GEOTECHNICAL INVESTIGATION MAJESTIC FREEWAY BUSINESS CENTER-BUILDINGS 14A &14B

NWC Harvill Avenue and Perry Street
Riverside County, California
for
Commerce Construction Co., L.P.

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November 24, 2021 (Revised January 11, 2022)

Commerce Construction Co., L.P.
13191 Crossroads Parkway North, 6th Floor
City of Industry, California 91746

Attention: Mr. Matthew Vawter
Vice President – District Manager

Project No.: **21G251-1R**

Subject: **Geotechnical Investigation**
Majestic Freeway Business Center-Buildings 14A & 14B
NWC Harvill Avenue and Perry Street
Riverside County (Perris), California

Dear Mr. Vawter:

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

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

Respectfully Submitted,
SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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1.0 EXECUTIVE SUMMARY

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

Geotechnical Design Considerations

- Artificial fill soils were encountered at Boring Nos. B-1, B-5, and B-8, extending from the ground surface to depths of 1½ to 4½± feet. The existing fill soils are considered to represent undocumented fill.
- Older alluvial soils were encountered at the ground surface and beneath the artificial fill soils and generally consist of silty sands and clayey sands, which possess variable strength.
- Val Verde Tonalite bedrock was encountered beneath the older alluvium at all of the boring locations, with the exception of Boring No. B-9 which was terminated within the older alluvial soils, extending at least to the maximum depth explored of 25± feet.
- The undocumented fill soils and upper-portion of the older 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 structures.
- Remedial grading will be necessary to remove the undocumented fill soils in their entirety and the upper portion of the older alluvial soils and replace these materials as compacted structural fill soils.

Site Preparation Recommendation

- 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 areas to remove the undocumented fill soils, which extend to depths of 1½ to 4½± feet at the boring locations, in their entirety. At a minimum, the building pad areas should be overexcavated to a depth of at least 8 feet below existing grade and to a depth of at least 5 feet below proposed pad grade, whichever is greater. Overexcavation within the foundation areas is recommended to extend to a depth of at least 5 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 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.
- The new parking area subgrade soils are recommended to be scarified to a depth of 12± inches, thoroughly moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Foundation Design Recommendations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 3,000 lbs/ft^2 maximum allowable soil bearing pressure.

- Reinforcement consisting of at least two (2) No. 5 rebars (1 top and 1 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab Design Recommendations

- Conventional Slab-on-Grade: minimum 6 inches thick.
- Modulus of Subgrade Reaction: k = 150 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 = 46) FOR BUILDING 14A					
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	3	4	6	6	8
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 46, f_c' = 3500 psi) FOR BUILDING 14A				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	7½
Compacted Subgrade (95% minimum compaction)	12	12	12	12

ASPHALT PAVEMENTS (R = 27) FOR BUILDING 14B					
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	11	12	14
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 27, f_c' = 3500 psi) FOR BUILDING 14B				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	7½
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. 21P445R, dated October 8, 2021 and Change Order No. 21G253-COR dated December 13, 2021. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slabs, and parking lot pavements along with site preparation recommendations and construction considerations for the currently 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 site is located at the northwest corner of Harvill Avenue and Perry Street in an unincorporated portion of Riverside County near Perris, California. The site is bounded to the north by Commerce Center Drive, to the west by Seaton Avenue, to the south by Perry Street, and to the east by Harvill Avenue. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in Appendix A of this report.

The site consists of a trapezoidal-shaped parcel, 18.96± acres in size. The site is presently vacant and undeveloped. The ground surface cover consists of exposed soil with sparse to moderate native grass and weed growth. Two small trees are present in the eastern area of the site.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth, and visual observations made at the time of the subsurface investigation, the overall site topography gently slopes downward to the northeast at a gradient of 2 to 3± percent. Based on our review of readily available historical aerial photographs and Google Earth, the site appears to have been previously rough graded, between 1978 and 1994. The northern area of the site includes steps sloping downward to the east. This area includes localized hills with a 5 to 6± foot difference in elevation.

3.2 Proposed Development

Based on the site plan provided to our office, the site will be developed with two (2) commercial/industrial buildings, identified as Buildings 14A and 14B. Building 14A will be located in the western half of the site and will be 200,152± ft^2 in size. Building 14B will be located in the eastern half of the site and will be 136,602± ft^2 in size. Dock-high doors will be constructed along a portion of the eastern building walls. The buildings will be surrounded by Portland cement concrete pavements in the loading dock areas, asphaltic concrete pavements in the parking and drive lane areas, concrete flatwork, and landscape planter areas throughout.

Detailed structural information has not been provided. We assume that the new buildings will be single-story structures of tilt-up concrete construction, typically supported on conventional shallow foundation systems with concrete slab-on-grade floors. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 4 to 7 kips per linear foot, respectively.

No significant amounts of below-grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. At the time of this report precise grading plans were not available. Based on the assumed topography, preliminary cuts and fills of up to 5 to 7± feet are expected to be necessary to achieve the proposed site grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of ten (10) borings and were advanced to depths of 10 to 20± feet below the existing site grades. All of the borings were logged during the drilling and excavation by members of our staff.

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

The approximate locations of the borings are indicated on the Boring Location Plan, included as Plate 2 in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Artificial Fill

Artificial fill soils were encountered at the ground surface at Boring Nos. B-1, B-5, and B-8, extending to depths of 1½ to 4½± feet below the existing site grades. The artificial fill soils generally consist of medium dense to very dense silty sands. The fill soils possess a disturbed and mottled appearance, resulting in their classification as artificial fill.

Older Alluvium

Older alluvium was encountered at the ground surface or beneath the fill soils, at all of the boring locations. The older alluvium generally consists of medium dense to very dense silty sands and clayey sands with varying amounts of silt, clay, and bedrock fragments. The older alluvium also possesses calcareous veining and nodules and some of the recovered samples were observed to be weakly to moderately cemented. The older alluvium extends to depths of 5½ to 12± feet at most of the boring locations, with the exception of Boring No. B-9 which was terminated in older alluvium at a depth of 10± feet.

Bedrock

Val Verde Tonalite bedrock was encountered beneath the older alluvium at all of the boring locations, with the exception of Boring No. B-9 which was terminated in the older alluvium at a depth of 10± feet. The bedrock consists of medium dense to very dense, gray brown fine to coarse grained tonalite. These materials are generally weathered and friable throughout the depths explored at the site. Tonalite bedrock materials extend to at least the maximum depth explored of 25± feet below the existing site grades.

Groundwater

Groundwater was encountered at Boring Nos. B-1 and B-5 within the weathered Val Verde Tonalite at depths of 16 and 22± feet, respectively. Based on the moisture contents of the recovered soil samples, the lack of groundwater present in all the other borings, and our experience with other projects in the area, a perched groundwater condition is believed to exist in the northern portion of the site, at the time of the subsurface investigation.

As a part of our research, we reviewed available groundwater data in order to determine groundwater levels for the site. Recent water level data was obtained from the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. The nearest monitoring well on record is located 0.6± miles northeast of the site. Water level readings within this monitoring well indicate a groundwater level 67.1± feet below the ground surface in March 2021.

Water level data was also obtained from the California State Water Resources Control Board, GeoTracker, website, http://geotracker.waterboards.ca.gov/. The nearest monitoring well on record is located 1.1± miles north of the site. Water level readings within this monitoring well indicate a high groundwater level of 12.7± feet below the ground surface in June 2006.

4.3 Geologic Conditions

Regional geologic conditions were obtained from the Geologic Map of the Steele Peak 7.5' Quadrangle, Riverside County, California, by Douglas M. Morton published by the California Department of Mines and Geology and United States Air Force, 2001. This map indicates that the site is underlain by early Pleistocene (Map Symbol Qvof) old alluvial valley deposits. Morton describes these deposits as predominantly composed of moderately indurated, slightly dissected, sandy alluvium, containing lesser silt, and clay-bearing alluvium. A portion of this map indicating the location of the subject site is included as Plate 3 in Appendix A.

Bedrock materials were encountered at all of the boring locations, depths of 5½ to 12± feet, with the exception of Boring No. B-9. Based on the mapping of the geologic formations present near the subject site, it is our opinion that the near-surface older alluvium is underlain by Val Verde tonalite (Map Symbol Kvt) formation. Morton describes this formation as gray-weathering, relatively homogeneous, massive to well-foliated, medium to coarse grained, hypautomorphic granular biotite-hornblende tonalite. The geologic conditions at the site are consistent with the mapped geologic conditions.

5.0 LABORATORY TESTING

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

Classification

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. The field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring and Trench 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 were tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-5 in Appendix C of this report.

Expansion Index

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

Sample Identification	**Expansion Index**	**Expansive Potential**
B-1 @ 0 to 5 feet	2	Very Low
B-10 @ 0 to 5 feet	8	Very Low

Maximum Dry Density and Optimum Moisture Content

Representative bulk samples have been tested for their maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plates C-6 and C-7 in Appendix C 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.

Soluble Sulfates

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

Sample Identification	**Soluble Sulfates (%)**	**Sulfate Classification**
B-10 @ 0 to 5 feet	0.001	Not Applicable (S0)

Corrosivity Testing

A representative sample of the near-surface soils was submitted to a subcontracted corrosion engineering laboratory to identify potentially 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.

Sample Identification	**Saturated Resistivity (ohm-cm)**	**pH**	**Chlorides (mg/kg)**	**Nitrates (mg/kg)**
B-10 @ 0 to 5 feet	6,000	6.7	6.4	5.3

R-value

R (resistance)-value testing was conducted on two (2) representative samples of the near-surface soils obtained from the subject site. The R-values were determined in accordance with CA Test Method 301. This test provides a measure of the pavement support characteristics of the soils, and is used in the pavement thickness design procedure. The result of the R-value testing is as follows:

Sample ID	**R-Value**
R-1 @ 0 to 5 feet	46
R-2 @ 0 to 5 feet	27

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

6.1 Seismic Design Considerations

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

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Furthermore, SCG did not identify any evidence of faulting during the geotechnical investigation. Therefore, the possibility of significant fault rupture on the site is considered to be low.

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 2019 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. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

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

The 2019 CBC Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool, a web-based software application available at the website www.seismicmaps.org. This software application calculates seismic design parameters in accordance with several building code reference documents, including ASCE 7-16, upon which the 2019 CBC is based. The application utilizes a database of risk-targeted maximum considered earthquake (MCE_R) site accelerations at 0.01-degree intervals for each of the code documents. The tables below were created using data obtained from the application. The output generated from this program is included as Plate E-1 in Appendix E of this report.

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.500
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.564
Site Class	---	C
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.800
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.810
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.200
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.540

Based on the presence of dense to very dense soils and bedrock, generally encountered in a majority of the boring locations, we have classified this site as Site Class C in accordance with ASCE 7-16, Chapter 20. Additionally, ASCE 7-16 allows for the determination of site-specific seismic design parameters in accordance with ASCE 7-16 Chapter 21 instead of using the code derived values presented above. Depending upon structural considerations, and the site classification of Site Class C, it may be desirable to perform a ground motion hazard analysis for this site in accordance with ASCE 7-16 Section 21.2. At the client's request, SCG can prepare a proposal to perform a ground motion hazard analysis.

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. Additionally, 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 alluvium and no evidence of a long-term groundwater table within the depths explored by the borings. Based on these considerations, liquefaction is not considered to be a design concern for this project.

6.2 Geotechnical Design Considerations

General

Artificial fill soils were encountered at some of the boring locations, extending from the ground surface to depths of 1½ to 4½± feet. These soils possess a mottled and disturbed appearance. Additionally, no documentation regarding the placement and compaction of these soils has been provided. The fill soils are therefore considered to be undocumented fill. Therefore, remedial grading is considered warranted within the proposed building area in order to remove all of the undocumented fill soils in their entirety and portion of the near-surface native alluvial soils, and replace these materials as compacted structural fill soils.

Settlement

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

Expansion

The near-surface soils generally consist of silty sands and clayey sands. Laboratory testing indicates that these materials have a very low expansion potential (EI = 2 and 8). Additionally, the near surface bedrock materials are composed of granite and do not possess appreciable plasticity. Based on these conditions, no design considerations related to expansive soils are considered warranted for this site. It is recommended that additional expansion index testing be conducted during subsequent geotechnical investigation and at the completion of rough grading to verify the expansion potential of the as-graded building pads.

Soluble Sulfates

The results of the soluble sulfate testing indicated a sulfate concentration of approximately 0.001 percent for the selected samples of the near-surface soils. This concentration is considered to be “not applicable” (S0) with respect to the American Concrete Institute (ACI) Publication 318-14 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building areas.

Corrosion Potential

The results of laboratory testing indicate that the on-site soils possesses a saturated resistivity value of 6,000 ohm-cm, and a pH value of 6.7. The test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Sulfides and redox potential are factors that are also used in the evaluation procedure. We have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH, and moisture content. Based on these factors, and utilizing the DIPRA procedure, the on-site soils are not considered to be corrosive to ductile iron pipe. Therefore, polyethylene protection is not expected to be required for cast iron or ductile iron pipes.

Based on American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary, reinforced concrete that is exposed to external sources of chlorides requires corrosion protection for the steel reinforcement contained within the concrete. ACI 318 defines concrete exposed to moisture and an external source of chlorides as “severe” or exposure category C2. ACI 318 does not clearly define a specific chloride concentration at which contact with the adjacent soil will constitute a “C2” or severe exposure. However, the Caltrans Memo to Designers 10-5, Protection of Reinforcement Against Corrosion Due to Chlorides, Acids and Sulfates, dated June 2010, indicates that soils possessing chloride concentrations greater than 500 mg/kg are considered to be corrosive to reinforced concrete. The results of the laboratory testing indicate a chloride concentration of 6.4 mg/kg. Although the soils contain some chlorides, we do not expect that the chloride concentrations of the tested soils are high enough to constitute a “severe” or C2 chloride exposure. Therefore, a chloride exposure category of C1 is considered appropriate for this site.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested sample possesses a nitrate concentration of 5.3 mg/kg. Based on this test result, the on-site soils are not 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 a more thorough evaluation of these test results.

Shrinkage/Subsidence

Removal and recompaction of the artificial fill and near-surface native soils is estimated to result in an average shrinkage of 4 to 14 percent. Shrinkage estimates for the individual samples range between 0 and 19 percent based on the results of density testing and the assumption that the on-site soils will be compacted to about 92 percent of the ASTM D-1557 maximum dry density. 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 trench 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 recommended that we be provided with copies of the preliminary grading and foundation plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

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

Site Stripping

Initial site preparation should include stripping of any surficial vegetation. This includes the removal of the sparse native grass, weeds, and shrubs present at the site. These materials should be disposed of off-site. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pads

Remedial grading will be necessary within the proposed building pad areas to remove the existing undocumented fill soils and a portion of the variable strength older alluvium. The fill soils extend to depths of 1½ to 4½± feet at the boring locations.

In addition, the overexcavation is also recommended to extend to a depth of at least 8 feet below existing grade and 5 feet below proposed building pad subgrade elevations, whichever is greater. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 5 feet below proposed foundation bearing grade.

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

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

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

The building pad areas may then be raised to grade with previously excavated soils or imported, very low expansive structural fill. All structural fill soils present within the proposed building area should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of proposed retaining and non-retaining site walls should be overexcavated to a depth of at least 3 feet below foundation bearing grade and replaced as compacted structural fill. Any undocumented fill soils within any of these foundation areas should be removed in their entirety. Erection pads for concrete tilt-up walls are considered part of the foundation system, and the recommended overexcavation should also be performed beneath erection pads. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

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

Treatment of Existing Soils: Flatwork, Parking and Drive Areas

Based on economic considerations, overexcavation of the existing soils in the new flatwork, 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 flatwork, parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations.

The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. Any such materials should be removed to a level of firm and unyielding soil. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to at least 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

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

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 0 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2019 CBC and the grading code of the city of Perris and/or the county of Riverside.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

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

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Perris and/or the county of Riverside. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

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

6.4 Construction Considerations

Excavation Considerations

Some of the near-surface soils generally consist of moderate strength silty sands and clayey sands. Some of these materials may be subject to moderate to severe caving within shallow excavations. Where caving does occur, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 2h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing unless founded in unweathered bedrock. Temporary excavation slopes should be made no steeper than 1h:1v in unweathered or slightly weathered bedrock. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

The near-surface possess appreciable silt and clay content. These soils may become unstable if exposed to significant moisture infiltration or disturbance by construction traffic. If grading occurs during a period of relatively wet weather, an increase in subgrade instability in localized areas should also be expected. The site should, therefore, be graded to prevent ponding of surface water and to prevent water from running into excavations.

Groundwater

Groundwater was encountered at Boring Nos. B-1 and B-5 within the weathered La Sierra Tonalite at depths of 16 and 22± feet, respectively. Therefore, groundwater is not expected to impact the grading or foundation construction activities. However, as previously stated, a perched groundwater condition is believed to exist in the northern portion of the site. If excavations to a depth near the encountered perched conditions are expected, or water intrusion is encountered during construction, consideration should be given to installing a cutoff trench.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pads will be underlain by structural fill soils used to replace existing undocumented fill soils. These new structural fill soils are expected to extend to depths of at least 5 feet below proposed foundation bearing grade. Based on this subsurface profile, the proposed structures may be supported on conventional shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 3,000 lbs/ft^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Two (2) No. 5 rebars (1 top and 1 bottom).
- 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. 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, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

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

Estimated Foundation Settlements

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

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: 3000 lbs/ft^3
- Friction Coefficient: 0.30

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 3,000 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floors of the proposed structures may be constructed as conventional slabs-on-grade supported on newly placed structural fill, extending to a depth of at least 5 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: k = 150 psi/in.
- Minimum slab reinforcement: Reinforcement is not expected to be required for geotechnical conditions. 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 area of the proposed slab where such moisture floor coverings will be used. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below

the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

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

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

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

6.7 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 as well as in the dock-high portions of the building. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near surface soils generally consist of silty sands with occasional clayey sands. Based on their classifications, the silty sand and clayey sands are expected to possess a friction angle of at least 30 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density.

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

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type: On-site Sands
Internal Friction Angle (ϕ)		30°
Unit Weight		133 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	45 lbs/ft^3
	Active Condition (2h:1v backfill)	72 lbs/ft^3
	At-Rest Condition (level backfill)	67 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/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

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

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

Seismic Lateral Earth Pressures

In accordance with the CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

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

Backfill Material

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

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

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

Subsurface Drainage

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

- A weep hole drainage system typically consisting of a series of 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.

6.8 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The on-site soils generally consist of silty sands and clayey sands. These soils are generally considered to possess fair to good pavement support characteristics. Based on the R-value testing performed as part of our scope for this project, the subsequent pavement designs are based upon an R-values of 27 and 46 for each associated building. 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 the completion of rough grading to verify the R-value of the as-graded parking subgrade.

The pavement recommendations presented below are based on the existing subgrade soils. We have also included options for pavement sections with an increase in the 28-day compressive strength of concrete and a daily truck traffic volume of one truck in per dock door and one truck out per dock door.

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 = 46) FOR BUILDING 14A					
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	3	4	6	6	8
Compacted Subgrade	12	12	12	12	12

ASPHALT PAVEMENTS (R = 27) FOR BUILDING 14B					
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	11	12	14
Compacted Subgrade	12	12	12	12	12

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

Portland Cement Concrete

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

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 46, f_c' = 3500 psi) FOR BUILDING 14A				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	7½
Compacted Subgrade (95% minimum compaction)	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 27, f_c' = 3500 psi) FOR BUILDING 14B				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	7½
Compacted Subgrade (95% minimum compaction)	12	12	12	12

The concrete should have a 28-day compressive strength of at least 3,500 psi. Any reinforcement within the PCC pavements should be determined by the project structural engineer. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness.

Alternative Portland Cement Concrete Pavement Design: Compressive Strength Increase

At the direction of the client, we are providing an alternative PCC pavement design which is based on the number of truck doors for the proposed buildings, and with the understanding that the project will not exceed one truck in and out per dock door. Based on the site plan provided to our office, the proposed buildings will be constructed with 27 dock doors along the eastern building wall for Building 14A and 21 dock doors along the eastern building wall for Building 14B. Therefore, the following pavement design is based on the understanding that the Building 14A will not exceed 54 heavy-truck loads per day and Building 14B will not exceed 42 heavy-truck loads per day, which is equivalent to 987,293 EALs (equivalent axle loads) for Building 14A and 767,895 EALs for Building 14B. These values are for a six-day work-week and a 20-year life-design for the project, which is equivalent to a TI of 8. It should be noted that the EALs for this project were based on a conversion of 2.93 EAL per truck for the PCC areas. If a different conversion factor is required by the tenant/owner, SCG should be contacted to provide revised pavement design recommendations.

The Portland cement concrete pavement design table below has been generated using the Pavement Designer, a web-based software application available at the website www.pavementdesigner.org. This software application calculates PCC sections in accordance with the American Concrete Institute (ACI) 330. Traffic spectrum "D" was utilized in the design of the pavements. The percent of slabs that are cracked at the end of the design life was assumed to be 5 percent, but a reliability percentage of 90 percent was provided by the client.

As previously discussed, the client requested that the number of heavy trucks per day utilized in our design be based on the number of dock doors for each building, and one truck in per dock door and one truck out per dock door. **If different values should be used, or if the number of dock doors changes for the proposed building**, SCG should be contacted to provide a new pavement design. The table below was created using data obtained from the application.

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 (PCC) pavement sections for proposed Buildings 14A and 14B are as follows:

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 46, f_c' = 3500 psi)	
Materials	Thickness (inches)
	Building 14A (27 Dock Doors)
PCC	7
Compacted Subgrade (95% minimum compaction)	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 27, f_c' = 3500 psi)	
Materials	Thickness (inches)
	Building 14B (21 Dock Doors)
PCC	7½
Compacted Subgrade (95% minimum compaction)	12

As directed by the client, the concrete should have a 28-day compressive strength of at least 3,500 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. The actual joint spacing and reinforcing of the Portland cement concrete pavements should be determined by the structural engineer.

7.0 GENERAL COMMENTS

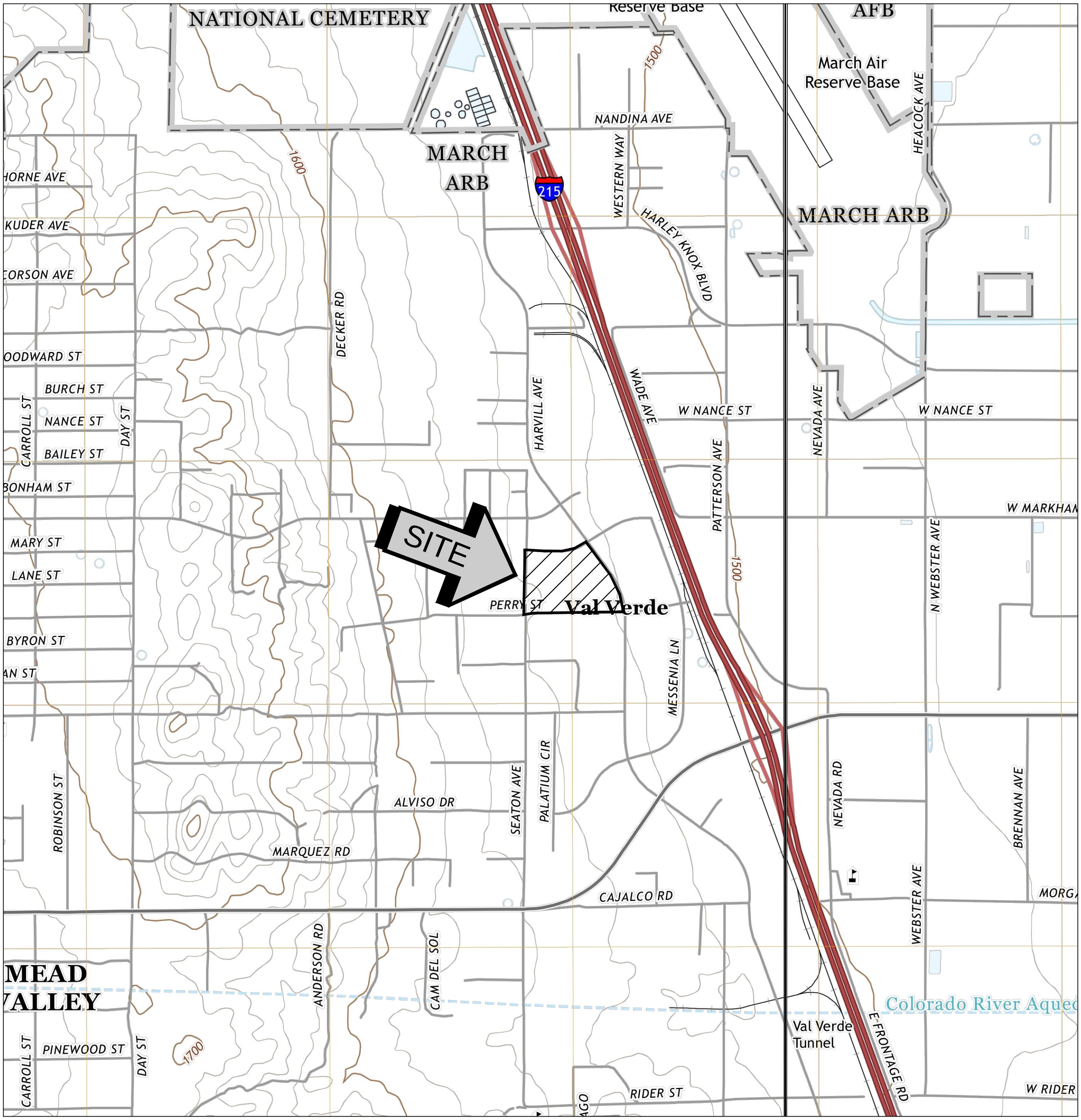
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, civil engineer, and/or structural engineer. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The client(s)' reliance upon this report is subject to the Engineering Services Agreement, incorporated into our proposal for this project.

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between trench locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted.

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

APPENDIX A



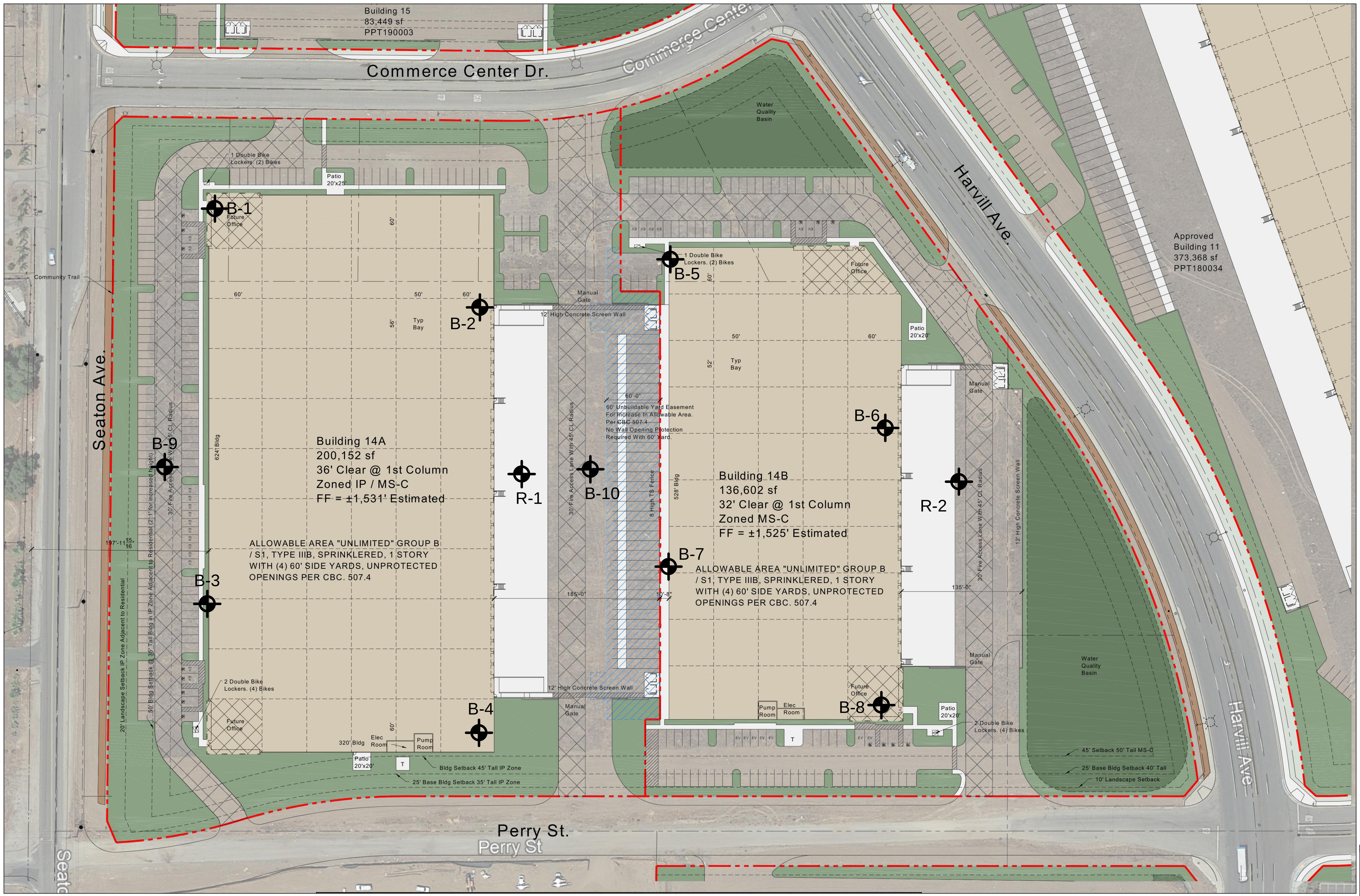
SOURCE: USGS TOPOGRAPHIC MAP OF THE STEELE PEAK AND PERRIS QUADRANGLES, RIVERSIDE COUNTY, CALIFORNIA, 2018.

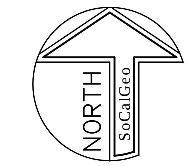
SITE LOCATION MAP

MAJESTIC FREEWAY BUS. CTR-BLDGS 14A &14B

RIVERSIDE COUNTY (PERRIS), CALIFORNIA

SCALE: 1" = 2000'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: MD CHKD: RF	
SCG PROJECT 21G251-1	
PLATE 1	





GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

NOTE: BASE MAP OBTAINED FROM GOOGLE EARTH.
SITE PLAN PREPARED BY COMMERCE CONSTRUCTION CO.

BORING LOCATION PLAN

MAJESTIC FREEWAY BUS. CTR-BLDGS 14A & 14B

RIVERSIDE COUNTY (PERRIS), CALIFORNIA

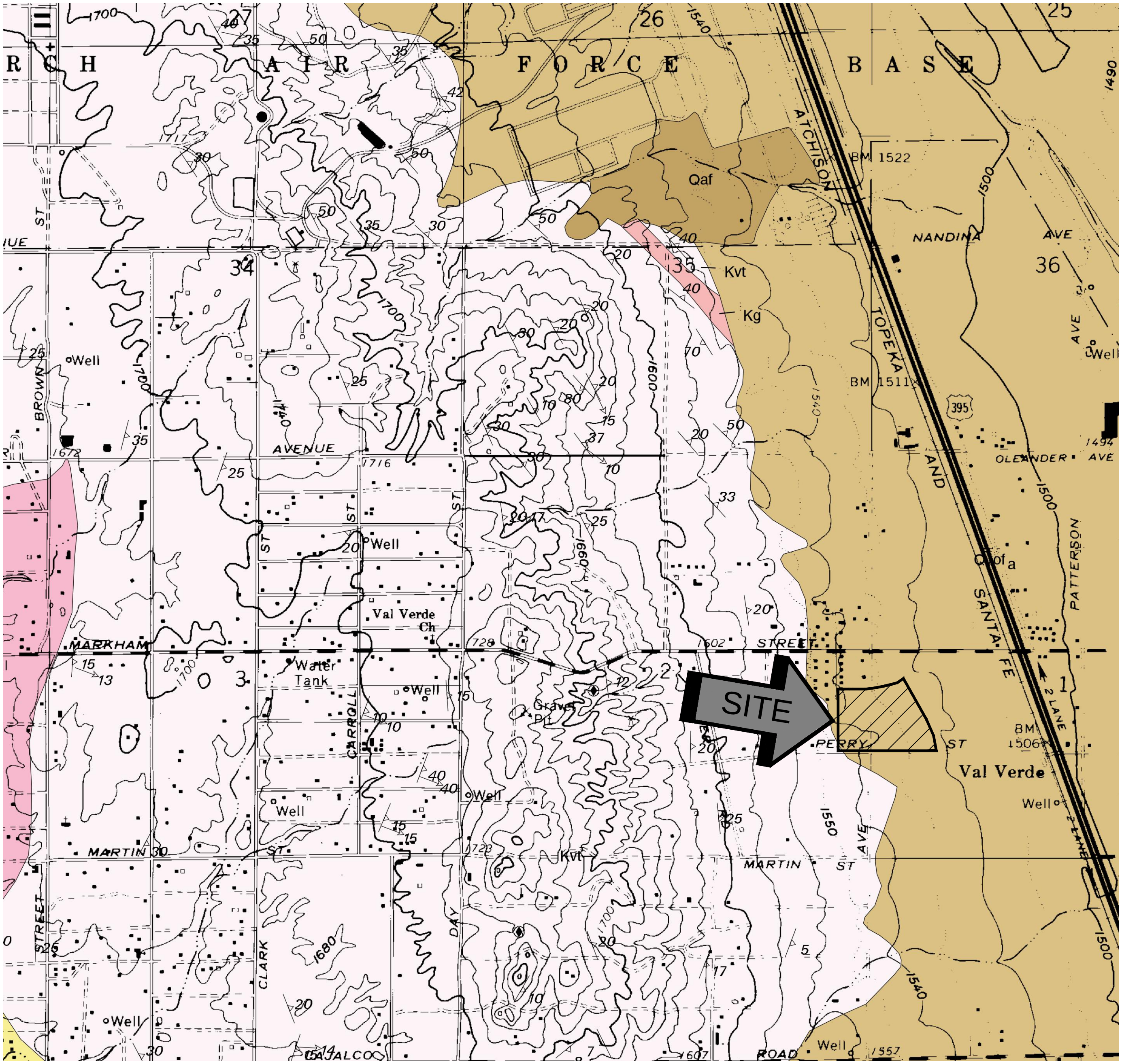
SCALE: 1" = 120'

DRAWN: MD
CHKD: RF

SCG PROJECT
21G251-1R

PLATE 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



DESCRIPTION OF MAP UNITS

Qvof

Very old alluvial fan deposits (early Pleistocene)—Mostly well-dissected, well-indurated, reddish-brown sand deposits. Commonly contains duripans and locally silcretes. Covers large areas adjacent to U.S. Highway 215 in northeastern part of quadrangle and flanking drainage followed by Cajalco Road

Kvt

Val Verde tonalite—Gray-weathering, relatively homogeneous, massive-to well-foliated, medium- to coarse-grained, hypautomorphic-granular biotite-hornblende tonalite; principal rock type of Val Verde pluton. Contains subequal biotite and hornblende, quartz and plagioclase. Potassium feldspar generally less than two percent of rock. Where present, foliation typically strikes northwest and dips moderately to steeply northeast. Northern part of pluton contains younger, intermittently developed, northeast-striking foliation. In central part of pluton, tonalite is mostly massive, and contains few segregational masses of mesocratic to melanocratic tonalite. Elliptical- to pancake-shaped, meso-to melanocratic inclusions are common

SOURCE: "GEOLOGIC MAP OF THE STEELE PEAK 7.5' QUADRANGLE, RIVERSIDE COUNTY, CALIFORNIA", BY DOUGLAS M. MARTIN.

GEOLOGIC MAP

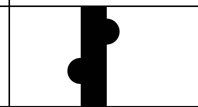
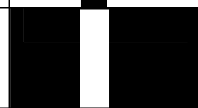
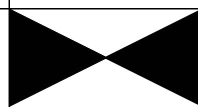
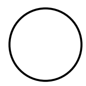
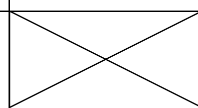
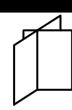
MAJESTIC FREEWAY BUS. CTR-BLDGS 14A & 14B

RIVERSIDE COUNTY (PERRIS), CALIFORNIA

SCALE: 1" = 2000'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: MD CHKD: RF	
SCG PROJECT 21G251-1	
PLATE 3	

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH:	Distance in feet below the ground surface.
SAMPLE:	Sample Type as depicted above.
BLOW COUNT:	Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3" indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.
POCKET PEN.:	Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.
GRAPHIC LOG:	Graphic Soil Symbol as depicted on the following page.
DRY DENSITY:	Dry density of an undisturbed or relatively undisturbed sample in lbs/ft^3.
MOISTURE CONTENT:	Moisture content of a soil sample, expressed as a percentage of the dry weight.
LIQUID LIMIT:	The moisture content above which a soil behaves as a liquid.
PLASTIC LIMIT:	The moisture content above which a soil behaves as a plastic.
PASSING #200 SIEVE:	The percentage of the sample finer than the #200 standard sieve.
UNCONFINED SHEAR:	The shear strength of a cohesive soil sample, as measured in the unconfined state.

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 21G251-1 | DRILLING DATE: 10/28/21 | WATER DEPTH: 16 feet
PROJECT: Majestic Freeway Bus. Ctr-Bldgs 14A & 14B | DRILLING METHOD: Hollow Stem Auger | CAVE DEPTH: 16 feet
LOCATION: Riverside County (Perris), California | LOGGED BY: Oscar Sandoval | READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	19			FILL: Light Brown Silty fine Sand, trace medium to coarse Sand, trace fine root fibers, mottled, medium dense-dry to damp	106	2					EI = 2 @ 0 to 5 feet
	X	21				113	3					
5	X	24			OLDER ALLUVIUM: Red Brown Silty fine Sand, trace medium to coarse Sand, trace fine root fibers, medium dense-damp	110	5					
	X	39			Red Brown Clayey fine Sand, trace medium to coarse Sand, trace Silt, trace fine root fibers, trace calcareous nodules, medium dense-damp	113	5					
10	X	35				117	5					
					VAL VERDE TONALITE (Kvt): Gray Brown fine to coarse grained Tonalite bedrock, weathered, friable, phaneritic, medium dense to very dense-damp to wet							
15	X	50/5"					4					
					@ 16 feet, Groundwater encountered during drilling							
20	X	50/4"					10					
25	X	50/4"					14					
					Boring Terminated at 25'							

TBL 21G251-1.GPJ SOCALGEO.GDT 11/24/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 21G251-1
DRILLING DATE: 10/28/21
WATER DEPTH: Dry
PROJECT: Majestic Freeway Bus. Ctr-Bldgs 14A & 14B
DRILLING METHOD: Hollow Stem Auger
CAVE DEPTH: 12 feet
LOCATION: Riverside County (Perris), California
LOGGED BY: Oscar Sandoval
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					OLDER ALLUVIUM: Red Brown Silty fine to medium Sand, trace coarse Sand, trace Clay, dense to very dense-damp to moist							
	X	33					6					
	X	74/9"					8					
5												
	X	50/5"					10					
	X	50					11					
10												
					VAL VERDE TONALITE (Kvt): Gray Brown fine to coarse grained Tonalite bedrock, weathered, friable, phaneritic, very dense-damp							
	X	75					4					
15												

Boring Terminated at 15'

TBL 21G251-1.GPJ SOCALGEO.GDT 11/24/21

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 21G251-1 | DRILLING DATE: 10/28/21 | WATER DEPTH: Dry
PROJECT: Majestic Freeway Bus. Ctr-Bldgs 14A & R4B | DRILLING METHOD: Hollow Stem Auger | CAVE DEPTH: 17 feet
LOCATION: Riverside County (Perris), California | LOGGED BY: Oscar Sandoval | READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION — SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					OLDER ALLUVIUM: Red Brown Silty fine Sand, trace medium Sand, trace calcareous nodules, cemented, dense to very dense-damp to moist							
	X	50/5"					3					
	X	39					8					
5												
	X	23			@ 6 feet, medium dense		9					
	X	80					9					
10												
					VAL VERDE TONALITE (Kvt): Gray Brown fine to coarse grained Tonalite bedrock, weathered, friable, phaneritic, very dense-damp							
	X	50/3"					2					
15												
	X	50/5"					4					
20												
					Boring Terminated at 20'							

TBL 21G251-1.GPJ SOCALGEO.GDT 11/24/21

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 21G251-1
PROJECT: Majestic Freeway Bus. Ctr-Bldgs 14A & 14B
LOCATION: Riverside County (Perris), California
DRILLING DATE: 10/28/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Oscar Sandoval
WATER DEPTH: Dry
CAVE DEPTH: 23 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (SURFACE ELEVATION: MSL)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					OLDER ALLUVIUM: Red Brown Silty fine to medium Sand, trace coarse Sand, cemented, dense to very dense-dry to moist							
		61				129	2					
		50/6"					7					Disturbed Sample
5		51				105	9					
		47				118	6					
		62			@ 9 feet, trace bedrock fragments	125	7					
10												
					VAL VERDE TONALITE (Kvt): Gray Brown fine to coarse grained Tonalite bedrock, weathered, friable, phaneritic, very dense-damp to moist							
		70/9"					4					
15												
		50/5"					4					
20												
		50/4"					9					
25												

Boring Terminated at 25'

TBL 21G251-1.GPJ SOCALGEO.GDT 11/24/21

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 21G251-1 DRILLING DATE: 10/28/21 WATER DEPTH: 22 feet
PROJECT: Majestic Freeway Bus. Ctr-Bldgs 14A & 14B DRILLING METHOD: Hollow Stem Auger CAVE DEPTH: 23 feet
LOCATION: Riverside County (Perris), California LOGGED BY: Oscar Sandoval READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					FILL: Brown Silty fine to medium Sand, trace coarse Sand, trace fine root fibers, mottled, medium dense-damp							
		38				102	5					
					OLDER ALLUVIUM: Red Brown Silty fine to medium Sand, trace coarse Sand, cemented, medium dense to very dense-damp to moist							
		50/5"				109	5					
5		76/11"				112	8					
		50/5"				120	6					
					VAL VERDE TONALITE (Kvt): Gray Brown fine to coarse grained Tonalite bedrock, weathered, friable, phaneritic, very dense-damp to wet							
		56/11"				108	5					
10												
		50/3"					3					
15												
		50/5"					6					
20												
					@ 22 feet, Groundwater encountered during drilling							Perched Groundwater
		50/5"					12					
25					Boring Terminated at 25'							

TBL 21G251-1.GPJ SOCALGEO.GDT 11/24/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 21G251-1 DRILLING DATE: 10/28/21 WATER DEPTH: Dry
PROJECT: Majestic Freeway Bus. Ctr-Bldgs 14A & 14B DRILLING METHOD: Hollow Stem Auger CAVE DEPTH: 17 feet
LOCATION: Riverside County (Perris), California LOGGED BY: Oscar Sandoval READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	20			OLDER ALLUVIUM: Red Brown Silty fine to medium Sand, trace coarse Sand, trace Clay, medium dense-moist		8					
5	X	18			Red Brown Clayey fine to medium Sand, trace coarse Sand, trace Silt, medium dense-moist		9					
	X	19			Red Brown Silty fine to medium Sand, trace coarse Sand, trace Clay, cemented, medium dense-moist		9					
10	X	76			@ 8½ feet, cemented, very dense		10					
15	X	50/5"			VAL VERDE TONALITE (Kvt): Gray Brown fine to coarse grained Tonalite bedrock, weathered, friable, phaneritic, very dense-damp		6					
20	X	50/4"					6					
					Boring Terminated at 20'							

TBL 21G251-1.GPJ SOCALGEO.GDT 11/24/21

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-7

JOB NO.: 21G251-1
PROJECT: Majestic Freeway Bus. Ctr-Bldgs 14A & 14B
LOCATION: Riverside County (Perris), California

DRILLING DATE: 10/28/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Oscar Sandoval

WATER DEPTH: Dry
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					OLDER ALLUVIUM: Red Brown Silty fine to coarse Sand, trace Clay, medium dense to very dense-damp							
	X	28					6					
	X	50/5"					5					
5												
	X	84/8"			@ 6 feet, little bedrock fragments		6					
					VAL VERDE TONALITE (Kvt): Gray Brown fine to coarse grained Tonalite bedrock, weathered, friable, phaneritic, very dense-damp							
	X	50/5"					6					
10												
	X	50/4"					3					
15					Boring Terminated at 15'							

TBL 21G251-1.GPJ SOCALGEO.GDT 11/24/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 21G251-1 DRILLING DATE: 10/28/21 WATER DEPTH: Dry
PROJECT: Majestic Freeway Bus. Ctr-Bldgs 14A & 14B DRILLING METHOD: Hollow Stem Auger CAVE DEPTH: 21 feet
LOCATION: Riverside County (Perris), California LOGGED BY: Oscar Sandoval READING TAKEN: At Completion

SURFACE ELEVATION: MSL

FIELD RESULTS: DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS: DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					FILL: Light Brown Silty fine Sand, trace medium Sand, trace fine root fibers, porous, mottled, medium dense-damp							
	X	34				113	3					
					OLDER ALLUVIUM: Red Brown Silty fine to medium Sand, trace coarse Sand, medium dense to very dense-damp							
	X	39				117	3					
5	X	31				113	5					
	X	50/5"					4					Disturbed Sample
					VAL VERDE TONALITE (Kvt): Gray Brown fine to coarse grained Tonalite bedrock, weathered, friable, phaneritic, very dense-damp							
	X	50/5"				112	4					
10												
	X	73/11"					4					
15												
	X	50/1"					3					
20												
	X	50/5"					4					
25												

Boring Terminated at 25'

TBL 21G251-1.GPJ SOCALGEO.GDT 11/24/21

TEST BORING LOG

BORING NO. B-9

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 21G251-1 | DRILLING DATE: 10/28/21 | WATER DEPTH: Dry
PROJECT: Majestic Freeway Bus. Ctr-Bldgs 14A & 14B | DRILLING METHOD: Hollow Stem Auger | CAVE DEPTH: 7 feet
LOCATION: Riverside County (Perris), California | LOGGED BY: Oscar Sandoval | READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					OLDER ALLUVIUM: Red Brown Silty fine to medium Sand, trace coarse Sand, medium dense to very dense-damp to moist							
	X	28					7					
	X	24			@ 3½ feet, trace Clay, very moist		17					
5												
	X	50/5"					12					
	X	43			@ 8½ feet, trace bedrock fragments		9					
10												
					Boring Terminated at 10'							

TBL 21G251-1.GPJ SOCALGEO.GDT 11/24/21

TEST BORING LOG **PLATE B-9**

BORING NO. B-10

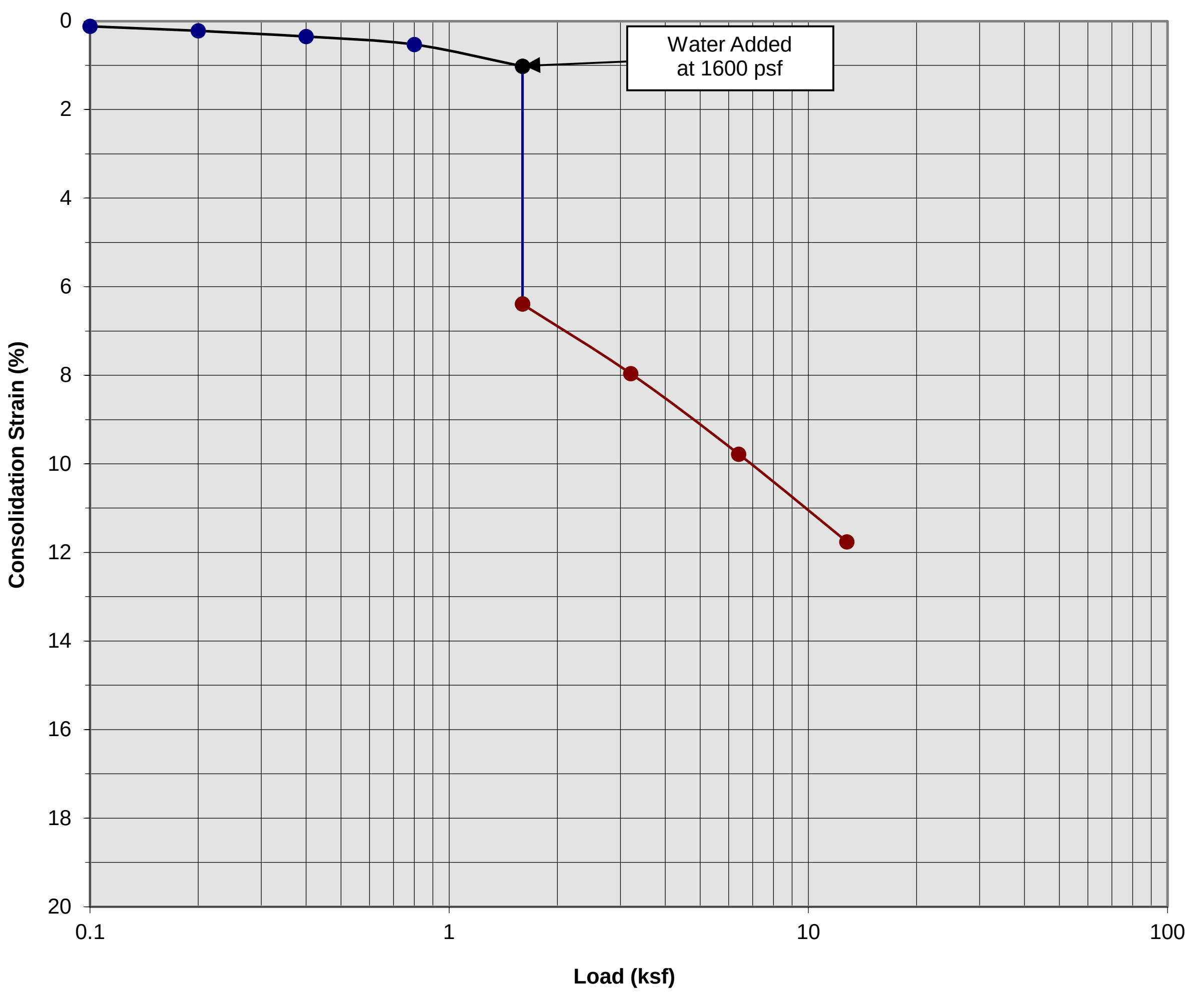
JOB NO.: 21G251-1
DRILLING DATE: 10/28/21
WATER DEPTH: Dry
PROJECT: Majestic Freeway Bus. Ctr-Bldgs 14A & 14B
DRILLING METHOD: Hollow Stem Auger
CAVE DEPTH: 9 feet
LOCATION: Riverside County (Perris), California
LOGGED BY: Oscar Sandoval
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					OLDER ALLUVIUM: Red Brown Silty fine to medium Sand, trace coarse Sand, trace Clay, medium dense to very dense-damp to moist							
	X	21					6					EI = 8 @ 0 to 5 feet
	X	74/10"					10					
5												
					VAL VERDE TONALITE (Kvt): Gray Brown fine to coarse grained Tonalite bedrock, weathered, friable, phaneritic, very dense-damp to moist							
	X	50/5"					2					
	X	50/5"					7					
10												
					Boring Terminated at 10'							

TEST BORING LOG

APPENDIX C

Consolidation/Collapse Test Results



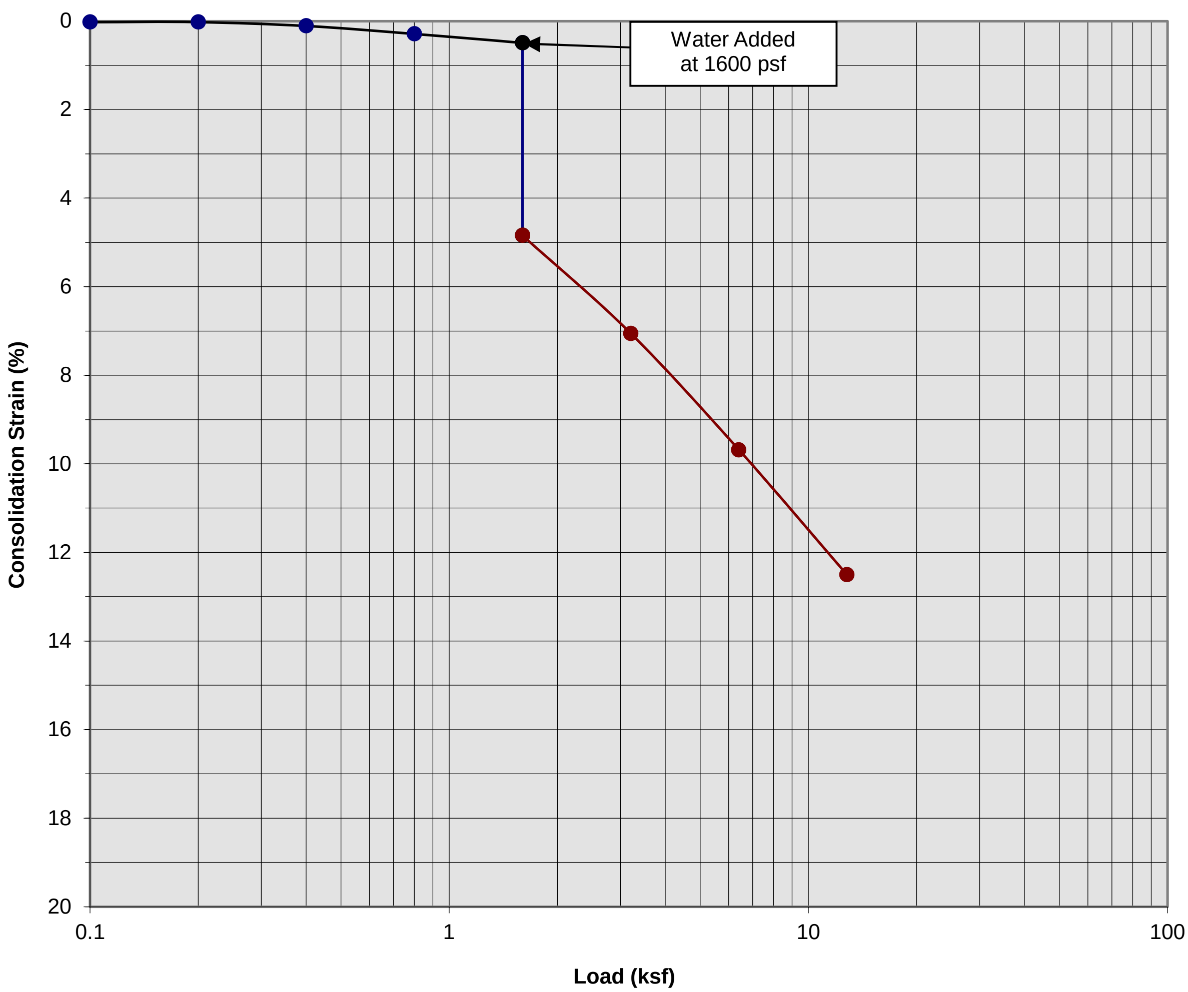
Classification: FILL: Light Brown Silty fine Sand, trace medium to coarse Sand

Boring Number:	B-1	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	3 to 4	Initial Dry Density (pcf)	113.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	128.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	5.37

Majestic Freeway Bus. Ctr-Bldgs 14A & 14B
Riverside County (Perris), California
Project No. 21G251-1
PLATE C-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

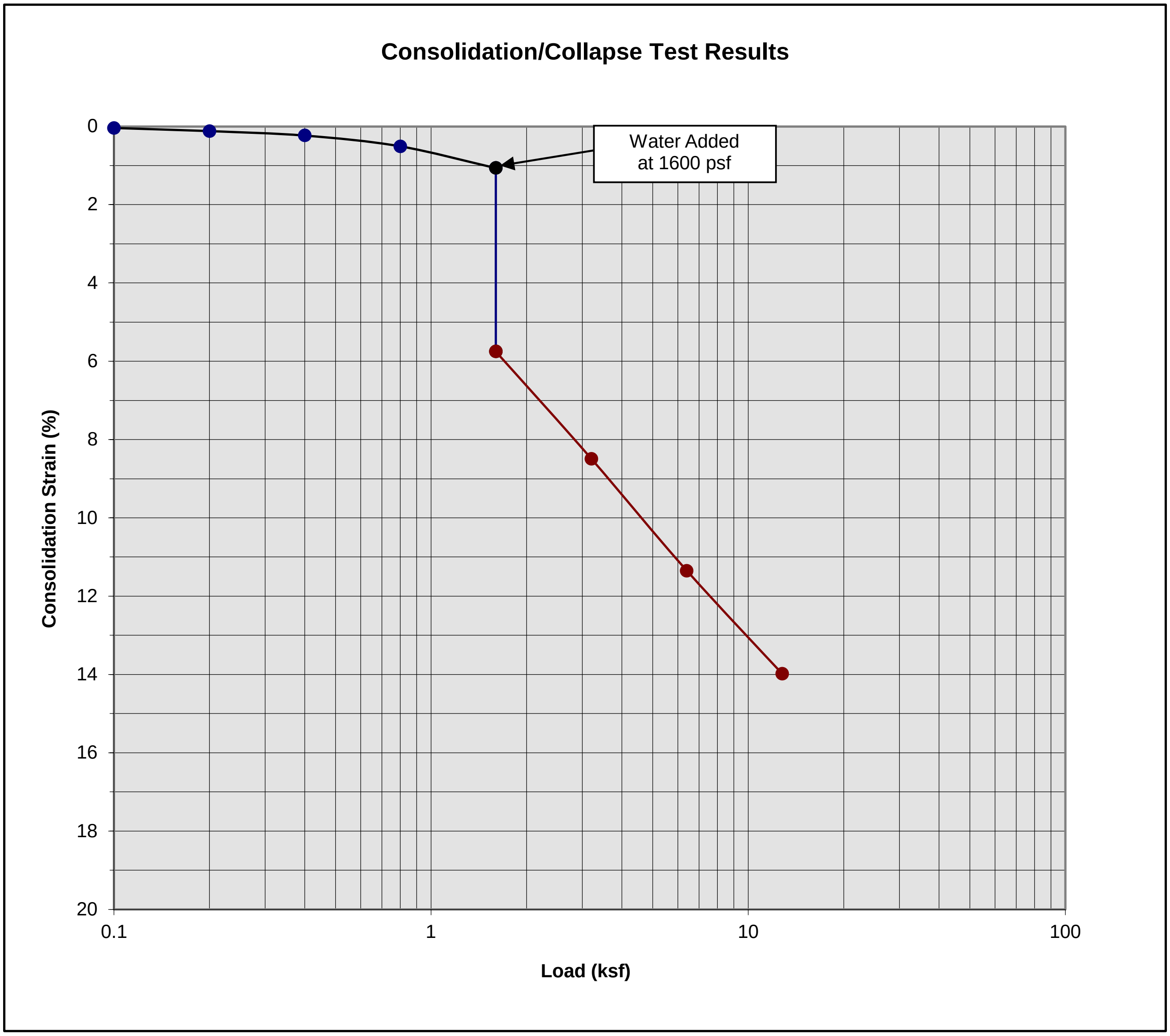


Classification: Red Brown Silty fine Sand, trace medium to coarse Sand

Boring Number:	B-1	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	5 to 6	Initial Dry Density (pcf)	110.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	125.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	4.35

Majestic Freeway Bus. Ctr-Bldgs 14A & 14B
Riverside County (Perris), California
Project No. 21G251-1
PLATE C-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

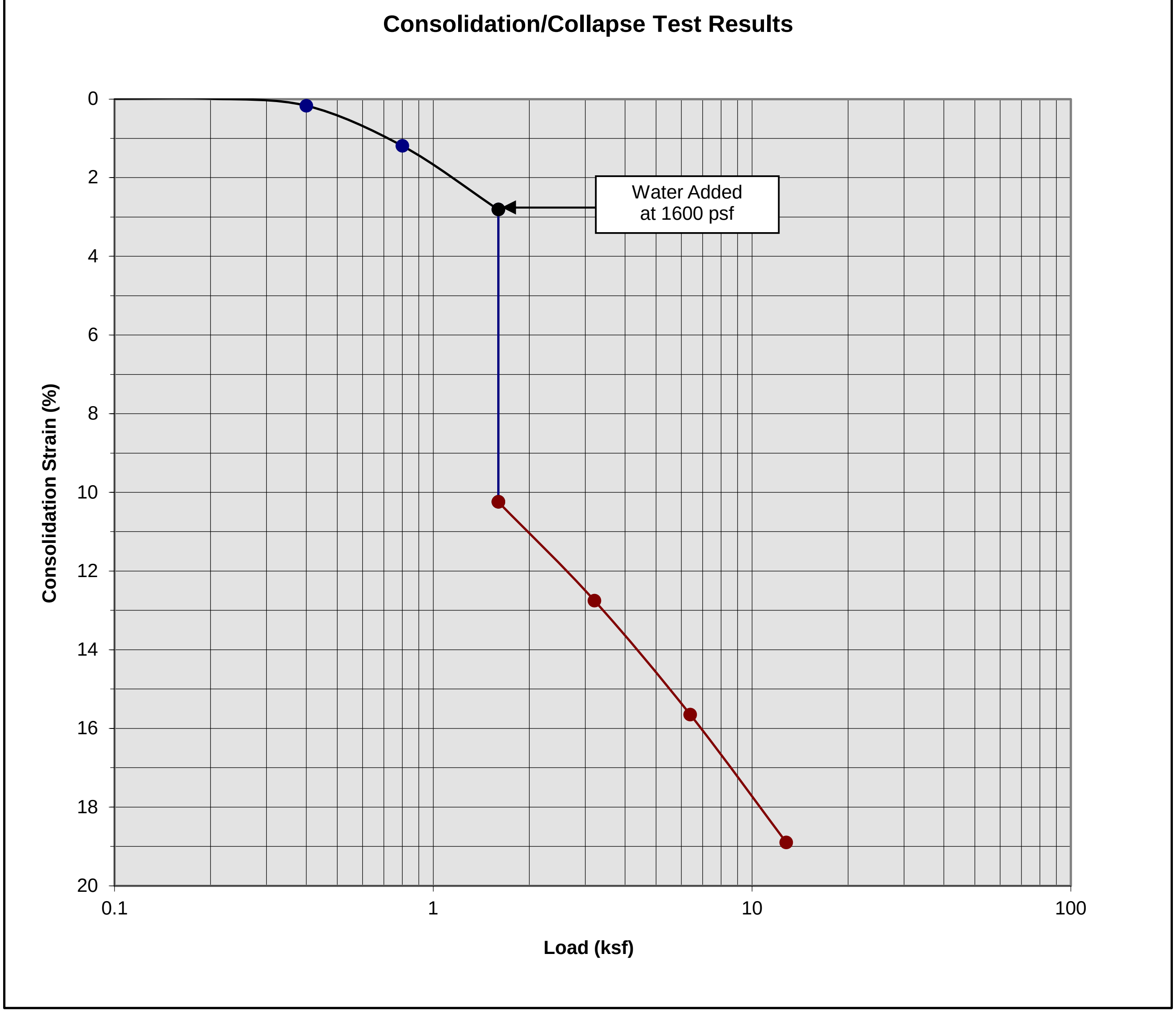


Classification: Red Brown Clayey fine Sand, trace medium to coarse Sand, trace Silt

Boring Number:	B-1	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	7 to 8	Initial Dry Density (pcf)	113.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	131.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	4.69

Majestic Freeway Bus. Ctr-Bldgs 14A & 14B
Riverside County (Perris), California
Project No. 21G251-1
PLATE C-3

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



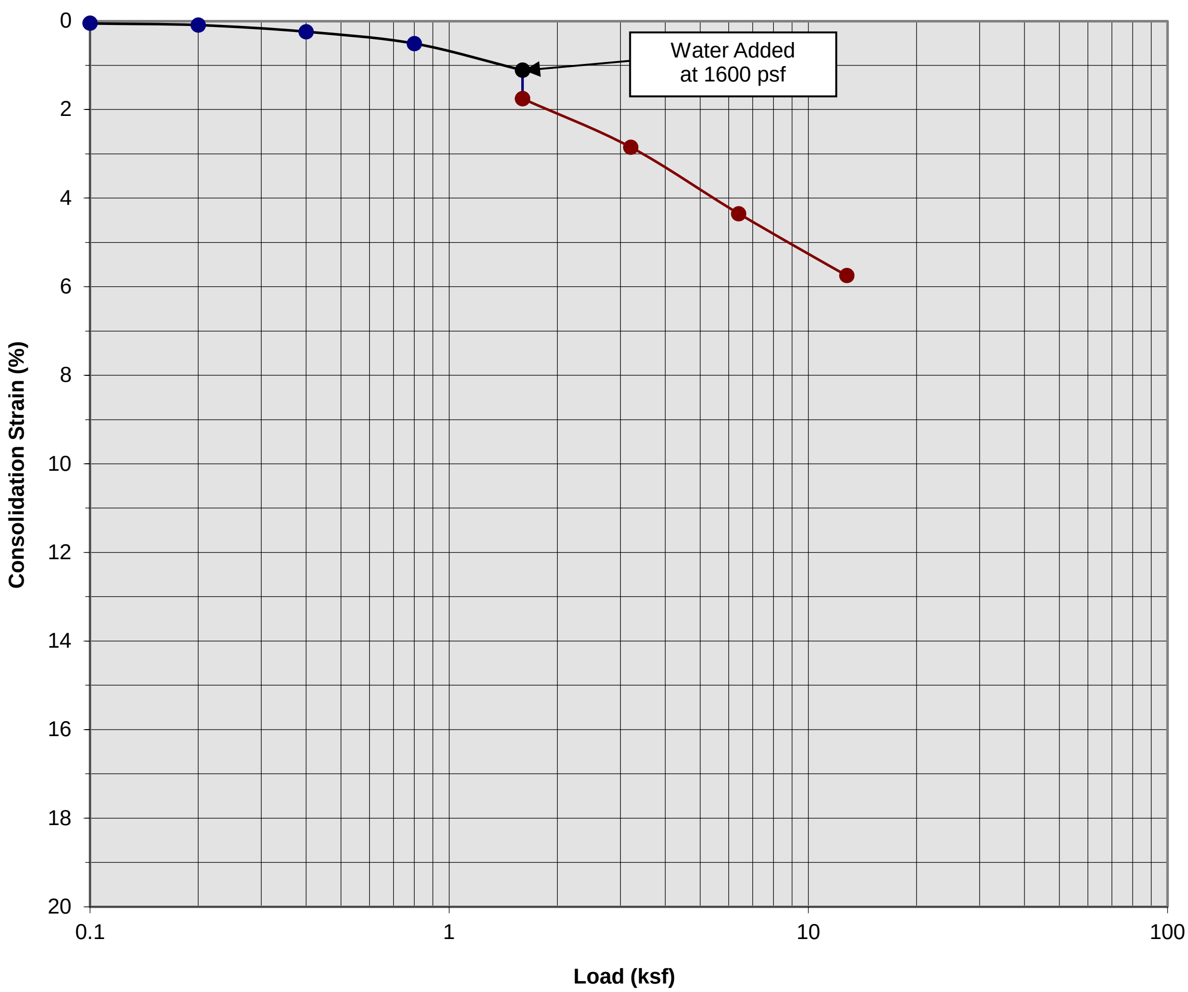
Classification: Red Brown Silty fine to medium Sand, trace coarse Sand

Boring Number:	B-8	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	3 to 4	Initial Dry Density (pcf)	117.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	144.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	7.43

Majestic Freeway Bus. Ctr-Bldgs 14A & 14B
Riverside County (Perris), California
Project No. 21G251-1
PLATE C-4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

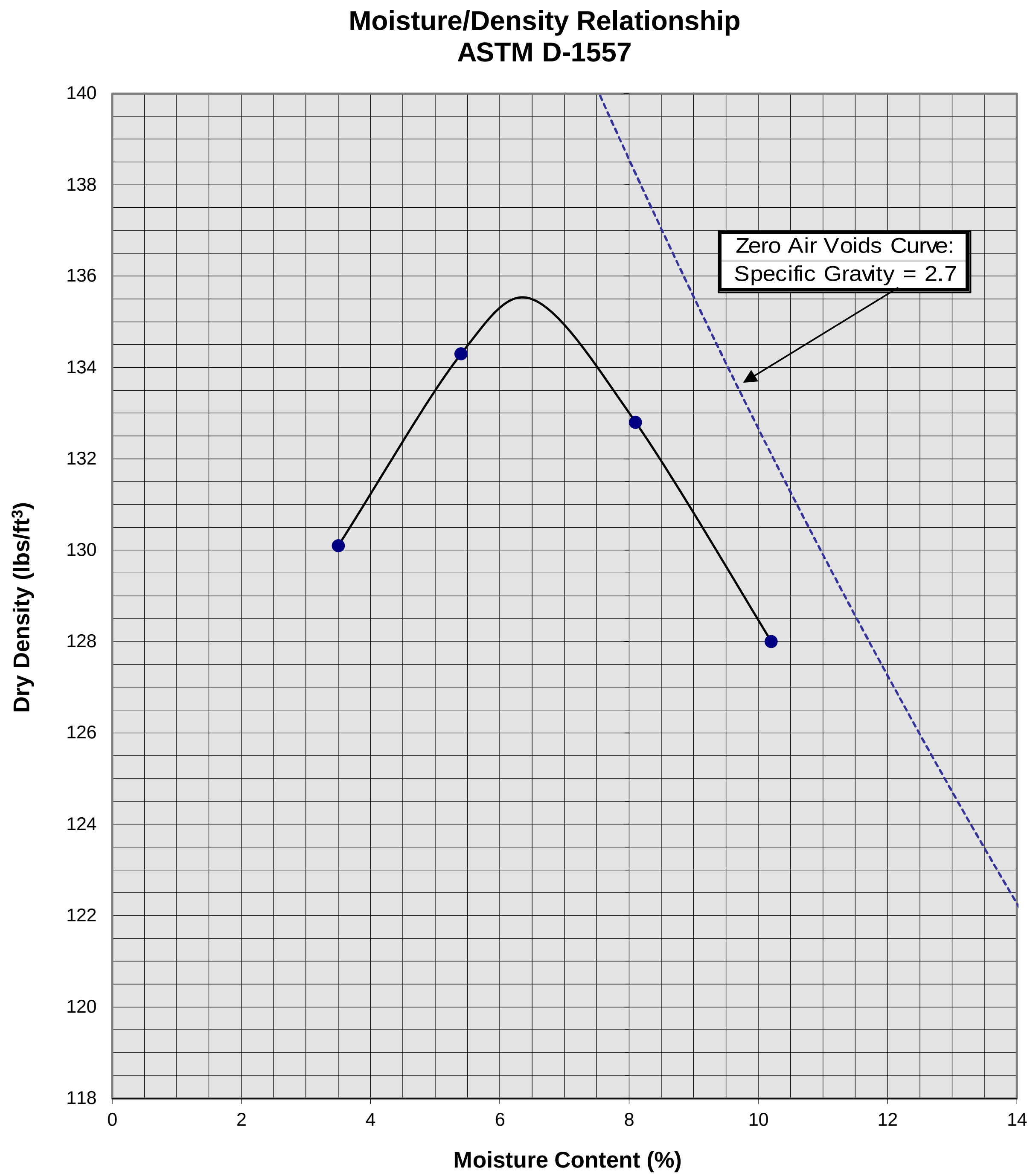


Classification: LAV VERDE TONALITE: Gray Brown fine- to coarse-grained Tonalite

Boring Number:	B-8	Initial Moisture Content (%)	4
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	9 to 10	Initial Dry Density (pcf)	112.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	119.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.64

Majestic Freeway Bus. Ctr-Bldgs 14A & 14B
Riverside County (Perris), California
Project No. 21G251-1
PLATE C-5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

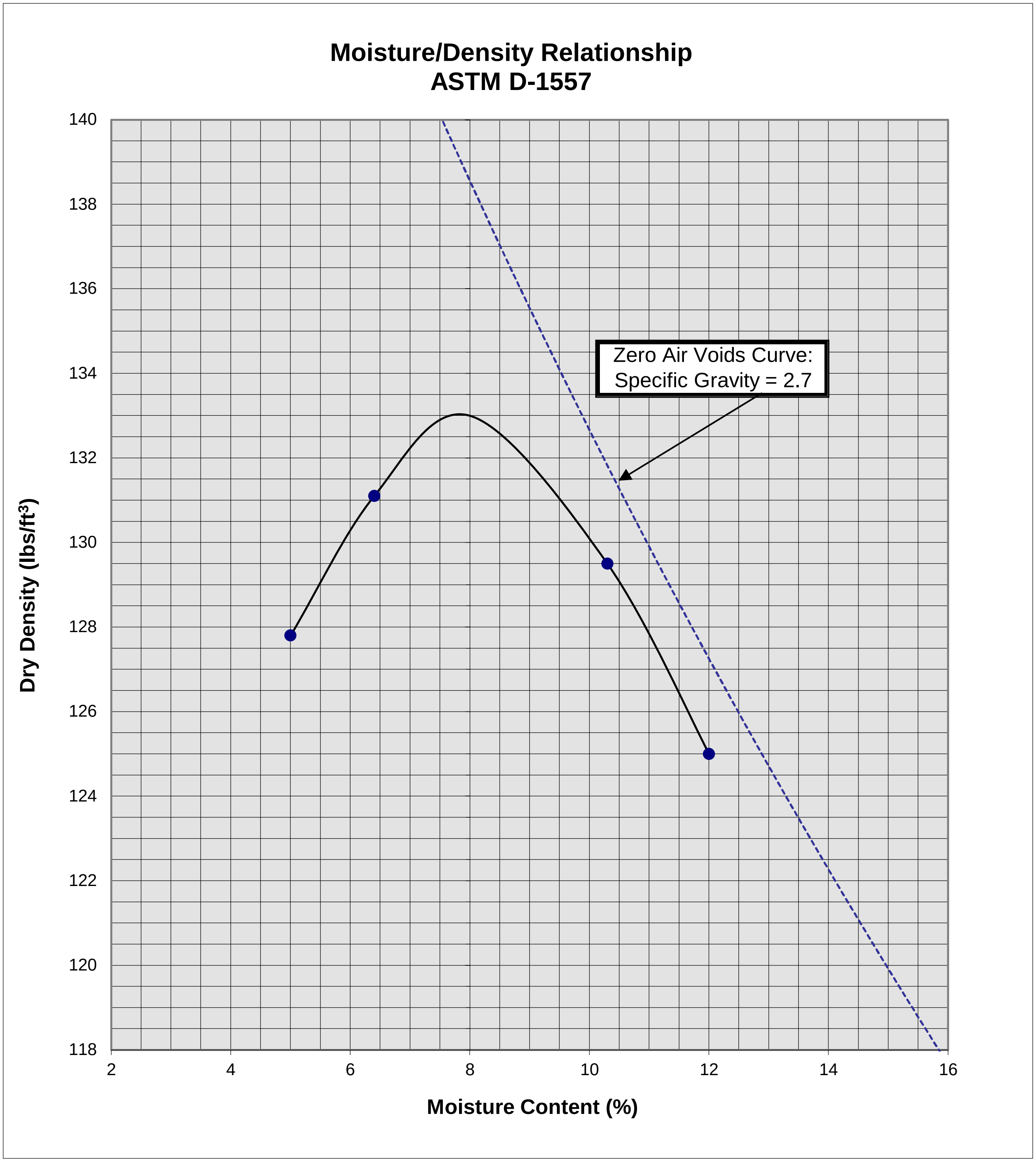


Soil ID Number	B-1 @ 0-5'
Optimum Moisture (%)	6.5
Maximum Dry Density (pcf)	135.5
Soil Classification	Light Brown Silty fine Sand, trace medium to coarse Sand

Majestic Freeway Business Center
Riverside County (Perris), California
Project No. 21G251-1

PLATE C-6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number		B-10 @ 0-5'
Optimum Moisture (%)		8
Maximum Dry Density (pcf)		133
Soil Classification	Red Brown Silty fine to medium Sand, trace coarse Sand, trace Clay	

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

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

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

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

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

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

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

Site Preparation

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

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

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

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

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

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

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

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

<u>Compacted Fills</u>

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

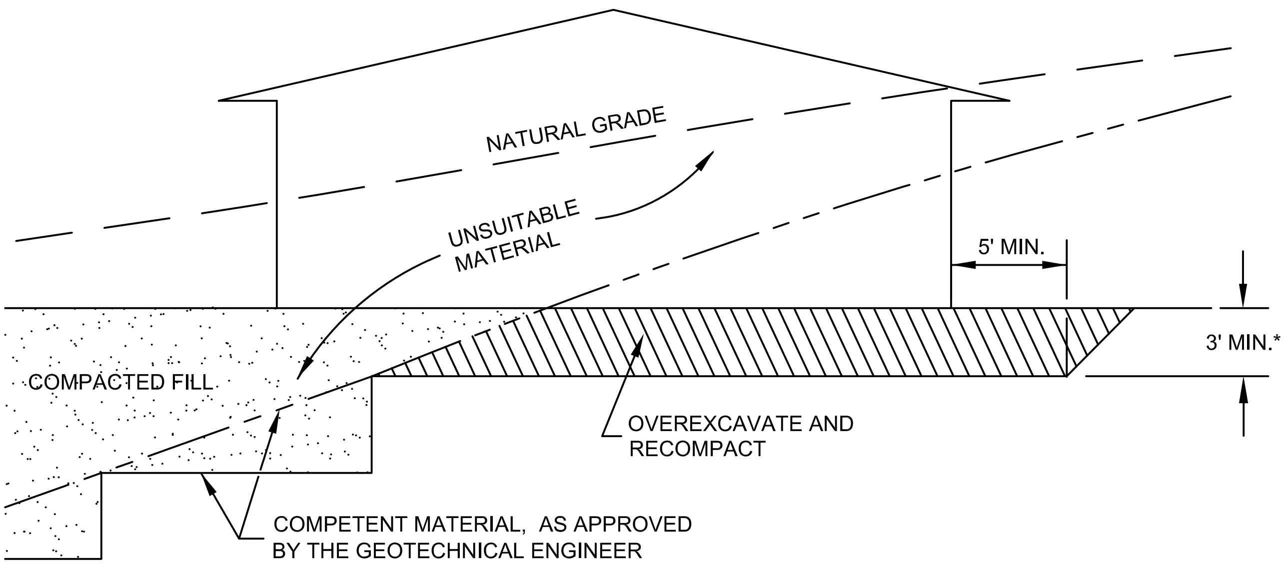
Subdrains

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

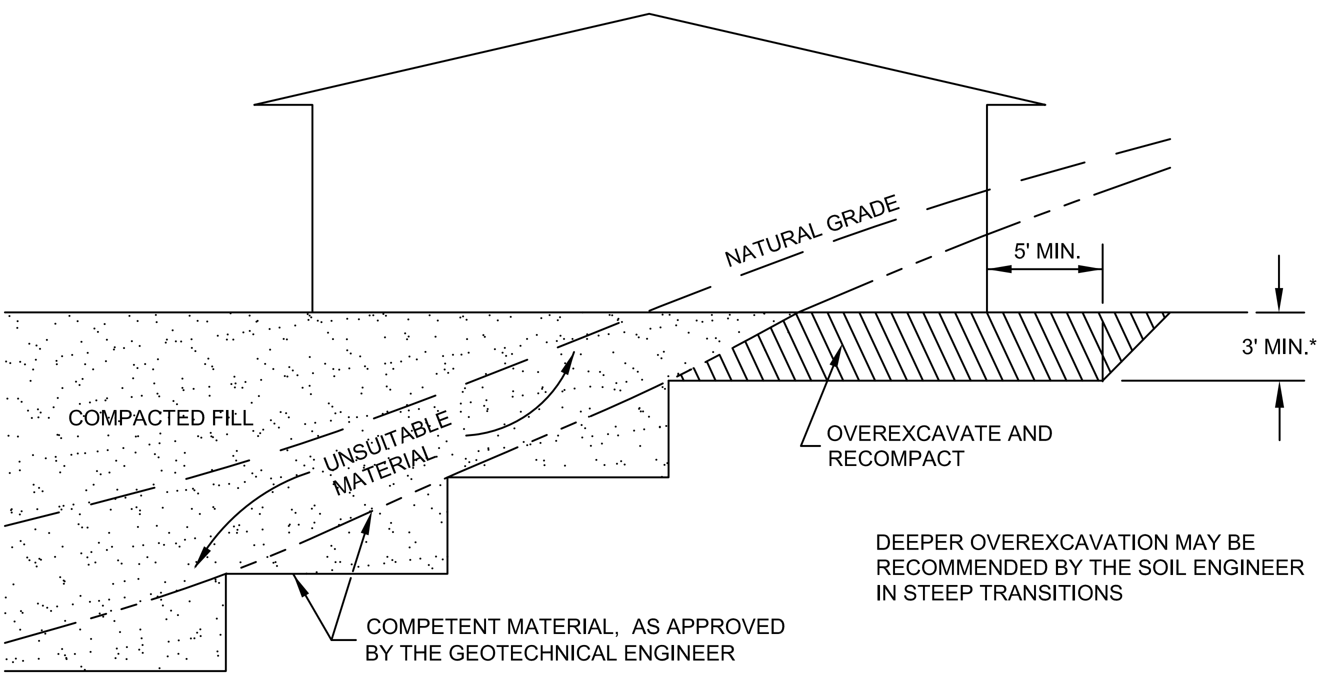
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

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

CUT LOT



CUT/FILL LOT (TRANSITION)



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

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

TRANSITION LOT DETAIL

GRADING GUIDE SPECIFICATIONS

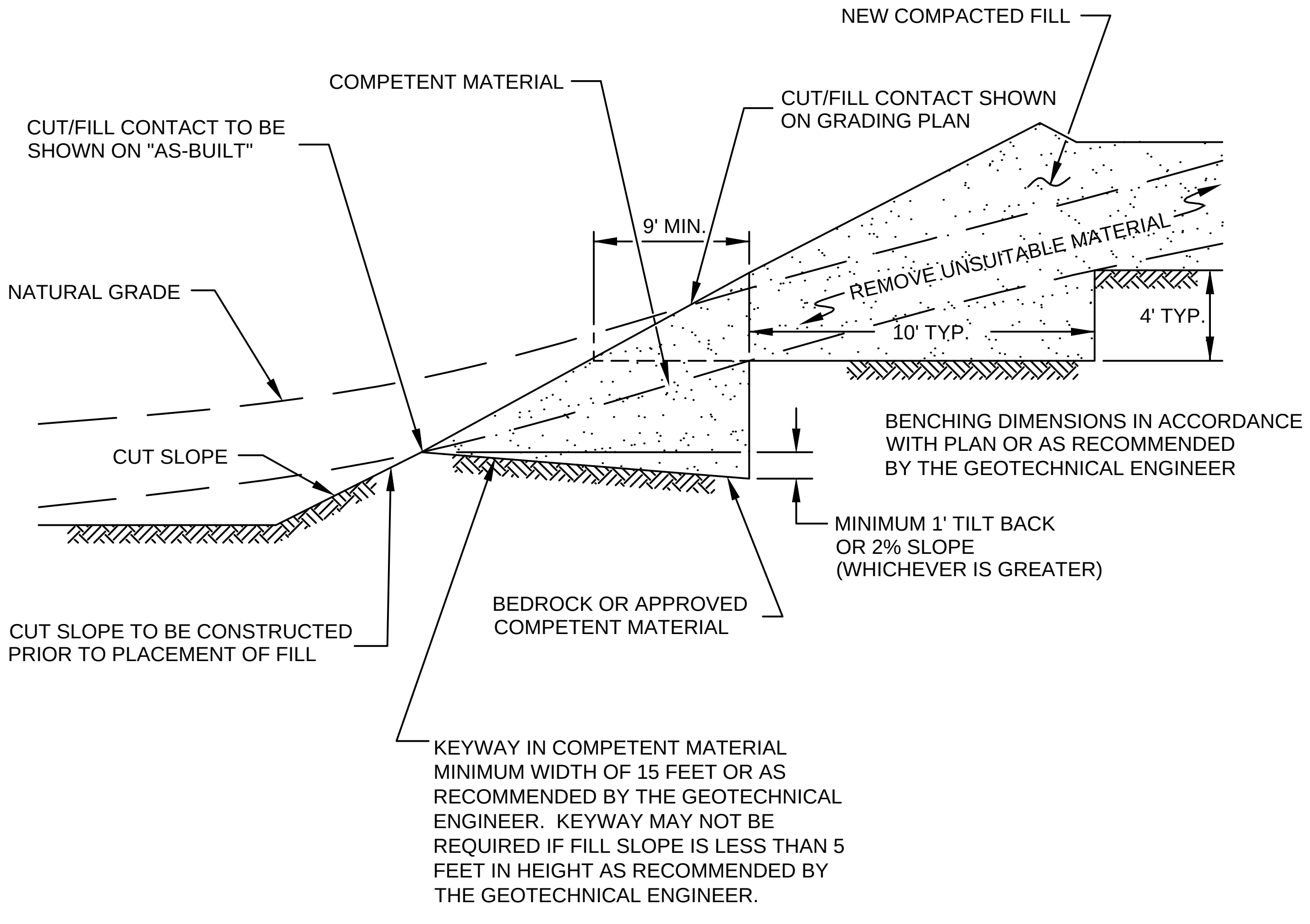
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

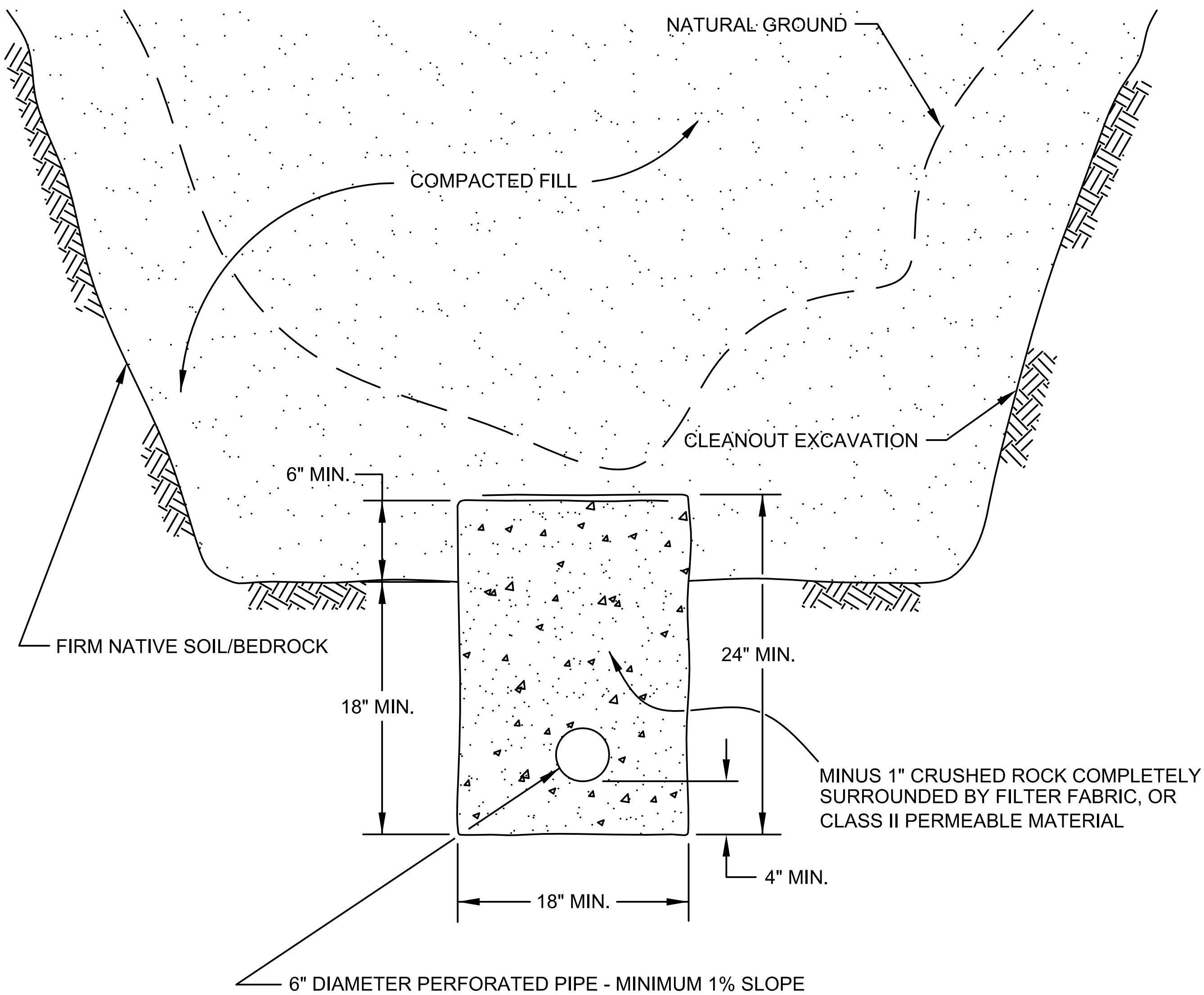
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SOUTHERN
CALIFORNIA
GEOTECHNICAL



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

GRADING GUIDE SPECIFICATIONS

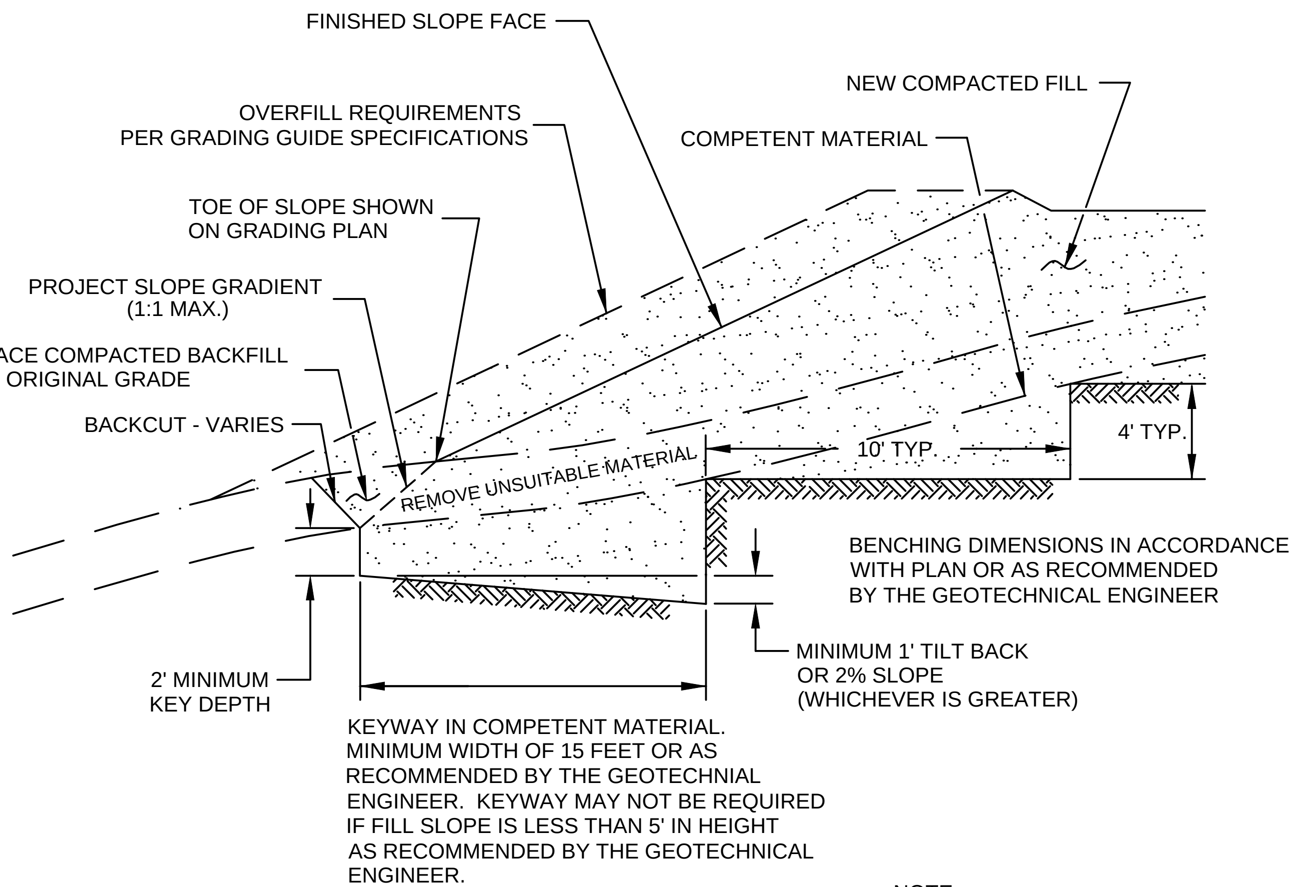
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



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

FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

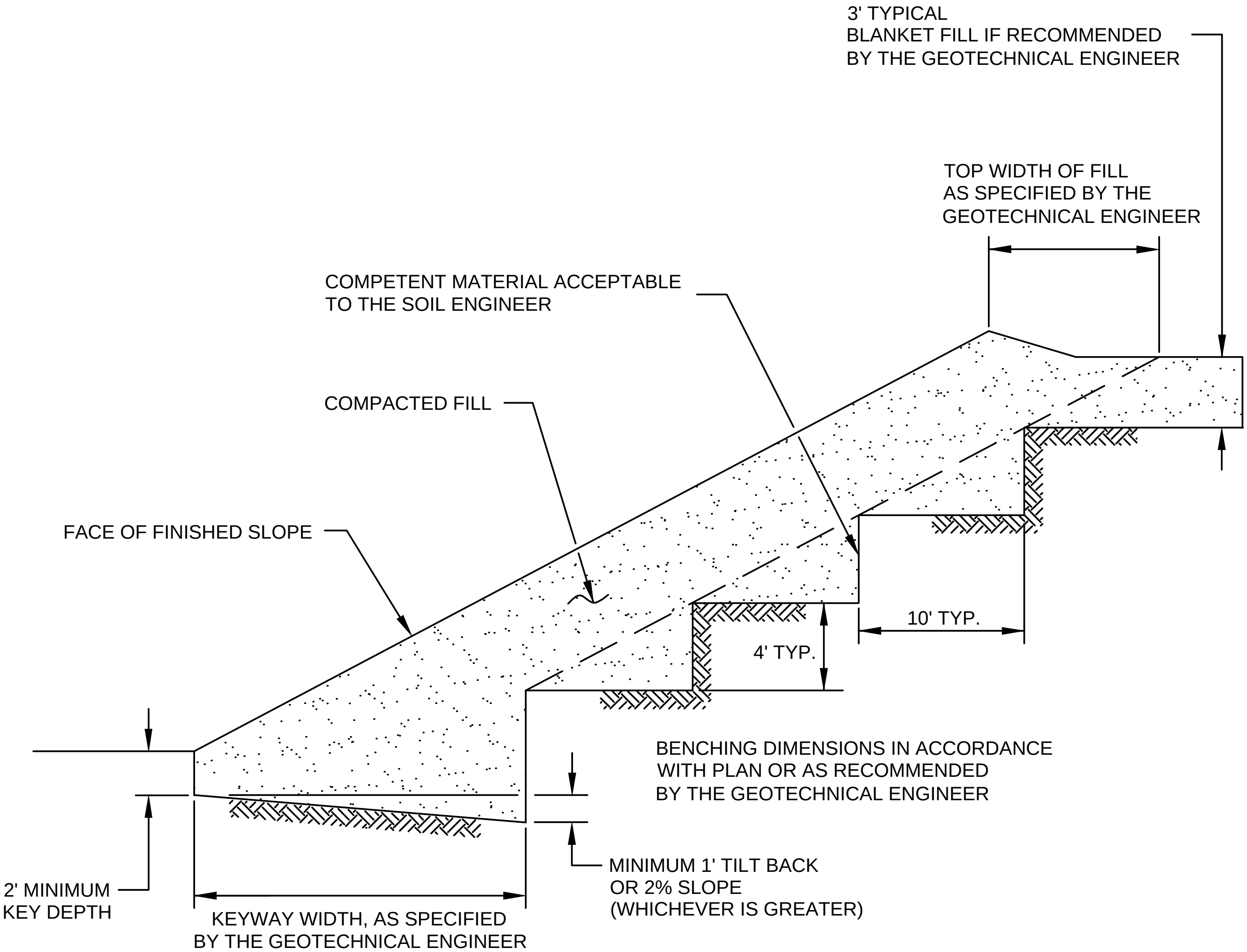
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

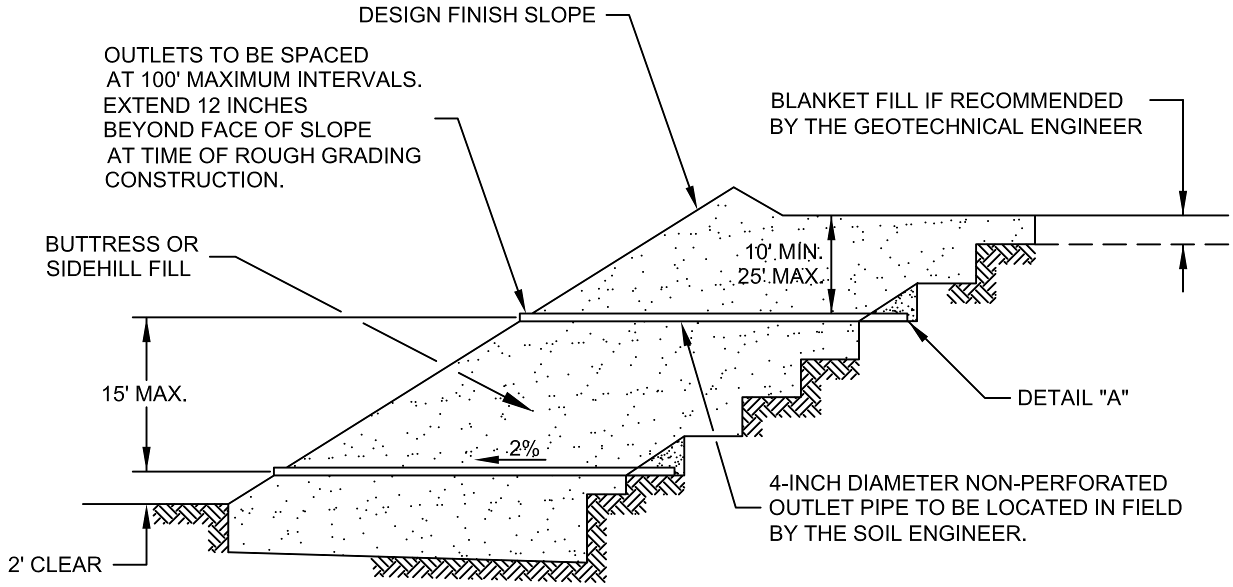
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

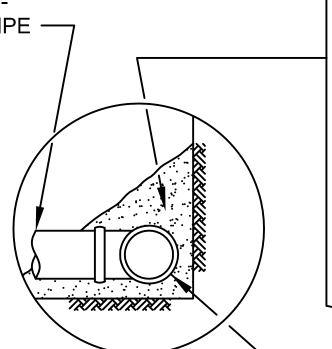
SAND EQUIVALENT = MINIMUM OF 50

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

FILTER MATERIAL - MINIMUM OF FIVE CUBIC FEET PER FOOT OF PIPE. SEE ABOVE FOR FILTER MATERIAL SPECIFICATION.

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

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



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

<u>DETAIL "A"</u>

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

SLOPE FILL SUBDRAINS

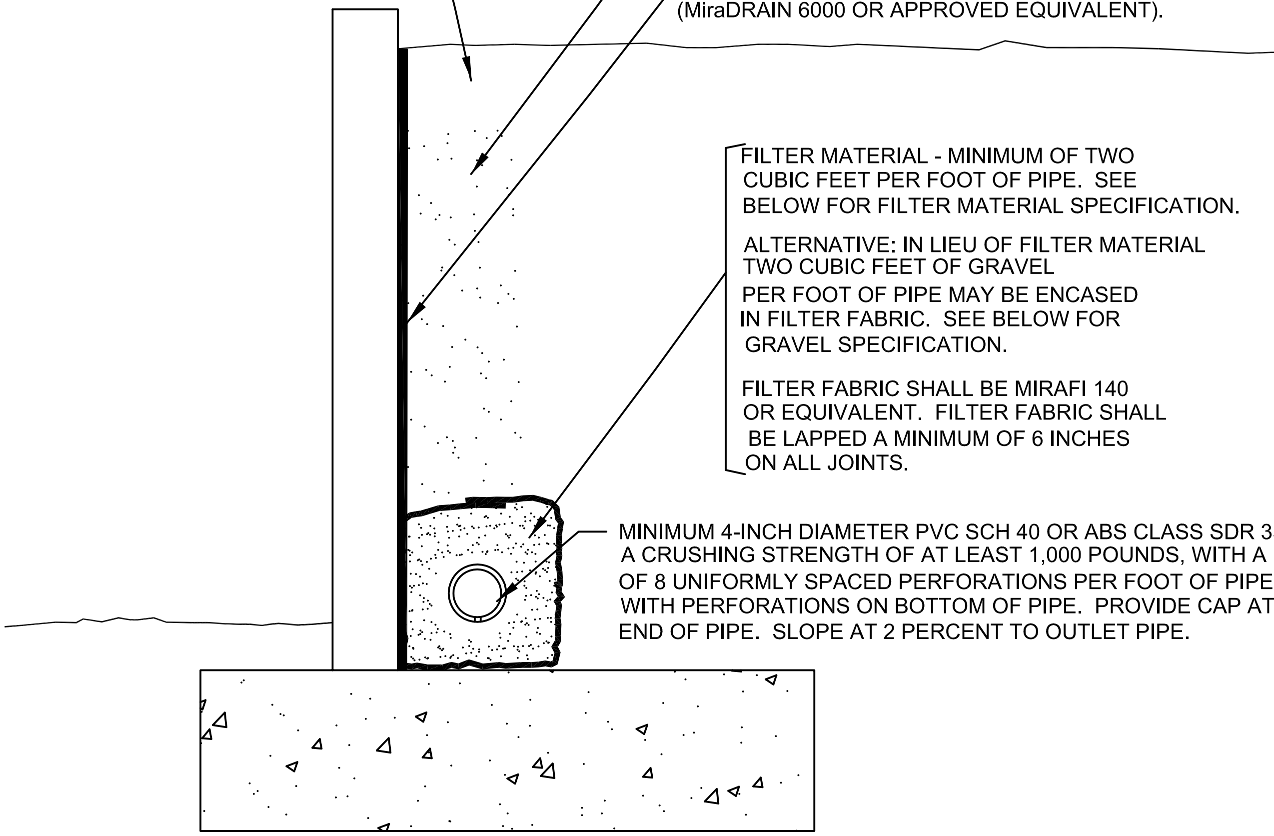
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR
APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

GRADING GUIDE SPECIFICATIONS

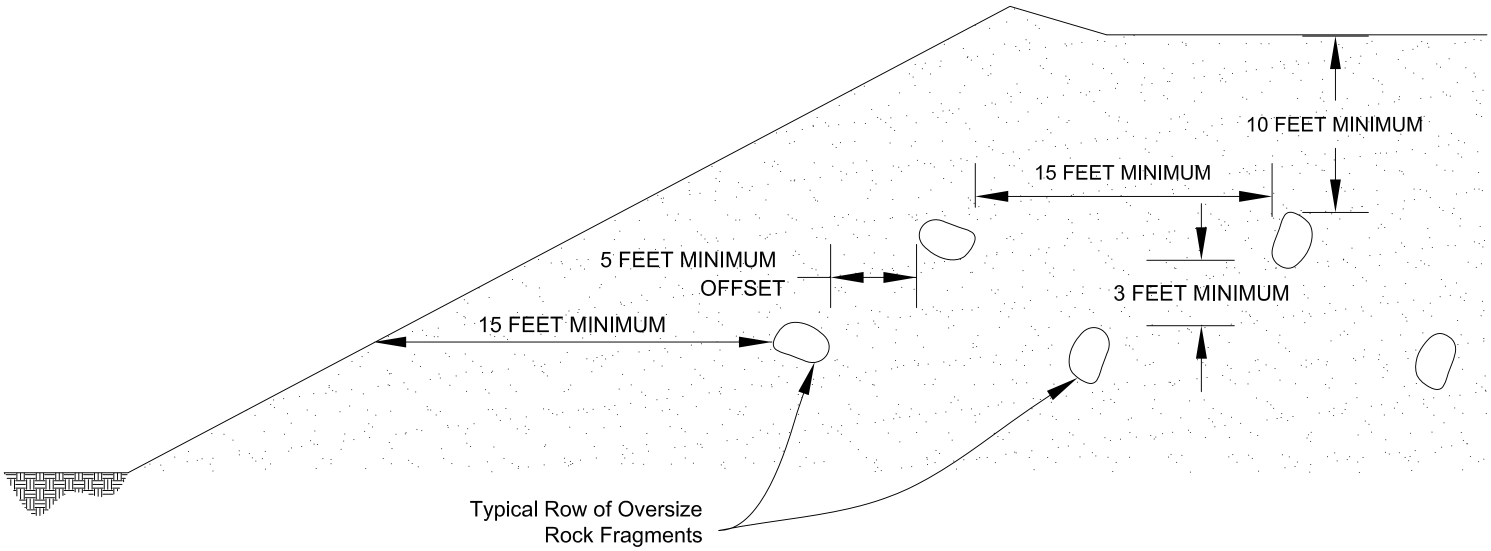
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

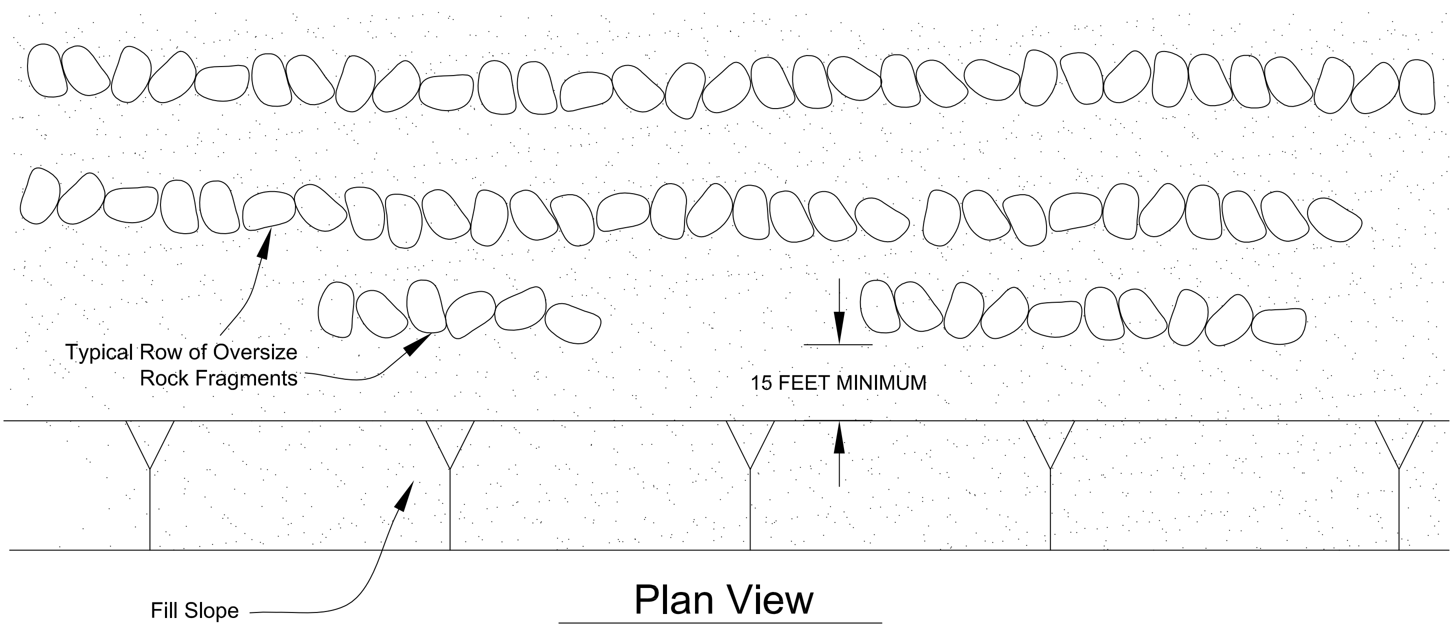
PLATE D-7

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

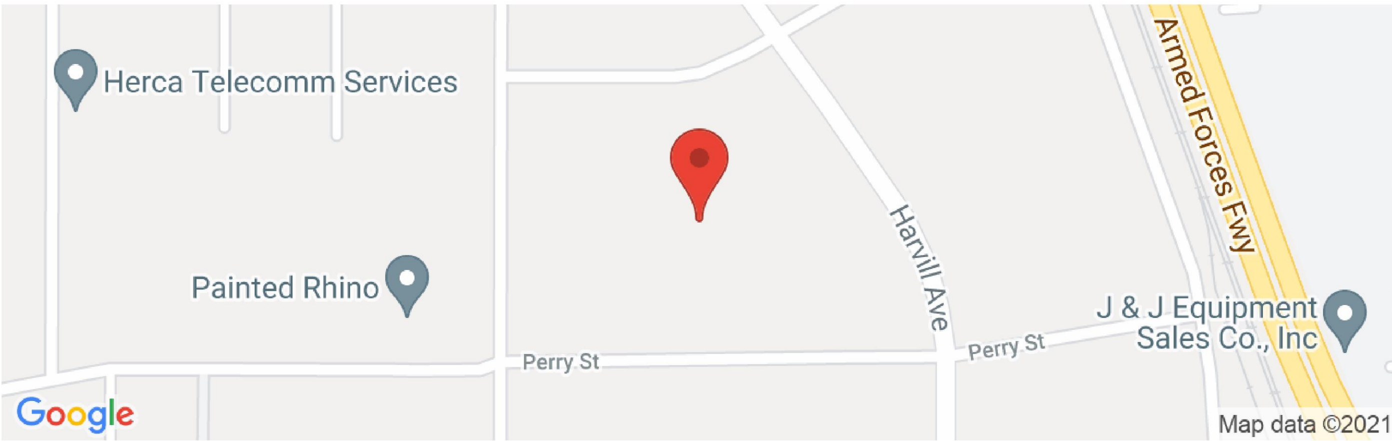
SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL

APPENDIX E

OSHPD

Latitude, Longitude: 33.849465, -117.259510



Date	11/15/2021, 11:52:28 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.564	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.8	Site-modified spectral acceleration value
S_{M1}	0.81	Site-modified spectral acceleration value
S_{DS}	1.2	Numeric seismic design value at 0.2 second SA
S_{D1}	0.54	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.436	Site amplification factor at 1.0 second
PGA	0.5	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.6	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.522	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.626	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.564	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.617	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.936	Mapped value of the risk coefficient at short periods
C_{R1}	0.915	Mapped value of the risk coefficient at a period of 1 s

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

SEISMIC DESIGN PARAMETERS - 2019 CBC
MAJESTIC FREEWAY BUS. CTR-BLDGS 14A & 14B
RIVERSIDE COUNTY (PERRIS), CALIFORNIA

DRAWN: MD
CHKD: RF
SCG PROJECT
21G251-1
PLATE E-1

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