO SCALE

DRAWN: JAS
 CHKD: GKM

PLATE D-2



**SOUTHERN
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 GEOTECHNICAL**



PIPE MATERIAL
ADS (CORRUGATED POLETHYLENE)
TRANSITE UNDERDRAIN
PVC OR ABS: SDR 35
SDR 21

DEPTH OF FILL OVER SUBDRAIN
8
20
35
100

**SCHEMATIC ONLY
NOT TO SCALE**

CANYON SUBDRAIN DETAIL GRADING GUIDE SPECIFICATIONS

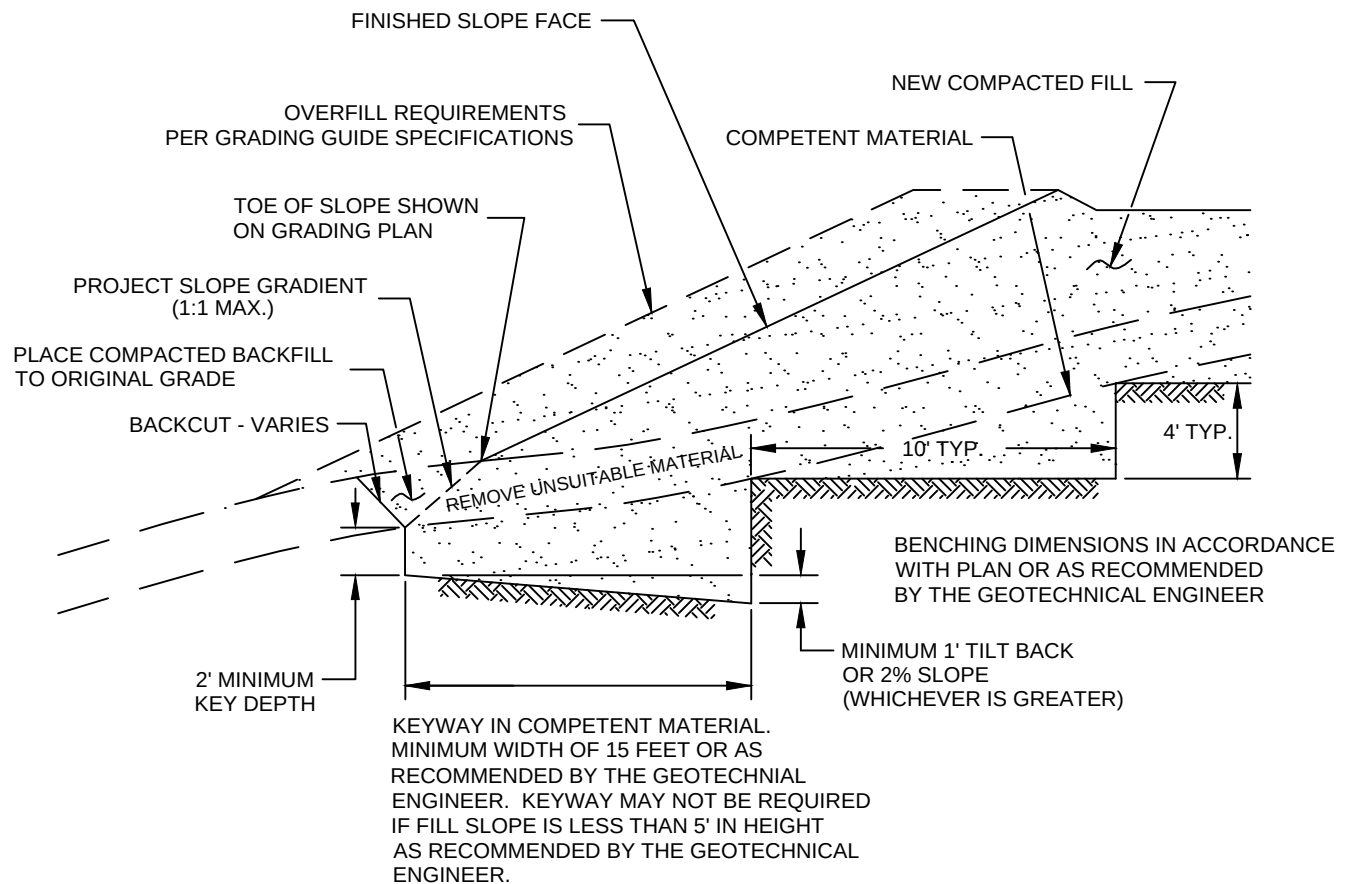
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**



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

FILL ABOVE NATURAL SLOPE DETAIL GRADING GUIDE SPECIFICATIONS

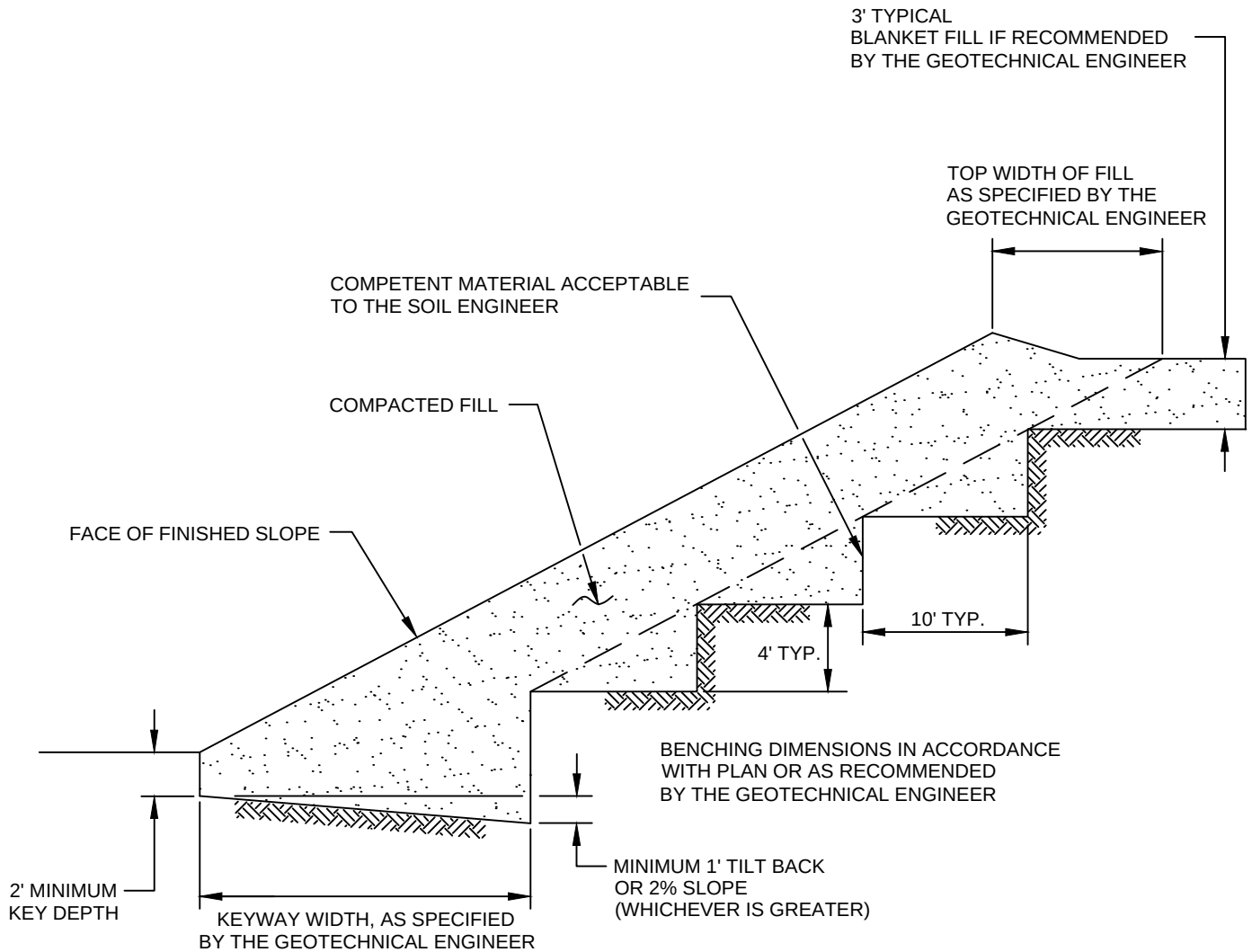
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

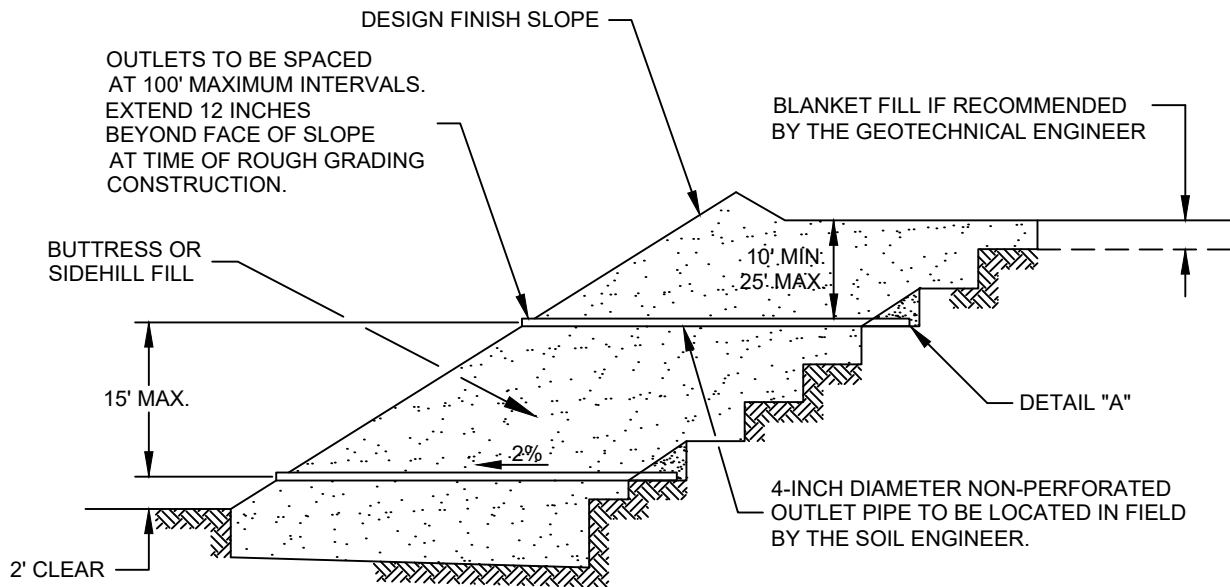
PLATE D-4



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



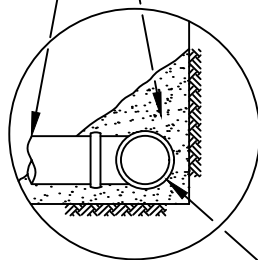
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8
SAND EQUIVALENT = MINIMUM OF 50	

OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW



DETAIL "A"

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.

NOTES:

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

SLOPE FILL SUBDRAINS	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-6	

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

WATERPROOFING AT FACE OF WALL IN ACCORDANCE WITH ARCHITECTURAL AND/OR STRUCTURAL DETAILS

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

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

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

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

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

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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8
SAND EQUIVALENT = MINIMUM OF 50	

RETAINING WALL BACKDRAINS GRADING GUIDE SPECIFICATIONS

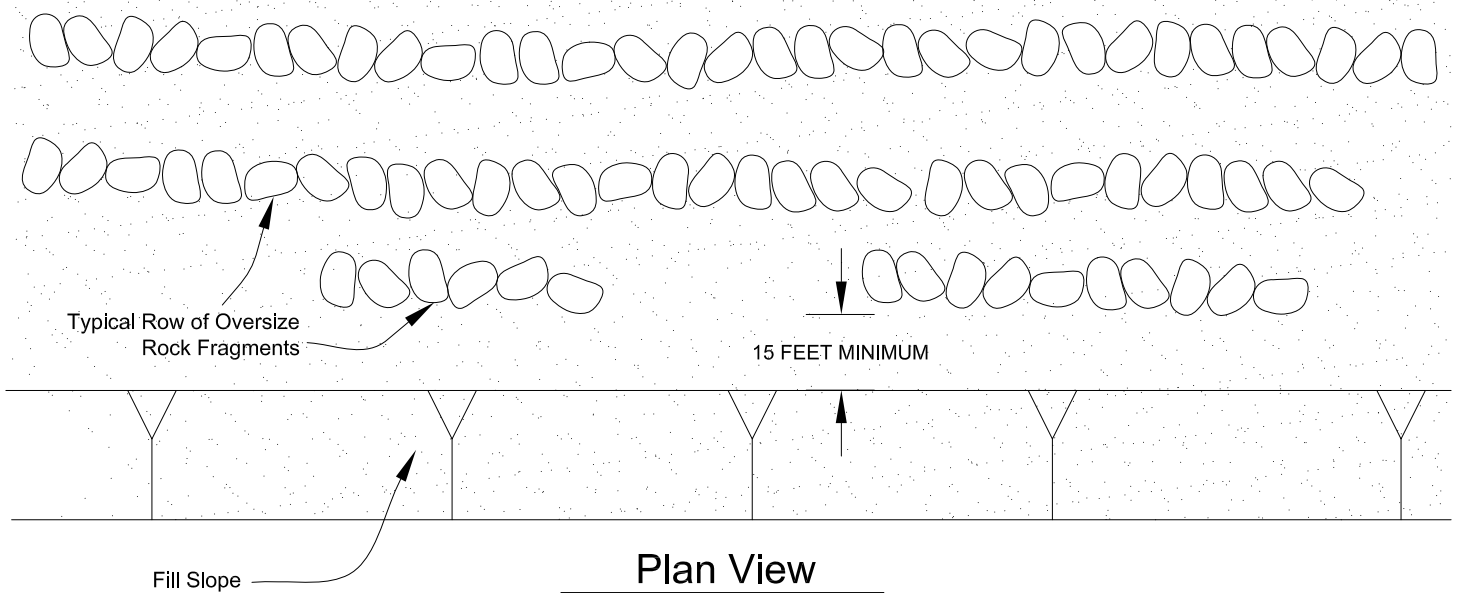
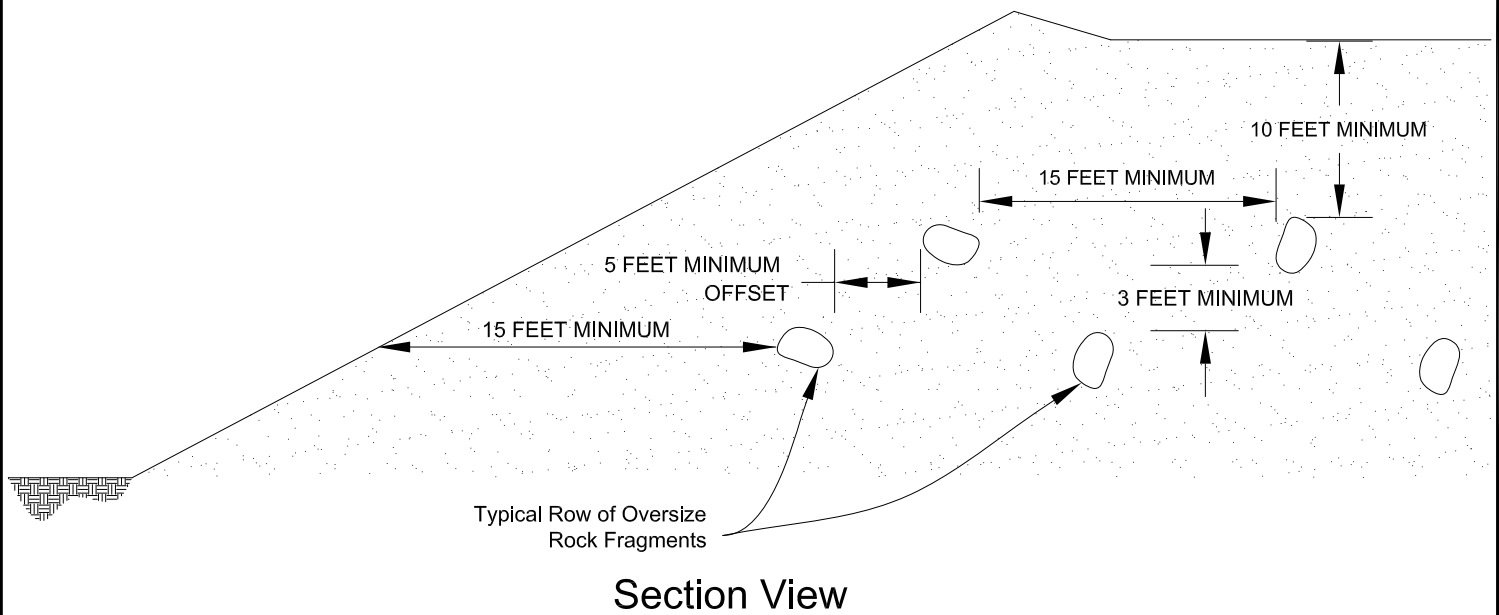
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-7



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**



PLACEMENT OF OVERSIZED MATERIAL
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
 CHKD: GKM

PLATE D-8



**SOUTHERN
 CALIFORNIA
 GEOTECHNICAL**

APPENDIX



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 Development
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 $1,075,680 \pm \text{ft}^2$ and $385,970 \pm \text{ft}^2$ 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 $25 \pm$ 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
S₁	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 (V₁₀₀)	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).

$$V_s = 100 / [(d1/v1) + (d2/v2) + ... + (dn/vn)]$$

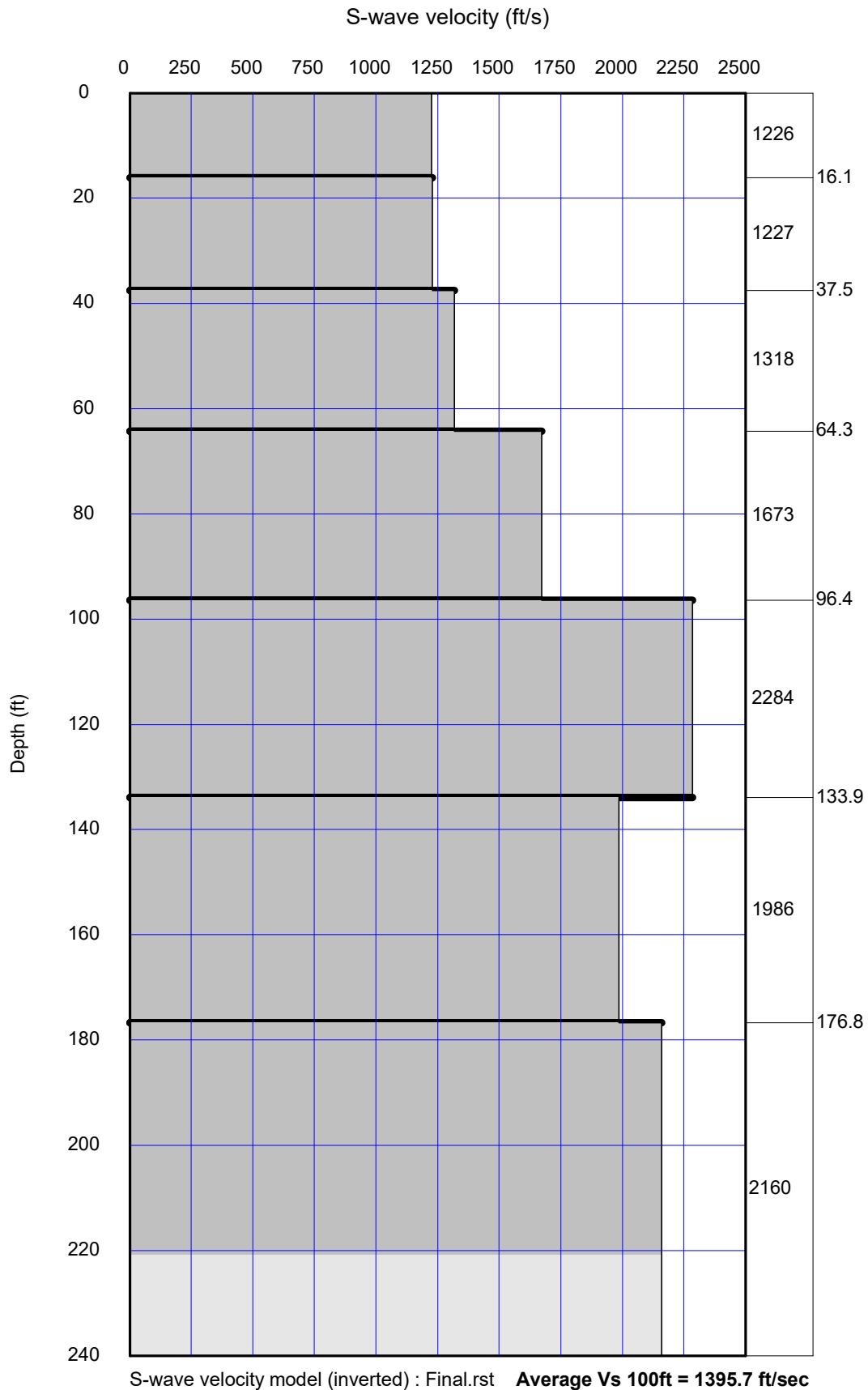
Where d1, d2, d3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n. The detailed shear-wave model displays these calculated layer boundaries/depths and associated velocities (feet/second) for the 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

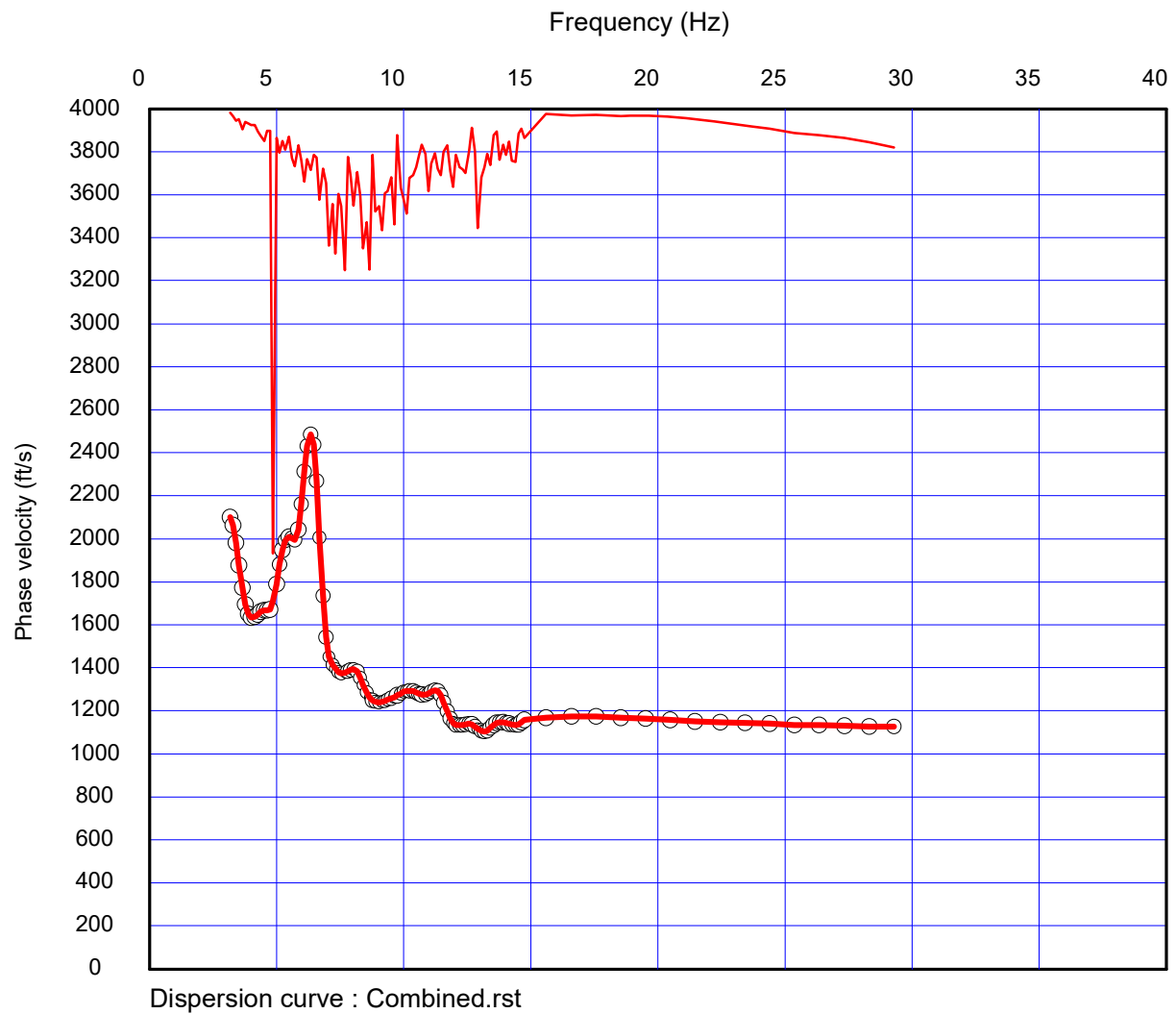
This survey was performed using “state of the art” geophysical equipment, techniques, and computer software. We make no warranty, either expressed or implied. It should be understood that when using these theoretical geophysical principles and techniques, sources of error are possible in both the data 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

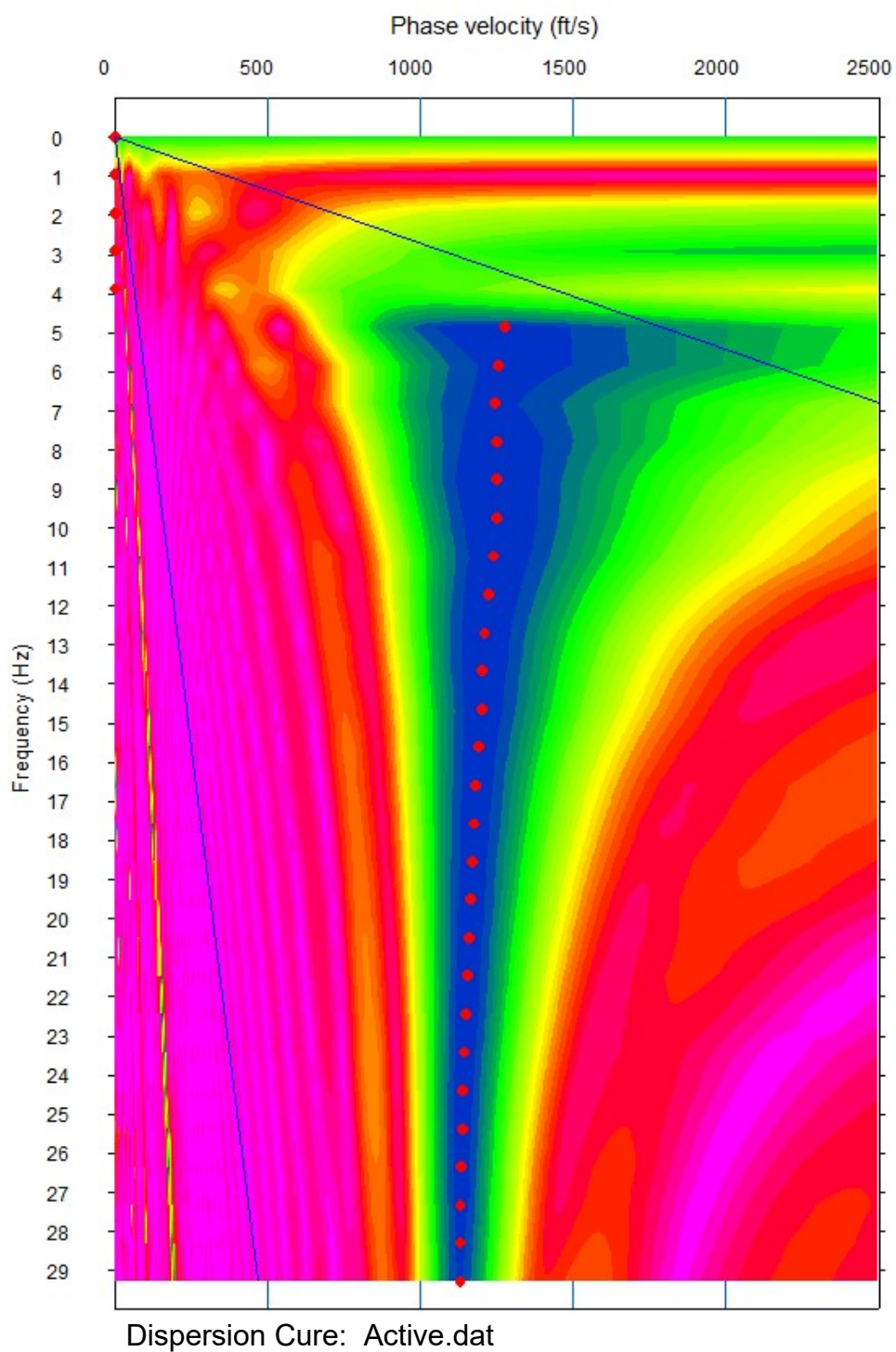


SHEAR-WAVE MODEL SW-1



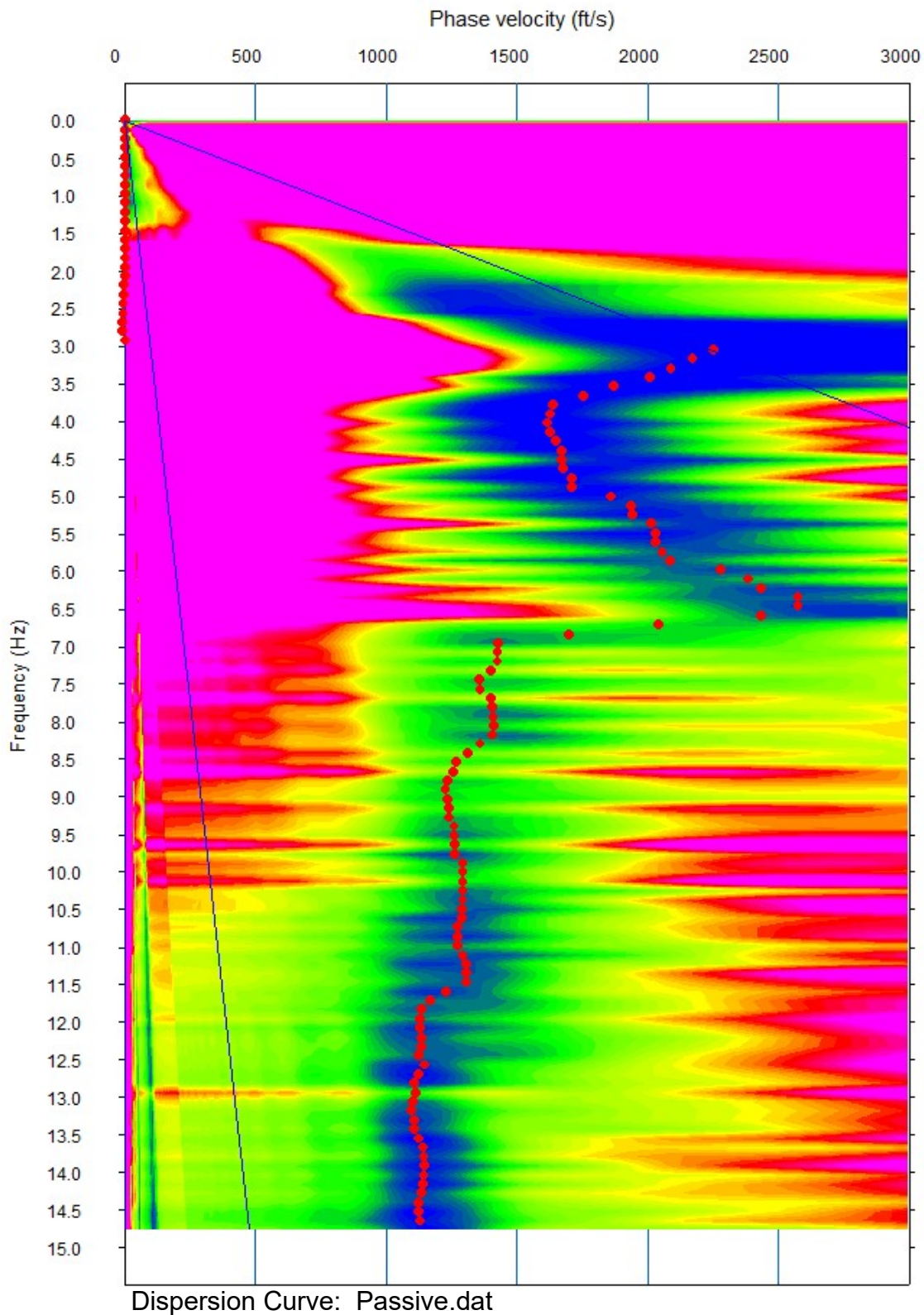
COMBINED DISPERSION CURVE

SEISMIC LINE SW-1



ACTIVE DISPERSION CURVE

SEISMIC LINE SW-1



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.

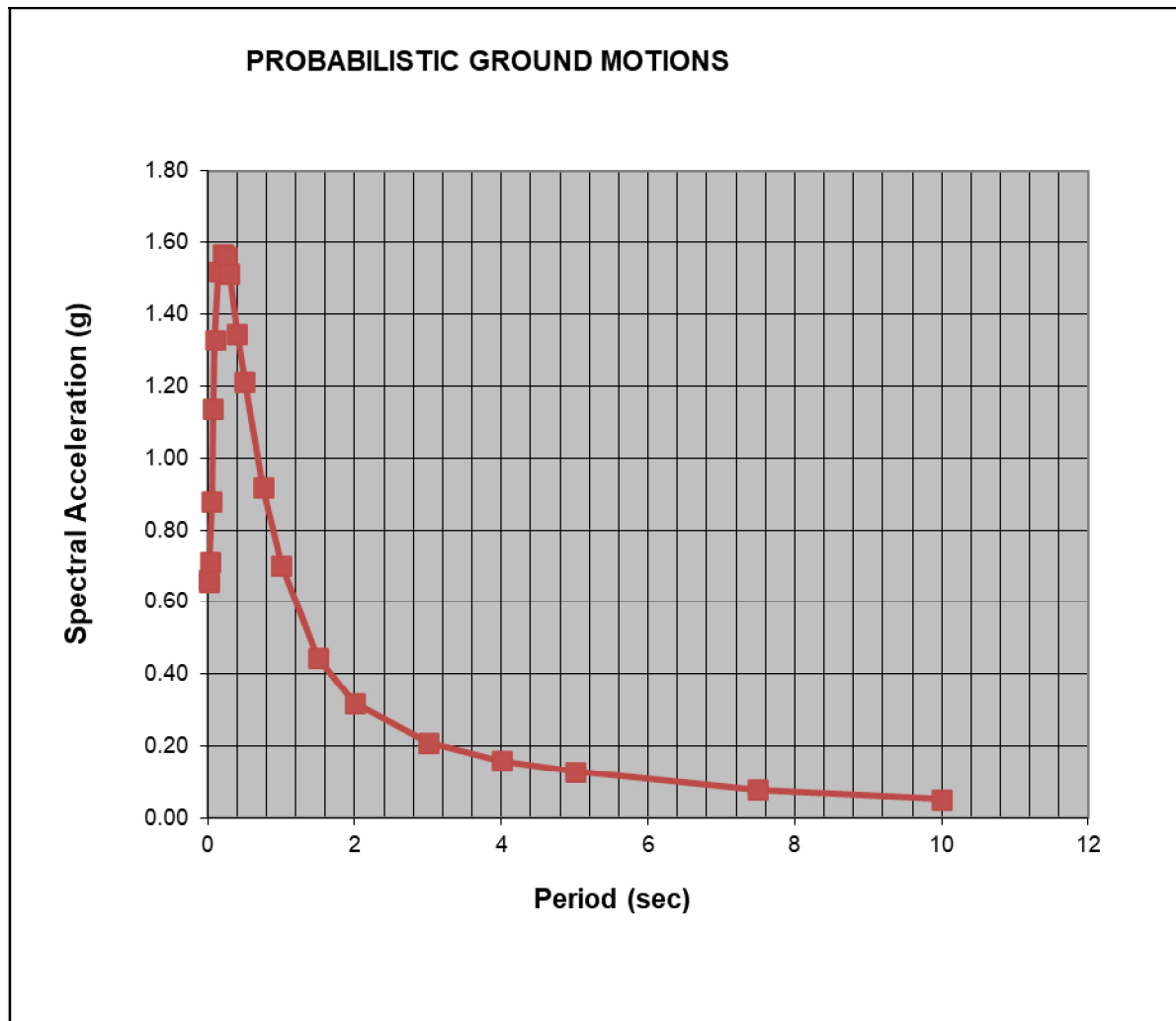
◆ Site Coefficients (CBC 1613.2.3)-

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

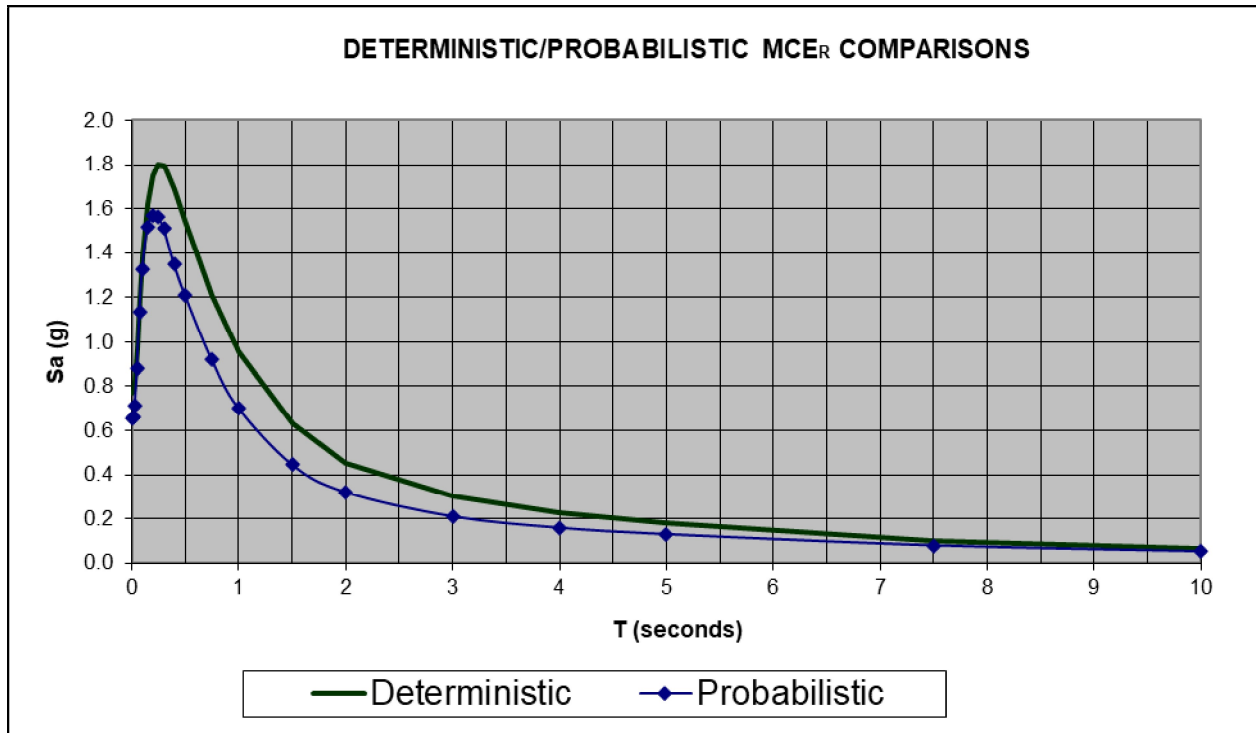
◆ **Site Specific MCE_R (ASCE 7 Section 21.2.3)-**

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

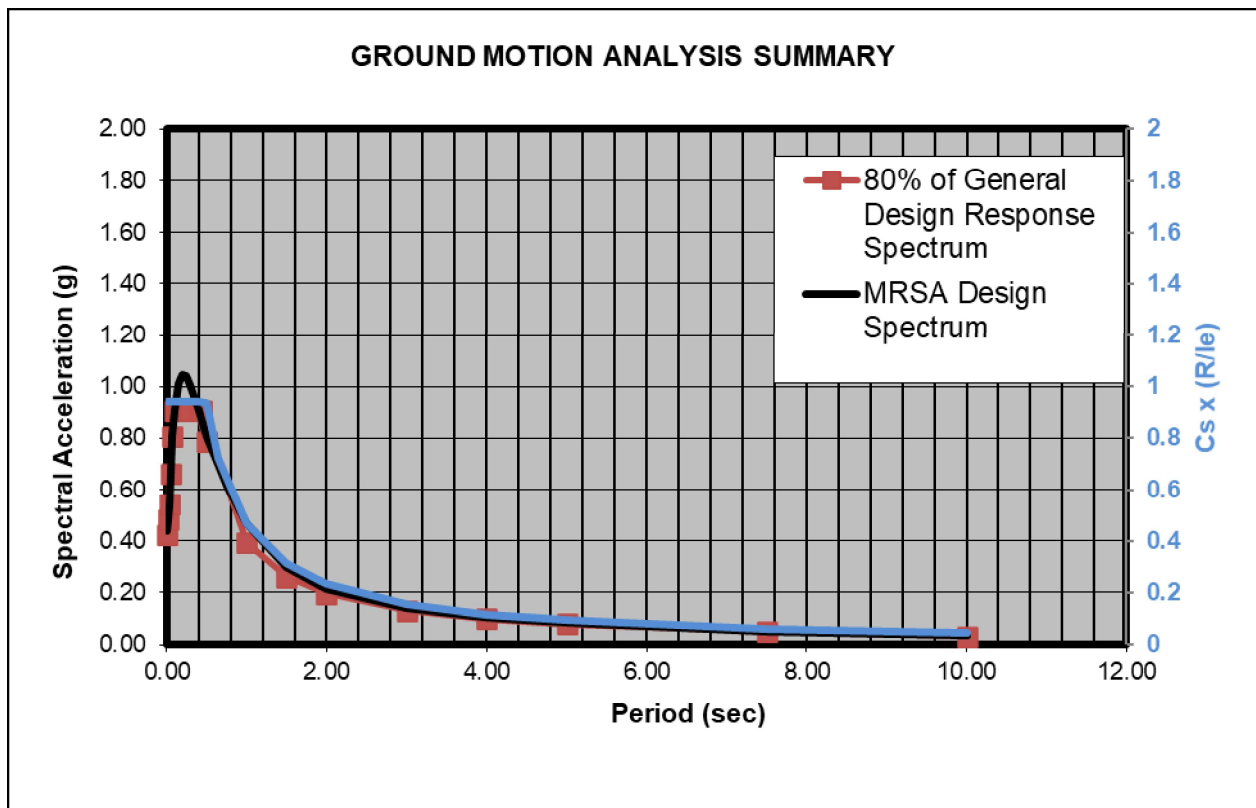
Period	Deterministic	Probabilistic	Lower Value (Site Specific MCE_R)	Governing Method	
T	MCE_R	MCE_R			
0.010	0.77	0.66	0.66	Probabilistic Governs	
0.020	0.78	0.66	0.66	Probabilistic Governs	
0.030	0.82	0.71	0.71	Probabilistic Governs	
0.050	0.97	0.88	0.88	Probabilistic Governs	
0.075	1.20	1.14	1.14	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: $S_a = 2/3 S_{aM}$, 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 be 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 $S_a * T$ 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 84th 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., $0.60g \times 0.80 = 0.48g$).

SEISMIC DESIGN PARAMETERS SUMMARY

Project: Ethanac and Dawson Roads Project Latitude: 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

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

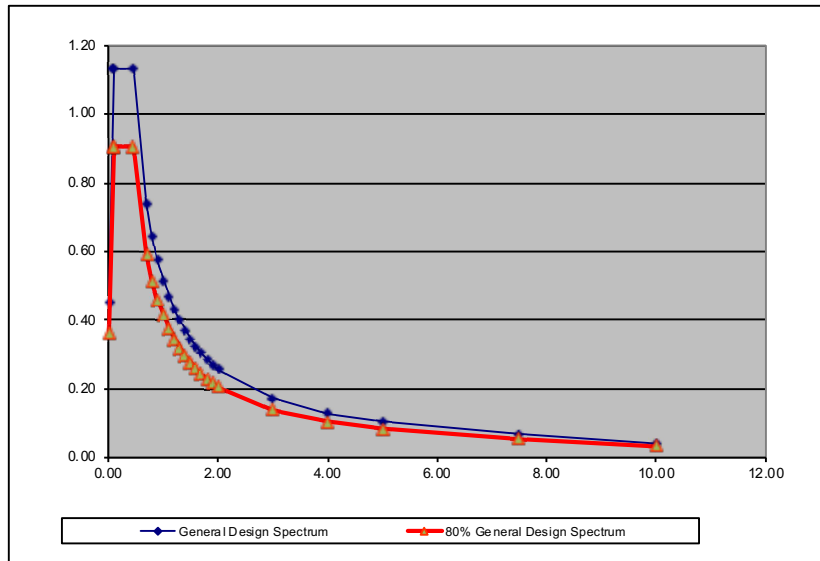
T_0	0.091	sec
T_S	0.456	sec
T_L	8	sec
PGA	0.5	g
F_{PGA}	1.2	
C_{RS}	0.935	
C_{R1}	0.918	

From Fig 22-12

From Table 11.8-1
Figure 22-17

Figure 22-18

Period (T)	S_a (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

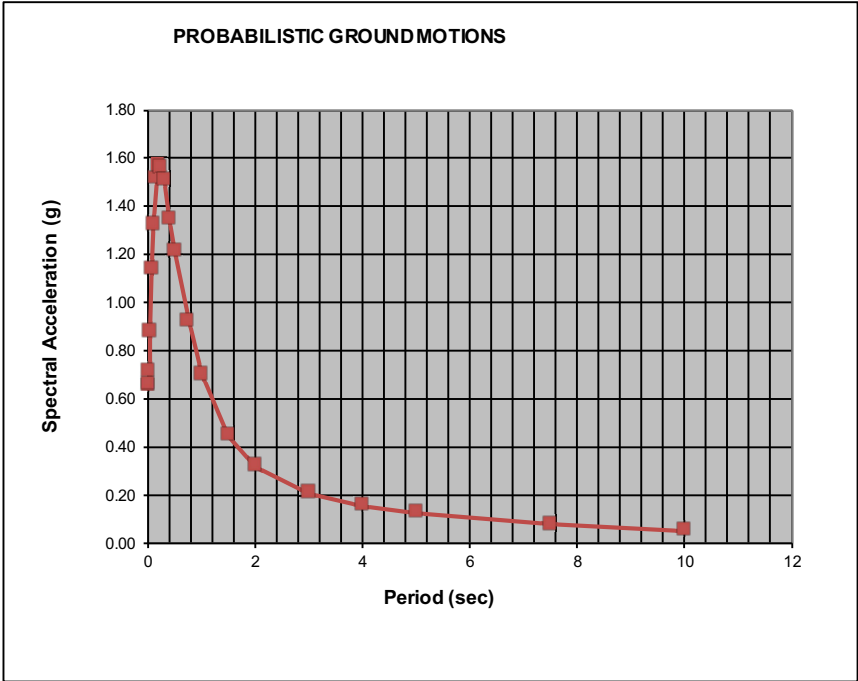
S _s =	1.68	1.57
S ₁ =	0.76	0.70
PGA	0.64 g	

Risk Coefficients:		
C _{RS}	0.935	Figure 22-18
C _{R1}	0.918	Figure 22-19
Fa	1.2	Table 11.4-1
Is Sa _(max) <1.2XFa?	NO	

Get from Mapped Values

Per ASCE7-16 - 21.2.3

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
R_x	= 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

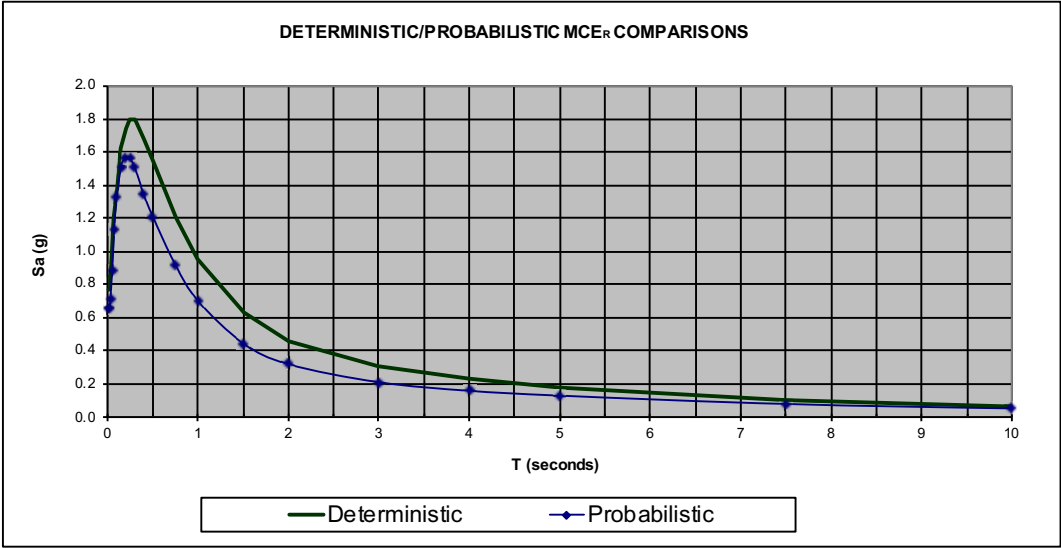
Deterministic Summary - Section 21.2.2 (Supplement 1)

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
Max Sa=	1.39	Per ASCE7-16 21.2.2	
Fa =	1.20		
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	Deterministic	Probabilistic	Lower Value (Site Specific MCE _R)	Governing Method
T	MCE _R	MCE _R		
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)

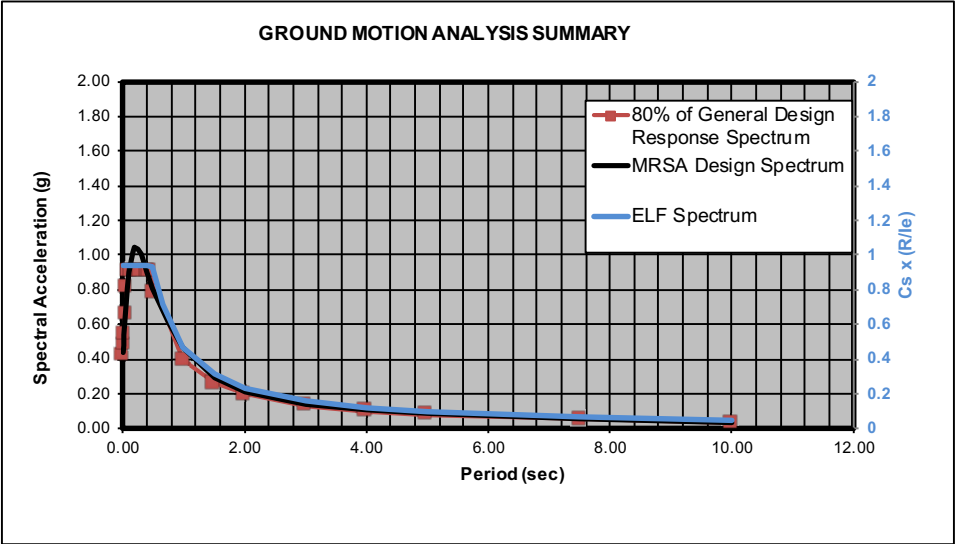
Period	2/3*MCE _R	80% General Design Response Spectrum (per ASCE 7-16 Figure 11.4-1)	Design Response Spectrum	TXSa
0.01	0.44	0.42	0.44	
0.02	0.44	0.48	0.48	
0.03	0.47	0.54	0.54	
0.05	0.59	0.66	0.66	
0.08	0.76	0.81	0.81	
0.10	0.89	0.91	0.91	
0.15	1.01	0.91	1.01	
0.20	1.05	0.91	1.05	
0.25	1.04	0.91	1.04	
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 T _{sa} from T=1s-2s =	0.47

S _{DS} =	0.94	S _{MS} =	1.412
S _{D1} =	0.47	S _{M1} =	0.702
T _s =	0.50		

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 F2

PALEONTOLOGICAL RESOURCES ASSESSMENT

Paleontological Resources Assessment
for the
Heritage Valley Business Center Project,
City of Menifee, Riverside County, California

October 2025

Prepared By:

Jamie Parra

Cultural and Paleontological Resources Management

Kimley-Horn and Associates, Inc.

3801 University Ave, Suite 300

Riverside, CA 92501

Kimley»Horn

A series of overlapping geometric shapes, primarily triangles and polygons, in shades of gray, dark gray, and maroon, arranged in a fan-like pattern pointing towards the bottom right corner of the page.

EXECUTIVE SUMMARY

Kimley-Horn and Associates, Inc. (Kimley-Horn) is under contract to complete a Paleontological Resources Assessment for the proposed Heritage Valley Business Center Project (Project) in the City of Menifee (City), Riverside County, California. The proposed Project proposes the development of one warehouse building, totaling approximately 829,755 square feet (sq. ft.) of warehouse space which includes 10,000 sq. ft. of office and 819,755 sq. ft. of ground floor warehouse on an approximately 41-acre site. The building is contemplated to have a structural height of approximately 60 feet and includes approximately 503 automobile parking spaces, approximately 258 truck trailer parking spaces, and approximately 111 dock high doors. The Project is subject to environmental review and consideration pursuant to the California Environmental Quality Act (CEQA) with the City of Menifee acting as the lead agency. As such, this study was completed to determine whether the Project area contains, or could reasonably contain, paleontological resources and to support the City's review of potential impacts to these resources as a result of the proposed Project.

Kimley-Horn conducted paleontological record searches at the Western Science Center (WSC) and Natural History Museum of Los Angeles County (NHMLAC). Kimley-Horn also reviewed the online paleontological database at University of California Museum of Paleontology (UCMP), geological maps, and geotechnical data for the Project area. Kimley-Horn did not identify any previously recorded fossil localities in the Project area. However, Kimley-Horn determined that the Project area has a high sensitivity for unrecorded paleontological resources. Specifically, the Project area is underlain by Pleistocene-age geologic deposits, and fossil localities have been identified in similar deposits across the Perris Valley and Menifee Valley. The City's General Plan also identifies the Project area as having a high paleontological sensitivity. As such, Kimley-Horn recommends that mitigation measures for a Paleontological Resources Impact Mitigation Program, paleontological monitoring, and data recovery are adopted for the Project. With these mitigation measures in place, any potential impacts paleontological resources would be less than significant.

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Figure 2: Project Area are Offsite Improvements Map

Figure 3: Project Area within the City of Menifee Paleontological Sensitivity Map

Figure 4: Geologic Map of the Project Area (Project Indicated with Red Star)

Figure 5: 1943 Topographic Map

Figure 6: 1961 Historic Aerial

Figure 7: 1985 Historic Aerial

Figure 8: 2009 Historic Aerial

TABLES

Table 1: Previously Recorded Fossil Localities at NHMLAC nearest to the Project

Table 2: Previously Recorded Fossil Localities at UCMP from the Pleistocene Epoch in Riverside County

APPENDICES

Appendix A: Paleontological Record Search Results

INTRODUCTION

Kimley-Horn and Associates, Inc. (Kimley-Horn) is under contract to complete a Paleontological Resources Assessment for the proposed Heritage Valley Business Center Project (Project) in the City of Menifee (City), Riverside County, California. The Project is subject to environmental review and consideration pursuant to the California Environmental Quality Act (CEQA) with the City of Menifee serving as the lead agency (Lead Agency). As such, this study was completed to determine whether the Project area contains, or could reasonably contain, paleontological resources and to support the Lead Agency's review of potential impacts to these resources as a result of the proposed Project.

PROJECT DESCRIPTION

The Project is generally bound by non-conforming single-family residential properties, a Riverside County Flood Control channel, Southern California Edison (SCE) utility corridor, and McLaughlin Road to the south; commercial, light industrial uses and Ethanac Road beyond to the north; Sherman Road to the east; and Trumble Road to the west. The Project area is generally located in the northeastern part of the City, within Riverside County, California (Figure 1). The Project area is comprised of ten parcels. The Project area is located approximately 0.25-mile (1,300 feet) east of Interstate 215 (I-215) and approximately 0.6 mile (2,900 feet) southwest of State Highway (SH) 74 (Figure 2).

The proposed Project proposes the development of one warehouse building, totaling approximately 829,755 square feet (sq. ft.) of warehouse space which includes 10,000 sq. ft. of office and 819,755 sq. ft. of ground floor warehouse on an approximately 41-acre site. The building is contemplated to have a structural height of approximately 60 feet and includes approximately 503 automobile parking spaces, approximately 258 truck trailer parking spaces, and approximately 111 dock high doors.

REGULATORY SETTING

Paleontological resources are limited, nonrenewable resources of scientific, cultural, and educational value and are afforded protection under federal and state laws and regulations. This study satisfies project requirements in accordance with state regulations, as well as guidelines and criteria specified by the Society of Vertebrate Paleontology (1995, 2010).

California Environmental Quality Act

CEQA is the principal statute governing environmental review of projects occurring in the state and is codified at Public Resources Code (PRC) Section 21000 et seq. CEQA requires lead agencies to determine if a proposed project would have a significant effect on the environment, including significant effects on paleontological resources. Guidelines for the Implementation of CEQA, as amended December 1, 2016 (Title 14, Chapter 3, California Code of Regulations 15000 et seq.), define procedures, types of activities, persons, and public agencies required to comply with CEQA.

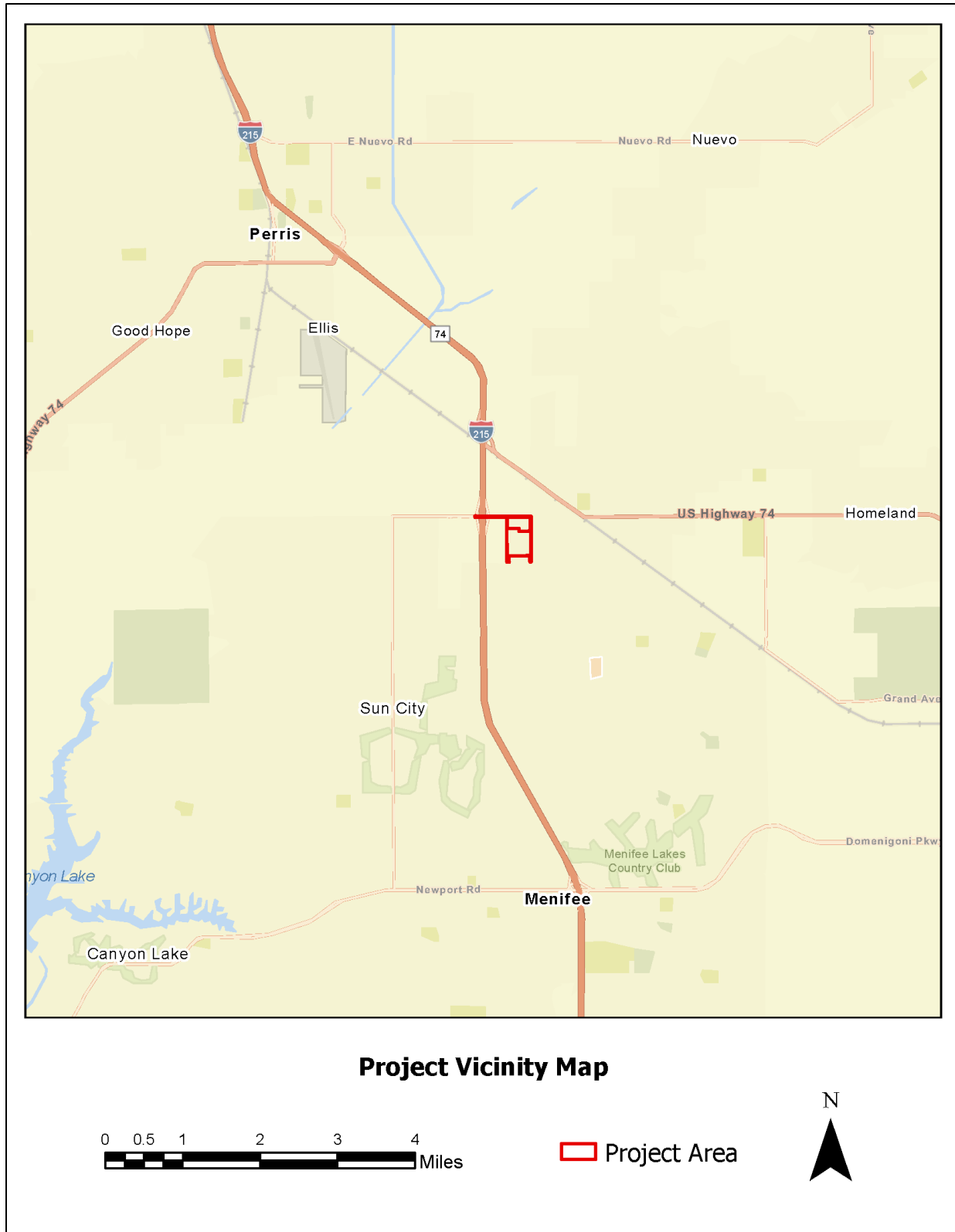


Figure 1: Project Vicinity Map

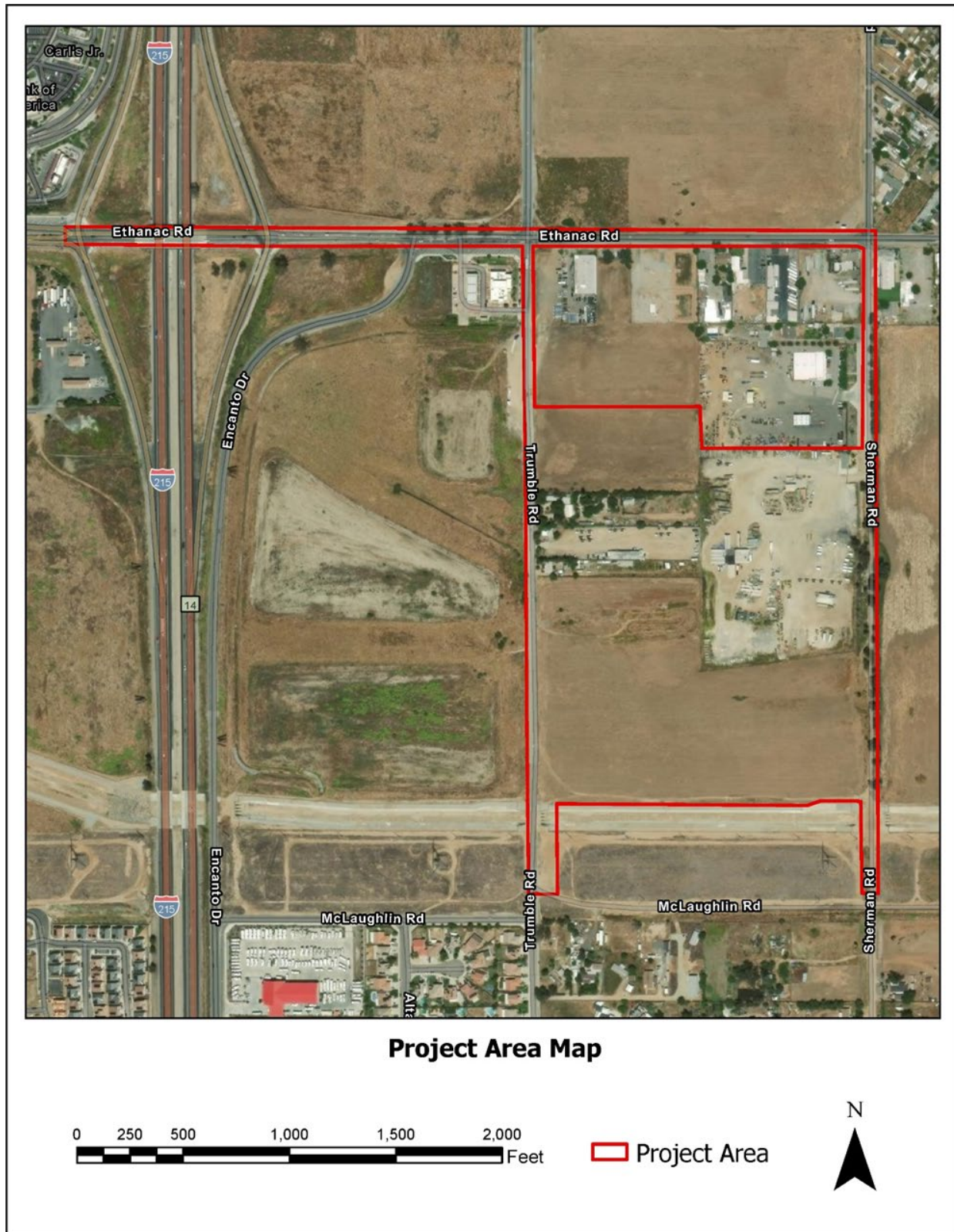


Figure 2: Project Area and Offsite Improvements Map

Public Resources Code Section 5097.5

Requirements for paleontological resource management are included in PRC Division 5, Chapter 1.7, Section 5097.5, and Division 20, Chapter 3, Section 30244. These statutes prohibit the removal, without permission, of any paleontological site or feature from lands under the jurisdiction of the state or any city, county, district, authority, or public corporation, or any agency thereof. Consequently, local agencies are required to comply with PRC Section 5097.5 for their own activities, including construction and maintenance, as well as for permit actions (e.g., encroachment permits) undertaken by others. PRC Section 5097.5 also establishes the removal of paleontological resources as a misdemeanor and requires reasonable mitigation of adverse impacts to paleontological resources from developments on public (state, county, city, and district) lands.

City of Menifee General Plan

The protection and preservation of paleontological resources within the City of Menifee are addressed in the City's General Plan (GP) within the Open Space and Conservation (OSC) Element (City of Menifee 2013, 2024) and the Draft Environmental Impact Report (EIR) prepared for the adoption and implementation of the GP (The Planning Center 2013). According to the GP, "paleontological and cultural resources are important for scientific, historic, and/or religious reasons to cultures, communities, groups, or individuals." As such, the City has established the goal, policies, and guidance for implementation to protect these resources. The specific policies listed within the GP focus on cultural resources, though their application to paleontological resources is implied. Furthermore, the GP includes a map that identifies paleontological resource sensitivity to directly address the paleontological resource category (Figure 3).

METHODS

All efforts made for the completion of this report were completed pursuant to requirements set forth in CEQA. This study is intended to identify whether paleontological resources are located within the Project area, whether any present paleontological resources are potentially significant pursuant to the above-referenced regulations and standards, and to develop specific recommendations that will address potential impacts to existing or potential paleontological resources. Tasks completed include:

- Paleontological record searches through the Western Science Center (WSC) and Natural History Museum of Los Angeles County (NHMLAC),
- Review of soil maps, geologic maps, and geotechnical data for paleontological sensitivity analysis,
- Review of University of California Museum of Paleontology (UCMP) online paleontological database,
- Review of historical maps and imagery to identify the extent of previous development and disturbance, and
- Development of recommendations for paleontological resources identified or potentially unrecorded within the Project area.

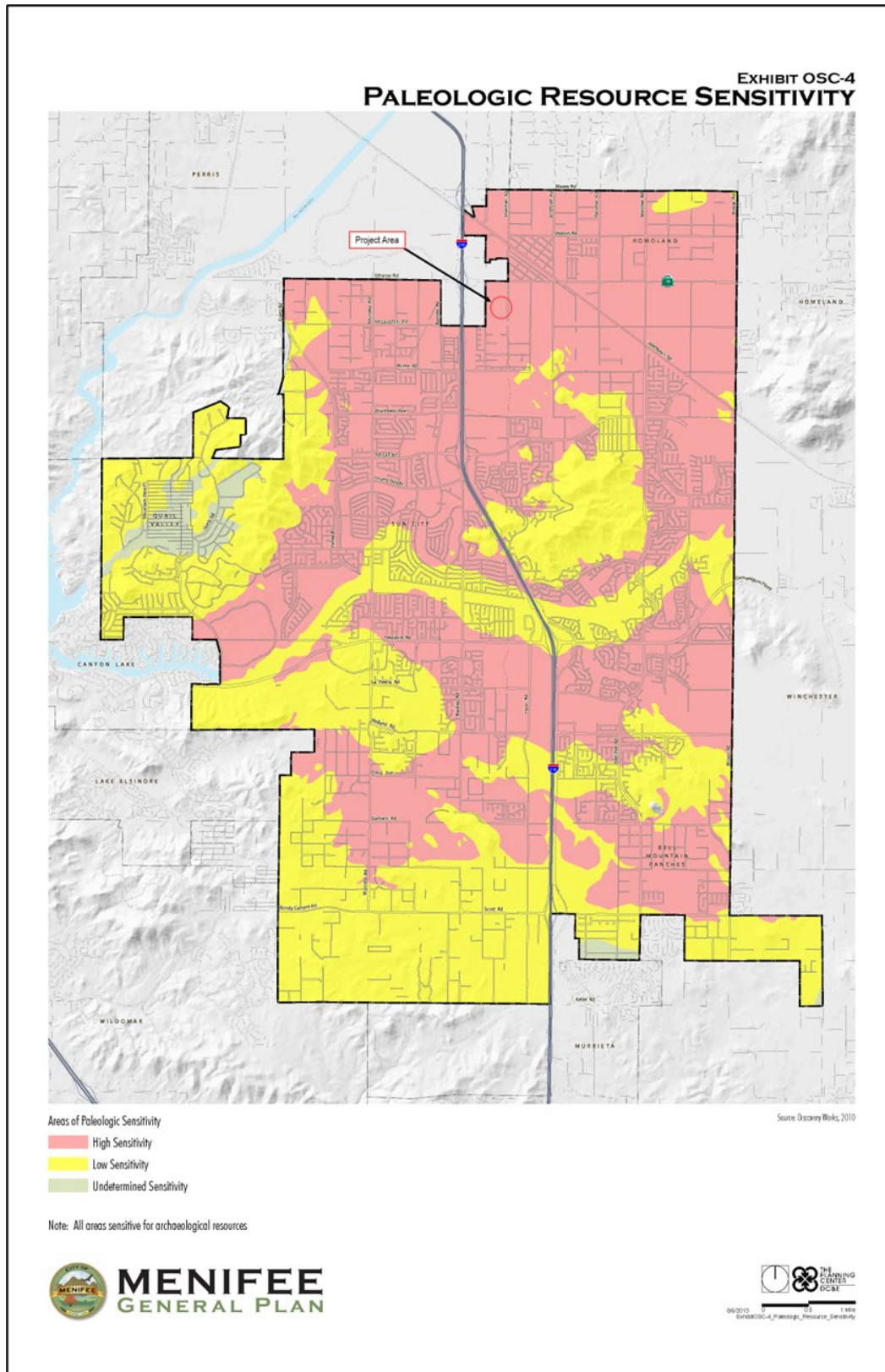


Figure 3: Project Area within the City of Menifee Paleontological Sensitivity Map

RESULTS

Environment and Geology

Kimley-Horn reviewed geologic maps, soil maps, geotechnical data, and literature to determine the paleontological sensitivity of the Project area. The Project area is located between the Perris Valley and Menifee Valley in western Riverside County. The San Jacinto River flows 2.3 miles to the west and offers the nearest natural water source, though it is now channelized. This region enjoys a semi-arid and Mediterranean climate with seasonal precipitation. No native vegetation currently inhabits the Project area, but this vicinity once supported diverse plant communities, such as native chaparral and coastal sage scrub. This area is characterized as a lower Sonoran Life Zone. Natural topography of the Project area is flat and situated at an elevation of 1425 feet above mean sea level (amsl). While the Project area is relatively flat, the City of Menifee is situated near many prominent hillsides, and its dynamic topography is one of its most identifiable features. However, no such geologic features are located in the immediate vicinity. Native soil composition at the Project area is comprised of primarily Monserate sandy loam with a small area in the southern portion comprised of Greenfield sandy loam (SoilWeb 2025). The nearest geological fault to the Project area is the Quaternary-aged Glen Ivy section of the Elsinore fault zone located approximately nine miles to the southwest (U.S. Geological Survey 2025). Geologic maps of the Romoland 7.5' quadrangle indicate that the Project area is underlain by late to middle Pleistocene alluvial fan deposits (Figure 4; Morton 2003). The Pleistocene epoch dates 2.58 million to 11,700 years ago. The deposits underlying the Project area are identified as Qof in Figure 4. The deposits are described by Morton (2003) as reddish brown, gravel and sand alluvial fan deposits that are indurated and commonly slightly dissected.

Kimley-Horn reviewed the geotechnical report that was prepared for the Project (Montes and Trazo 2025). The geotechnical investigation was conducted in three phases between 2020 and 2022 and involved a total of 17 borings to depths of 15-25 feet below ground surface. Artificial fill soils were encountered to depths of 2.5-4.5 feet below the surface in some of the borings. The authors reported that the fill soils generally consisted of medium dense to very dense silty fine sands, clayey fine sands, and fine sandy silts, with varying medium to coarse sand and clay content. Older alluvium dating to the Pleistocene epoch was encountered beneath the artificial fill soils or at the ground surface at all of the boring locations, extending to at least the maximum depth explored of 25 feet below ground surface. The older alluvium generally consisted of medium dense to very dense silty fine to coarse sands, clayey fine to coarse sands, and very stiff to hard silty clays, fine sandy clays, and clayey silts with varying medium to coarse sand content (Montes and Trazo 2025). Given that Pleistocene-age deposits are present at the surface or near the surface across the entire Project area, excavation for the proposed Project will reach these sediments that possess a high paleontological sensitivity.

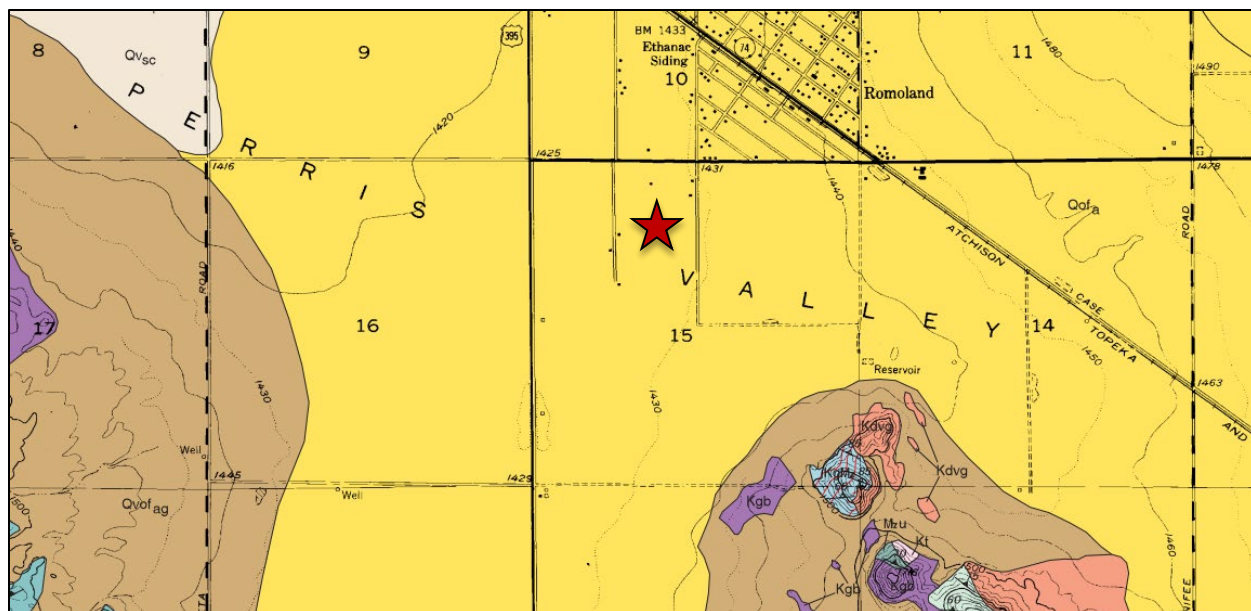


Figure 4: Geologic Map of the Project Area (Project Indicated with Red Star)

Record Searches

Kimley-Horn submitted record search requests for the Project area and vicinity to two paleontological repositories in southern California on May 30, 2025. The WSC responded on July 3, 2025, and the NHMLAC responded on June 8, 2025. A copy of their responses is included in **Appendix A** of this report. The results of the record search were negative for known specimens within the Project area. While there are no known fossil localities that lie directly within the Project area, the NHMLAC cited positive findings in sediments of the same composition in other areas of the Perris Valley and Menifee Valley. Table 1 summarizes the closest known findings within LACMNH records.

Table 1: Previously Recorded Fossil Localities at NHMLAC nearest to the Project

Locality Number	Location	Formation	Taxa	Depth	Distance from Project
LACM VP 5168	East Bay Section of Canyon Lake	Unknown formation (Pleistocene; clay)	Horse (<i>Equus</i>)	Unknown	Approximately 5 miles southwest of the Project
LACM VP 6059, CIT 570 - 572	Lake Elsinore and vicinity	Unknown formation (Pleistocene)	Horse (<i>Equus</i>); peccary (<i>Platygonus</i>); camel (<i>Camelops</i>)	Unknown	Approximately 9 miles southwest of the Project
LACM VP 7261	Skinner Reservoir, Auld Valley	Unknown formation (Pleistocene,	Elephant clade (<i>Proboscidea</i>); ungulate (<i>Ungulata</i>)	Unknown	Approximately 12 miles

Locality Number	Location	Formation	Taxa	Depth	Distance from Project
		arenaceous silt)			southeast of the Project
LACM VP 4540	Vicinity of Jackrabbit Trail & Gilman Springs Road; San Jacinto Valley	Unnamed formation (Pleistocene, gravel pit)	Horse Family (<i>Equidae</i>)	Unknown	Approximately 12 miles northeast of the Project
LACM VP 7456	Hwy 79 and Butterfield Stage Rd., Pauba Valley near Temecula	Alluvium (interbedded silty clay, sandy silt, and silty to coarse-grained sand)	Garter snake (<i>Thamnophis</i>); pocket gopher (<i>Thomomys</i>); deer mouse (<i>Peromyscus</i>); snails (gastropods)	Unknown, collected during earth-moving activities for construction	Approximately 18 miles south of the Project

It is important to note that the management of paleontological records is not delineated based on geographical region and, as such, there is no single repository or specific group of repositories responsible for the management of paleontological records within Riverside County. As such, it is possible that additional records exist for the general vicinity of the Project area that are not included in the records provided by the WSC or NHMLAC. However, given the regional proximity of the repositories to the Project area, as well as their long operating history in the region and emphasis on paleontological resource management, it is likely that a higher percentage of records for the vicinity of the Project area are held at these locations. As such, the results in Table 1 are generally considered a strong sample of fossil localities for the region.

Database Review

Kimley-Horn reviewed the UCMP online paleontological database to identify any previously recorded paleontological resources in or near the Project area. The database can be searched by county, geological age, and/or formation. As such, Kimley-Horn reviewed the records for Riverside County that date to the Pleistocene epoch. The UCMP reported 28 fossil localities throughout Riverside County that date to the Pleistocene epoch as shown in Table 2.

Table 2: Previously Recorded Fossil Localities at UCMP from the Pleistocene Epoch in Riverside County

Locality Number	Locality Name	Formation	Museum Specimens
-3240	Fossil Canyon	Bautista Beds	<i>Equus bautistensis</i>
-3241	Horse And Deer Pits	Bautista Beds	<i>Odocoileus; Equus bautistensis</i>
-3242	Pedata Pit	Bautista Beds	<i>Equus bautistensis</i>
-3243	Blacksand Station	Bautista Beds	<i>Equus bautistensis; Megalonyx; Antilocapra; Odocoileus; Tapirus merriami; Unidentified Mammalia</i>
-3244	Blacksand Top	Bautista Beds	<i>Equus bautistensis</i>
-3245	Slide Region	Bautista Beds	<i>Equus bautistensis; Capromeryx</i>
-3246	Highest Hill	Bautista Beds	None
-3247	Meeks Neighborhood	Bautista Beds	<i>Odocoileus; Antilocapra; Lepus</i>
B6352	N/A	N/A	None
P363	Lakeview	N/A	None
PA1236	CPV Sentinel Energy I	Ocotillo Conglomerate	None
PA1237a	CPV Sentinel Energy IIA	Ocotillo Conglomerate	None
PA1237b	CPV Sentinel Energy IIB	Ocotillo Conglomerate	None
PA1238	CPV Sentinel Energy IV	Ocotillo Conglomerate	None
PA1239	CPV Sentinel Energy V	Ocotillo Conglomerate	None
PA1240	CPV Sentinel Energy VI	Ocotillo Conglomerate	None
PA1241a	CPV Sentinel Energy VII	Ocotillo Conglomerate	None
PA1241b	CPV Sentinel Energy VIII	Ocotillo Conglomerate	None
RV6875	RF-19	Temecula Arkose	None
RV8601	Corona East	N/A	<i>Microtus californicus; Microtus; Neotoma; Unidentified Mammalia</i>
RV9612	San Timoteo Paleomag. Site 202A	San Timoteo	<i>Sorex leahyi; Sylvilagus hibbardi; Thomomys gidleyi; Perognathus; Peromyscus hagermanensis; Peromyscus complexus; Sigmodon minor; Mimomys</i>
V6004	Blythe	N/A	None

Locality Number	Locality Name	Formation	Museum Specimens
V7006	Carr Ranch	N/A	<i>Gopherus</i>
V7007	Bastenchury Ranch	N/A	None
V65248	Riverside	N/A	<i>Mammuthus</i>
V65338	Bautista Fm General	Bautista Beds	<i>Equus bautistensis</i> ; <i>Antilocapra</i> ; <i>Equus</i> ; Unidentified <i>Mammalia</i>
V67109	Whittier Canon	Bautista Beds	None
V99828	Blythe Energy Turtles	Chemehuevi	<i>Gopherus agassizii</i>

Kimley-Horn also reviewed available historic and topographic imagery to ascertain the level of prior and existing disturbance that would impact the potential for paleontological resources. Historical data and maps indicate that the Project area had a road constructed in the general vicinity of the current alignment by 1943 (Historic Aerials 2025; Figure 5). The earliest available historic imagery from 1961 shows the roads within the Project area, multiple single-family residences within the Project area, and intensive agricultural development (Figure 6). Imagery from 1985 shows relatively few changes to the Project area apart from landscaping changes to the residential parcels (Figure 7). By 2009, much of the Project area had been converted from agricultural to industrial/commercial use (Figure 8). Few changes to the Project area have occurred between 2009 and present day. In summary, the entire Project area has been subject to some surface-level disturbances associated with agricultural use. Additionally, approximately half of the Project area has been developed for residential and industrial use, which involved grading of the parcels.

CONCLUSIONS AND RECOMMENDATIONS

In conclusion, Kimley-Horn did not identify any previously recorded fossil localities in the Project area but determined that the Project area maintains a high sensitivity for unrecorded paleontological resources. The Project area is underlain by Pleistocene-age geologic deposits, and fossil localities have been identified in similar deposits across the Perris Valley and Menifee Valley. The City of Menifee General Plan identifies the Project area as a location of high paleontological sensitivity (City of Menifee 2024). The Project's geotechnical investigation revealed that the Pleistocene deposits are present at the surface or near the surface (2.5-4.5 feet below surface) across the entire Project area. Excavation for the proposed Project will reach these sediments that possess a high paleontological sensitivity. Any paleontological resources identified in the Project area would be considered scientifically significant.

Therefore, mitigation measures are recommended for a Paleontological Resources Impact Mitigation Program, paleontological monitoring, and data recovery. With these mitigation measures in place, any potential impacts to paleontological resources would be less than significant. An example of such mitigation measures are included below:

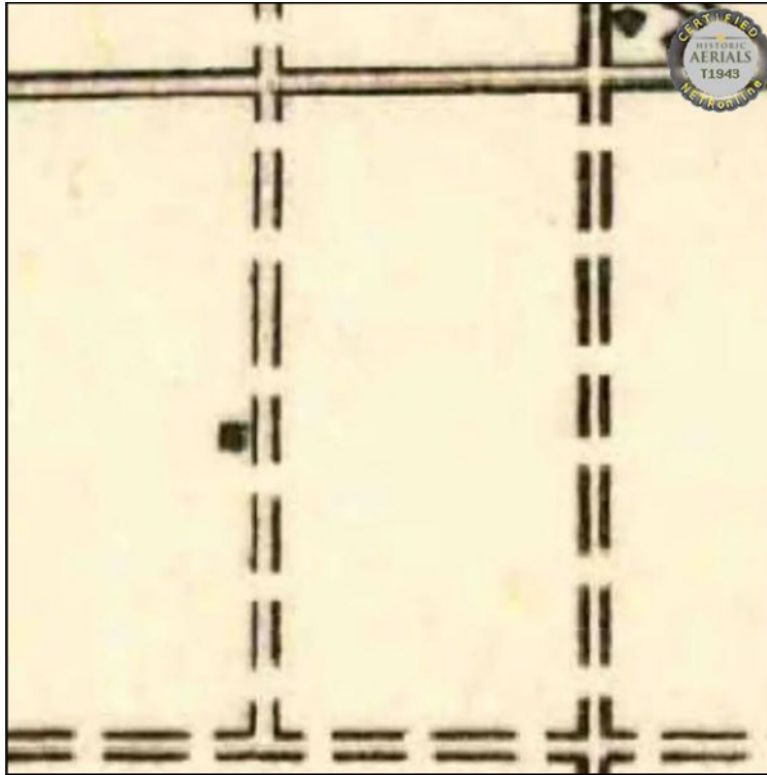


Figure 5: 1943 Topographic Map



Figure 6: 1961 Historic Aerial



Figure 7: 1985 Historic Aerial



Figure 8: 2009 Historic Aerial

- **GEO-1 - Paleontological Resources Impact Mitigation Program**

The Applicant will submit a Paleontological Resources Impact Mitigation Program (PRIMP) prepared by a qualified paleontologist to the City of Menifee prior to the issuance of a grading permit. A qualified paleontologist is defined as an individual with an M.S./M.A. or Ph.D. in paleontology or geology who is familiar with paleontological procedures and techniques, and who is knowledgeable in the geology and paleontology of the area.

- **GEO-2 – Paleontological Monitoring**

A qualified paleontologist will attend preconstruction meetings to consult with the grading and excavation contractors concerning planned depths, excavation schedules, paleontological field techniques, and safety issues. In addition, all onsite construction personnel will receive Worker Education and Awareness Program (WEAP) training prior to the commencement of excavation work. All ground-disturbing activities associated with Project construction occurring within previously undisturbed fossil bearing formations will be monitored by a qualified paleontologist or qualified paleontological monitor. A paleontological monitor is defined as an individual who has experience in the collection and salvage of fossil materials and works under the direction of a qualified paleontologist. If fossils are discovered, the paleontologist (or paleontological monitor) will recover them. In most cases, this fossil salvage can be completed in a short period of time; however, some fossil specimens, such as a complete large mammal skeleton, may require an extended salvage period. In these instances, the paleontologist (or paleontological monitor) will be allowed to temporarily direct, divert, or halt grading to allow recovery of fossil remains in a timely manner. Because of the potential for the recovering of small fossil remains, such as isolated mammal teeth, it may be necessary to set up a screen-washing operation on site.

- **GEO-3 – Paleontological Data Recovery**

Fossil remains collected during the monitoring and salvage portion of the program will be cleaned, repaired, sorted, and catalogued. Prepared fossils, along with copies of all pertinent field notes, photos, and maps, will be deposited (as a donation) in a scientific institution with permanent paleontological collections located within Riverside County (or, if no repository is available, adjacent Counties). A final data recovery report will be completed that outlines the results of the paleontological monitoring program. This report will include discussions of the methods used, stratigraphic section(s) exposed, fossils collected, and significance of recovered fossils. The report will be submitted to the City upon completion.

REFERENCES

City of Menifee

- 2013 General Plan. Open Space and Conservation Element.
2024 Open Space & Conservation Element Osc-3: Natural Landforms.
<https://www.cityofmenifee.us/874/OSC-3-Natural-Landforms#:~:text=The%20City%20of%20Menifee%20is,traveling%20northward%20across%20McCall%20Boulevard.>

Historic Aerials

- 2025 Historic Aerial Viewer. <https://www.historicaerials.com/viewer.>

Montes, Pablo Jr., and Robert G. Trazo

- 2025 Update Geotechnical Investigation Heritage Valley Business Center - Building 2, Trumble Road, South of Ethanac Road, Menifee, California for Core5 Industrial Partners.
Prepared by Southern California Geotechnical.

Morton, Douglas M.

- 2003 Geologic Map and Digital Database of the Romoland 7.5' Quadrangle, Riverside County, California: U. S. Geological Survey Open-File Report 03-102,
[http://pubs.usgs.gov/of/2003/0102/.](http://pubs.usgs.gov/of/2003/0102/)

SoilWeb

- 2025 SoilWeb. [https://casoilresource.lawr.ucdavis.edu/gmap/.](https://casoilresource.lawr.ucdavis.edu/gmap/)

The Planning Center

- 2013 Draft Environmental Impact Report for the City of Menifee General Plan.

U.S. Geological Survey

- 2025 U.S. Quaternary Faults.
<https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=5a6038b3a1684561a9b0aadf88412fcf.>

Appendix A

Paleontological Record Search Results



July 3rd, 2025

Kimley-Horn
Jamie Nord
3801 University Ave, Ste. 300
Riverside, CA 92501

Hello,

This letter presents the results of a record search conducted for the Heritage Valley Project located in the City of Menifee, Riverside County, CA. The project is located north of McLaughlin Road, south of Ethanac Road, east of Trumble Road, and west of Sherman Road on Township 5 South, Range 3 West, Section 18 on the *Romoland, CA* USGS 7.5 minute quadrangle.

The geologic units underlying the project area is mapped as alluvial deposits from the late to middle Pleistocene epoch (Morton, Bovard, and Morton 2003). Pleistocene alluvial units are considered to be highly paleontologically sensitive. The Western Science Center does not have localities within the project area or within a 1 mile radius; however it does have localities in similarly mapped units across Southern California.

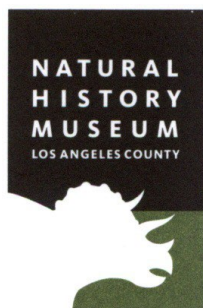
Any fossil specimen from the Heritage Valley Project would be scientifically significant. Excavation activity associated with the development of the project area would impact the paleontologically sensitive Pleistocene alluvial units, and it is the recommendation of the Western Science Center that a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

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

Sincerely,

A handwritten signature in black ink, appearing to read 'Brittney Stoneburg', with a stylized, flowing script.

Brittney Elizabeth Stoneburg, MSc
Collections Manager



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Research & Collections

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June 8, 2025

Kimley-Horn
Attn: Jamie Parra

re: Paleontological resources records search for the Heritage Valley Project

Dear Jamie:

I have conducted a thorough search of our paleontology collection records for the locality and specimen data for proposed development at the Heritage Valley Project area as outlined on the portion of the Romoland USGS topographic quadrangle map that you sent to me via e-mail on May 30, 2025. We do not have any fossil localities that lie directly within the proposed project area, but we do have fossil localities nearby from similar sedimentary deposits that may occur in the proposed project area, either at the surface or at depth.

The following table shows the closest known localities in the collection of the Natural History Museum of Los Angeles County (NHMLA).

Locality Number	Location	Formation	Taxa	Depth
LACM VP 5168	East Bay Section of Canyon Lake	Unknown formation (Pleistocene; clay)	Horse (<i>Equus</i>)	Unknown
LACM VP 6059, CIT 570 - 572	Lake Elsinore and vicinity	Unknown formation (Pleistocene)	Horse (<i>Equus</i>); peccary (<i>Platygonus</i>); camel (<i>Camelops</i>)	Unknown
LACM VP 7261	Skinner Reservoir, Auld Valley	Unknown formation (Pleistocene, arenaceous silt)	Elephant clade (Proboscidea); ungulate (Ungulata)	Unknown
LACM VP 7456	Hwy 79 and Butterfield Stage Rd., Pauba Valley near Temecula	Alluvium (interbedded silty clay, sandy silt, and silty to coarse-grained sand)	Garter snake (<i>Thamnophis</i>); pocket gopher (<i>Thomomys</i>); deer mouse (<i>Peromyscus</i>); snails (gastropods)	Unknown, collected during earth-moving activities for construction
LACM VP 4540	Vicinity of Jackrabbit Trail & Gilman Springs Road; San Jacinto Valley	Unnamed formation (Pleistocene, gravel pit)	Horse Family (Equidae)	Unknown

VP, Vertebrate Paleontology; IP, Invertebrate Paleontology; bgs, below ground surface

This records search covers only the records of the NHMLA. It is not intended as a paleontological assessment of the project or project area for the purposes of CEQA or NEPA. Potentially fossil-bearing units are present in the project area, either at the surface or in the subsurface. As such, NHMLA recommends that a paleontological assessment of the project area be conducted by a paleontologist meeting Federal (43 Code of Federal Regulations Part 49.110) or Society of Vertebrate Paleontology standards.

Sincerely,

A handwritten signature in black ink that reads "Alyssa Bell". The signature is written in a cursive, flowing style. The letters are connected, and the overall appearance is that of a personal or professional signature.

Alyssa Bell, Ph.D.
Natural History Museum of Los Angeles County